N Sustainability & Climate Change

INTENTIONALLY BLANK

TECHNICAL NOTE - Wave overtopping

JBA Project Code	2016s5126
Contract	East Rhyl Coastal Defence Scheme
Client	Balfour Beatty
Day, Date and Time	05/04/2018
Author	Johan Skanberg-Tippen
Reviewer / Sign-off	Graham Kenn
Subject	Wave overtopping methodology

Project Title: East Rhyl Coastal Defence Scheme			Sheet No: I
Subject: Wave overtopping – Design frontage			Calc No: n/a
Job No: 2016s5126			Version:1.0
Developed By: Johan Skanberg-Tippen	Date: 05/04/2018	Revised By: Johan Skanberg-Tippen	Date: 22/06/18
Checked By: Graham Kenn	Date: 21/06/2018	Approved By: Graham Kenn	Date: 22/06/18

1 Introduction

JBA have been commissioned to develop the detailed design of a rock armour revetment and new sea wall by Balfour Beatty (BB) on behalf of Denbighshire County Council (DCC). The purpose of this document is to present the methodology and assumptions applied within the wave overtopping calculations for the coastal frontages at East Rhyl.

2 Wave overtopping assessment

The complexity of the physical processes leading to wave overtopping introduces a high degree of uncertainty into its quantification. As a result, the overtopping caused by individual waves is not typically calculated; instead the average overtopping rate for a particular sea-state is estimated using empirical or physical models.

The wave overtopping Artificial Neural Network (ANN)[1] - the industry best practice tool - was used in this study along with the latest release of the wave overtopping guidance, EurOtop II (2016)[2]. Results have also been checked against the older well-known release of the ANN as well as empirical formulae.

Estimated wave overtopping rates produced by the ANN are based on a dataset of physical model tests which are affected by scale effects, the accuracy of measurement equipment and wave generation techniques. There is also the potential for limited data for particular schematisations, for example overtopping for low freeboard structures, as limited model tests are available within the database. As a result, it is important that the results of the ANN are used with a degree of engineering judgement and caution.

The ANN tool can be applied to different defence profiles, the geometric properties of which are characterised using 22 parameters including: crest height (Rc); armour height (Ac); armour width (Gc); berm submergence (hb); berm width (B); beach slope (m); upper slope (αu); lower slope (αd); upper slope roughness (γfu); lower slope roughness (γfu); size of upper structure elements (Dd); size of lower structure elements (Du); (see Figure 2-1).

1 Artificial Neural Network, http://overtopping.ing.unibo.it
2 EurOtop II (2016), Manual on wave overtopping of sea defences and related structures, second edition

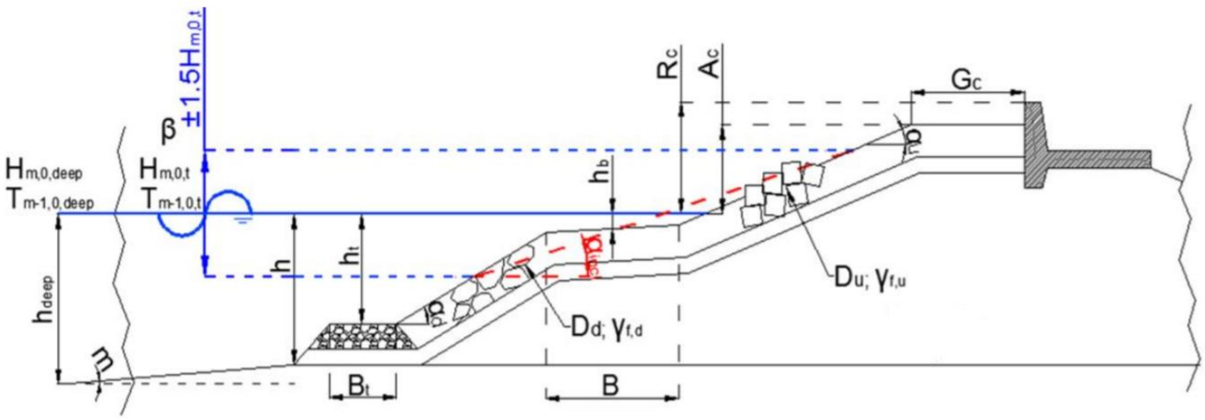

Figure 2-1: Schematisation of a coastal defence (extracted from ANN_tool_user_Manual_V1.0.pdf[1]).

2.1 Wave overtopping frontages

The two separate methods for calculating wave overtopping have been implemented for the **adjacent frontages** (golf course frontage, and west of splash point) and the **design frontage** (splash point to the slipway). This Technical Note discusses the methodology implemented for the design frontage along with the input conditions for both adjacent study sites. For information and results about the adjacent frontage see technical note: *ER-JBA-00-00-TN-C-0003-S1-P01-Overtopping_adjacent_frontage*.

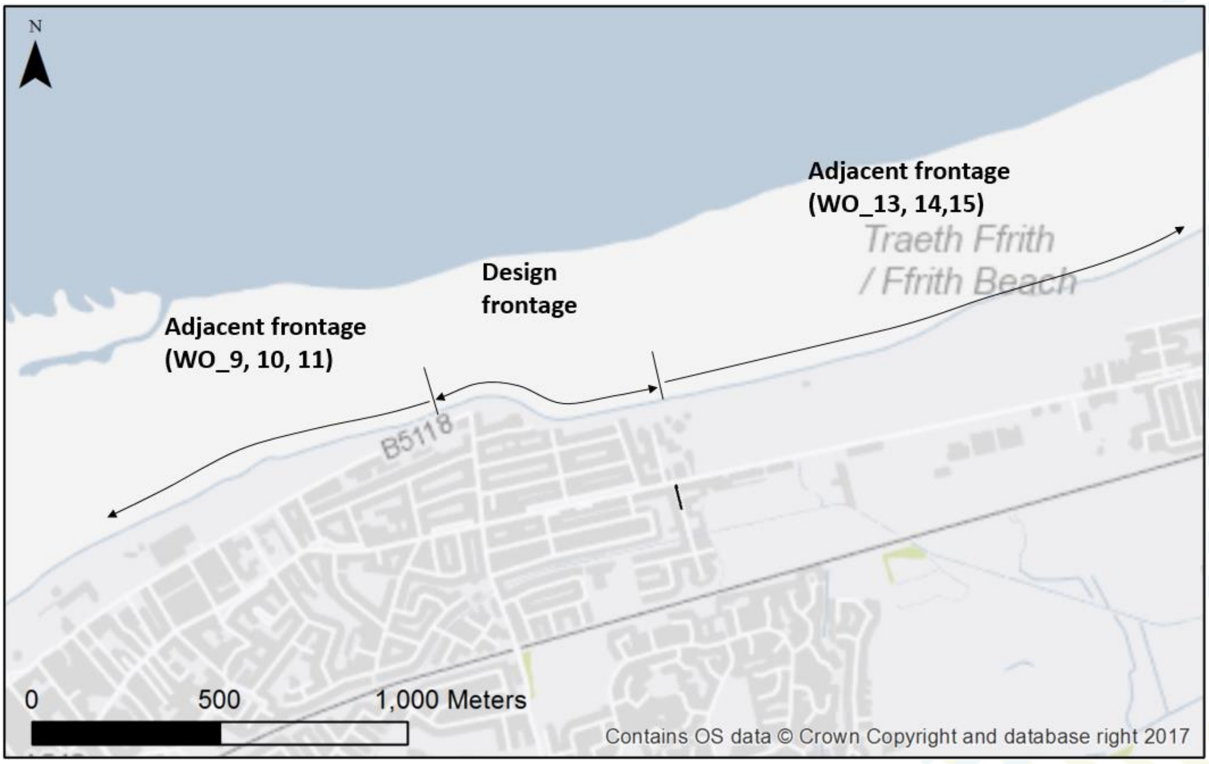

Figure 2-2: Overview of adjacent and design frontages.

2.2 Wave overtopping design process overview

The wave overtopping process flow chart is presented in Figure 2-3 which shows the input data and processing required for both the design and adjacent frontages. The wave overtopping methodology (schematisation and analysis) in this report is specific to the design of the rock armour coastal flood defence.

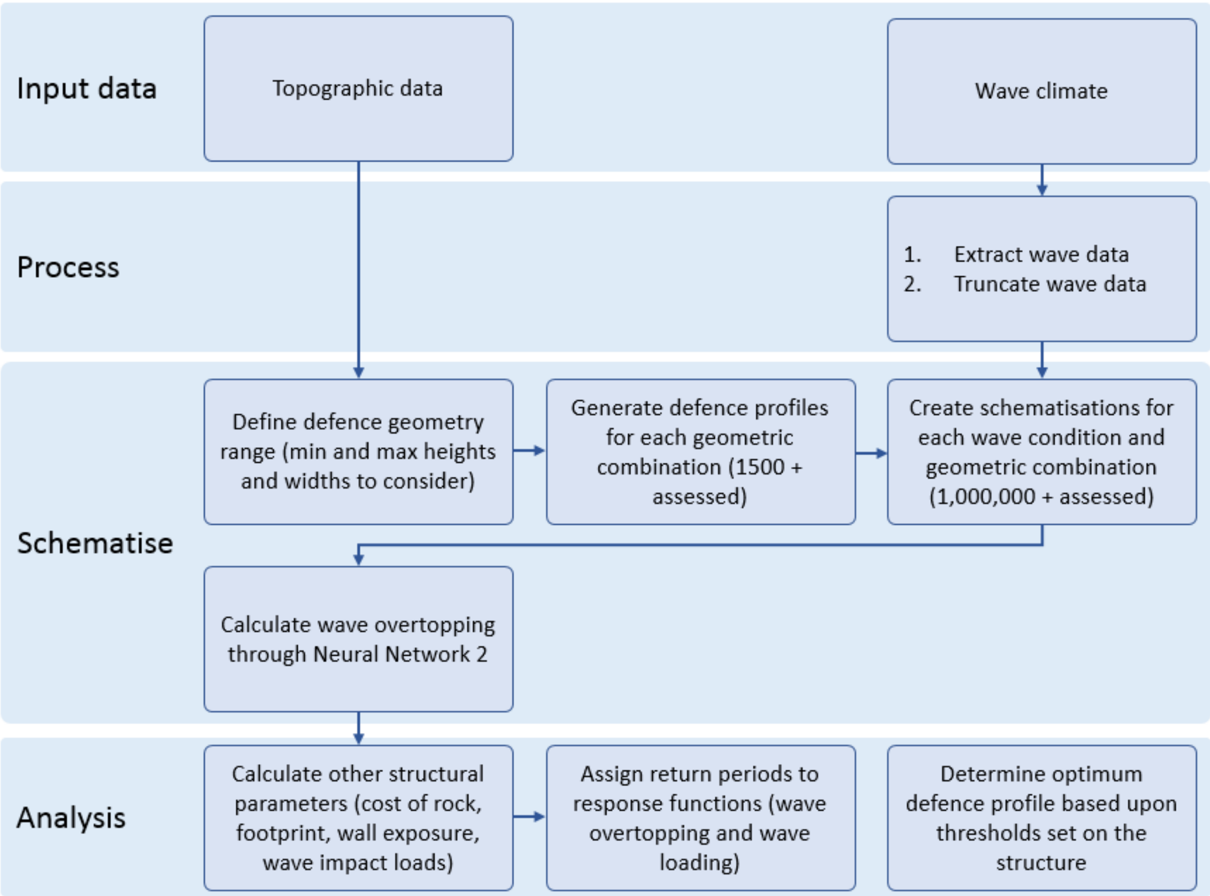

Figure 2-3: Design frontage process flow chart.

3 Input data and processing

To calculate baseline wave overtopping rates two principle data sets are required; the topographic profile of each coastal defence and the wave climate extracted at the defence toe.

3.1 Topographic data

Topographic data is used to 'schematise' a coastal defence to define its key geometric features, such as the height, width, slope and roughness of a defence. Accurately reproducing a defence profile in a wave overtopping schematisation is crucial to achieve representative wave overtopping estimates, minor differences in the defence height or width of a defence will greatly affect the results of the wave overtopping process. All schematisations produced for the East Rhyl coastal defence scheme have been based upon latest topographic survey data produced in the autumn of 2017 (Grantham Coates Surveys Ltd, 2017[3]).

The survey data has been used to create schematisations across the whole design frontage with the exception of the frontages to the east and west of splash point and the golf course frontage where the survey data was not available. In these locations wave overtopping calculations have been produced from older topographic data as used in previous modelling studies (JBA, 2015[4]). Figure 3-1 shows the extent of the topographic data produced in December 2017.

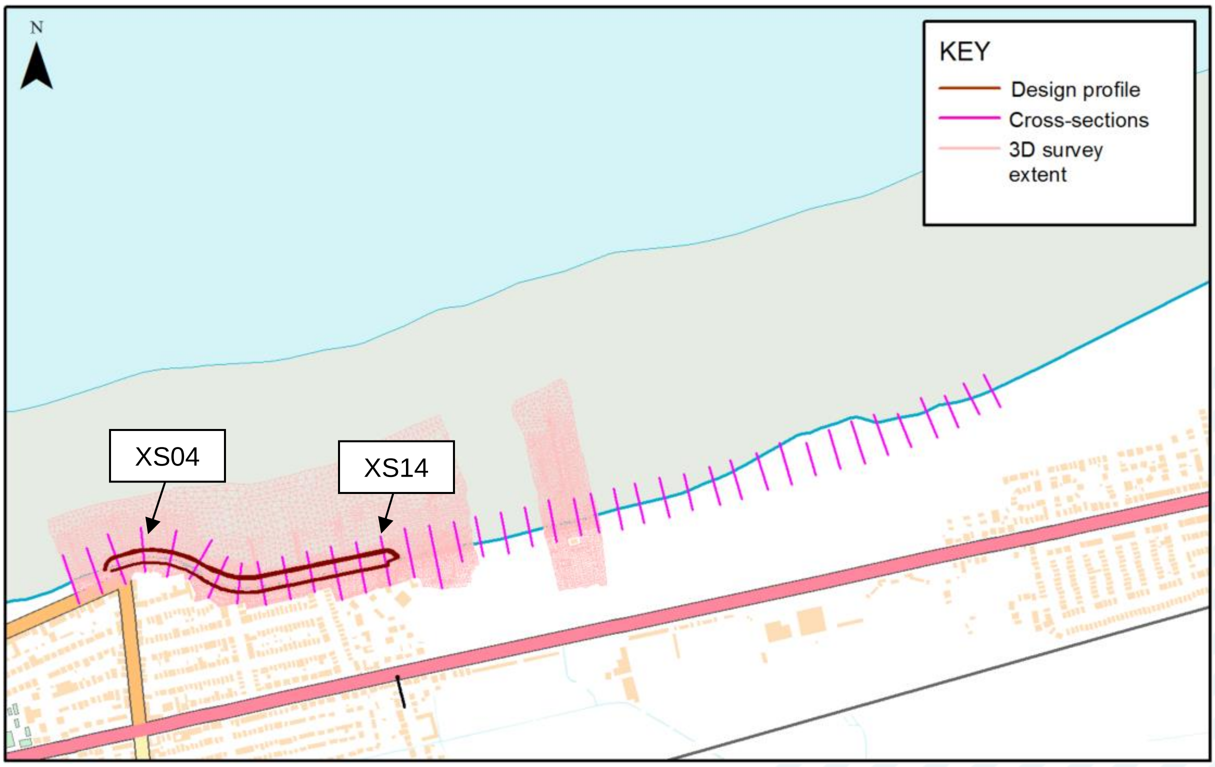

Figure 3-1: Extent of topographic survey produced in December 2017.

