Appendix 15.5

Coastal Processes Report

Irish Water

Arklow Wastewater Treatment Plant Project

Coastal processes assessment

247825-00

Issue | 29 August 2018

Job number 247825-00

Ove Arup & Partners Ireland Ltd

Arup
50 Ringsend Road
Dublin 4
D04 T6X0
Ireland
www.arup.com

ARUP

Contents

1 Introduction

1.1 Aim

As part of the Arklow Wastewater Treatment Plant Project (the proposed development), Arup has been commissioned to design revetment upgrade works for the length of the rock revetment adjacent to the proposed wastewater treatment plant site (WwTP site) as well as a storm water overflow (SWO) which will discharge at the toe of the revetment and a long sea outfall, discharging treated effluent, extending into the Irish Sea by approximately 900 m.

This report examines the existing coastal processes in the area and assesses the likely significant effects that the proposed revetment, the long sea outfall and the SWO at the WwTP may have on the coastal system either during the construction and/or operation of the proposed development. This report supports the Environmental Impact Assessment Report (EIAR) which has been prepared for the proposed development.

It should be noted that this report examines the effect of the proposed revetment, SWO and sea outfall works included as part of the proposed development on the dynamic coastal system only. The assessment of any other relevant aspects is documented separately in the EIAR [1].

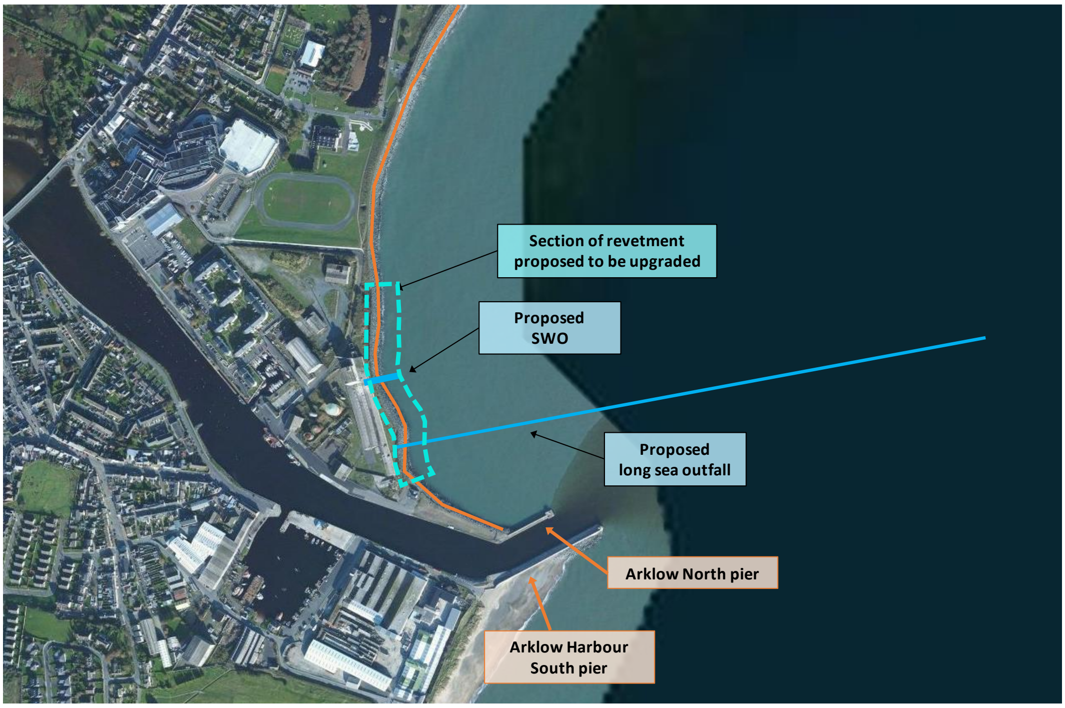
1.2 Site

The WwTP site is located due north of the entrance to Arklow Harbour, which is at the mouth of the Avoca River. The rock revetment that is currently in place runs between the river mouth and Arklow North Beach (see Figure 1).

The section of the existing revetment that is proposed to be upgraded, as well as the approximate location of the proposed outfalls is shown in Figure 2.

Figure 1: Location of the proposed WwTP development (Source: Google Maps - ©2014 Google).

Figure 2: Overview of the proposed revetment, SWO and sea outfall works (Source: Google Maps - ©2014 Google).



2 Proposed development

2.1 Scope of works

The proposed development will include a new Wastewater Treatment Plant and associated infrastructure, including the upgrade of an existing rock revetment, a SWO and a long sea outfall.

This section refers to the elements of the proposed Arklow WwTP development that are relevant for assessing the likely significant effects on the existing coastal processes, i.e. the rock revetment, the SWO and the long sea outfall.

A summary of the reasonable worst case based on the specimen design of these elements as well as the reasonable worst case construction methodology that is envisaged the contractor would follow is presented herein.

2.2 Procurement strategy

Overview

Irish Water intends to procure the detailed design and construction of the proposed development using a Design and Build contract. This form of contract has the benefit of encouraging innovation and value engineering, particularly for a project of this nature and scale, by giving the contractor ownership of both the detailed design and construction phases. Design and Build contracts traditionally also lead to shorter construction programmes. Under this form of contract the successful contractor will ultimately be responsible for the final detailed design of the proposed development, within the constraints as outlined herein.

The contractor is required to comply with all of the performance requirements set out in the tender documentation including the statutory consent approvals which may be granted by An Bord Pleanála, Department of Housing Planning and Local Government, EPA and other statutory stakeholders.

Design

Irish Water has developed a specimen design of the proposed development for assessment within this EIAR. This EIAR has considered the likely significant effects on the environment associated with our specimen design. The contractor will develop this design further, including final dimensions and details of the various elements, in accordance with the proposed mitigation measures, and any conditions that may be prescribed as part of the consent for the proposed development, ensuring that there is no material change in terms of significant effects on the environment.

As such, the assessment herein is considered to be the ‘reasonable worst case scenario’ in terms of significant environmental effects with regard to the overall planning boundary of the proposed development. The detailed design by the contractor should seek to identify opportunities for reducing further any significant adverse environmental effects where practicable.

Construction

The approach to construction describes the main construction activities that are relevant for the reasonable worst case assessment of likely significant environmental effects. The approach is considered to be the reasonable worst case scenario, given the existing site constraints, the adjacent land uses and the various construction methodologies which could be considered by the contractor. The construction of the proposed development will require a combination of marine, riverine and land-based works.

It will be the responsibility of the contractor (under the obligations of the contract) to ensure compliance with those measures that have been outlined in this EIAR to avoid and/or reduce significant adverse effects that have been identified. Where the contractor diverts from the methodologies and working areas outlined herein and defined in the granted planning consent, it will be the responsibility of the contractor to obtain the relevant licenses, permits and consents for such changes.

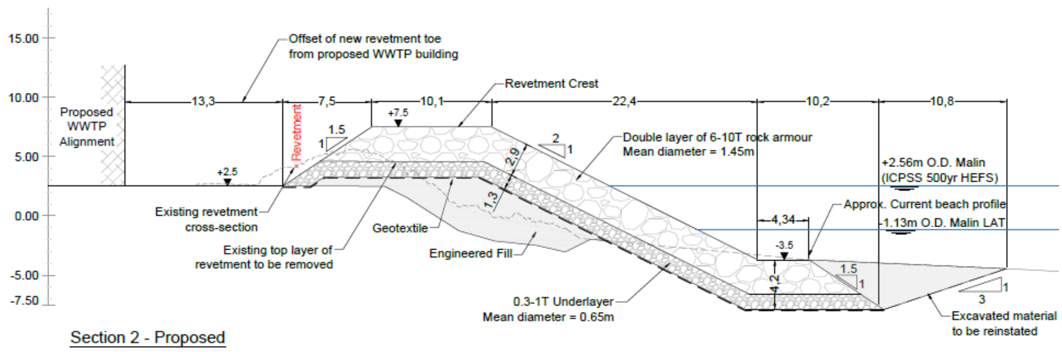
2.3 Revetment

2.3.1 Design

The section of the existing rock armour revetment located adjacent to the site will be upgraded as part of the proposed development. It is proposed that the existing structure will be removed and subsequently replaced along a distance of approximately 350m, as shown in the revetment layout drawings [2] and [3].

The upgraded revetment will consist of a double layer of rock armour of approximately 6 - 10 tonnes (t) with a slope of 1:2 on an underlayer of rock armour of approximately 0.3 - 1t. The designed crest level will be approximately 1 to 3m above the level of the existing revetment crest. Further details on the proposed structure can be found in the revetment cross section drawings [4], [5] and [6] (See Figure 3).

Figure 3: Typical cross section of the proposed revetment.



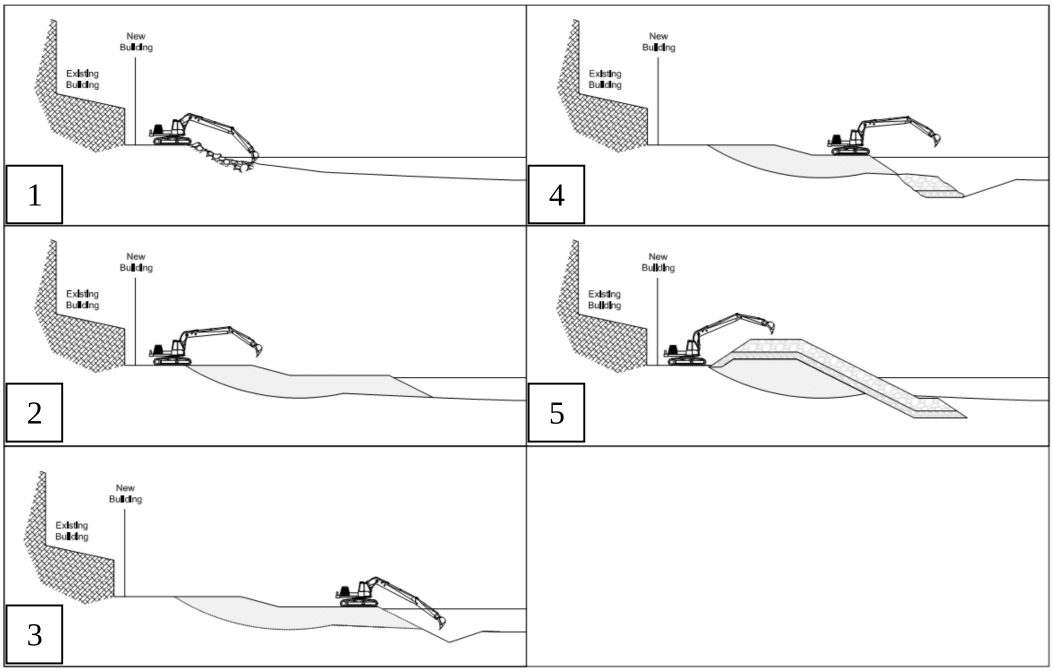
2.3.2 Construction methodology

It is envisaged that the removal of the existing rock revetment and its subsequent upgrade will be carried out in a staged process, in sections of approximately 15 to 25m.

A schematic summary of the envisaged construction methodology for an individual section of revetment is provided in Figure 4. The existing rock armour will be removed from crest to toe, with the installation of the new rock armour carried out from toe to crest to upgrade the revetment. Both construction processes will be carried out by the use of suitable excavators, and a temporary platform may be needed throughout the process.

Suitable fill and rock armour material will be required to upgrade the revetment. It is also envisaged that part of the excavated soil will be reinstated at the toe location, while the rest will be transported to a suitable facility off-site.

Figure 4: Envisaged procedure for the removal and subsequent replacement of the rock revetment.



2.4 SWO and Long Sea Outfall

2.4.1 Design

As part of the proposed development, a Storm Water Overflow (SWO) and a long sea outfall will be provided. These structures will start from the WwTP site, cross through/ underneath the revetment and discharge to the Irish Sea.

While the SWO will discharge excess storm flows in shallow waters, the long sea outfall will continue perpendicular to the revetment for approximately 900 m offshore, discharging the treated effluent at an approximate seabed level of -15m Chart Datum (CD) in relatively deep waters. The proposed locations of the SWO and long sea outfall are shown in Figure 2.

The following sections include a brief description of the proposed SWO and long sea outfall.

2.4.1.1 Storm Water Overflow (SWO)

The SWO pipeline will consist of precast concrete elements with an internal diameter of approximately 2.0m. The pipeline will be routed through the upgraded section of the revetment and the outlet structure will comprise a precast base slab, a headwall and wingwalls installed at the crest of the toe of the revetment. The outlet from the SWO pipe will be fitted with appropriate non-return valves.

2.4.1.2 Long sea outfall

It is expected that the long sea outfall will likely comprise high density polyethylene (HDPE) pipes with an internal diameter of approximately 555mm. Its longitudinal and transversal configuration will be defined based on the construction methodology ultimately followed by the contractor. For the purpose of this coastal assessment, we have considered all relevant construction methods.

2.4.2 Construction methodology

2.4.2.1 Storm Water Overflow (SWO)

The installation of the SWO will take place during the construction of the revetment in the section crossed by this pipeline.

The works will likely require the installation of a temporary sheet pile cofferdam to allow the works to be carried out in the dry. After dewatering, the existing rock armour will be removed and the trench for the SWO which will likely consist of precast concrete elements will be excavated on the landside. This will be followed by the placement of the bedding layer and the laying of the pipeline in the trench. The outlet from the SWO will be installed at the crest of the toe of the revetment. Subsequently, the backfilling and construction of the remaining section of the revetment will be carried out.

During the construction of the cofferdam for the short sea outfall, the depth of excavation will be below the water table. It will therefore be necessary to prevent groundwater and marine water ingress or dewater the water bearing sand and gravel soil likely to be present (based on the ground conditions at the WwTP site). Considering the high permeability of the sand and gravels, groundwater exclusion will be achieved by installing deep temporary sheet pile walls (approximately 15m beneath ground level).

Even with these measures, some dewatering from the areas of excavation may be necessary to remove residual groundwater within the sheet pile wall, manage surface water and to manage any small amounts of seepage through the sheet pile wall. The groundwater encountered may be contaminated due to the historical use of the site (as documented in Chapter 14 of the EIAR) and if this is the case, it will not be possible to discharge directly into the Irish Sea. The strategy for removing contaminated groundwater from the site is likely to comprise either tankering off site to a suitable licenced facility or treatment on site (See section 14.3 of Chapter 14 of the EIAR).

2.4.2.2 Long sea outfall

Introduction

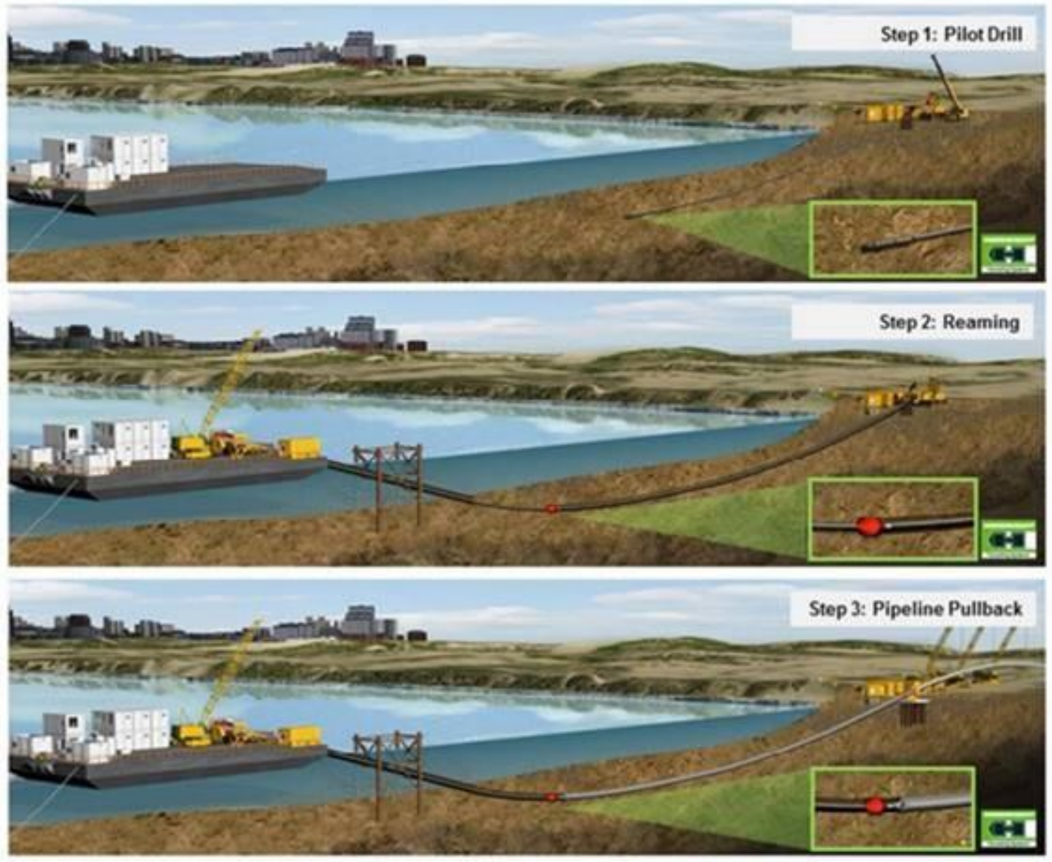
As outlined in Chapter 5 of the EIAR [1], it is envisaged that the contractor will follow one of the following methods for the installation of the long sea outfall:

- Horizontal Directional Drilling (HDD) method;
- Float and flood method; and
- Bottom-pull method.

Horizontal Directional Drilling method

Construction of the outfall would be carried out by the use of a drilling rig located in either the WwTP site or on a barge or jack-up platform near the seaward end of the outfall. The installation would comprise three phases: drilling of a pilot boring, pre-reaming and pipe positioning, illustrated in Figure 5.

Figure 5: Typical HDD process for a sea outfall (Source: Stevens [7]).



First, a drill rig would be positioned at a designated launch point (i.e. within the WwTP site), from which pilot boring would be carried out. The pilot boring would be undertaken to excavate along the alignment of the outfall.

Following the pilot boring, a reamer would be used to enlarge the hole in order to accommodate the outfall. Subsequently, the pipe positioning phase would take place, during which the outfall pipeline would be laid out at the exit point and connected to the previous hollow pipe.

It is noted that there is no need to install scour protection along the route of the outfall in this case.

More details regarding the HDD method can be found in Chapter 5 of the EIAR.

Float and flood method

The use of the float and flood method would require the formation of trenches and the placement of suitable material to support and protect the long sea outfall once it is in position.

Trenching and placement of bedding layer

Like the installation of the SWO, a temporary sheet pile cofferdam would likely be required to facilitate the installation of the outfall at the location of the revetment. This section of the outfall would be routed underneath the upgraded revetment and would consist of a HDPE pipeline laid within a concrete culvert.

The installation will take place prior to the construction of the revetment. The dewatering methodology would follow that for the SWO outlined above.

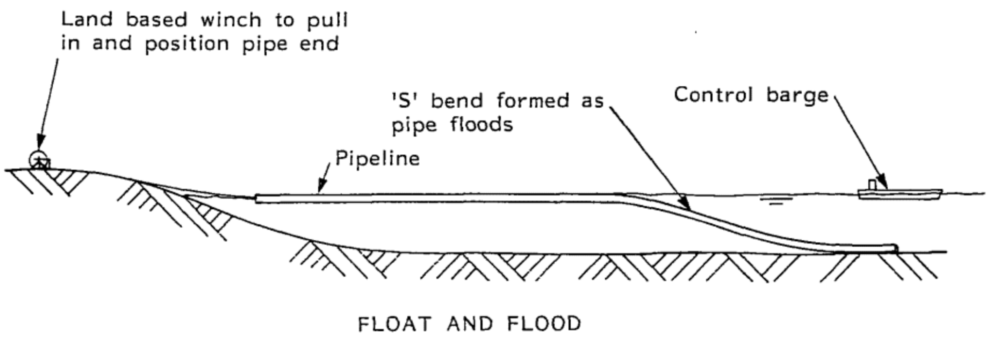
Prior to the installation of the marine section of the pipeline, the trench in which the outfall is to be laid would be excavated along its route. The total volume of seabed material to be removed to form the trench is estimated to be c. 18,000m^3. This excavated/ dredged material will be left to the side of the trench. It is anticipated that approximately 50% of the material would be later reused as fill material whilst the rest may be naturally dispersed. The dredging equipment that will be used will depend on the contractor, but it is envisaged that either backhoe dredgers or grab dredgers will be used.

Once the seabed material has been removed and the trench has been formed, the imported bedding material would be placed along the bottom of the trench to form the bedding layer.

Installation of the outfall pipeline

The float and flood method, also known to as the ‘S-Bend method’ would involve floating and towing the entire marine section of the outfall pipeline into position on the surface of the sea and the subsequent lowering down of the pipe into the trench as illustrated in Figure 6.

Figure 6: Flood and float method of installing the outfall (Source: WRC [8]).



Sections of the outfall pipe would be assembled on land and readied for moving to the water. The pipe and diffuser would be sealed temporarily while full of air, which provides the buoyancy necessary to float.

The pipeline would then be floated into the water using barges, which would tow and manoeuvre the outfall into position. The lowering operation would be achieved by replacing the air with water, which causes the outfall to sink into position. The rate of submergence would be controlled by the rate of air release.

Additional weight would be added where required (e.g. by using concrete ballast collars) in order to provide the negative buoyancy needed to sink the pipeline and place it in the bottom of the trench.

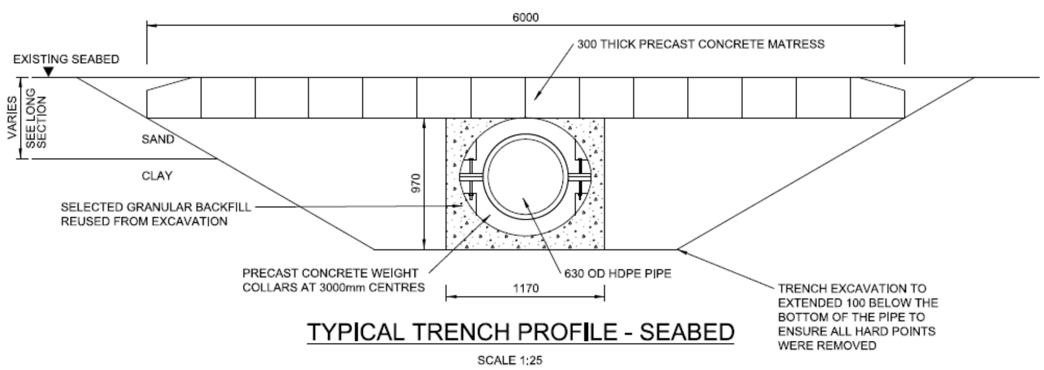
Backfilling the Trench

Once the outfall is laid in place, the fill material and the scour protection would be placed to surround the outfall pipe. Figure 5 below shows an indicative detail for the trench and scour protection.

As previously mentioned, the fill material will be comprised of seabed material as well as imported material. The excavated seabed material, previously placed parallel to the trench, and the imported material, brought by barges, would be placed back into the trench most likely by the use of backhoe or grab dredgers, or similar equipment. Given the nature of the contract, the exact equipment that will be used will be determined by the contractor.

To ensure against potential medium/long term effect from scour, suitable protection of the pipeline is required. A concrete mattress layer of approximately 300mm thickness is proposed for this purpose. The concrete mattress will finish at existing bed level.

Figure 7: Typical detail for scour protection of an outfall.



The total duration of the works is estimated to be 3-4 months (dependent on weather conditions).