3.1.1 Beach levels

The condition (size) of a beach in front of a hard structure significantly affects wave overtopping rates over coastal defences. Large 'healthy' beaches help to naturally dissipate wave energy in the surf zone seaward of a defence, reducing wave heights before they impact on a defence. Conversely, 'unhealthy' small beaches dissipate very little wave energy allowing for larger waves to travel further inshore impacting and overtopping coastal defences with more force.

Wave overtopping calculations that are only concerned with present day beach levels can easily be schematised based upon topographic and lidar survey data. However,

[3] Grantham Coates Surveys (2017), East Rhyl Coastal Protection Scheme Topographic Survey
[4] JBA (2015), 2015s2677_Rhyl Coastal Risk Assessment-Phase 1_Final_Report_1p0.pdf

as beach levels change over time it is important to account for any trends that might occur between present day levels and the 100-year design scenario.

The Technical Report *ER-JBA-00-00-TN-C-0002-S1-P01-Future_wave_conditions* discusses the details of beach trend analysis conducted at Rhyl, from which it concludes that beach levels at the design frontage are likely to lower by approximately 1.3m beyond present day levels in the year 2117. Numerical wave transformation modelling[5] has accounted for this change by using an adjusted bathymetry dataset, and defence schematisations have adjusted the beach levels fronting the toe of the defence to match the lowered bathymetry dataset, thereby accounting for the predicted 1.3m lowering in the 2117 scenario.

3.2 Hydrodynamic data

The wave climate at the toe of coastal defences consist of wave height, wave period, wave direction and extreme water level all of which are crucial in accurately calculating wave overtopping rates.

Each of these wave parameters have been attained through a full numerical modelling process (SWAN) and extracted at specific locations around the coastline at 'toe' points specific to each defence overtopping schematisation. Details around the SWAN modelling process can be found in the report *ER-JBA-00-00-RP-MO-0002-S1-P01-Coastal_modelling_report*. An example of the extracted wave data at the toe of the defence is presented in Table 3-1, representing the 2117 wave conditions for the 200-year wave height, wave period and Extreme Sea Level (ESL) in isolation. Note, this dataset is presented as an example and is not used as the design wave conditions.

Table 3-1: 200-year, 2117 wave conditions for the water level, wave height and wave period in isolation.

200-year x	Run ID	Hm0 (m)	Tm-1,0 (s)	Wave direction (°)	ESL (mAOD)
ESL	13433	2.44	6.24	314	6.21
Hm0	50581	2.81	6.77	329	6.07
Tm	9696	2.76	6.98	334	5.38

3.2.1 Climate change

By selecting a design life of 100 years, it is important to factor in the predicted effects of climate change. The design has incorporated an allowance for climate change for the following:

- Still water levels; and
- Wind and swell driven waves.

The full effects of climate change from the UKCP09 projections have been applied within the SWAN modelling process and have been adjusted for each of the epochs 2017, 2037, 2057, 2077, 2097 and 2117. The wave climate in each epoch has then been applied

[5] JBA (2018), ER-JBA-00-00-RP-MO-0002-S1-P01-Coastal_modelling_report

within the overtopping calculation process. See report *ER-JBA-00-00-RP-MO-0002-S1-P01-Coastal_modelling_report* for full details of climate change and its effect on the wave climate.

3.2.2 Tidal data

Tide data has been used within the wave overtopping modelling process to determine the overtopping rates across the tidal cycle at each coastal defence. For all overtopping calculations, the tidal cycle used within the TUFLOW modelling process[6] was used to calculate the adjusted wave height and water levels at different stages of the tide in the nearshore. Figure 3-2 shows an example of the local tide cycle adjusted for an ESL condition along with predicted depth-limited wave heights; Table 3-2 shows the primary tide levels at Rhyl.

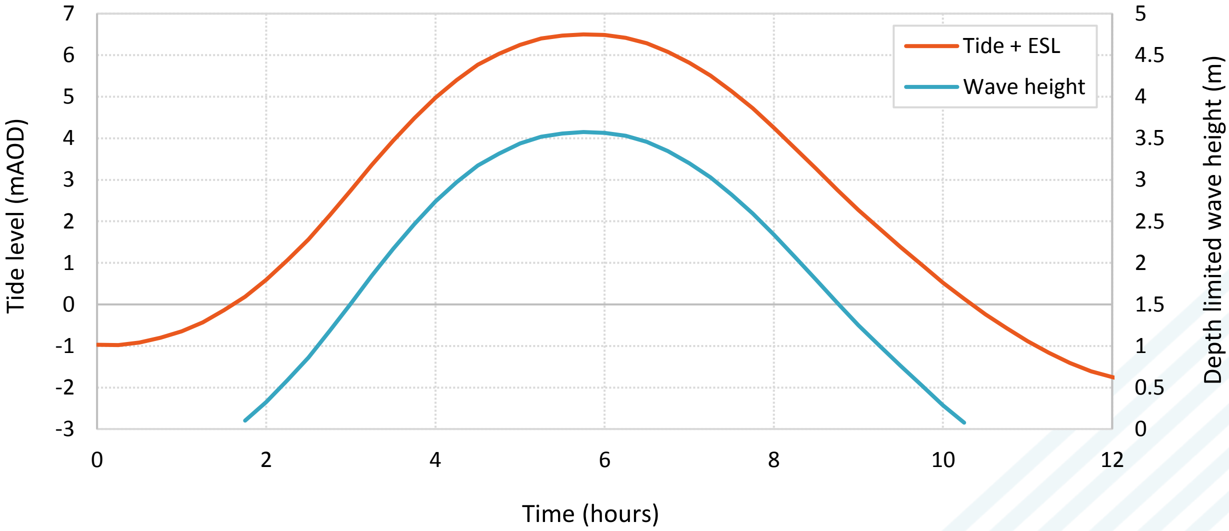

Figure 3-2: East Rhyl Extreme water level + tidal cycle

Table 3-2: Rhyl tide levels

Tide level	ESL (mAOD)
Highest Astronomical Tide (HAT)	5.03
Mean High Water Springs (MHWS)	3.97
Mean High Water Neaps (MHWN)	2.17
Mean Sea Level (MSL)	0.22
Mean Low Water Neaps (MLWN)	-1.70
Mean Low Water Springs (MLWS)	-3.50
Lowest Astronomical Tide (LAT)	-4.56

[6] JBA (2018), ER-JBA-00-00-RP-MO-0001-S1-P01-Wave_inundation_modelling_report

3.2.3 Multivariate wave conditions

Coastal defences experience the greatest wave overtopping rates when large wave heights are combined with large extreme water levels, and it is often the case that large wave heights or water levels in isolation will not cause as severe overtopping. In classic Joint Probability (JP) wave overtopping calculations are undertaken on a range of 200-year events with varying water levels and wave heights. The governing load case is then the JP combination that produces the highest overtopping rate for the defence schematisation. Typically, the JP condition which strikes a balance between a high-water level and large wave height will result in the worst-case overtopping rates. Figure 3-3 presents a hypothetical worst-case wave overtopping condition along with a JP relationship between wave height and water level.

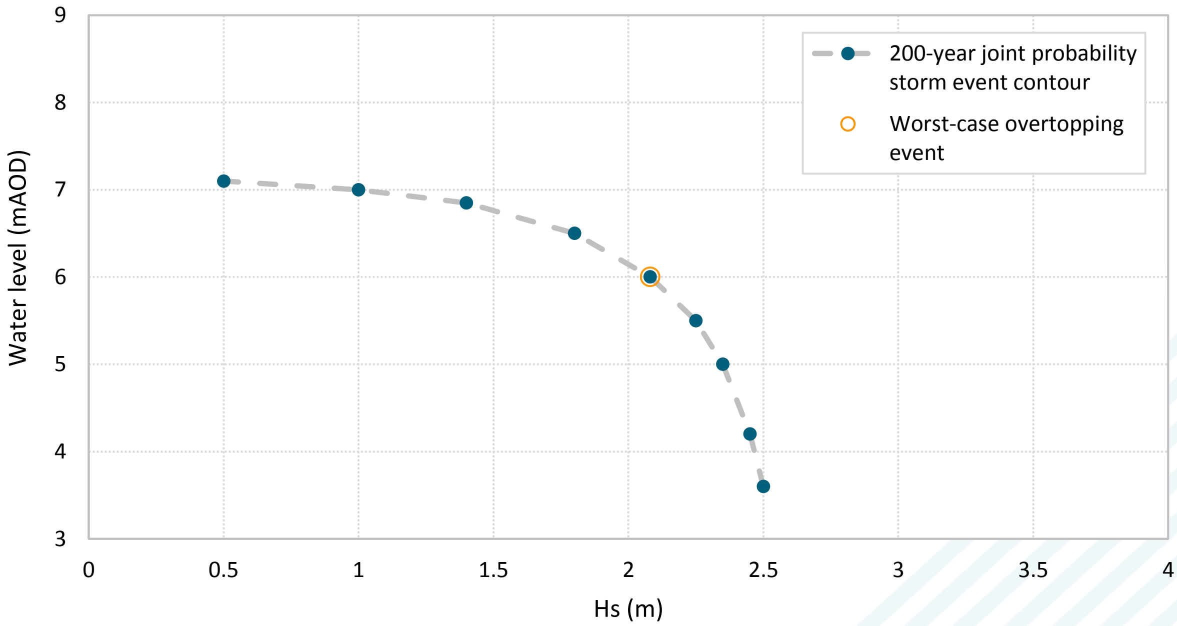

Figure 3-3: Graphical representation of a joint probability contour, demonstrating the interdependency between water level and wave height. ***The figure is for illustration purposes only and does not represent actual wave data.***

The analysis undertaken at East Rhyl has been completed using Multivariate (MV) analysis of wave conditions as opposed to the classic JP approach, as part of an industry wide shift in statistical modelling of coastal processes. This means that rather than considering an envelope of 5 to 10 design wave conditions representing a series of 200-year storm events, a much larger dataset, with over 172,000 wave and water level combinations, ranging from a 10 in 1-year to a 1 in 10,000-year event are studied. Each wave condition within this larger MV dataset does not directly have a return period associated with it, rather a return period is assigned to the response function of interest (wave overtopping calculation), which can be achieved once the overtopping rate has been calculated for all wave conditions.

This new methodology is considered to be more accurate, as the return period is associated with the wave overtopping magnitude (*response function)* instead of the *storm* *event*, which has been proven in recent studies to have the potential underestimate wave overtopping and flood risk (Gouldby, Wyncoll, Panzeri et al., 2017[7]).

Figure 3-4 below shows an example comparison of the JP wave conditions vs. the larger MV wave dataset, demonstrating the theoretical difference in 200-year wave overtopping rates. Here it can be seen that the wave condition from the MV approach has both a larger wave height and water level, which ultimately results in more severe wave overtopping rates.

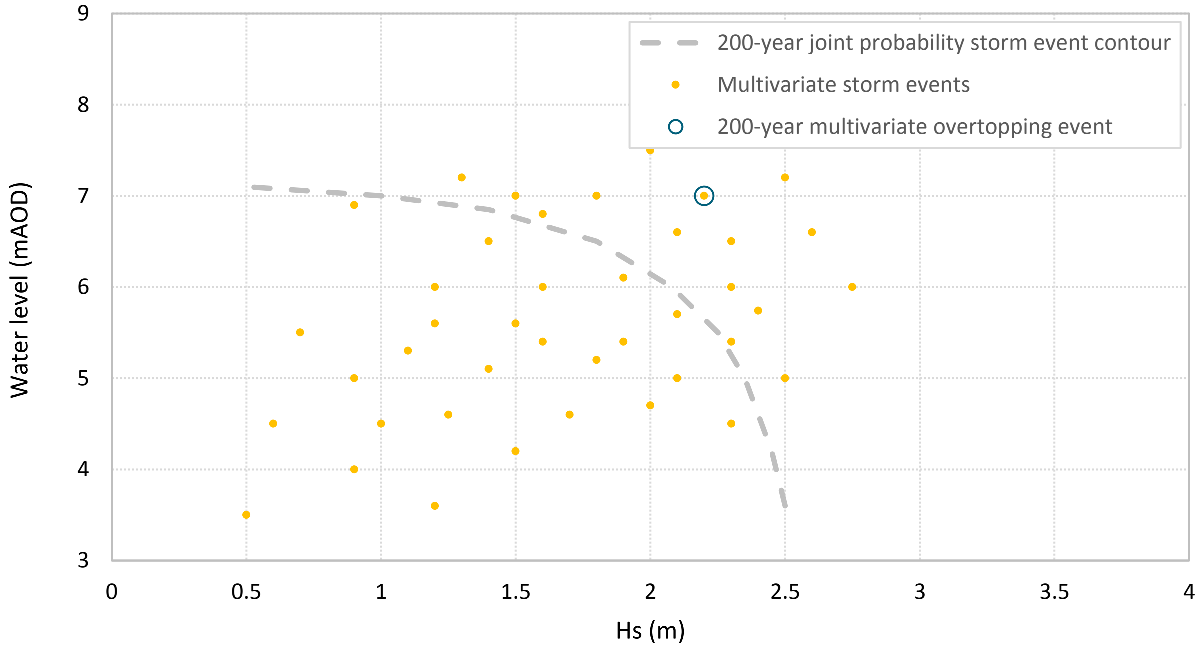

*Figure 3-4: Comparison of a joint probability contour against multivariate extreme events. Note, the mulivarite 200-year overtopping event exceeds the wave conditions predicted by the joint probability approach. **The figure is for illustration purposes only and does not represent actual wave data.***

3.2.3.1 Truncating multivariate data

With the multivariate approach to modelling wave overtopping conditions, the dataset for each toe extraction point contains 172,000+ wave conditions per epoch. Processing all these wave conditions within the overtopping process provides results for all return periods from a 10,000-year overtopping event down to 10 in 1-year events.

As an understanding of these lower return period events is not required and is computationally time consuming, the full wave data set for each location was first processed through a truncation algorithm which reduces the 172,000+ runs down to ~500 runs, containing only the most extreme wave conditions with overtopping return periods from a 1 in 20-year event up to a 10,000-year event.

This truncation algorithm assimilates all wave conditions containing the most extreme wave heights and water levels in both isolation and combination, whilst discarding all other smaller events. This process results in a greatly reduced dataset which contains only those wave conditions that result in the largest overtopping rates. This truncation

[7] Gouldby, Wyncoll, Panzeri et al. (2017), Multivariate extreme value modelling of sea conditions around the coast of England, HR Wallingford

procedure was developed and validated successfully against separate multivariate datasets.

For each wave overtopping schematisation within the Rhyl project the truncated wave dataset has been used. An example of this data subset is presented in Figure 3-5 underlaid by a full MV dataset.