Bottom-pull method

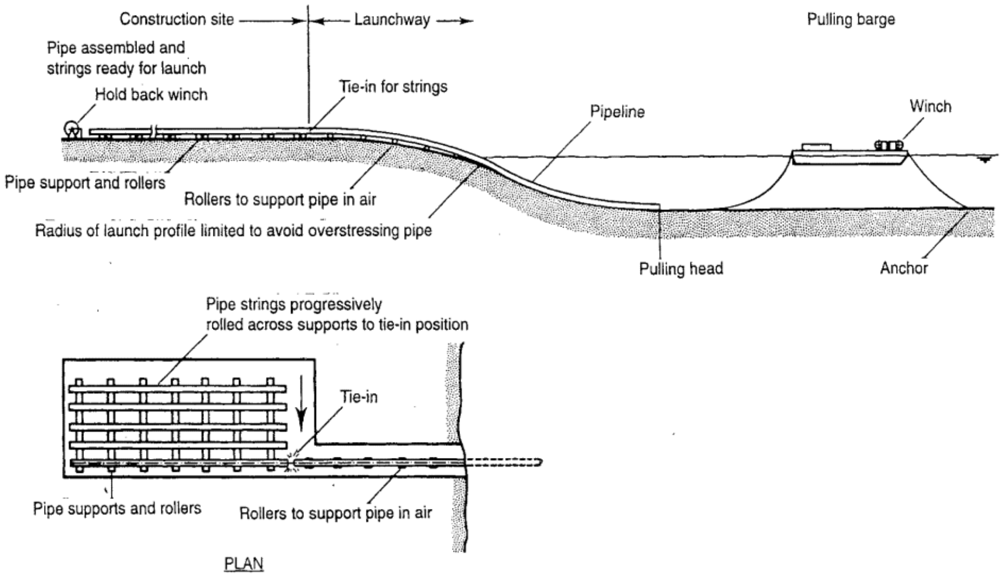
Overview

The use of the bottom-pull method would, in a similar manner to the float and flood method, require the formation of trenches and the placement of suitable bedding material to support and protect the positioned pipeline. The revetment crossing, trenching, placement of the bedding layer, scour protection, backfilling of the trench and the diffuser assembly procedures would also be the same as described in the Float and flood method section above. Laying of the outfall would be undertaken as described below.

Installation of the outfall pipeline

The bottom-pull method would involve joining and pulling sections of the outfall pipeline towards the sea by using a barge. The pipes would be pulled into place by the barge as illustrated in Figure 8.

Figure 8: Bottom pull method of installing the outfalls (Source: CIRIA [9]).



Small sections of the outfall pipe would be arranged on land (within the WwTP site) and readied for placing on rollers. The rollers would be aligned with the route of the outfall and the location of the revetment crossing to ensure that the correct pipe alignment is achieved. The sections of the pipe would be joined in sequence to make pipe strings that could be placed onto the rollers. The number and length of the pipe strings would be determined by the contractor based on the space that is made available within the WwTP site.

The pipe strings would be pulled by winches mounted on a barge anchored offshore in a stepped process. The first pipe string would be pulled towards the sea then the next string would be moved across the rollers and joined to the first string at the tie-in position. This procedure would be repeated until all the strings have been joined and the outfall pipe has been laid in position. Following the completion of pulling, the culvert (i.e. the interface between the outfall and the revetment) would be installed.

The total duration of the works is estimated to be 4-5 months (dependent on weather conditions).

Diffuser assembly

Once the long sea outfall has been laid, by whichever method (HDD, float and flood or bottom-pull), the diffuser would be assembled on the seaward end of the outfall. The diffuser arrangement would include up to 6 diffusers of approximately 0.16m diameter at a spacing of c. 10m intervals.

The diffuser would be prefabricated on land and placed on the seabed by barge as one complete unit. The exact procedure and depths of backfill required would depend on the equipment available from the contractor along with programme and cost considerations, however it is anticipated that this would be undertaken from the barges and it will likely require open excavation of the seabed, along the length of the diffusers.

3 Site conditions

3.1 Metocean conditions

This section presents a summary of the Metocean conditions relevant to the study area. Detailed information can be found in the Wave Modelling report included in Appendix A.

3.1.1 Tidal levels

The relevant tidal levels, based on information from the relevant Admiralty Tide Tables for Arklow Harbour, are shown in Table 1 below.

Table 1: Tide levels in Arklow Harbour.

Tidal Level	Referred to Chart Datum	Referred to OD Malin
Mean High Water Springs (MHWS)	1.4m	0.28m
Mean High Water Neaps (MHWN)	1.2m	0.08m
Mean Low Water Neaps (MLWN)	0.9m	-0.22m
Mean Low Water Springs (MLWS)	0.6m	-0.52m
Lowest Astronomical Tide (LAT)	0m	-1.12m

3.1.2 Extreme sea level

The extreme water level estimated at the site for both wave modelling and revetment design is 2.56mOD Malin or 3.68m Chart Datum. This level was obtained from the Irish Coastal Protection Strategy Study (ICPSS) [10] which includes the future scenario assessments of extreme coastal water levels. This predicted water level includes a combination of storm surge and extreme tidal levels, based on both numerical modelling and statistical analysis of historic tide gauge data. The High End Future Scenario levels also allows for land movement and +1.00m sea level rise due to climate change by the year 2100.

3.1.3 Currents

According to previous studies (See Ref. [11]) the oceanography at the site can be described as energetic with strong tidal currents, brief slack waters, large tidal excursions and good dispersive characteristics.

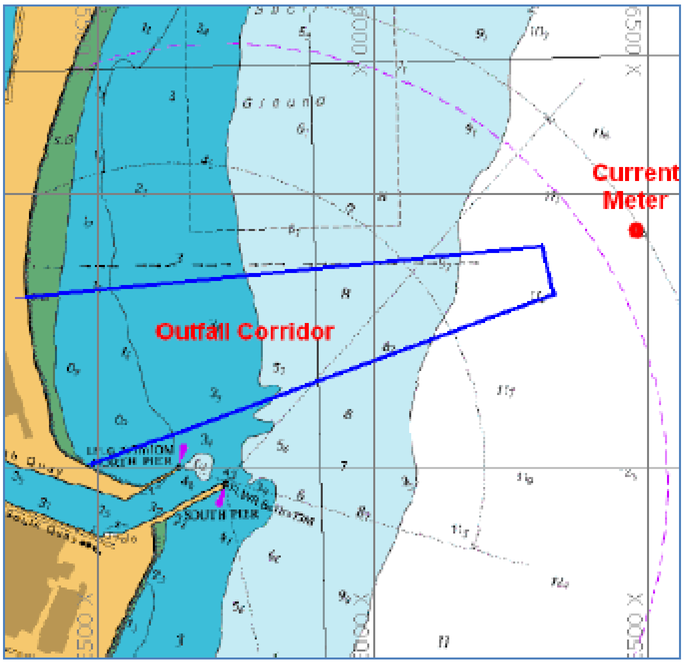
Table 2 below summarises depth averaged current speed and drogue trajectory data derived during the 1985 Irish Hydrodata study.

Table 2: Summary depth averaged currents.

Tide	Current Speeds (m/s)	
	Flood	**Ebb**
Spring	0.66	0.59
Neap	0.42	0.35

According to this information, a recording current meter was also deployed for 30 days during the 1985 survey. This was located approximately 1000m east-northeast from the harbour mouth on the (then) proposed outfall line (See Figure 9). It was positioned at a height of 1.5m above the seabed. The 95%'ile exceedance speed recorded at the current meter location was 0.05m/s. This indicates that the durations of slack water at the site are limited.

Figure 9: Location of the current meter.



3.1.4 Avoca river

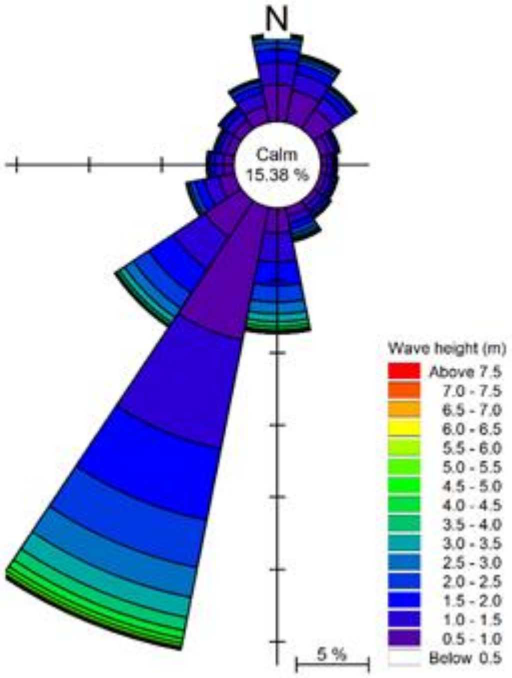
According to Ref. [11], the flow characteristics of the Avoca River based on the EPA Hydrometric data system are: DWF = 0.8 m^3/s, 95%'ile = 3.09 m^3/s and 50%'ile = 15 m^3/s. According to the Irish Hydrodata report, the river flow is assumed to be low so that there is no beneficial momentum from the river plume which would carry the wastewaters further offshore. Therefore, its influence in comparison with tidal effects, waves and currents is considered negligible in terms of coastal processes. Sediments transported by the Avoca river and an evolution of these in time has not been assessed.

3.1.5 Wave and wind data

The directional wave distribution of the offshore wave climate is represented in Figure 10.

Due to the orientation of the coastline at the study area and its surroundings in relation to the offshore waves, only waves approaching from the NE (northeast) to SE (southeast) are considered relevant for this assessment.

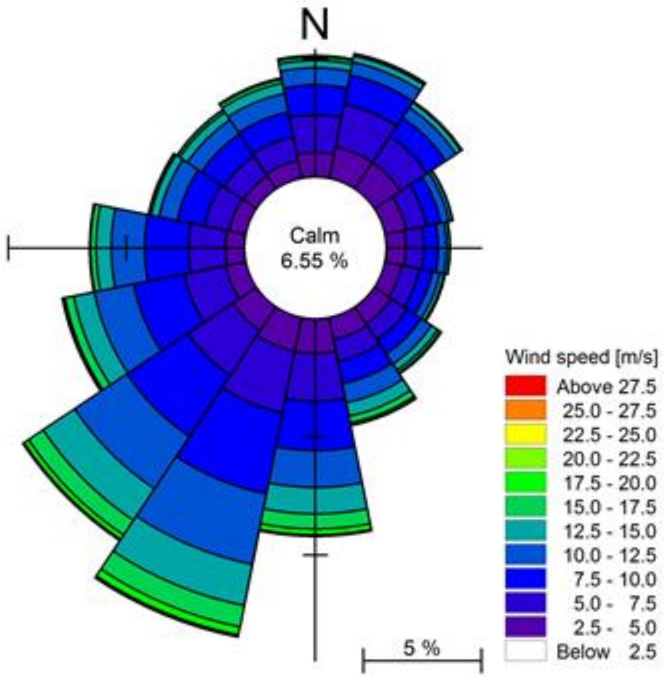
Figure 10: Offshore Wave Rose – All directions.



The directional wind distribution of the full wind rose is shown in Figure 11.

As for the offshore wave data, only offshore wind from the NE to SE has been considered for this assessment due to the orientation of the coastline at and near the study area. For these sectors, the predominant directions are NE and SE. Maximum wind speed values are roughly 25m/s for the south easterly directions.

Figure 11: Offshore Wind Rose – All directions.



3.1.6 Wave modelling results

Wave propagation from offshore to the proposed location for the revetment was modelled using MIKE21-SW.

Table 3 shows the model results for various combinations of wind/wave data and direction for the 500 year return period events for each of the relevant directions. The model results presented correspond to points located 20m offshore of the existing revetment crest.

Table 3: MIKE21-SW modelling results. Displayed are the return period (Tr), wave height (Hs), wave period (Tp) and Wind speed (Wsp) for a water level corresponding to Tr=500y.

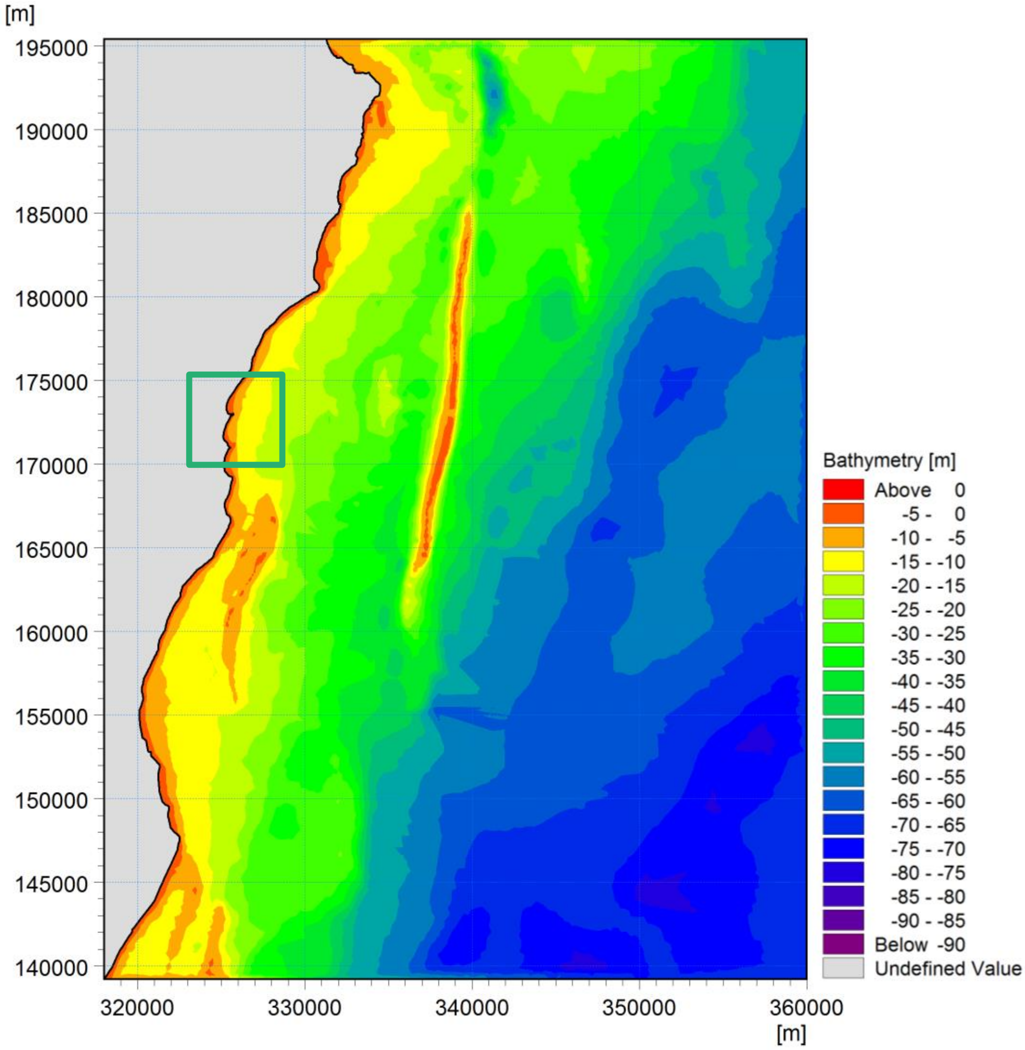
Direction	T_r [y]	Offshore (approximately -60m CD)		20m from existing revetment crest (approximately -3m CD)		
		H_s [m]	T_p [s]	W_{sp} [m/s]	H_s [m]	T_p [s]
NE	500	5.5	8.6	24.3	2.9	9.5
ENE	500	5.0	8.3	20.7	2.8	9.5
E	500	5.6	8.7	24.4	3.0	9.9
ESE	500	6.2	9.1	25.0	3.0	10.5
SE	500	7.1	9.7	23.0	3.0	11.4

The results show that storms from all the tested directions give similar resulting nearshore wave heights despite having significantly higher input offshore conditions from the south easterly directions.

This can be explained partly due to the presence of the Arklow Bank parallel (as can be observed in Figure 12) to the coast on which waves likely break and dissipate energy, and partly due to the shallow waters adjacent to the site ('depth limited wave conditions').

This effect was confirmed in the Irish Coastal Protection Strategy Study (Ref. [10]), where it was noted that "The banks that lie off the east coast of Ireland have a significant effect on the inshore wave climate at the shoreline of the study area. Even at high tide the banks reduce the height of the higher waves passing over and thus protect the shoreline."

Figure 12: Bathymetric model derived from the Admiralty Charts and bathymetric survey (site located in the middle of green square).



3.2 Ground conditions

3.2.1 Introduction

The following sections provide an overview of the ground conditions at the WwTP onshore site and the site of the proposed long sea outfall.

Ground conditions at the location of the proposed rock revetment are anticipated to be broadly in line with the onshore ground conditions at the site, however given the coastal location, the conditions identified in the vicinity of the long sea outfall may also be relevant over parts of the revetment footprint.

3.2.2 Rock revetment

A geotechnical interpretation of the ground conditions based on Ground Investigations undertaken within the site as well as an assessment of publicly available baseline information was carried out by Arup in 2018 as part of the EIAR and design development.

The following conclusions are relevant to the present assessment:

- The expected ground stratigraphy across the site is Made Ground over Sands and Gravels (Glacial Deposits) over Clay (Glacial Till) over Bedrock. Taking into account the excavation depth required for the revetment works, only Made Ground and Sands and Gravels are expected to be encountered during construction.
- It must be noted that, while the Sands and Gravels were generally described in the ground investigations as medium dense to dense, loose deposits were also encountered at some parts of the site.
- The footprint of the proposed development is within an area of reclaimed land dating back to the mid 1800's. The area was reclaimed using local deposits of sand and gravel. The site has a history of industrial use and from review of the ground investigation there is a risk of encountering contaminated ground across the site. There is a potential risk of encountering contamination in the ground underlying the existing revetment. (Refer to Chapter 14 EIAR [1]).

3.2.3 Long sea outfall

A geotechnical interpretation of the ground conditions around the footprint of the long sea outfall as well as an assessment of publicly available baseline information was carried out as part of the EIAR and design development.

The stratigraphy and material properties for the main geological units expected to be encountered across the area were derived as part of this study, and the associated geotechnical risks were identified.

The following conclusions are considered relevant to this assessment:

- Based on the results from the ground investigation undertaken along the outfall alignment, the anticipated ground conditions in the area are medium dense Sands and Gravels over Clay, over dense Sands and Gravels over Bedrock. The upper layer of Sands and Gravels are likely to comprise material of marine deposits, with the underlying materials being of glacial origin.
- The only geotechnical unit expected to be encountered during the outfall installation works is comprised of medium dense to dense marine sand and gravel deposits.

They were generally encountered as extensive granular deposits along the alignment of the marine outfalls of the proposed development. However, it must be noted that loose sands and gravels were occasionally encountered within the first 2m of the seabed. The underlying layers are not expected to be encountered given the limited trench depth.

- With regard to contamination of the existing material, it was estimated that a limited volume of material (approximately 18,000m^3) is proposed for excavation of the outfall, of which almost half would be classed as uncontaminated and inert. However, the environmental testing undertaken in the contaminated material indicates that the marine sediments are only very slightly contaminated at relatively low levels for some specific parameters (Refer to Chapter 14 EIAR [1]).
- The reusability potential of the seabed material that will be excavated during the trenching works will be confirmed by the contractor. Based on the existing information, it was concluded that the first 4m of the seabed are expected to be suitable as a General Granular Fill (Class 1) material.
- This Granular Fill is considered appropriate for use as the filter layer, but not as the bedding layer. As a preliminary estimate, it is considered that approximately 50% of the filter layer will likely be comprised of this material.
- The in situ materials will likely not be suitable for use as bedding material, and hence the bedding layer material will need to be imported to the site.

4 Environmental constraints

In this section, those areas considered to be most susceptible to a potential change in the existing coastal processes, as a result of the proposed development are presented.

Specific consideration has been given to those Natura 2000 sites protected under the provisions of Council Directive 92/43/EEC (Habitats Directive) and Council Directive 79/409/EEC (Birds Directive), as amended and codified in Council Directive 2009/147/EC.

These sites include:

- Buckroney – Brittas Dunes and Fen SAC (Site Code 000729) which lies approximately 4.5km to the north at its closest point;
- Kilpatrick Sandhills SAC (Site Code 001742) which lies approximately 6.5km to the south at its closest point; and
- Part of Magharabeg Dunes SAC (Site Code 001766) which also lies within 15km of the proposed development.

The potential impacts (from any potential changes in coastal processes) on ecological receptors is assessed in the EIAR in Chapter 11, with potential impacts on Natura 2000 sites assessed in the Natura Impact Statement for the proposed development.

5 Coastal processes

5.1 Uniform units

A desktop assessment of the coastal areas has been carried out. The coastline in the vicinity of the site consists of beaches limited by headlands. Barriers such as headlands accompanied by change in orientation of the adjoining areas suggest limited exchange of sediment between them. Some uniform units in terms of areas with similar orientation and limited by headlands have been identified in the vicinity of the area of study and are as follows:

- Kilmichael Point to Mizen Head; and
- Mizen Head to Wicklow Head.

Their location is shown in Figure 13.

Figure 13: Relevant uniform units within the study area.

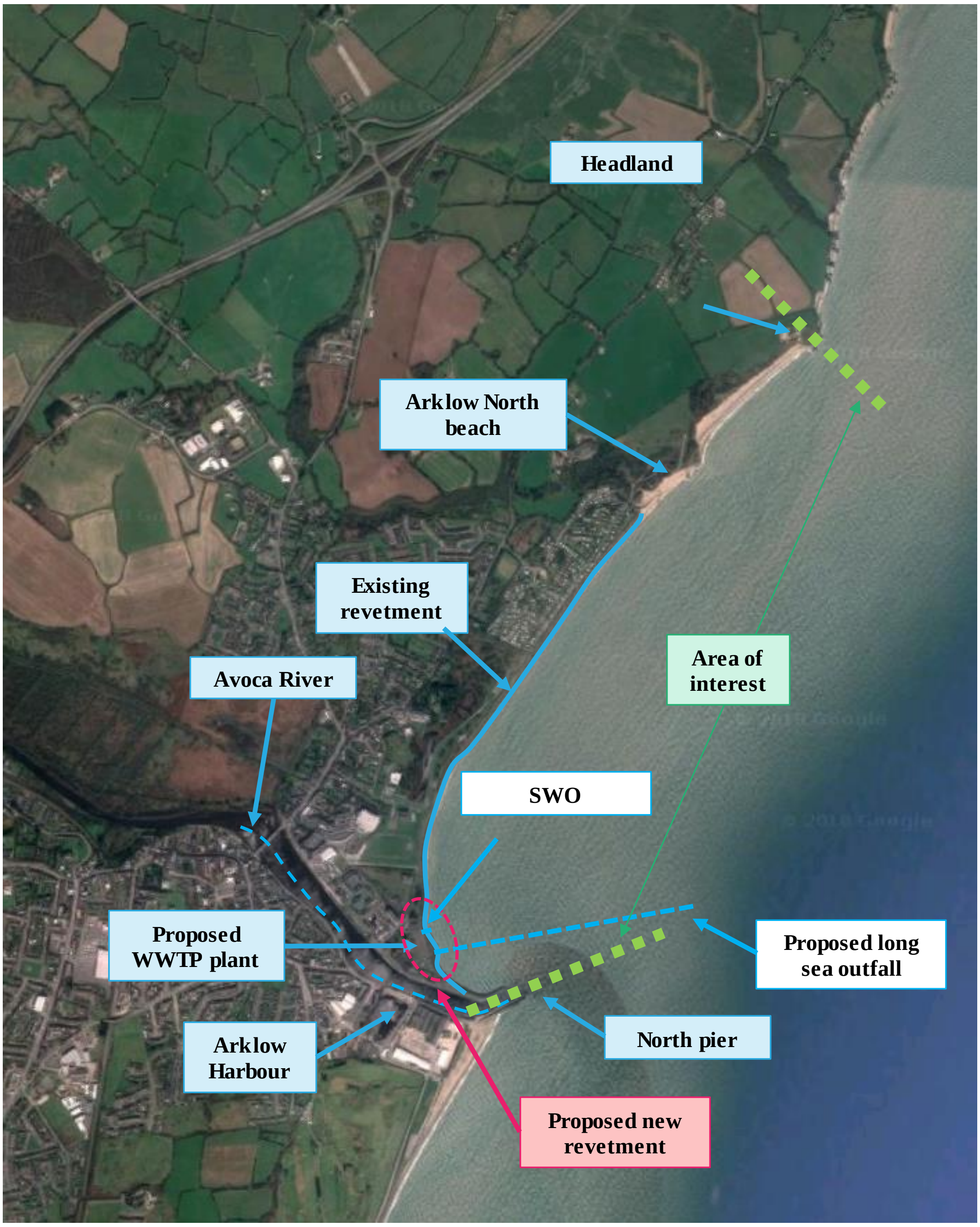
The proposed development is located within the Kilmichael Point to Mizen Head area, in a stretch of coastline that is limited to the south by breakwaters which protect the entrance to Arklow harbour, and to the North by the headland located at the north end of the Arklow North Beach. The extent and features of this sub-physiographic unit defined as an Area of Interest are shown in Figure 14.