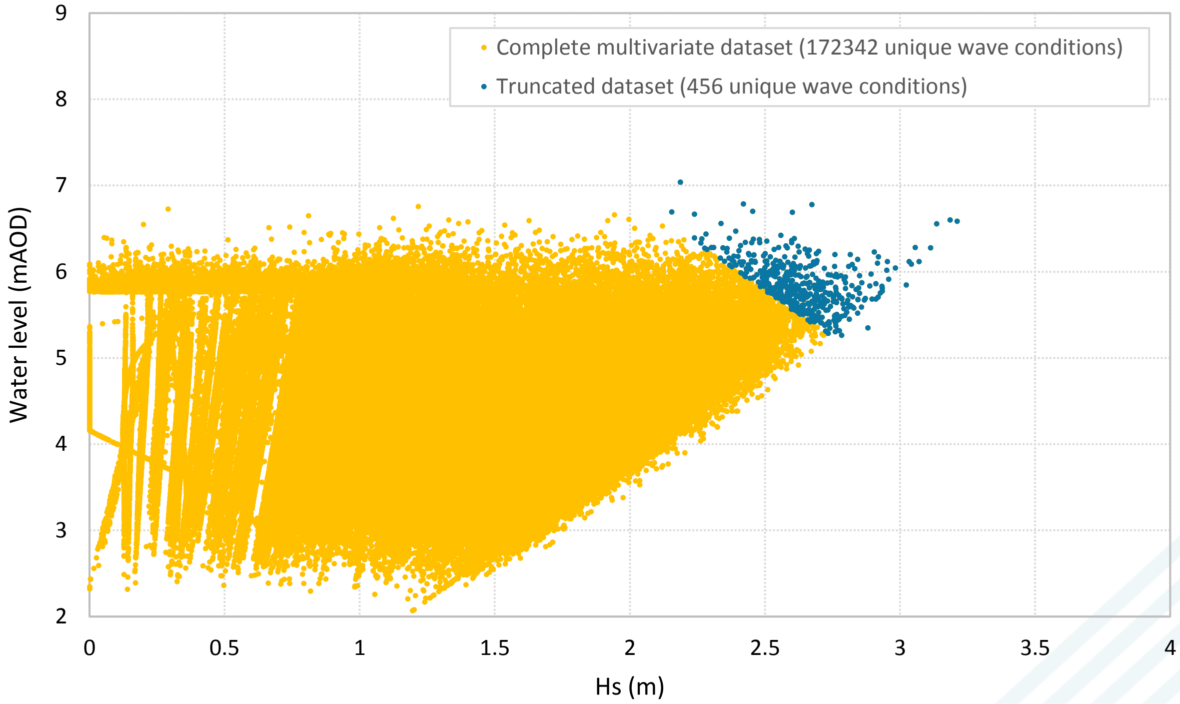

Figure 3-5: A comparison of a complete multivariate dataset highlighted with the truncated dataset used within wave overtopping analysis.

3.2.3.2 Wave data

With a unique set of multivariate wave conditions being analysed for each defence schematisation there is no individual 200-year wave input condition. However, following the wave overtopping analysis at each location the wave conditions resulting in the 200-year overtopping event can be extracted. The details of this are presented within the results (section 4.7).

In isolation, the extreme wave climate (wave height, wave period and water level) for XS04 are presented in Table 3-3. These values are unique to XS04 but are representative of general the wave climate along the Rhyl frontage.

Table 3-3: 200-year, 2117 wave conditions for the water level, wave height and wave period in isolation.

200-year, 2117	Run ID	Hm0 (m)	Tm-1,0 (s)	Wave direction (°)	ESL (mAOD)
ESL	13433	2.44	6.24	314	6.21
Hm0	50581	2.81	6.77	329	6.07
Tm	9696	2.76	6.98	334	5.38

3.2.4 Wave extraction and depth limiting conditions

To calculate wave overtopping rates requires an understanding of the transformed wave climate at the toe of the defence. It is thus necessary to transform wave data to the nearshore of the structure toe within the SWAN modelling processes (details of this process are available in *ER-JBA-00-00-RP-MO-0002-S1-P01-Coastal_modelling_report*).

Due to the limitations of numerical modelling, on average the wave extraction nodes are situated approximately 40m seaward of the actual coastal defence, as presented in Figure 3-6. This is because numerical models are unstable at model boundaries and result is poor accuracy results; to counter this, extraction points tend to be taken at least two cells from the boundary.

While this wave climate data could be used in the overtopping analysis, a considerable amount of transformation can be expected between the 40m offshore location and the toe of the defence. Therefore, some analysis of this transformation is required. No refraction or diffraction has been accounted for in this zone, but depth limitation has been applied to produce representative wave heights and lengths at the toe of the defence.

For the overtopping analysis for the existing coastal defences, the wave data has been adjusted to account for higher beach levels near the toe of the defence through the introduction of depth limiting criteria. This was done by applying a breaker criterion of 0.55 to the local water depth fronting the structure, overriding any wave heights which were greater than this. A breaker criterion of 0.55 was considered to be a suitable value as the foreshore fronting the Rhyl coastline has a slope ranging from 1 in 50 up to 1 in 100, where beaches and foreshores with a slope of 1 in 50 upward typically have breaker indexes in the range of 0.5, as stated in the Rock Manual (2007[8]). A conservative estimation of 0.55 was therefore adopted.

In the case of the design frontage the extraction point 40m offset from the existing coastal defence coincides with the toe location of the proposed rock armour revetment. As such no depth limiting modifications to the wave data were undertaken as this would already be factored into the SWAN model outputs.

Figure 3-6 shows the approximate location of each of the toe locations used for wave extraction.

[8] CIRIA (2007), The Rock Manual: The use of Rock in Hydraulic Engineering - A guide to good practice

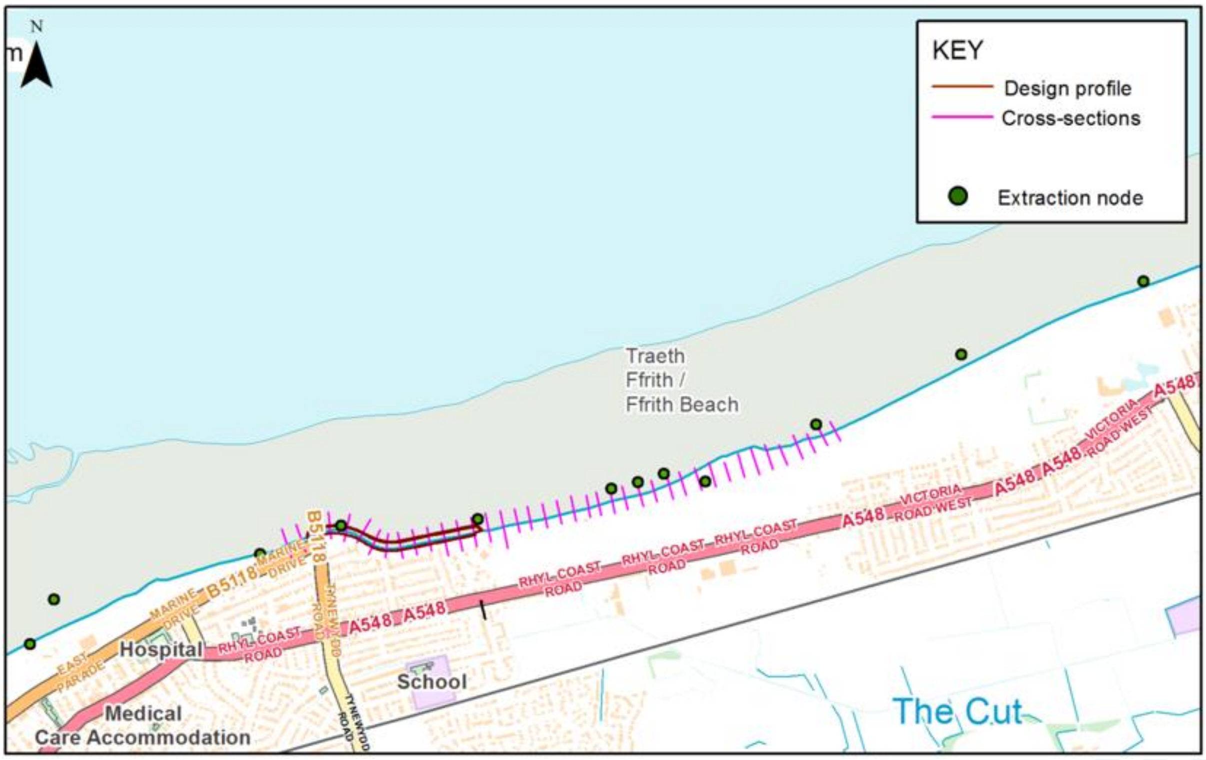

Figure 3-6: Wave extraction locations.

4 Wave overtopping methodology – Design frontage

4.1 Design life and level of protection

The structure has been designed to achieve the following design standards:

- **Design life**: 100 years
- **Design storm event:** 1 in 200-year wave overtopping event (including climate change to the year 2117)

4.2 Performance standards

For coastal flood defences, the performance standards can typically be split into two areas, the still water level performance and wave overtopping performance.

Performance standard 1 – still water level flood risk

Based upon the wave modelling results, the 1 in 200-year still water level (with the effects of climate change) has a maximum level of ~6.2mAOD. The existing defence crest levels across the design frontage vary from 7.25 to 7.40mAOD, and existing promenade levels are in the order of 6.4 to 6.6mAOD. Based on this prediction, it is considered that there will be no risk of flooding to Rhyl caused solely by static water/tide levels (from the design storm event). Therefore, it can be assumed that all coastal flooding risks will be caused by the potential for wave overtopping over any proposed defences.

Performance standard 2 – wave overtopping risk

The performance standard of this coastal defence will be measured against its ability to reduce wave overtopping. The design of the revetment will limit the rate of wave overtopping during the 1 in 200-year wave overtopping event in 100 years' time (2117) to a value that is deemed acceptable for design.

4.3 Tolerable discharges

The design of a coastal defence to offer wave overtopping protection against a storm event, needs to consider the use of tolerable discharges to limit the volume of wave overtopping to levels that are deemed acceptable to the usage behind the defence. Table 4-1 and Table 4-2 summarise the guidance for safe overtopping levels for vehicles, pedestrians and overtopping induced damage provided within the European Wave Overtopping Manual II[5].

Table 4-1: Limits for overtopping for people, vehicles and grass covered landward slopes (EurOtop II, 2016).

Hazard type and reason	Mean discharge q (l/s/m)	Max volume Vmax (l/m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m H_{m0} < 0.5 m	 0.3 1 10-20 No limit	 600 600 600 No limit
Cars on seawall / dike crest, or railway close behind crest. H_{m0} = 3 m H_{m0} = 2 m H_{m0} = 1 m	 <5 10-20 <75	 2000 2000 2000
Highways and roads, fast traffic.	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Table 4-2: Limits for wave overtopping for structural design of breakwaters, seawalls, dikes and dams (EurOtop II, 2016).

Hazard type and reason	Mean discharge q (l/s/m)	Max volume Vmax (l/m)
Rubble mound breakwaters; Hm0 > 5 m; no damage	1	2,000-3,000
Rubble mound breakwaters; Hm0 > 5 m; rear side designed for wave overtopping	5-10	10,000-20,000

Grass covered crest and landward slope; maintained and closed grass cover; Hm0 = 1 – 3 m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; H_{m0} = 0.5 – 3 m	0.1	500
Grass covered crest and landward slope; H_{m0} < 1 m	5-10	500
Grass covered crest and landward slope; H_{m0} < 0.3 m	No limit	No limit

Typically, a coastal defence may be designed to limit wave overtopping in design storm events, such that people can safely access the promenade at the rear of the defence where overtopping waves will impact. With wave heights in the order of 2-2.5m in the nearshore, this would mean that a threshold on wave overtopping rates would be set to less than 1 l/s/m and a Vmax of 2,000-3,000 l/m. As 1 l/s/m is a very stringent threshold, no structural damage or damage to the promenade grass covered promenade embankments would be expected (the threshold for this is 5 l/s/m, providing that it is well maintained).

Instead of adopting a 1 l/s/m threshold, 5 l/s/m has been adopted on the following premise:

- Due to the extremely high wave overtopping rates at the adjacent coastal frontages (to the east and west of the design frontage), DCC have committed that all public access to the promenade in these areas will be strictly forbidden under most storm events. Analysis produced within this study suggest that safe wave overtopping rates (1 l/s/m) will be exceed in present day 1 in 10-year return periods or less.
- It is likely that flood water from the adjacent golf course frontage will encroach onto the design frontage and provide a risk to pedestrian access regardless of the overtopping threshold set.
- The design frontage relies on the existing set-back wall acting as an impermeable flood barrier, meaning that access to and from the promenade will have to be closed during a storm event.
- Modelling results have shown that wave overtopping rates greater than 1l/s/m do not measurably influence the flood extents and depths local to Garford due to large overtopping rates observed at the adjacent frontage.
- Increasing the tolerable overtopping threshold at the design structure to 5l/s/m will reduce the required size of the coastal defence, reducing construction costs and carbon footprint which would be better used for future schemes looking to extend the coastal protection into the adjacent frontage.
- A threshold of 5l/s/m with a Vmax of 2,000-3,000 l/m is the limit at which damage may be expected to grassed embankments which backs the landward face of the defence. Under the design scenario with the effects of climate change (2117) a 200-year overtopping rate will be set to not exceed 5l/s/m. The probability of this overtopping rate in lower epochs (present day to 2097) will be greater than a 200-year event.

4.4 Design methodology

The design frontage was assessed for wave overtopping in two principle locations relating to the areas of greatest wave exposure – XS04 and XS14 (Figure 4-1). At these locations various defence geometries were assessed to determine the most cost-efficient solution that met the performance standard.

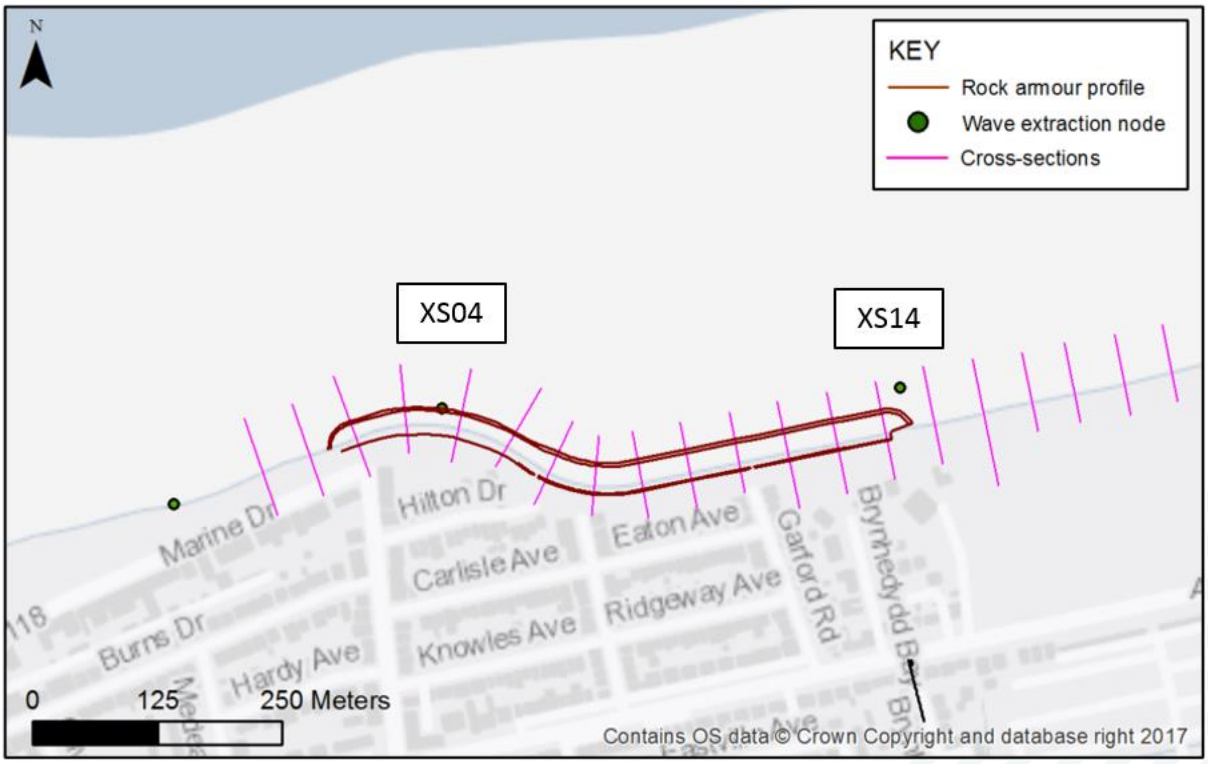

Figure 4-1: Wave assessment locations.