This headland and the change in coastline orientation of the areas to the north limit partially the coastal processes within this area. The existing and proposed works are also involved in the coastline characterisation. It includes (from South to North):

- The river breakwaters (piers);
- The upgraded revetment; including the SWO and long sea outfall;
- The existing revetment; and
- The Arklow North Beach.

The existing 2.2km revetment starts at the northern pier at the harbour mouth and continues in a northerly direction, and then in a north-easterly direction as far as Arklow North Beach. The section of the revetment that is proposed to be upgraded is located near the existing revetment's southern end, immediately adjacent to the WwTP site. The extent of the proposed upgrade revetment is approximately 350m. The proposed long sea outfall alignment is also represented in Figure 14.

Figure 14: Extent and features of the Area of Interest (extent limited by dashed green lines).



5.2 Historical evolution

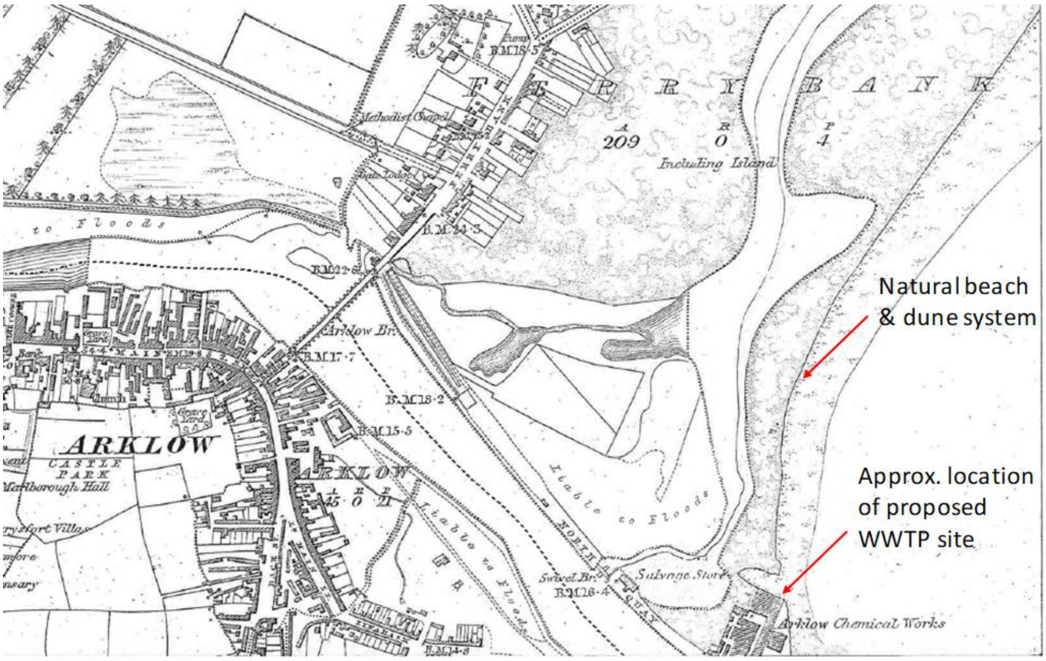
5.2.1 Background

A report produced by J.P. Byrne & Partners in April 1990 [12] following the construction of the existing revetment outlines the historical coastal defences at this location as well as details of the construction of this revetment. A paper presented by the same authors in February 1990 [13] gives further details of the previous coastal defences at the site.

Below is a summary of the relevant information from the aforementioned reports in relation to previous sea defences in this area.

Prior to the construction of Aklow harbour in 1860, the area under consideration was the estuary of the Avoca river. This formed a dynamic coastal system which included a natural beach and dune system. This system of sand dunes developed behind the beach, acting as a natural sea defence for the area. This is shown in Figure 15.

Figure 15: Ordnance Survey Map, Sheet 40, published 1880 [14].

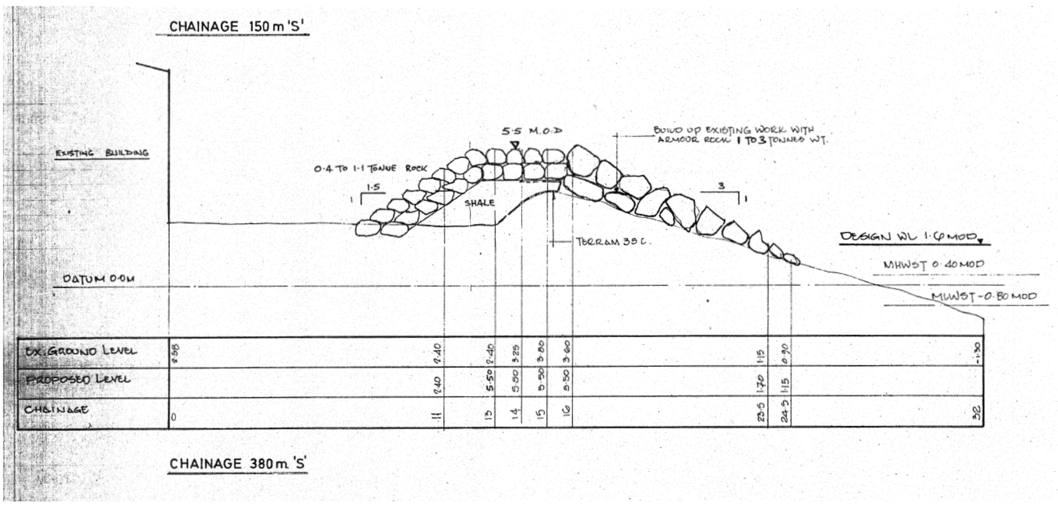


After the development of Kynoch munitions factory in 1912, an increased level of shore protection was required beyond that provided by the natural dune system. This additional shore protection took the form of a three-tiered timber piled solution to retain sand. This was reported as very efficient but was not maintained. A historical image of the remnant of these structures is shown below in Figure 16.

Figure 16: Beach and dune system with vertical piles remaining from previous defence structure. Date unknown.

In 1972, a rubble revetment was constructed to protect the Wallboard Factory at the location of the WwTP site, over a distance of about 400m. According to the J.P. Byrne & Partners report, this took the form of an earthen embankment with gabion and rubble rock protection and was later extended northwards of the site. A storm in December 1989 is known to have caused severe damage to this defence, in particular the natural sand dunes, and extensive flooding behind it. This led to the construction of the existing revetment in 1990 (Figure 17). This revetment was designed for a significant wave height of 2.85m and a water depth of 3m, however the design wave period is not specified in the design report. The design water depth allowed for 1m of beach scour.

Figure 17: Section of the 1990 revetment at the proposed WwTP location (Source: Byrne, K.P. and Motherway, F.K. [13]).



Arup carried out an inspection of this revetment on 28 February 2017. This inspection, combined with a subsequent assessment of the 1990 design, also by Arup, concluded that the protection currently offered by the existing revetment is insufficient to protect the proposed WwTP, particularly noting the critical infrastructure proposed for the site. Major rehabilitation works to the structure were recommended. During the inspection visit, undertaken at low tide, it was noted that the toe of the revetment, and seabed level, were not visible. Therefore, it was concluded that a loss of the beach which existed in front of the revetment at the time of its construction had occurred in this area in the time period between 1990 and 2018.

An image of the existing revetment taken during a project site visit undertaken in April 2018 is shown in Figure 18.

Figure 18: Photograph of the existing revetment taken from the north pier of Arklow Harbour towards the proposed WwTP site in April 2018.

5.2.2 Desk study coastal evolution

5.2.2.1 Introduction

There is historic evidence of a beach located in front of the current revetment which at present does not exist. This evidence was found in historical photographs and a paper presented at Engineer's Ireland [13] which states that the previously existing beach presented continuous erosion of about 1.5m between 1930 and 1980, with increased erosion rates following that period in particular years. Reasons cited for this erosion are as follows:

- The construction of the north and south piers at the entrance to Arklow Harbour (1860);

- Large quantities of sand from the North beach area were exported to England from 1930 to 1945; and
- Dismantling of the previously existing wooden coastal defence structures during World War II.

However, the information from [13] suggests that the beach was still fairly visible at the time the revetment was designed, as it is stated that the revetment was designed to be as far back from the existing beach as possible so that it would continue to be an amenity for the town. It was also made clear in the study that the performance of the coastal protection structure would be related to the erosion of the previously existing beach in front of the revetment. Tests confirmed that the reduced rear rock armour would be damaged if the beach was completely eroded and waves of around 3m height attacked the structure. Possible methods of resisting erosion recommended at that time were the provision of beach structures such as groynes and/or beach nourishment. Hard coastal protection measures such as groynes appear not to have been implemented based on visual observations.

As shown in the satellite imagery and observed during site visits, the beach which is known to have previously existed, is no longer visible at low tide.

Information from available satellite imagery and bathymetric surveys between 1985 and the present date have been studied in order to assess the historical coastal evolution at the site area and the adjacent areas from the construction of the first revetment and this is discussed below.

5.2.2.2 Satellite imagery

Satellite imagery from Google maps was used to examine shoreline retreat from 1984, when the first revetment was already in place, to the present day. A comparison of aerial photographs from 1984 to the present date is shown in Figure 19. Some retreat of the coastline (loss of emerged beach) appears to have taken place between the 1984 and 2005 images (Figure 20), while the coastline remains relatively stable from this date forward, due to the presence of the hard defence structure.

The phase of the tide in these images is unknown, and therefore beach material loss cannot be assessed quantitatively, but given the fact that the beach is no longer visible at low tide conditions the evolution suggests a loss of beach material after the construction of the existing revetment.

Figure 19: Comparison of satellite imagery of the Arklow coastline dating, from left to right: 1984, 1993, 2005, 2011 and 2017 (Source: Google Earth).

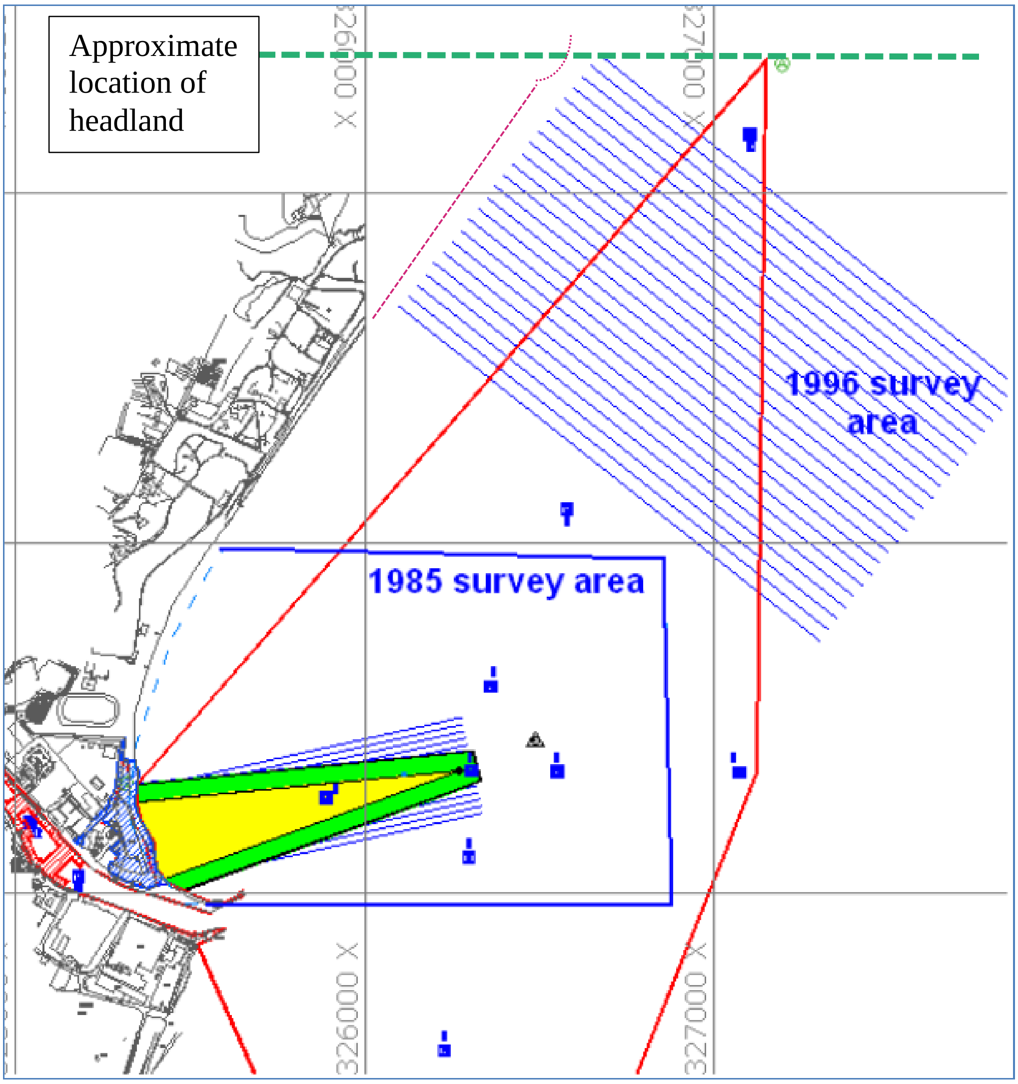
Figure 20: Zoom in comparison of satellite imagery from 1984 (left) and 2005 (right). Source:

5.2.2.3 Historical bathymetric surveys

A comparison of two available bathymetric surveys dating 1985 and 1996 by Irish Hydrodata Ltd and 2016 survey GSI INFOMAR data was carried out by Byrne Looby Consulting Engineers in 2017 [15]. The information contained in this study, summarised below, is relevant for assessing the evolution of the seabed profile.

In this study, the levels reported in each of the surveys for a number of cross sections taken within and near the WwTP site are compared. The full extent of the 1985 and 1996 surveys used for the assessment can be observed in Figure 21.

Figure 21: Survey areas (Source: Irish Hydrodata Limited report [15]).



Survey areas

In the area proposed for the upgraded rock revetment, long sea outfall and SWO at the WwTP specifically, 15 cross sections were analysed (profile lines no.1 to no.15, [15]). One of these profile lines falls close to/along the proposed alignment of the outfall (profile line no. 12).

Figure 23, Figure 24 and Figure 25 show the levels reported for profile line no.12 and the two lines that represent the study boundaries (i.e., profile lines no. 1 and no. 15). Their location is shown in Figure 22.

These figures suggest loss of material at the seabed in front of the existing revetment; the 2016 seabed level appears lower than the 1985 seabed level by 0.5 – 2m. In profile 12 (location of long outfall), the maximum seabed level difference is shown to be approximately 1m. This value, given the time difference between the two surveys (30 years), suggests that the sediment transport processes in this specific area are limited.

Figure 22: Approximate location of the relevant profile lines from the survey comparison study [15].

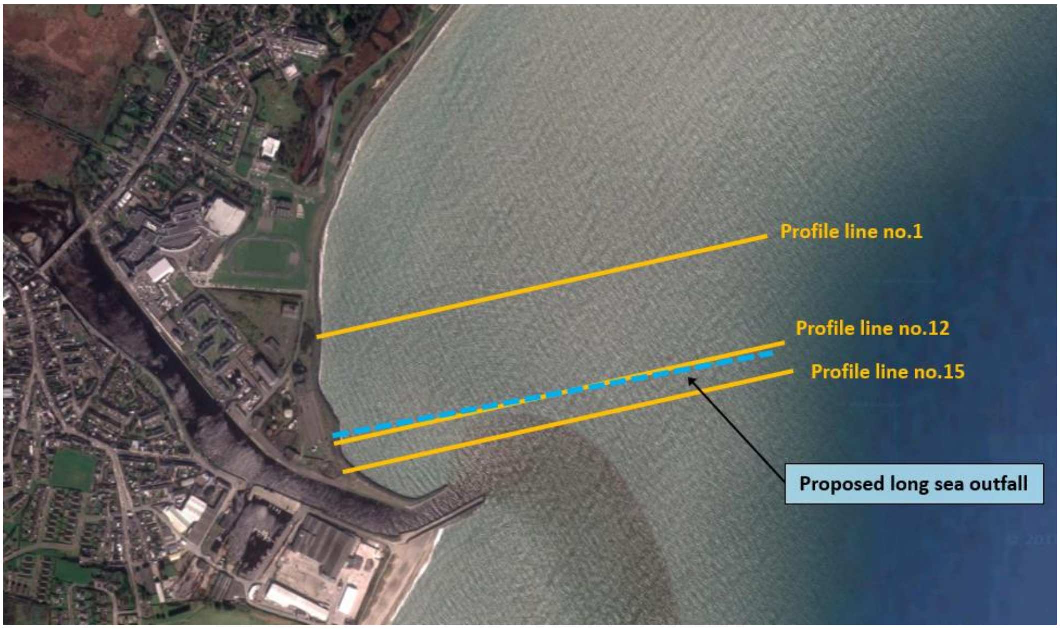


Figure 23: Comparison of 1985 and 2016 bathymetric surveys for profile line no. 1 (Source: Irish Hydrodata Limited report [15]).

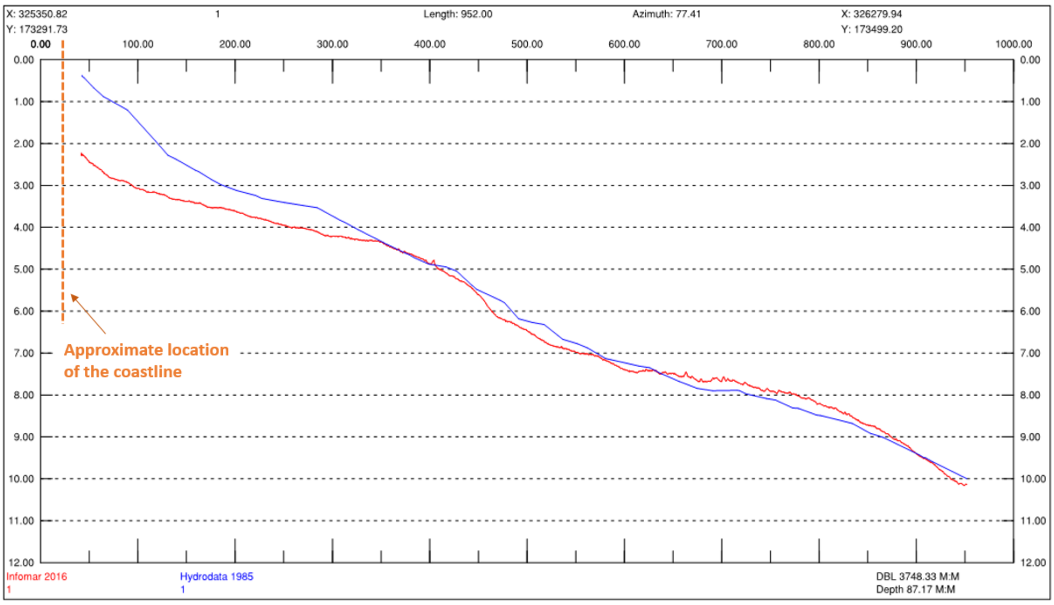


Figure 24: Comparison of 1985 and 2016 bathymetric surveys for profile line no. 12 (Source: Irish Hydrodata Limited report [15]).

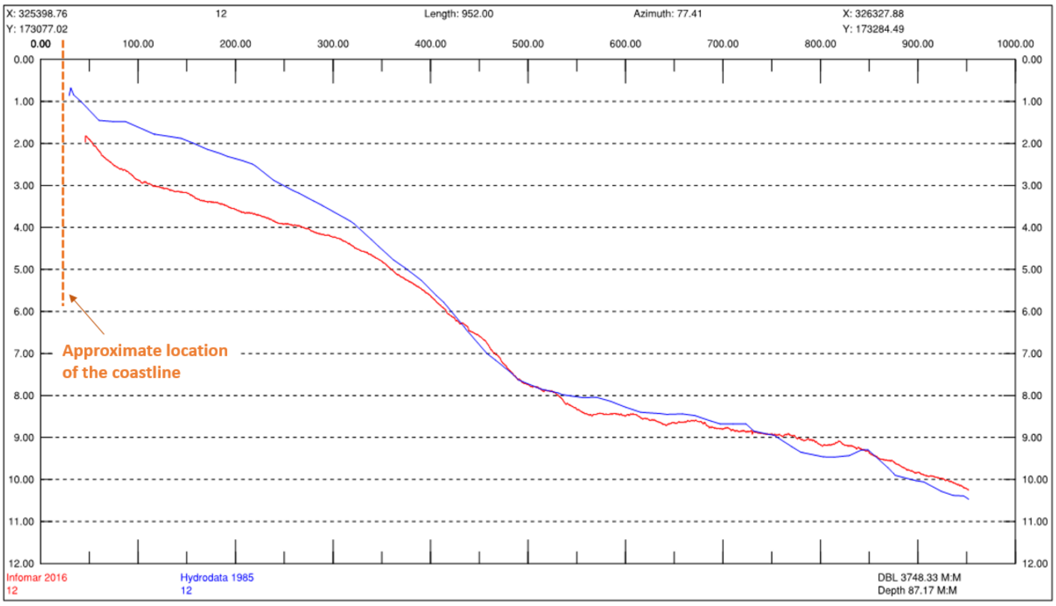
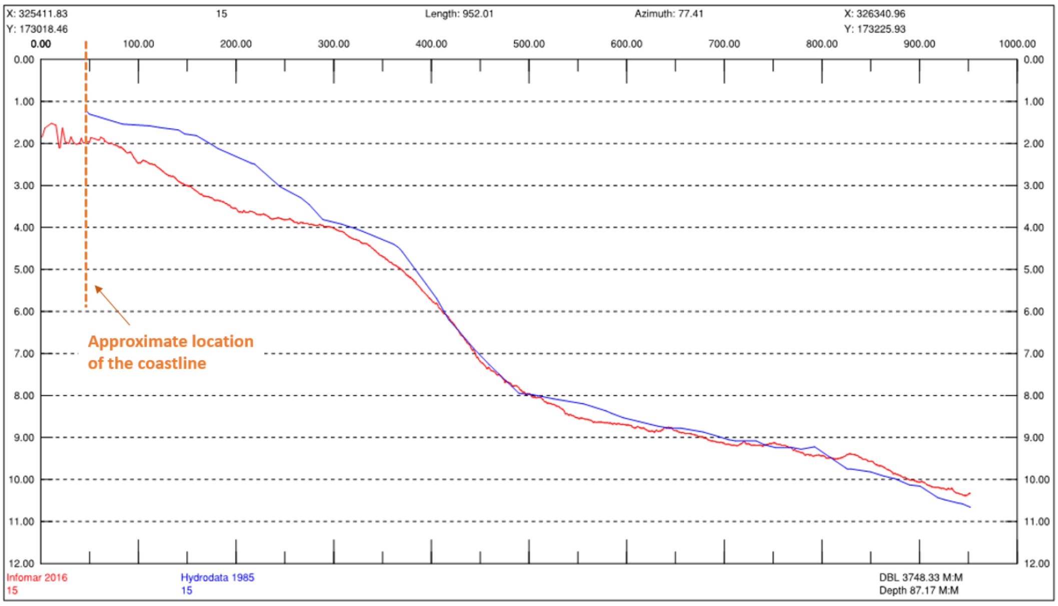


Figure 25: Comparison of 1985 and 2016 bathymetric surveys for profile line no. 15 (Source: Irish Hydrodata Limited report [15]).



The figures shows that the seabed erosion continues to approximately 400m offshore (depth of approx. 6m). Beyond this point, the seabed is shown to remain relatively stable between the survey dates.

Seabed lowering is shown to be higher in the northern sections, which could be explained by the shelter provided to the southern end of the revetment by Arklow harbour piers.

In the area surveyed to the north (1996 area) a larger volume of loss material can be observed. The seabed lowering and loss of material seem to be concentrated in depths from 7 to 11m and gain of material can generally be observed in deeper areas. This could be partially explained by seasonal changes (e.g winter and summer profiles), however, there seems to be an overall loss of material in the 20 years of comparison for these surveys (1996 vs 2016). Northern and southern profiles of this north area are shown for clarity in Figure 26 and Figure 27. This suggests offshore sediment transportation in this area.

Figure 26: Survey 1996 northern section (Source: Irish Hydrodata Limited report [15]).

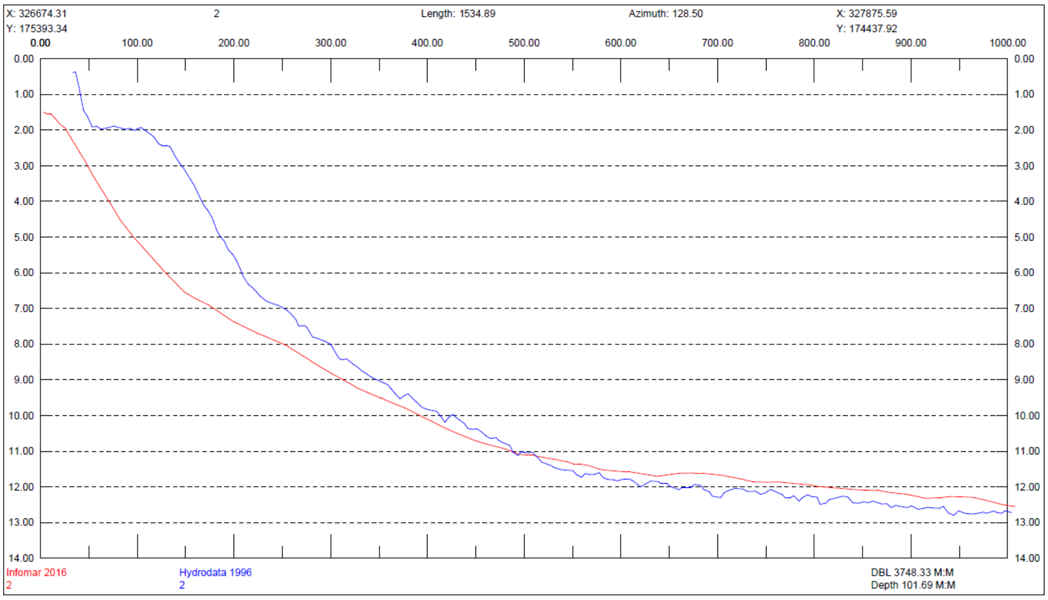
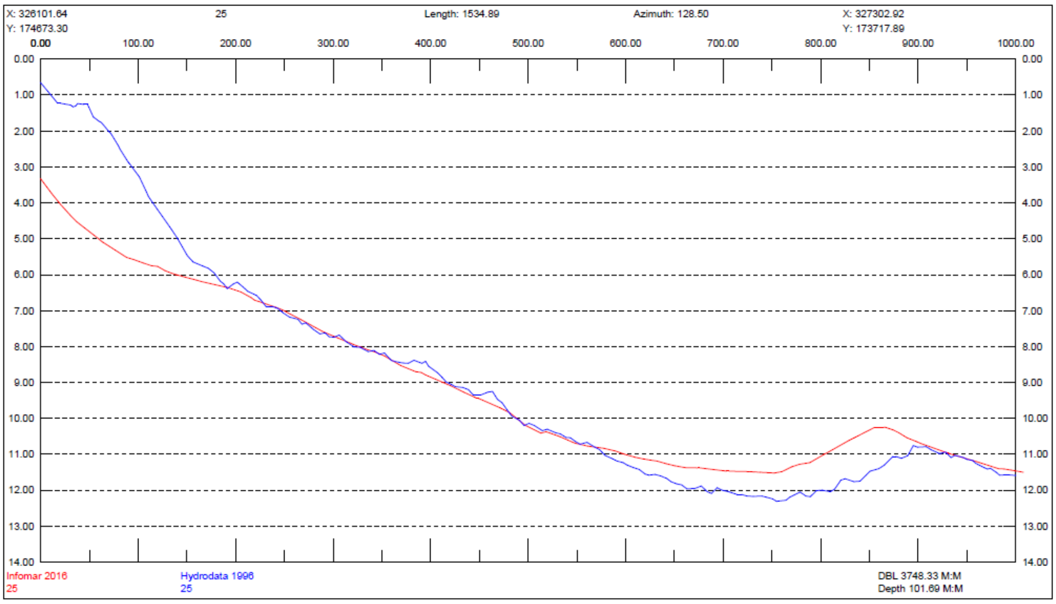


Figure 27: Survey 1996 southern section (Source: Irish Hydrodata Limited report [15]).



5.2.3 Conclusions

Several works are known to have been undertaken within or near the study coastline before the construction of the existing rock revetment, such as the removal of sand from the beach, installation and later dismantling of tiered timber piles, and the construction of the north and south piers at the entrance to Arklow Harbour. There is evidence of continuous historical beach erosion along the coastline beside the site (loss of previously existing beach) during this period. Coastal protection structures had been recommended in the past (See Ref. [13]); however, there is no visual evidence that these have been constructed.

The construction of the revetment in 1972 and 1990 caused the coastline to be rigidized. However, while the coastline is fixed since then, there is evidence of an ongoing natural loss of seabed material. As outlined, previously, the beach in front of the rock revetment at the site is no longer visible. The seabed lowering in front of the revetment at the location of the site between 1985 and 2016 ranges between 0.5m and 2m.

5.3 Existing coastal processes

5.3.1 Coastal processes

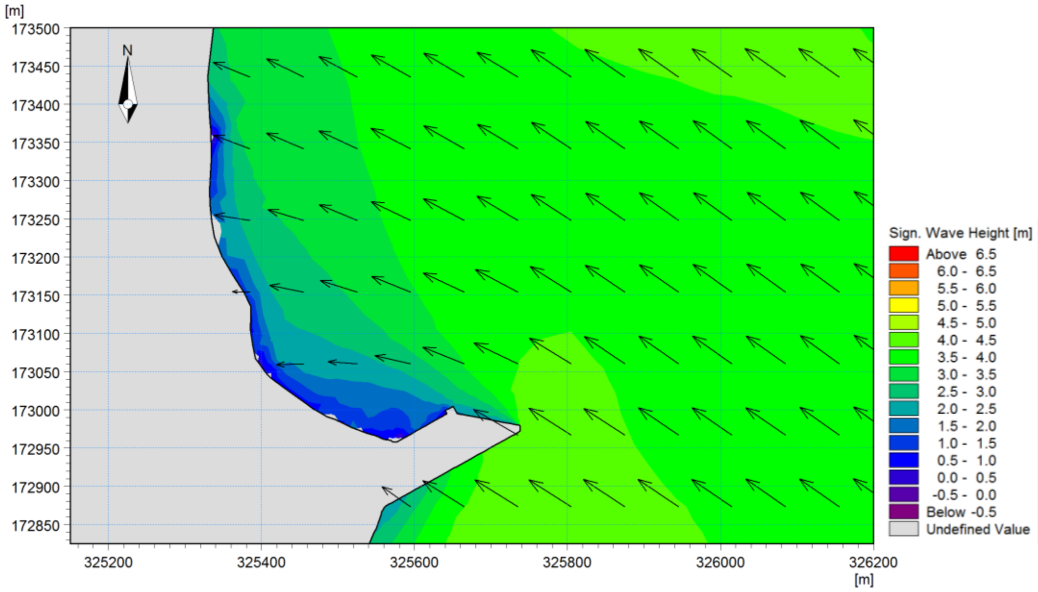
5.3.1.1 Introduction

As outlined in Section 3.1.5, the predominant directions of offshore waves along the study coastline are north-east and south-east.

It was assessed for the previous design of the revetment carried out by JP Byrne (Ref. [13]) that the main longshore drift of sediments in the Area of Interest moves from South to North. Therefore, the two piers which form the entrance of the Arklow harbour act as a barrier to sediment transport from the south. Hence, accretion of sediment is expected to still be occurring to the south of the Area of Interest (south to Arklow Harbour entrance) with further loss of sediments in the north.

However, the existence of the harbour entrance also provides shelter to the revetment at the site location from wave action from the second quadrant directions (south to east south-east). This shelter effect means that the stretch of the proposed upgraded revetment is more protected from wave action from the directions coming from the second quadrant (east-southeast to south) than the section of revetment to the north of the site. This can be seen in Figure 28 which is extracted from the Wave Modelling report (Appendix A) - it is shown that significantly reduced waves reach the revetment specifically at the WwTP site given its proximity to the harbour entrance. As also highlighted in Section 5.2, the coastline which is protected by the revetment does not retreat due to the presence of this hard structure. The stable coastline created by the revetment also means that there is a very limited sediment source existing in the Area of Interest apart from the seabed material and the unprotected areas to the north.

Figure 28: Wave height distribution with waves from the south-east. Zoomed in on Arklow revetment site location.

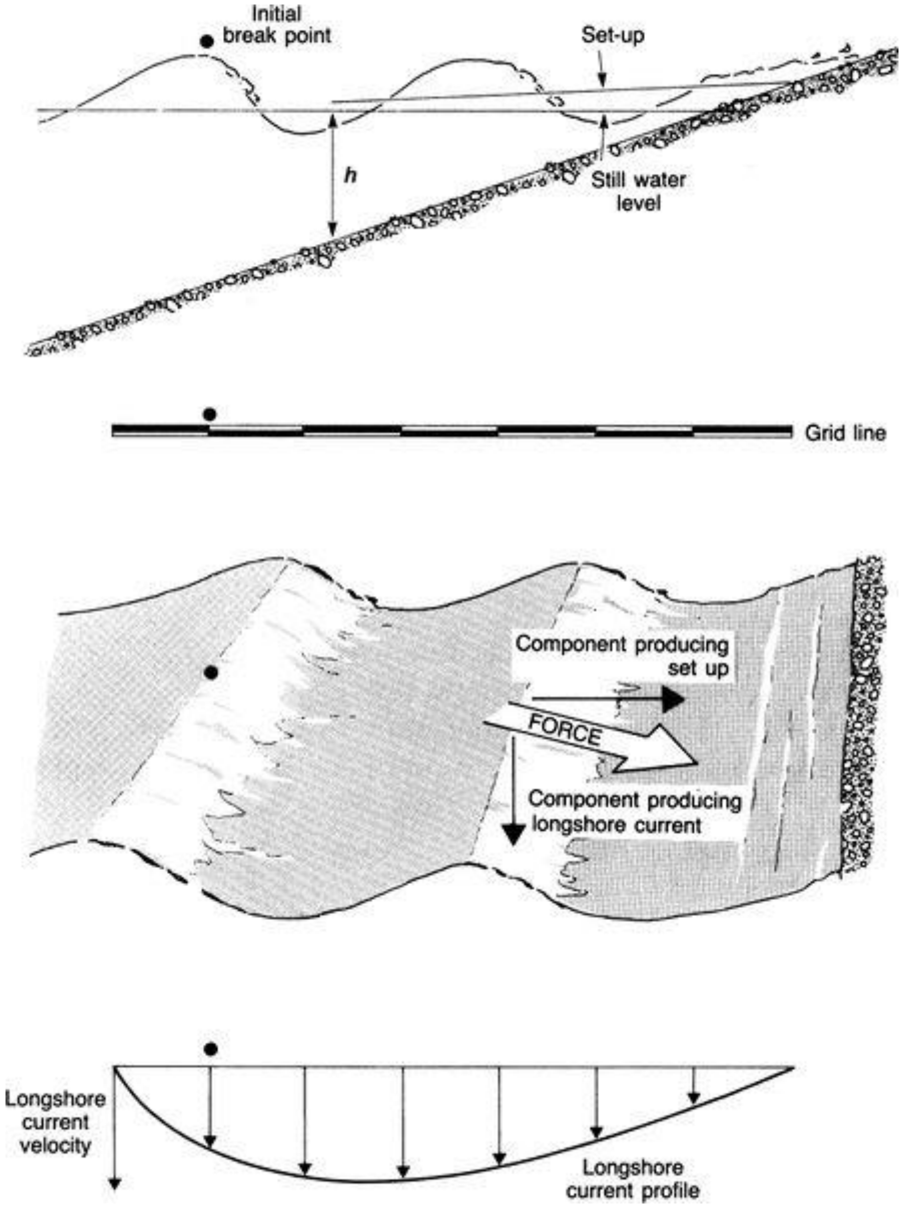


Surf Zone Processes

Waves start to break at some distance offshore of the shoreline/revetment. The area between the wave breaking point and the shoreline is known as the surf zone. The swash zone extends from the surf zone to the waves run-up level on the beach. In this region, the height of an individual wave is largely controlled by the water depth.