4.5 Defence geometry

Within the design process numerous defence geometries have been tested to determine which structure combination offers the most cost-efficient and sustainable solution whilst meeting the required wave overtopping performance standard.

Table 4-3 and Figure 4-2 show the range of geometric parameters that have been considered within design analysis.

*Table 4-3: Defence geometry assessment range. * Following initial assessments, it was found that narrow crested structures were able to effectively meet overtopping performance requirements thereby excluding wider and more costly structures from analysis.*

Structure element	Minimum dimension	Maximum dimension	Increment	Number of combinations
Top of recurve wall level (Rc)	7.5mAOD	8.0mAOD	0.1m	5
Top of rock armour crest level (Ac)	6.5mAOD	8.0mAOD	0.1m	15

Rock armour crest width (Gc)	4m	9m*	1m	5
Rock armour front slope	1 in 2	1 in 3	0.25	5
Rock armour toe level	2.6mAOD	2.6mAOD	-	1
Rock armour toe width	5.5m	5.5m	-	1
Structure permeability	0.55	0.55	-	1
		Total number of defence combinations:		1875

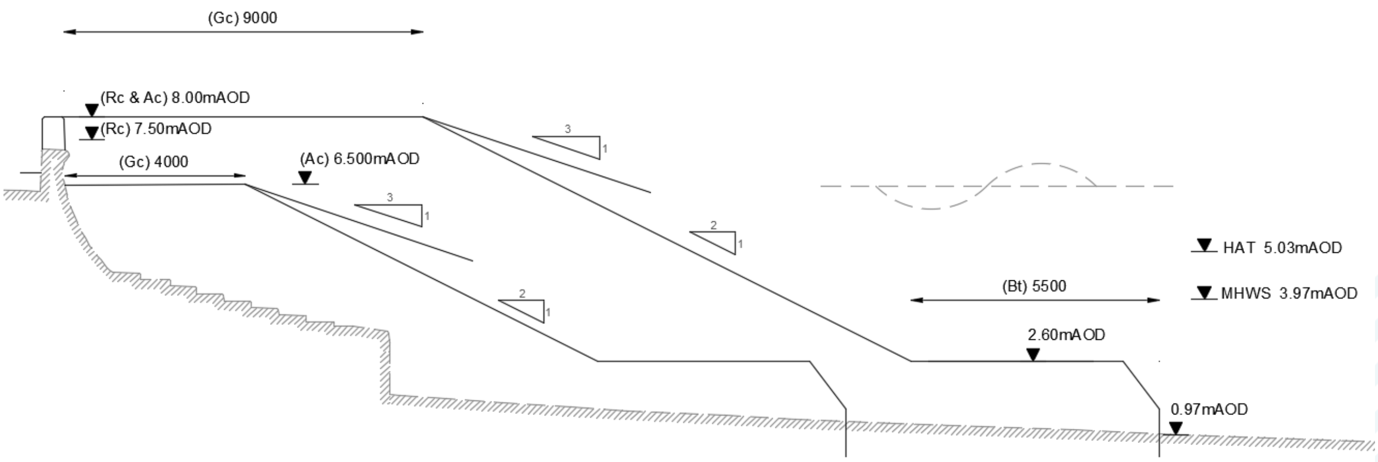

Figure 4-2: Defence geometry assessment range with 2117 beach levels.

4.6 Schematisation

All 1875 No. defence combinations have been schematised against the multivariate wave data extracted at the two locations being analysed (XS04 and XS14).

Location XS04 has a truncated wave dataset of 456 wave conditions, and XS14 having 522 conditions, with a collective 978 conditions. Therefore, a total of 1,833,750 (XS04 = 855,000 + XS14 =978,750) schematisations were produced.

4.6.1 Schematisation validity

Each schematisation has been assessed within the latest release of the ANN (2016) and the input parameters were separately assessed for validity against the limits of the training data, whilst also ensuring the error value (E) output by the ANN was within acceptable limits (requiring a value < 0.5). Where schematisations were out of bounds, adjustments were made to the input data which were then passed back through the ANN.

Wave steepness was the only parameter which was out of range of the ANN training data. Wave periods were too short (in some cases) and were then re-calculated to bring them back into range of the ANN. This was achieved by increasing wave periods to decrease wave steepness. As the wave periods were increased the results would produce slightly larger wave overtopping rates and was therefore considered a conservative and acceptable modification.

4.6.2 Additional calculations

Along with the output of wave overtopping rates from the ANN, separate calculations of wave impact loads on the rear impermeable recurve wall, rock armour revetment footprint, cross-sectional area and cost were also produced. These separate calculations were produced for each defence profile which provided a means of sorting the results in terms of a desired criteria.

Wave impact loads were calculated using the Pedersen method (1996) (The Rock Manual, 2007). To optimise the 1875 No. defence combinations, a means of assessing the efficiency of the structure was required. Cross sectional area, structure footprint and a raw material cost, were used to allow a ranking of all combinations. The cross-sectional area and footprint were derived from the schematised defence profile, and costs calculated from a simplified rate of £55/tonne of placed rock armour.

4.7 Design frontage wave overtopping results

The output from the analysis produced for the design frontage provides a comprehensive dataset of wave overtopping rates, wave impact loads and scheme costs for each schematisation assessed. The data allows for an insight into the wave overtopping rates for all overtopping return periods from a 1 in 10,000-year event, down to a 1 in 20-year event (depending upon the data truncation range).

To determine the most efficient defence geometry tested a number of filtering criteria were applied to the final results which were based upon engineering thresholds set by the wider design team; these thresholds include:

- Wave overtopping rates in the 200-year event < 5 l/s/m
- Exposed height of recurve wall <= 0.3m
- Wave impact loads in the 200-year event <20 kN/m (wave impact calculations were separately validated following this process)

The top 10 results (most efficient structures) from the overtopping analysis are presented in Table 4-4 each of these defence geometries meet the thresholds set and are sorted by cost (low to high). Note, the defence geometries presented are *after* 0.3m settlement over the course of 100-years, and as a result the defence will need to be built to pre-settlement levels. Defence combination 1 has been selected as the preferred optimum defence solution, and defence combination 39 represents the performance of the pre-settlement (+0.3m) revetment.

Table 4-4: Wave overtopping table of results for the top 10 most cost-efficient defence geometries.

No.	Top of rock level, Ac (mAOD)	Top of wall level, Rc (mAOD)	Slope (1 in x)	Crest width, Gc (m)	Cross-sectional area (m2)	Footprint (m)	Approximate cost (£m)	200-year overtopping rate (l/s/m)	200-year wave loading (kN/m)
1	**7.4**	**7.7**	**3.0**	**5**	**24.90**	**70.14**	**6.27**	**4.03**	**19.69**
2	7.4	7.6	3.0	5	24.90	70.14	6.27	4.91	17.77
3	7.2	7.5	3.0	6	25.30	70.92	6.34	4.06	19.29

4	7.7	7.9	3.0	4	24.80	71.00	6.34	4.11	17.69
5	7.7	8.0	3.0	4	24.80	71.00	6.34	3.30	19.59
6	7.8	8.0	2.5	5	23.50	71.38	6.38	4.48	19.35
7	7.5	7.6	3.0	5	25.20	72.10	6.44	4.91	15.27
8	7.5	7.7	3.0	5	25.20	72.10	6.44	4.03	17.12
9	7.5	7.8	3.0	5	25.20	72.10	6.44	3.26	18.97
10	7.6	7.8	2.5	6	24.00	72.83	6.51	4.46	18.85
39	**7.7**	**8.0**	**3.0**	**5**	**25.8**	**76.10**	**6.80**	**2.09**	**17.56**

Figure 4-3 compare the overtopping rate for the top 3 defence solutions, and Figure 4-4 shows the comparison of overtopping performance pre and post settlement for the chosen solution.

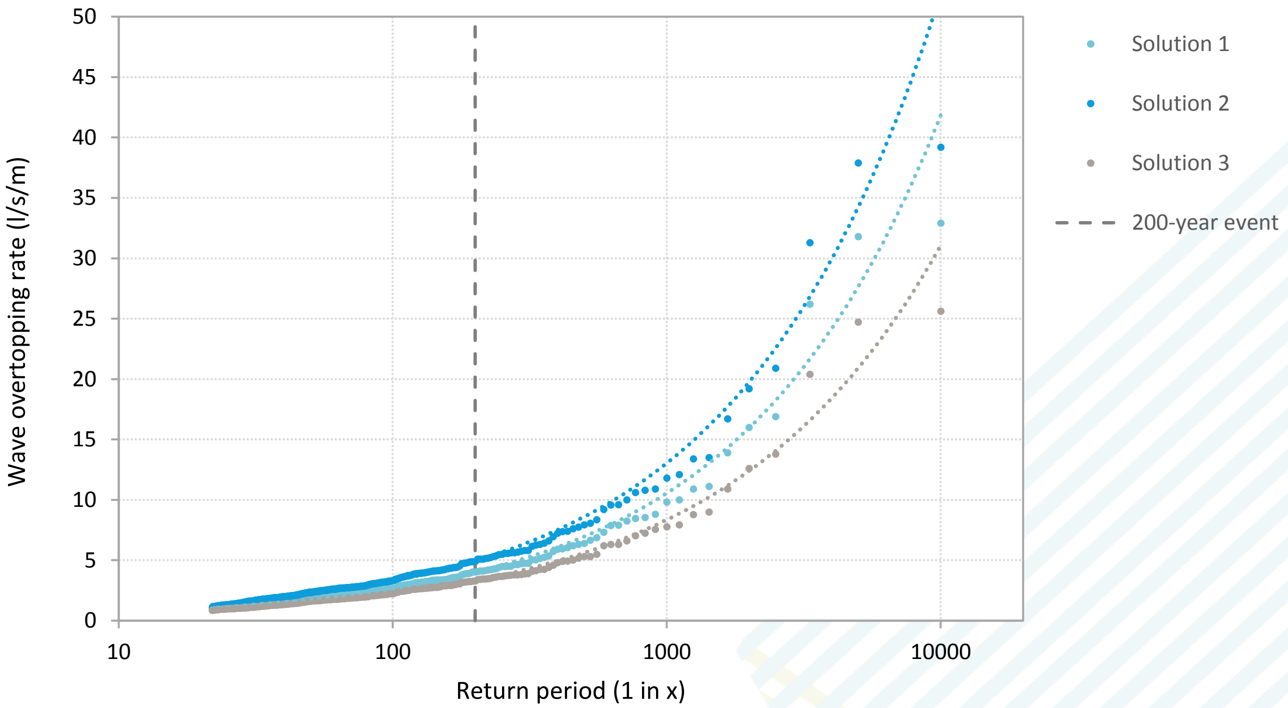

Figure 4-3: A comparison in wave overtopping performance of solutions 1, 2 and 3 presented in the results table.

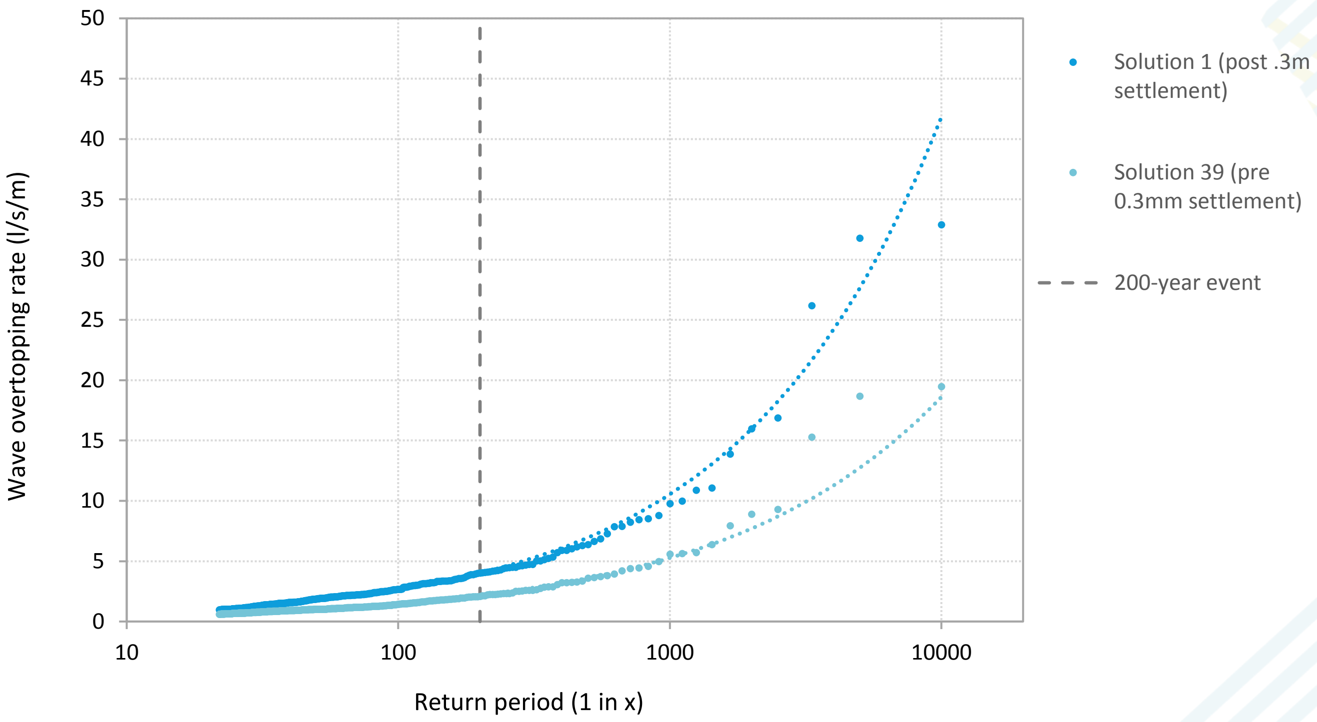

Figure 4-4: A comparison of the selected design structure (solution 1) and the as-built pre-settlement structure (solution 39).

The finalised defence geometry progressed within design is presented in Table 4-5, with the overtopping performance both pre and post 0.3m settlement. **It should be noted that the defence geometry has subsequently been modified as a result of the findings from the physical model.**

Table 4-5: Final selected defence geometry (construction levels, pre-settlement).

	Top of rock level, Ac (mAOD)	Top of wall level, Rc (mAOD)	Slope (1 in x)	Crest width, Gc (m)	10,000-year overtopping rate (l/s/m)	5,000-year overtopping rate (l/s/m)	1000-year overtopping rate (l/s/m)	500-year overtopping rate (l/s/m)	200-year overtopping rate (l/s/m)
Post-settlement	7.4	7.7	3.0	5	32.90	31.80	9.80	6.40	4.03
Pre-settlement	7.7	8.0	3.0	5	19.50	18.70	5.61	3.61	2.09

Table 4-6: Post-settlement 200-year overtopping wave condition.

	Wave ID	Hm0,t (m)	Tm-1,0,t (s)	ESL (mAOD)	Dir (degrees)
Post-settlement 200-year overtopping wave condition	30648	2.887	7.029	5.839	324.3

4.8 Design structure standard of protection results

Through the development of the East Rhyl TUFLOW flood inundation model it became apparent that a large percentage of the flooding was caused from the undefended golf course frontage to the east. It was demonstrated in these first stages of modelling that should no overtopping be allowed at all (0 l/s/m) over the designed rock armour revetment significant flooding would still occur. This led to the need to understand what standard of protection could be achieved by the coastal flood defence and what extension would be required to meet the 2117, 200-year design target.

Two principle processes were undertaken to determine the standard of protection for the rock armour flood defence:

1. Establish the wave overtopping rate over the golf course frontage that would not result in Garford Road flooding,
2. Extrapolate the return period of this wave overtopping event from the multivariate data set.

4.8.1 Maximum wave overtopping rate

To determine the standard of protection offered by the rock armour revetment (in conjunction with the adjacent frontages) it is first necessary to establish how much wave overtopping can be tolerated from the adjacent frontage before Garford Road floods. This was done by incrementally producing TUFLOW flood inundation models for different overtopping rates to determine the point at which flooding does not occur. Figure 4-5 presents a simplified approach of this process, reducing the input wave overtopping rates until no flooding at Garford Road occurs. For each model run, the 5 l/s/m threshold was applied over the design frontage. The process was then repeated for different revetment extensions (300 to 900m) until the upper wave overtopping limit was known. Table 4-7 shows the predicted maximum allowable overtopping rates through the adjacent frontage before flooding of Garford Road occurs.

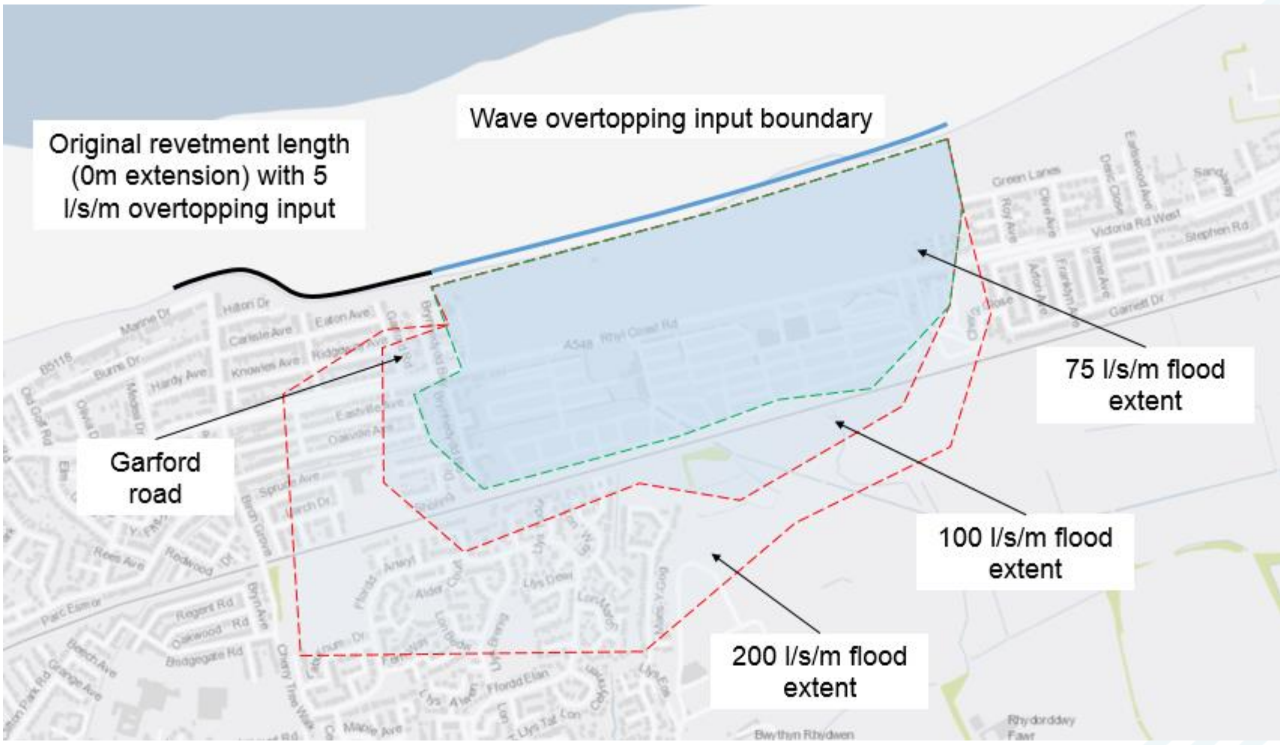

Figure 4-5: Visualisation of flood extents for 3 different wave overtopping inputs (50, 100, 200 l/s/m). ***Note, this image is for illustration purposes only and does not represent accurate flood extents.***

Table 4-7: Maximum allowable wave overtopping rates from the adjacent frontage before flooding to Garford Road occurs.

Defence combination	Maximum allowable wave overtopping rate (l/s/m)
Defence around splash point with golf course wall	75
Defence with golf course wall and 300m revetment extension	100
Defence with golf course wall and 400m revetment extension	120
Defence with golf course wall and 500m revetment extension	135
Defence with golf course wall and 600m revetment extension	170
Defence with golf course wall and 700m revetment extension	215
Defence with golf course wall and 800m revetment extension	260
Defence with golf course wall and 900m revetment extension	350

4.8.2 Standard of protection – assigning a return period to allowable overtopping rates

The second stage of the assessment associates a return period to the maximum allowable wave overtopping rate for each defence extent. To do this, a lookup table was produced to find the return period associated with an average overtopping rate of interest from the multivariate results for the golf course frontage (XS14, 24, 26, 28, 39). This return period provides an estimate of the standard of protection offered by the defence option, any higher return periods would result in flooding to Garford Road (as discussed in

4.8.1). The process was then repeated for each epoch and each defence extension. This process is presented in Figure 4-6.

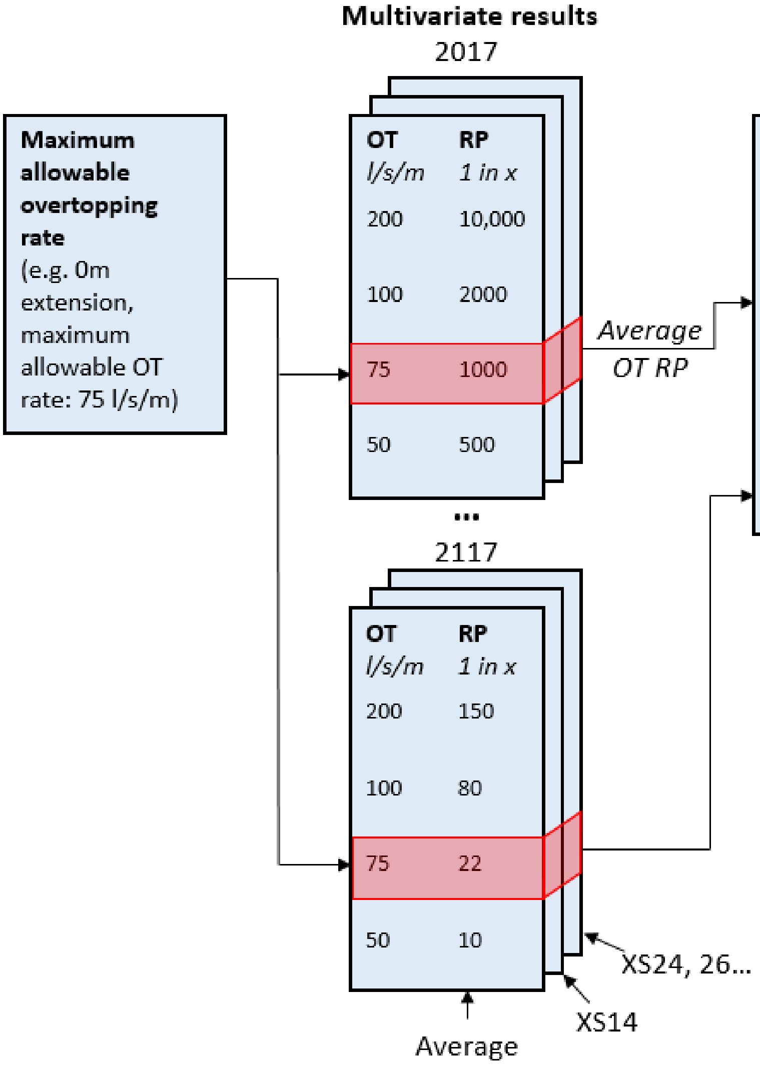

Figure 4-6: Standard of protection flowchart. The return period associated with the average overtopping rate of interest is extracted to inform the standard of protection able to be provided by the flood defence.

Table 4-8 presents the results of the calculations produced for the original un-extended revetment profile. Here the wave overtopping rates for each cross-section are presented which result in an average overtopping rate of 75 l/s/m (maximum allowable before flooding to Garford Road occurs). This is provided along with the multivariate identification number and the corresponding return period associated with 75 l/s/m. The standard of protection under each epoch is therefore equivalent to the return period of the event that does not exceed the maximum overtopping rate.

Table 4-8: Overtopping return period calculation example for the original un-extended defence length. For each epoch the return period is found for wave conditions that result in an average wave overtopping rate of 75 l/s/m – the maximum rate before Garford Road floods.

Epoch	XS14 (l/s/m)	XS24 (l/s/m)	XS26 (l/s/m)	XS28 (l/s/m)	XS39 (l/s/m)	Average (l/s/m)	Multivariate ID No.	Corresponding return period of wave overtopping rate (1 in x)
2117	86	93	102	46	57	75	455	22
2097	97	90	96	45	51	75	260	38
2077	99	86	91	48	53	75	115	87
2057	99	84	90	49	56	75	45	222
2037	96	85	90	50	53	75	16	625
2017	85	98	79	57	56	75	10	1000

The results for the standard of protection for each defence combination is presented in Table 4-11 under the full 1.3m predicted beach drawdown. Further sensitivity analysis was conducted on how the standard of protection varies with different beach levels and it was concluded that 0.3m beach drawdown over the 100-year design life is the most realistic scenario. Table 4-9 and Table 4-10 present the change in standard of protection offered by a 0.3m and 0.8m drawdown respectively, the former being akin to present day conditions. Note: intermediate epochs results are interpolated between known data points.

4.8.3 Conclusions

- Managing golf course frontage beach levels significantly helps to reduce overtopping rates. DCC will need to commit to managing the beach at or above 0.3m drawdown to achieve the desired standard of protection.
- Under no/low beach drawdown, the rock armour revetment without extension offers a 200-year standard of protection up to the epoch 2077 and is still close to this standard until 2087.

JBA Project Code	2016s5126
Contract	East Rhyl Coastal Defence Scheme
Client	Balfour Beatty
Day, Date and Time	05/04/2018
Author	Johan Skanberg-Tippen
Reviewer / Sign-off	Graham Kenn
Subject	Wave overtopping methodology

Table 4-9: Standard of protection for scheme extensions with 0.3m beach drawdown (approximately 0m of sheet pile exposed).

Defence combination (1.3m drawdown)	Present day	2027	2037	2047	2057	2067	2077	2087	2097	2107	2117
Defence around splash point, Garford road with golf course wall	1 in 2500-year	1 in 1667-year	1 in 1250-year	1 in 952-year	1 in 769-year	1 in 488-year	1 in 357-year	1 in 190-year	1 in 130-year	1 in 70-year	1 in 48-year
Defence with golf course wall and 300m revetment extension	1 in 5000-year	1 in 3333-year	1 in 2500-year	1 in 1538-year	1 in 1111-year	1 in 870-year	1 in 714-year	1 in 364-year	1 in 244-year	1 in 122-year	1 in 81-year
Defence with golf course wall and 400m revetment extension	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2857-year	1 in 2000-year	1 in 1111-year	1 in 769-year	1 in 571-year	1 in 455-year	1 in 208-year	1 in 135-year
Defence with golf course wall and 500m revetment extension	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 3333-year	1 in 2500-year	1 in 1429-year	1 in 1000-year	1 in 769-year	1 in 625-year	1 in 278-year	1 in 179-year
Defence with golf course wall and 600m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2857-year	1 in 2000-year	1 in 1176-year	1 in 833-year	1 in 500-year	1 in 357-year
Defence with golf course wall and 700m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2500-year	1 in 1667-year	1 in 1000-year	1 in 714-year
Defence with golf course wall and 800m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 4000-year	1 in 2500-year	1 in 1429-year	1 in 1000-year
Defence with golf course wall and 900m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 4000-year	1 in 2500-year

Table 4-10: Standard of protection for scheme extensions with 0.8m beach drawdown (approximately 0.5m of sheet pile exposed).

Defence combination (1.3m drawdown)	Present day	2027	2037	2047	2057	2067	2077	2087	2097	2107	2117
Defence around splash point, Garford road with golf course wall	1 in 1429-year	1 in 1176-year	1 in 1000-year	1 in 714-year	1 in 556-year	1 in 282-year	1 in 189-year	1 in 92-year	1 in 61-year	1 in 43-year	1 in 33-year
Defence with golf course wall and 300m revetment extension	1 in 3333-year	1 in 2000-year	1 in 1429-year	1 in 1111-year	1 in 909-year	1 in 606-year	1 in 455-year	1 in 192-year	1 in 122-year	1 in 71-year	1 in 50-year
Defence with golf course wall and 400m revetment extension	1 in 10000-year	1 in 4000-year	1 in 2500-year	1 in 1538-year	1 in 1111-year	1 in 870-year	1 in 714-year	1 in 313-year	1 in 200-year	1 in 105-year	1 in 71-year
Defence with golf course wall and 500m revetment extension	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2222-year	1 in 1429-year	1 in 1111-year	1 in 909-year	1 in 435-year	1 in 286-year	1 in 156-year	1 in 108-year
Defence with golf course wall and 600m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 5000-year	1 in 3333-year	1 in 1818-year	1 in 1250-year	1 in 870-year	1 in 667-year	1 in 313-year	1 in 204-year
Defence with golf course wall and 700m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 4000-year	1 in 2500-year	1 in 1538-year	1 in 1111-year	1 in 714-year	1 in 526-year
Defence with golf course wall and 800m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2500-year	1 in 1667-year	1 in 1111-year	1 in 833-year
Defence with golf course wall and 900m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2500-year	1 in 1667-year

Table 4-11: Standard of protection for scheme extensions with 1.3m beach drawdown (approximately 1m of sheet pile exposed).

Defence combination (1.3m drawdown)	Present day	2027	2037	2047	2057	2067	2077	2087	2097	2107	2117
Defence around splash point, Garford road with golf course wall	1 in 1000-year	1 in 769-year	1 in 625-year	1 in 328-year	1 in 222-year	1 in 125-year	1 in 87-year	1 in 53-year	1 in 38-year	1 in 28-year	1 in 22-year
Defence with golf course wall and 300m revetment extension	1 in 1250-year	1 in 1176-year	1 in 1111-year	1 in 769-year	1 in 588-year	1 in 290-year	1 in 192-year	1 in 99-year	1 in 67-year	1 in 43-year	1 in 31-year
Defence with golf course wall and 400m revetment extension	1 in 2500-year	1 in 1667-year	1 in 1250-year	1 in 1053-year	1 in 909-year	1 in 488-year	1 in 333-year	1 in 167-year	1 in 111-year	1 in 63-year	1 in 44-year
Defence with golf course wall and 500m revetment extension	1 in 3333-year	1 in 2222-year	1 in 1667-year	1 in 1333-year	1 in 1111-year	1 in 741-year	1 in 556-year	1 in 256-year	1 in 167-year	1 in 88-year	1 in 60-year
Defence with golf course wall and 600m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 2857-year	1 in 1667-year	1 in 1333-year	1 in 1111-year	1 in 541-year	1 in 357-year	1 in 182-year	1 in 122-year
Defence with golf course wall and 700m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 2500-year	1 in 1429-year	1 in 1053-year	1 in 833-year	1 in 426-year	1 in 286-year
Defence with golf course wall and 800m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 4000-year	1 in 2500-year	1 in 1538-year	1 in 1111-year	1 in 870-year	1 in 714-year
Defence with golf course wall and 900m revetment extension	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year +	1 in 10000-year	1 in 6667-year	1 in 5000-year	1 in 2000-year	1 in 1250-year

INTENTIONALLY BLANK

JBA consulting

Flood Consequence Assessment

East Rhyl Coastal Defence Scheme

Final Report

October 2018

www.jbaconsulting.com

JBA Project Manager

Sam Wingfield
Mersey Bank House
Barbauld Street
Warrington
WA1 1WA

Revision history

Revision Ref/Date	Amendments	Issued to
Draft / 15th October 2018		Balfour Beatty and Denbighshire CC
Final / 22nd October 2018		Planning Application consultees

Contract

This report describes work commissioned by Balfour Beatty, on behalf of Denbighshire County Council as part of the East Rhyl Coastal Defence commission. Balfour Beatty's representative for the contract was Graham Manners. Sam Wingfield of JBA Consulting carried out this work.

Prepared by .... Samuel Wingfield BSc MRes MCIWEM C.WEM CEnv

Principal Analyst

Reviewed by .... Mike Williamson BSc MSc EADA FRGS CGeog

Chartered Senior Analyst

Purpose

This document has been prepared as a Draft Report for Balfour Beatty and Denbighshire County Council. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA Consulting has no liability regarding the use of this report except to the client.

Copyright

Carbon footprint

A printed copy of the main text in this document will result in a carbon footprint of 124g if 100% post-consumer recycled paper is used and 157g if primary-source paper is used. These figures assume the report is printed in black and white on A4 paper and in duplex.

JBA is aiming to reduce its per capita carbon emissions.

Contents

List of Figures

List of Tables

Abbreviations

ANN	Artificial Neural Network
AoD	Above Ordnance Datum
DCC	Denbighshire County Council
ES	Environmental Statement
FCA	Flood Consequence Assessment
FMfSW	Flood Map for Surface Water
JBA	Jeremy Benn Associates
LFRMS	Local Flood Risk Management Strategy
NRW	National Resources Wales
OBC	Outline Business Case
PAR	Project Appraisal Report
PFRA	Preliminary Flood Risk Assessment
PPW	Planning Policy Wales
RoFSW	Risk of Flooding from Surface Water
SLR	Sea Level Rise
SMP	Shoreline Management Plan
SoP	Standard of Protection
SWMP	Surface Water Management Plan
TAN	Technical Advice Note
uFMfSW	updated Flood Map for Surface Water

1 Introduction

1.1 Terms of reference

JBA Consulting were commissioned by Balfour Beatty on behalf of Denbighshire County Council (DCC) to design a coastal defence scheme for the reduction of wave overtopping flood events at East Rhyl.

As part of the planning application for the proposed scheme an Environmental Statement (ES) has been produced. One of the topics covers flood risk, this Flood Consequence Assessment (FCA) will form part of the ES and will also be submitted to Natural Resources Wales (NRW) for approval.

1.2 FCA Requirements

1.2.1 Environmental Impact Assessment (EIA) Scoping and Screening Response

The proposed scheme was submitted to NRW for EIA Scoping and Screening on the 23rd of January 2018. NRW's response, received on the 14th of March 2018 (NRW document reference SC1801), states:

NRW TE welcome the flood risk improvements that the proposals will offer to the local community.

NRW TE advise that a site-specific Flood Consequences Assessment (FCA) produced in accordance with TAN15: Development & Flood Risk should be undertaken to support and inform development proposals at the planning application stage. The FCA should assess the flood risks to, and the potential flood risks arising from the proposed development (including offsite impacts), over the lifetime of the scheme.

NRW TE note that the proposals will involve the complete and/or partial demolition of existing coastal defences, and it will be of importance to ensure that standard of flood protection is maintained at all times during construction stages.

NRW also advised that Welsh Government has recently updated their Climate Change Allowance Guidance for Risk Management Authorities, which was published on 01 February 2018[1].

1.2.2 Pre-planning meeting with NRW

Following the EIA Scoping and Screening feedback, JBA and Balfour Beatty had a meeting with Ryan Knowles (NRW, Development and Flood Risk Technical Specialist).

The meeting discussed the development sequence and how the Standard of Protection (SoP) could be maintained throughout. The outcome was a series of questions for the FCA to answer and requests for what the FCA should cover, listed below:

- What time of year will the existing wall be taken down?
- How long will the construction take?
- For how long will there be a gap between the old wall being taken down and new wall constructed?
- The existing SoP should be maintained throughout construction.

Options were discussed for maintaining the SoP between the old wall being taken down and the new wall being constructed. It was also confirmed that wave action will be dissipated by the new defence and this reduction in risk will not lead to flood risk elsewhere.

1 https://gov.wales/topics/environmentcountryside/epq/flooding/nationalstrategy/guidance/climateguide/?lang=en

1.2.3 Aims and scope of this FCA

The scope of this FCA will therefore be to:

- Assess the flood risks to, and the potential flood risks arising from the proposed development (including offsite impacts), over the lifetime of the scheme. See sections 4.1.5 and 4.1.9
- Describe the sequence of construction including the timings and time of year the wall will be taken down. See section 4.1.6
- Show how the standard of flood protection is maintained at all times during construction stages. See sections 4.1.7 and 4.1.8
- Address the SoP the scheme provides, considering climate change guidance from the latest and previous guidance. See section 4.1.10

2 Background to the flood defence scheme

2.1 Site description

Rhyl is a seaside resort town on the coast of Denbighshire, North Wales (Figure 2-1). The town has historically been protected from coastal flooding by a range of defence structures, which in the east of the town are now failing their performance standards and design lives.

In East Rhyl, the existing defences overtopped in 2013, causing significant damage and disruption to residential and commercial properties. The coastal defence scheme has been designed to provide the Garford Road area of East Rhyl with a 1 in 200 year SoP up until 2077. Properties between Splash Point and the end of Rhyl Golf Course will also benefit (See Figure:2-1 and Appendix A).

Figure 2-1: Location map of East Rhyl

3 Planning policy and flood risk

3.1 Planning policy

3.1.1 Planning Policy Wales

Planning Policy Wales (PPW) Edition 9 was published in November 2016 and sets out the land use planning policies of the Welsh Government and the planning system in Wales. This is supported by Technical Advice Notes (TANs), including TAN 15: Development & Flood Risk.

3.1.2 Technical Advice Note 15: Development and Flood Risk

TAN 15 forms the technical guidance to PPW on development and flood risk. It sets out a long-term approach to the role of development and flood risk and its contribution towards sustainability principles (Section 4.3 of PPW), with regards to flood risks arising from both river and coastal flooding, and from additional runoff from development.

The overarching aim of TAN 15 is to take a precautionary approach and direct development away from areas at high risk of flooding where possible. It clearly states that where development must be considered in high risk areas, those developments must be considered against the justification and acceptability tests set out in TAN 15.

In TAN 15, the whole of Wales is divided into three flood zones (flood risk from Zone A, which represents areas at little or no risk, to Zone C, which shows areas at high risk of flooding or an extreme event). Zone C is further subdivided into C1 and C2 indicating whether the area is defended or not. These flood zones are described fully in Table 3-1.

Description of Zone	Zone	Use within the Precautionary Framework
Considered to be at little or no risk of fluvial or coastal/tidal 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 NRW 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. In accordance with The Welsh Government letter to Chief Planning Officers of 9 January 2014, DCC will need to now also consider the impaction of climate change into account in terms of development planning.
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 a justification test, including acceptability of consequences. Emergency services and highly vulnerable development should not be considered.

Table 3-1: Tan 15 Zone Classifications

Figure 3-1 indicates that the site location for the proposed coastal defence scheme is located within Zone C1. This area is located on 'the floodplain but has previously been developed and served by significant infrastructure'. The proposed development is water compatible flood defence infrastructure so justification for construction in this zone is not required.

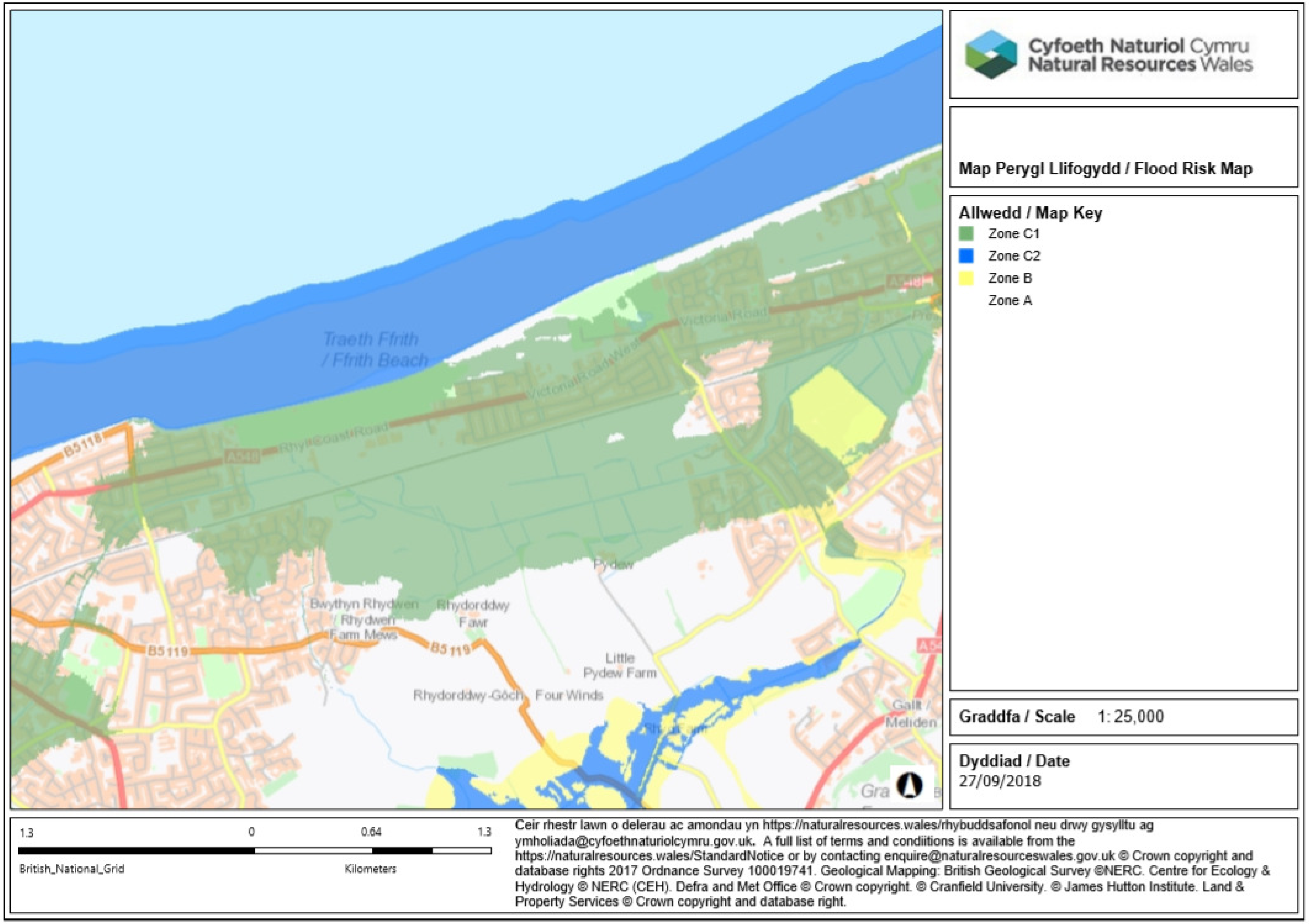

Figure 3-1: Flood Zones at the site

4 Assessment of flood risk

4.1 Coastal

4.1.1 History of flooding

East Rhyl is at risk of coastal flooding and recent events have been severe.

- In February 1990 a storm event led to 108 residential properties flooding in Rhyl and Prestatyn, caused by the overtopping of sea defences.
- 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 deep flooding of 130 residential properties and 400 people had to be evacuated from their homes and others had to be rescued by boat.

4.1.2 Source of risk

The primary source of flooding is wave overtopping. Still water flooding is not a flood risk issue along this frontage as the promenade height is above the 1 in 10,000-year extreme sea level. However, flood modelling has shown that the wave overtopping risk is set to increase due to:

- The effectiveness of the existing defences as their condition declines and they eventually collapse (see 'current management')
- Climate change / sea level risk (SLR) bringing higher surges alongside the wave height.
- The natural lowering of beach levels across the frontage bringing larger nearshore wave conditions.

Recent beach lowering means larger waves are now reaching the coastal defences during common (everyday) storm conditions. In the very nearshore, wave height is limited by their depth; as this depth increases the wave height is correspondingly able to increase. This will lead to greater rates of overtopping as time goes on.

The Garford Road residential area is particularly vulnerable to flooding from wave overtopping as it is in a low-lying basin. Modelling shows that this location will flood at the 1 in 30-year event. By 2077, with sea level rise, it is estimated that this area will flood on average once every five years. This would make the residential community unsustainable and without intervention the only option would be managed retreat. This shows that action is needed to protect East Rhyl now and in the future in order to sustain this community and encourage investment in Rhyl as a popular tourist destination. This aligns with the Shoreline Management Plan (SMP) policy to 'hold the line'.

The Figures in Appendix 1 show flood extents in the area now and in the future.

4.1.3 Modelling undertaken

Detailed wave transformation and wave inundation modelling has been undertaken to understand flood risk during the development of the scheme. This type of modelling was required to gain greater accuracy concerning the amount of overtopping the coastline could be exposed to, and if there are any outflanking risks if overtopping volumes exceeds the residual allowance modelled in the 2016 Project Appraisal Report (PAR) / Outline Business Case (OBC).

Overtopping caused by individual waves is not typically calculated; instead the average overtopping rate for a particular sea-state is estimated using empirical models, this was the approach at the original 2016 PAR stage. The wave overtopping Artificial Neural Network (ANN) was used in the OBC along with the latest release of the wave overtopping guidance, EurOtop II (2016).

Through the development of the inundation model, it became apparent that a large percentage of the flooding came from the undefended golf course frontage to the east.

The following reports describe the wave transformation and inundation modelling and are available upon request:

- East Rhyl Costal Defence Scheme - Wave overtopping methodology, June 2018, JBA Consulting
- East Rhyl Coastal Defence Scheme - Wave inundation modelling report, April 2018, JBA Consulting

4.1.4 Current management

DCC is responsible for coastal risk management along the Denbighshire frontage. Rhyl has been protected from coastal flooding in the past by a range of defence structures, the performance standards of these defences are now being exceeded. The coastal defences in Rhyl comprise a composite of a number of sections, varying in form, material and age. This ranges from historic vertical concrete sea walls (circa 110 years old), to the latest defences incorporating re-curved seawalls constructed between 2009 and 2015 as part of the West Rhyl Coastal Defence Scheme.

At East Rhyl, a number of timber groynes were constructed to control the longshore sediment transport. These have deteriorated over time and are now in various states of disrepair. Sediment modelling during the development of the OBC and scheme design showed that the groynes are not situated at an optimum angle and are limited in their ability to retain beach levels. Other beach control structures are located towards the east of the defence scheme and include a rock groyne field and vegetated sand dunes.

The current flood defences at East Rhyl are managed by inspections every year and ongoing reactive maintenance through revenue funds. Whilst in relatively good condition for their age, the greatest threat to the existing seawall is the exposed toe conditions due to beach lowering. If beach levels drop sufficiently, the defence foundations may be undermined, which could cause a sudden failure. Visual inspections have already exposed sheet pile (likely to be the defence foundations) due to wave conditions from recent storm events. It is unlikely these foundations were designed to resist these wave forces. At some stage, a breach could form as a result of the continued scour of the defence wall toe. Over the longer term, due to the combination of scour, beach lowering and increased wave loads, a complete failure of the seawall could occur.

This means that capital investment will be required in the short to medium term to prevent the stepped revetment and flood wall being undermined as ongoing maintenance through revenue funds cannot sustain the current defences.

Flood events are currently managed through:

- 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.
- Event response: Upon receipt of an NRW coastal alert/warning, or through anecdotal or observed wave overtopping by emergency responders.

4.1.5 The proposed new scheme

The proposed scheme is a rock revetment with a new upstand wall (see Appendix A2 for a schematic drawing). The scheme will firstly place rock armour over the existing concrete stepped structure to dissipate wave energy arriving at the structure.

Following the placement of rock, the proposed scheme includes the demolition and replacement of the recurve wall so that the design life of the whole structure can be guaranteed for 100 years.

This option also depends on the storage capacity of the golf course for residual overtopping events in the future. This storage area was completed separately before this project and DCC are currently investigating their responsibilities for this flood storage area under the Reservoirs Act (1975[2]).

The proposed scheme will significantly reduce flood risk by wave overtopping through the construction of the new rock revetment and sea wall. The scheme will not increase flood risk elsewhere as the purpose of the rock revetment is to dissipate wave energy. There will be no displacement of tidal flood waters.

The figures in Appendix A2 show the reduction in risk over the 100-year design life. Section 4.1.10 of this report provides details on the SoP the scheme provides.

4.1.6 Sequence of construction and timings

Against the current programme **the construction will take approximately three years, between summer 2019 and summer 2022.** The following lays out the proposed sequence of construction and dates:

- A concrete buttress for a new sea wall will be constructed ahead of the rock armour revetment works.
- In preparation for construction of the rock armour revetment, the beach levels would be lowered to 0 m Above Ordnance Datum (AOD) at the location of the revetment toe. Geotextile would then be laid directly into the excavation.
- The rock armour revetment would be constructed in approximately 10 m bays per tidal cycle. All tidal work schedules would be assessed two weeks in advance with tidal working shifts decided also allowing for weather conditions. It is anticipated that tidal works would continue until three hours before high tide.
- Following completion of the rock armour revetment works, the demolition of the existing sea wall would be undertaken as a new pre-cast wall is placed onto the new concrete buttress. This is to ensure that the rock armour is in place to provide protection against wave overtopping.
- It is estimated that **the existing sea wall will start to be taken down in April 2021. There will be a 55 week gap** between the old wall coming down and the new wall being complete.
- The new concrete sea wall will be precast off-site and transported to the site or the Garford Road compound. The reinforced concrete wall base would be cast in situ behind the new sea wall.

The scheme significantly reduces flood risk to residential areas. NRW’s feedback in the EIA Scoping response and the pre planning FCA meeting was for the scheme to maintain the current SoP during construction. The next two sections describe how this will be possible for the two sources of flood risk, namely; 1) wave overtopping; and 2) still water (surge) overtopping.

4.1.7 Maintaining SoP – wave overtopping

The current flood defences at East Rhyl include a concrete stepped revetment and an upstand wall on the top of promenade. The photographs shown in Figure 4-1 show the current revetment and wall and their ineffectiveness during a storm event.

[2] https://www.legislation.gov.uk/ukpga/1975/23

Figure 4-1: The current revetment and wall and their ineffectiveness during a storm event

The proposed construction sequence will be to put in place the new rock revetment before the existing wall is taken down. This means the risk of wave overtopping will be significantly reduced for the full duration of the scheme construction, even with the wall down.

Calculations have been undertaken to compare the wave overtopping rate with the rock revetment in place and the wall removed, against the existing situation. These calculations have been produced using the latest industry guidance and tools (EurOtop II and the wave overtopping Artificial Neural Network). Wave data and return periods are based from the emulated wave multivariate data in the nearshore, local to the cross-section analysed at East Rhyl.

The results are shown in Table 4-1 below. This shows that even with no wave wall, the rock revetment significantly reduces wave overtopping flood risk compared to the existing situation. It can therefore be concluded that the SoP from wave overtopping will not be reduced but improved with the scheme in its temporary state.

Return period (1 in x years)	Defence with existing wall (l/s/m)	New rock revetment, no wall (l/s/m)
500	38	19.10
200	26	12.10
100	18	8.01
50	13	5.22
25	8	2.75
10	3	1.03
5	1	0.57
2	0.5	0.32

Table 4-1: Calculations comparing wave overtopping rates

The next section looks at still water wave overtopping.

4.1.8 Maintaining SoP – still water (surge) overtopping

The primary source of flood risk at East Rhyl is wave overtopping, the promenade and natural height of the land are well above extreme sea levels normally used to assess flood risk.

During construction, the **frontage level of the promenade will fall no lower than 6.35 m AOD**. The present day extreme sea level predictions show that this minimum level is greater than the 1 in 10,000-year event as shown in Table 4-2 below.

The risk of still water flooding along this frontage can effectively be discounted due to the promenade height. The existing wave wall is not currently providing still water overtopping protection. It was designed to reduce wave overtopping. Wave overtopping will be significantly reduced from the present situation, even with the wall taken down due to the new rock revetment that will be in place first.

Return period 1 in x-years	Extreme sea level (mAOD)
10,000	6.33
5000	6.07
2000	5.96
1000	5.59
200	5.34
100	5.31
25	5.26
10	5.10
5	4.90
2	4.18

Table 4-2: Extreme sea level by return period

The tidal still water boundary was calculated through the generation of a design tidal graph. This is a time-series that quantifies how sea levels change through time during an extreme event. These design tidal graphs were used to drive the still water component of flooding at the model offshore boundary.

Derivation of the design tidal graphs required two principal sources of information:

- Extreme still water sea level estimates taken from the latest coastal extreme guidance for the UK (CFB[3]) for the return periods of interest
- A design surge shape taken from the latest coastal extreme guidance for the UK

Figure 4-2 shows the heights of the existing promenade level and wave wall. The proposed height of the rock revetment can also be seen.

[3] Coastal flood boundary (CFB) conditions for UK mainland and islands. Project: SC060064/TR2: Design sea levels. Flood and coastal erosion risk management research and development programme. Environment Agency

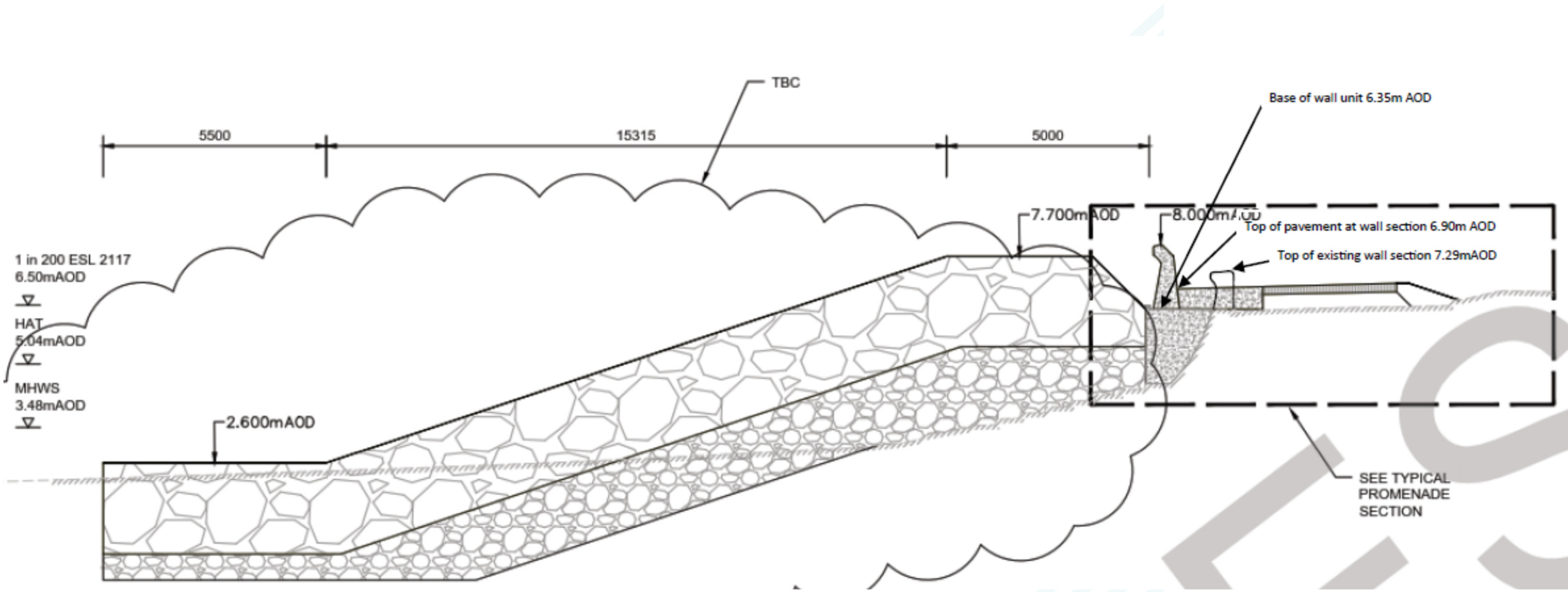

Figure 4-2: The heights of the existing promenade and wave wall, with proposed height of rock revetment.

4.1.9 Flood risks during construction

This section describes the risks and mitigation should a storm even occur during construction.

In general, construction within tidal zones has been simplified where practicable to reduce time spent in an exposed coastal environment. For example, the buttress in-situ wall is unreinforced, simplifying the construction process; the new sea wall has been designed to be precast with an in-situ heel and downstand, reducing the amount of formwork, reinforcement and wet concrete poured on site thus increasing the speed of installation.

Beach access steps have also been designed to be precast and installed on a foundation made of filter layer material and unreinforced concrete, minimising the time spent on site for its construction.

Other steps that will be taken to reduce risks during a storm are listed below:

- Prior to construction, a Storm Action Plan will be developed to prevent public use of the promenade during storm event. If a storm event is forecast, the promenade will be closed by existing secondary flood gates. Signage will be installed to warn of associated hazards. The new step access flood gates will also be closed to prevent excessive overtopping and restrict public access to the beach.
- The contractor will register for NRW flood forecasting warnings.
- The contractor will provide lifesaving equipment, training and toolbox talks covering working in a tidal environment.
- Plant and materials will be removed from at risk areas overnight and there will be careful consideration of material placement scheduling to ensure materials at risk of tidal/wave displacement are appropriately protected at the earliest opportunity.
- Precast units will require propping if a large storm is forecast.
- Site storage of fuel and chemicals shall be above any flood water level and where possible away from high-risk locations.

4.1.10 SoP and climate change guidance comparison

Since completion of the PAR and commencement of detailed design, Welsh Government have developed updated climate change guidance, which they require to be adopted for all Coastal Risk Management projects commencing after December 2017. The guidance below has now replaced 'Adapting to Climate Change'[4].

https://gov.wales/topics/environmentcountryside/epq/flooding/nationalstrategy/guidance/climateguide/?lang=en?

This new guidance from Welsh Government means sea level rise predictions now align with the climate change guidance for development planning in TAN 15. There is now consistency across scheme design and development planning.

The new guidance predicts a higher rate of sea level rise which means that the proposed design SoP would not be as high under the new guidance.

The design of the scheme had already started by the time the guidance was released so the scheme follows Welsh Government's transitional arrangements which allows the scheme, at this stage, to progress if the design is too far advanced to redesign using the new guidance.

The SoP that the scheme offers has been shown in the table and figure below against both sets of guidance.

The Garford Road area of Rhyl currently floods at the 1 in 30-year event today decreasing to a 1 in 5-year event in 2078. The new scheme provides an

[4] Adapting to Climate Change: Guidance for Flood and Coastal Erosion Risk Management Authorities in Wales, December 2017

improvement to that standard against both sets of guidance up to 2078. By the end of the scheme's design life it will offer a 1 in 48-year and 1 in 5-year SoP against the TAN 15 climate change guidance.

	Present day	2038	2078	2118
UKCP09	1 in 2500 yr	1 in 1250 yr	1 in 357 yr	1 in 48 yr
FCDPAG3 (TAN15)	1 in 1667 yr	1 in 833 yr	1 in 69 yr	1 in 5 yr

Table 4-3: Comparison of the schemes SoP against climate change guidance

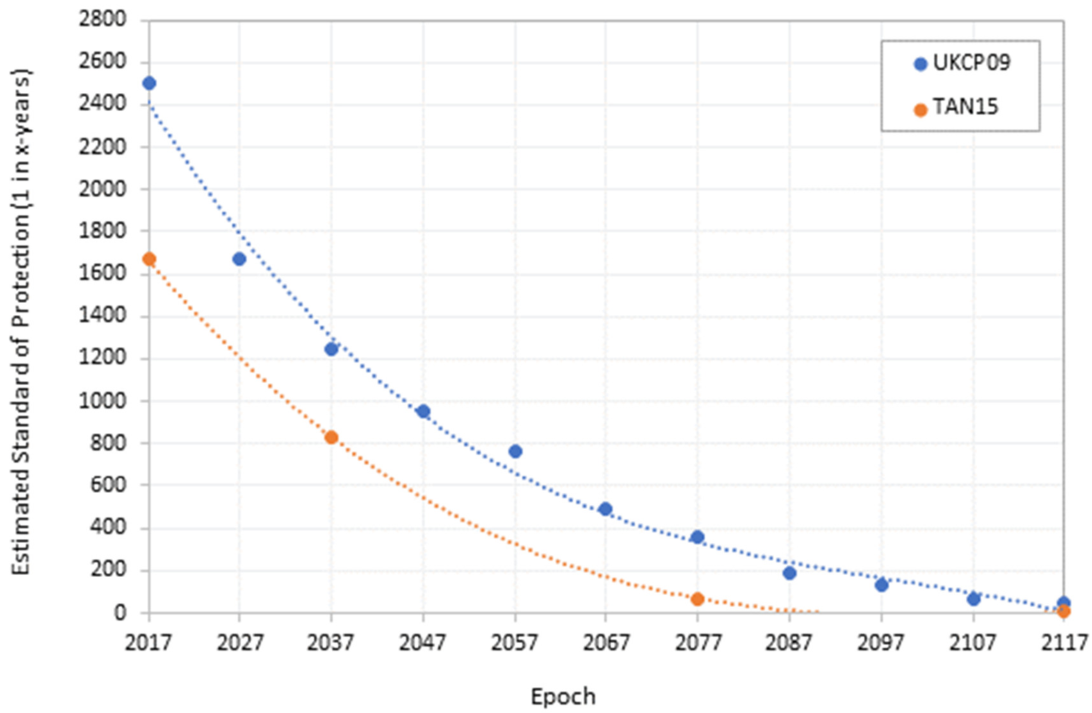

Figure 4-3: Comparison of fall in SoP with climate change against both sets of guidance

4.2 Fluvial flood risk

There is no known fluvial flood risk in the scheme study area.

Figure 4.4 shows NRW's Floodmap Flood Zones, showing the scheme location is at risk form the 1 in 200 year tidal flood event. These undefended Flood Maps also show the areas that benefit from the existing flood defences at East Rhyl.

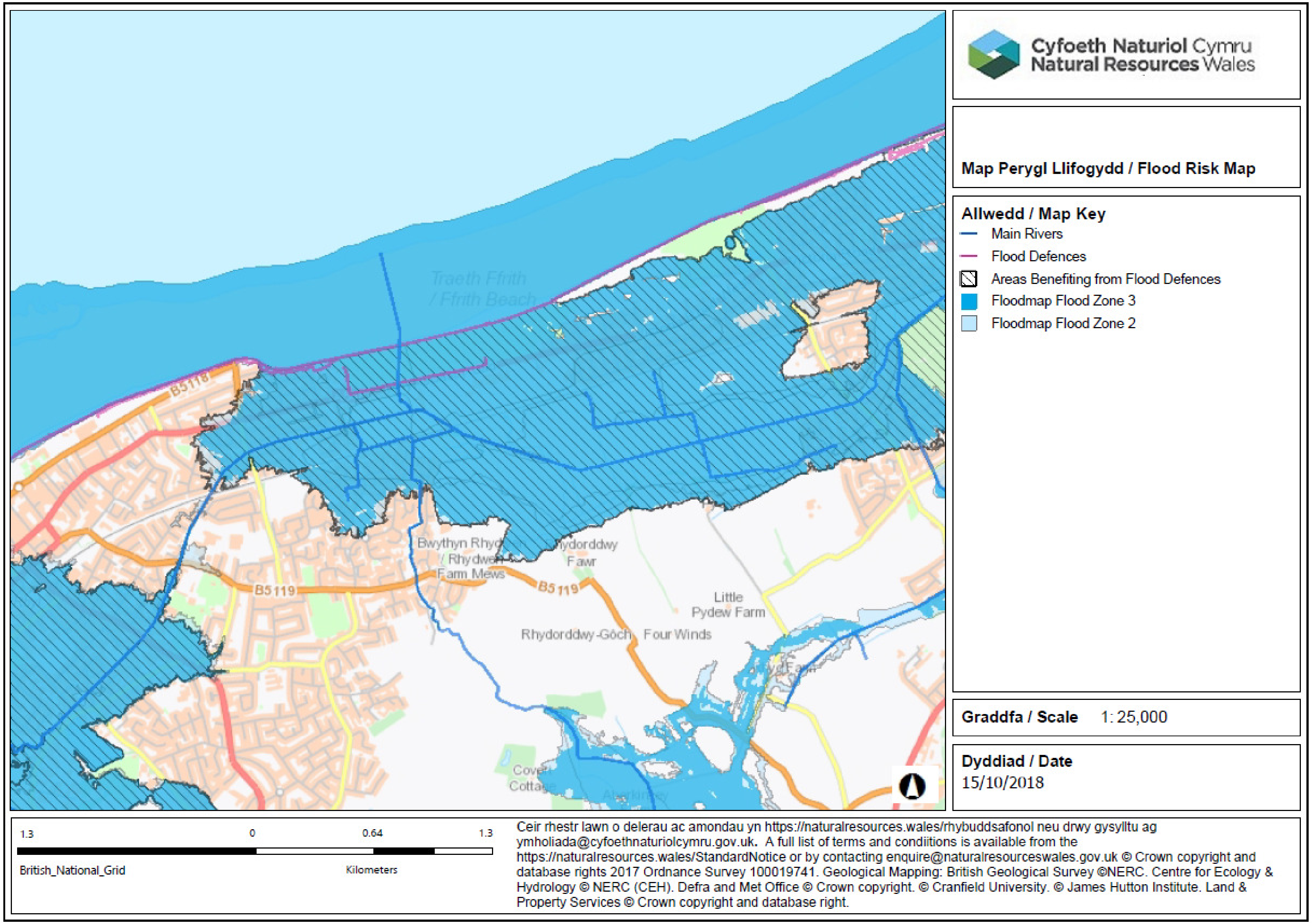

Figure 4-4: Risk of Flooding from Rivers and Sea

4.3 Surface water flood risk

DCC stated in the Preliminary Flood Risk Assessment (PFRA) that the local surface water drainage system has generally been designed to accommodate a 1 in 5 to 1 in 30-year storm event.

Figure 4-6 highlights the risk of flooding from surface water within the Rhyl area. This map shows that the location of the proposed scheme is primarily at low risk of flooding from surface water with some parts being at medium and high risk of surface water flooding.

There are no historic records of properties being flooded by surface water however the new scheme will provide a new promenade with new surface water drainage (see Figure 4.5 below).

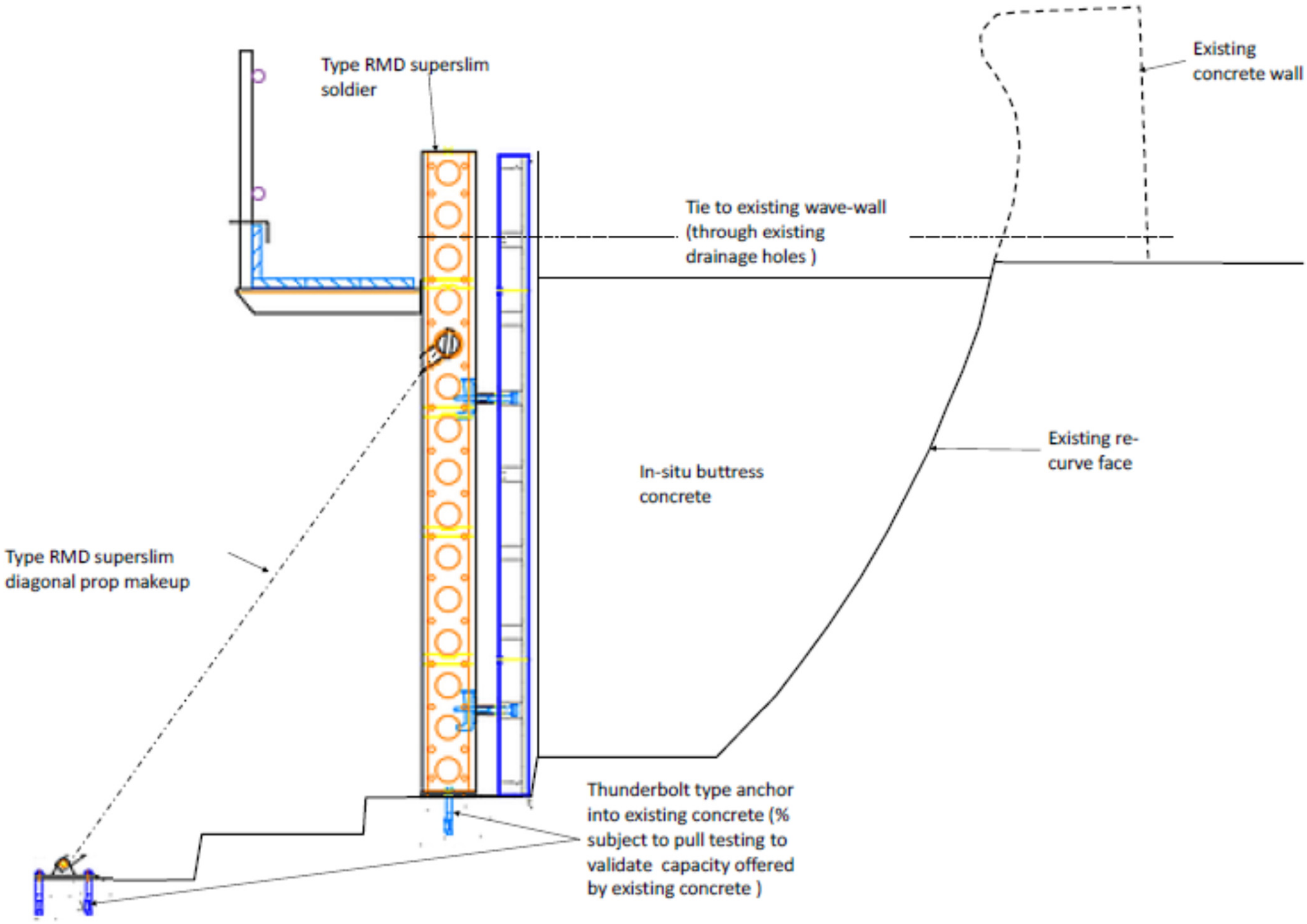

Figure 4-5: Surface water drainage for the proposed scheme

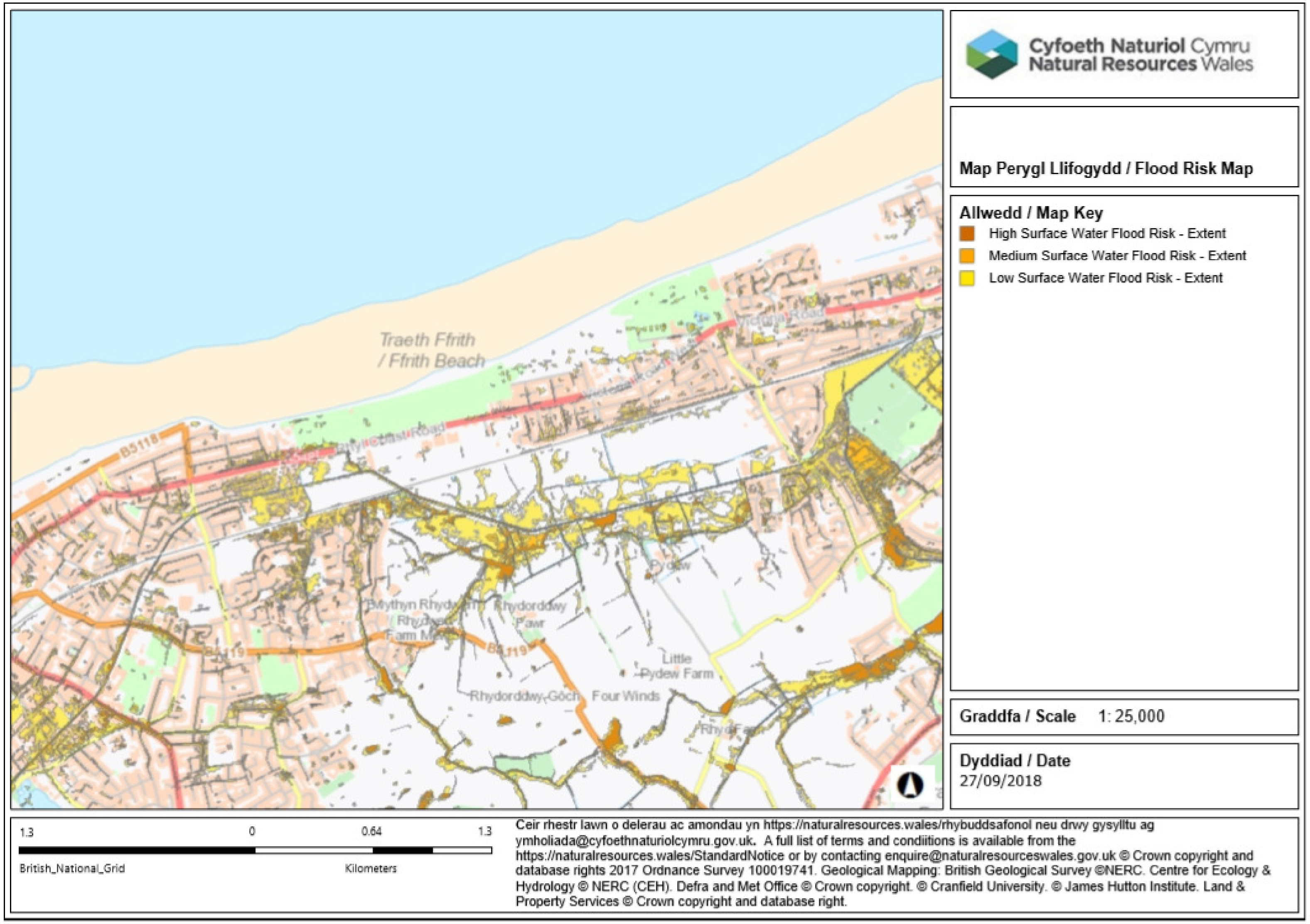

Figure 4-6: Risk of Flooding from Surface Water for the Rhyl area

JBA consulting

4.4 Groundwater flood risk

During the design stage the potential risk of groundwater flooding was investigated using GeoStudio SEEP/W finite element modelling package. The modelling showed that even with repeated flood events, groundwater flooding will not occur due to seepage beneath the embankment.

Appendices

A Plans

A.1 Flood extent maps

A.2 Proposed scheme plan

JBA consulting

Offices at

Coleshill
Doncaster
Dublin
Edinburgh
Exeter
Glasgow
Haywards Heath
Isle of Man
Limerick
Newcastle upon Tyne
Newport
Peterborough
Saltaire
Skipton
Tadcaster
Thirsk
Wallingford
Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE
United Kingdom

+44(0)1756 799919
info@jbaconsulting.com
www.jbaconsulting.com
Follow us:

Jeremy Benn Associates Limited

Registered in England 3246693

JBA Group Ltd is certified to:
ISO 9001:2015
ISO 14001:2015
OHSAS 18001:2007

JBA consulting

Offices at

Coleshill
Doncaster
Dublin
Edinburgh
Exeter
Glasgow
Haywards Heath
Isle of Man
Limerick
Newcastle upon Tyne
Newport
Peterborough
Saltaire
Skipton
Tadcaster
Thirsk
Wallingford
Warrington

Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE
United Kingdom

+44(0)1756 799919
info@jbaconsulting.com
www.jbaconsulting.com
Follow us:

Jeremy Benn Associates Limited

Registered in England 3246693

JBA Group Ltd is certified to:
ISO 9001:2015
ISO 14001:2015
OHSAS 18001:2007