The mechanics of this progressive breaking are very complex. It involves turbulence in the breaking area and also a momentum force which may be resolved into two components. The component which lies parallel to the shoreline causes 'longshore current'. The component which is perpendicular to the shoreline produces an increase in the depth of water above the still water level called the 'set up'. Therefore, from a coastal context sediment transport can be separated into two different components - longshore and cross-shore transport. Offshore currents may also occur.

Figure 29: Schematic representation of the longshore current and set up with respect to the breaking point (Source: Reeve, D., Chadwick, A. and Fleming, C. [16]).



Sediment transport of the material which forms the coastline occurs through two different key mechanisms: bed load transport (material rolls or moves at the seabed when sheer stress is exceeded) and suspended load transport (material is suspended in the water column and moves above the seabed). Bed load represents a small fraction of longshore transport compared to suspended load transport [17].

Sediment may be transported by unbroken waves and/or currents, however most transport takes place in the surf and swash zones.

The breaking zone has been estimated based on the Goda 1985 method (Coastal Engineering Manual (CEM) [18] Part II) and the wave study (Appendix A). The maximum breaking wave heights have been estimated based on an approximate average seabed slope of 1V:70H (based on information from the available bathymetric surveys), for return periods of 1, 5 and 10 years.

The significant wave heights offshore (Ho) have been given as an input to the formulation. The results are shown in Table 4.

Table 4: Estimation of the wave height based on Goda 1985 (Coastal Engineering Manual (CEM) Part II [18]).

Tr (years)	**Ho (from wave analysis) (m)**	**Water depth at breaking (m)**
5	3.7	8.14
10	4.2	9.24

The breaking depth has been subsequently estimated for two tide conditions: Mean High Water Springs (MHWS) and Mean Low Water Springs (MLWS). A sea level rise of 1m due to potential effects of climate change has been accounted for in the calculations (see Appendix A for more details).

Table 5: Resulting breaking depths for the two tide scenarios and three return periods considered in the assessment.

Tide scenario	**Tide level (m CD)**	**Breaking contour (m CD)**	
		Tr = 5 years	**Tr = 10 years**
MHWS (m CD) + 1m of sea level rise	2.4	-5.74	-6.84
MLWS (m CD) + 1m of sea level rise	1.6	-6.54	-7.64

The resulting contour lines for the different return periods for the MHWS scenario (i.e. the scenario that gives the widest breaking area) are depicted in Figure 30. These lines represent the seaward extent of the breaking area for each of the return periods considered.

Return periods have been selected to identify the surf zone in average conditions (e.g. relatively low return periods which reflect an average behaviour). Based on these results and considering a return period of five years and climate change it could be expected that the surf zone, where most of the sediment transport occurs in this scenario due to wave breaking and currents, would be limited offshore to the bathymetric line of -6.5m CD. For annual average conditions, the extent of the surf zone is limited to shallower waters, closer to the coastline.

Figure 30: Approximate extent of the estimated breaking zones corresponding to return periods of 5 and 10 years.

5.3.1.2 Estimation of the sediment transport patterns

The potential direction of longshore sediment transportation has been estimated using the CERC formula (presented in the Shore Protection Manual [19]) and the results from the wave propagation model [1].

The resulting potential directions for sediment transport within the Area of Interest are shown in Table 6 for each of the directional sectors with influence on the study area. Estimated directions for the net longshore sediment transport for each wave sector have been assessed for 5 locations at the nearshore (bathymetric contour of -4m CD). These locations were superimposed along with the wave directions for the east south-east direction including shadow effect of the harbour entrance (see Figure 31).

Figure 31: locations for sediment transport calculations.

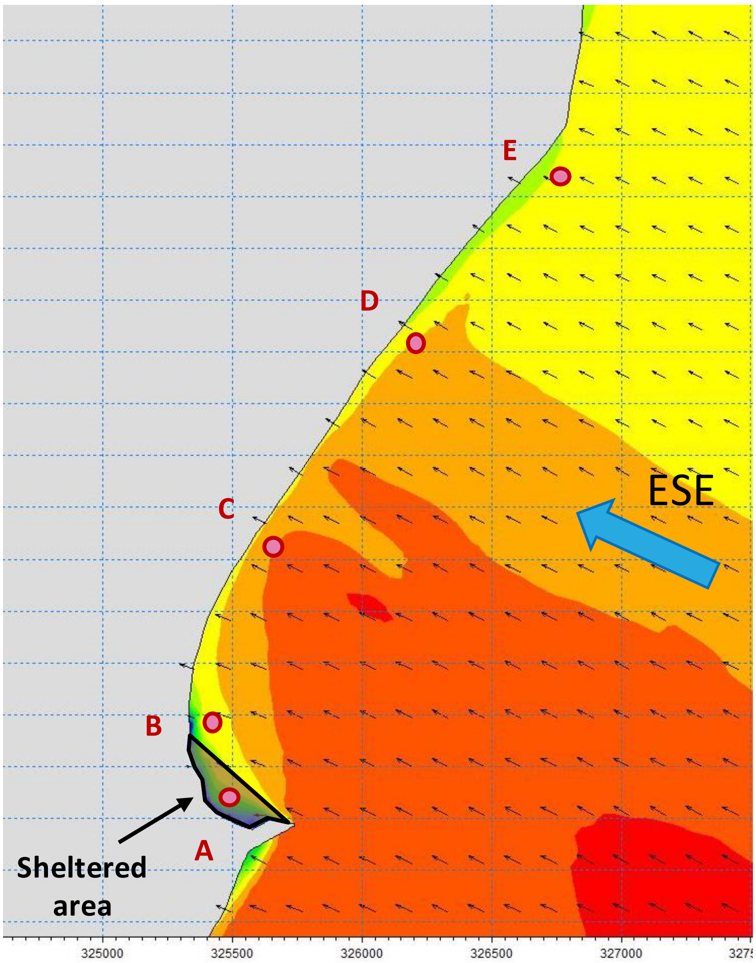


Table 6: Direction of potential sediment transport for each case assessed (red arrow indicates from north to south, green arrow indicates from south to north and yellow indicates neutral).

Points		NE			ENE			E			ESE			SE			SSE			S		
	α_r (°)	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**	α_b (°)	α'_b (°)	**Q**
A	70	70	**0**	⇨	80	**10**	⇧	95	**25**	⇧	70	**0**	⇨	70	**0**	⇨	70	**0**	⇨	70	**0**	⇨
B	90	70	**-20**	⇩	80	**-10**	⇩	95	**5**	⇧	110	**20**	⇧	90	**0**	⇨	90	**0**	⇨	90	**0**	⇨
C	120	80	**-40**	⇩	85	**-35**	⇩	100	**-20**	⇩	120	**0**	⇨	125	**5**	⇧	140	**20**	⇧	160	**40**	⇧
D	130	85	**-45**	⇩	90	**-40**	⇩	105	**-25**	⇩	120	**-10**	⇩	135	**5**	⇧	140	**10**	⇧	160	**30**	⇧
E	135	80	**-55**	⇩	90	**-45**	⇩	100	**-35**	⇩	120	**-15**	⇩	135	**0**	⇨	140	**5**	⇧	160	**25**	⇧

The CERC method assumes that there is an adequate and unlimited source of sediment on the seabed which can be suspended and transferred due to wave action depending on the direction of incoming waves. The formula also assumes bathymetry to be straight and parallel to the coastline. The study area does not consist of a beach- the only beach is the North Arklow Beach, therefore, these potential sediment transport patterns would only affect the nearshore seabed material.

The CERC formula has only been applied qualitatively to assess the potential directions of sediment transport along the coastline.

5.4 Coastal processes affected by the upgraded revetment

5.4.1 Construction phase

It has been considered that the only likely effect that the upgrading works could have on the existing coastal dynamics is the dispersion of material at the location of the revetment works during construction.

The excavation of material from the seabed is limited at the toe of the proposed revetment and the volume of material is expected to be small.

The assessment of likely significant effects associated with the potential presence of potential contaminants within and on the existing revetment is outside the scope of this study, and is covered in Chapter 15 the EIAR [1].

5.4.1.1 Potential effects within the area of interest

As previously mentioned, the coastline at the Area of Interest is mostly covered by the existing rock revetment along approximately 2.2km, while the northern area is comprised of an unprotected sandy beach to the north – the North Arklow Beach.

The potential transport of any suspended material will be mostly confined within the surf zone (approximately limited by the bathymetric contour of -6.5m CD, as outlined in Section 5.3). Moreover, the coastal section of the site is sheltered from the second quadrant directions by the entrance of Arklow harbour. Therefore, any potential dispersion of the material is expected to be naturally deposited within the Area of Interest and mostly limited by both the harbour entrance at the south and the natural headland at the north.

It is important to note that the excavated sediment may either be reinstated in front of the toe of the revetment or disposed of at a suitably licensed facility off-site.

Therefore, the likely effect of dispersing material on coastal processes is considered to be not significant within the Area of Interest during construction of the revetment upgrade.

5.4.1.2 Potential effects outside the area of interest

Based on the envisaged construction methodologies, the site conditions and the existing coastal processes described in the previous sections, the following is noted:

- The areas to the south of the Area of Interest are not affected by the new changes being introduced in the site due to the longshore drift being from south to north, the location of the upgrade of the revetment and the existing barrier of the Arklow harbour entrance.
- The change in orientation of the adjoining Area of Interest to the north limits the influence of any potential sediment transport derived from the excavation in the site.
- The excavation required during construction is limited to the area of the toe of the upgraded revetment. The excavation of the toe extents approximately up the -5m CD bathymetric line.
- The excavated sediment obtained during excavation of the existing seabed in front of the existing revetment may either be reinstated at the toe location or disposed of at a suitably licensed facility off-site

Thus, the likely effect of dispersing material is considered to be not significant in relation to coastal processes outside the Area of Interest during construction of the revetment upgrade.

5.4.2 Mitigation and monitoring

No mitigation or monitoring measures are proposed with respect to the construction of the revetment.

5.4.3 Operational phase

5.4.3.1 Potential effects within the area of interest

No significant effects on existing coastal processes are likely within the Area of Interest during the operational phase given the coastline has already been stabilised by the existing rock armour revetment. Further, the alignment of the upgraded revetment will generally follow the existing revetment alignment.

The revetment upgrade will ensure coastal protection within the site for a 500 year return period storm event as it has been designed to protect against wave overtopping and satisfy functional and safety requirements.

The design ensures that the upgraded revetment can withstand the expected incident waves. The upgraded revetment is a porous flexible structure where wave energy can be partly absorbed and dissipated. For this reason, local wave reflections are expected to be minimum and similar to those currently experienced. The upgraded revetment, being parallel to the coastline and located in the shadow of the Arklow harbour entrance, does not impose a barrier, or an obstruction to the predominant longshore sediment transport patterns.

In this regard, no change in sediment transport is expected with the upgraded revetment from that which exists currently.

Therefore, the likely effect of the existence of the upgraded revetment is considered to be not significant in relation to coastal processes within the Area of Interest during operation of the proposed development.

5.4.3.2 Potential effects outside the area of interest

The areas south of the Area of Interest are not affected by the revetment upgrade predominantly due to the barrier imposed by the Arklow harbour entrance and prevailing longshore drift from south to north. Given the change in orientation of the adjoining area to the north of the Area of Interest, it is concluded that any local potential changes in the Area of Interest will have no effect to the north. Furthermore, the assessment of the effects within the Area of Interest apply (see section 5.4.3.1).

Therefore, the likely effect of the existence of the upgraded revetment is considered to be not significant in relation to coastal processes outside the Area of Interest during the operation of the proposed development.

5.4.4 Mitigation and monitoring

The revetment and its toe will be monitored to ensure its performance. The revetment will be monitored by Irish Water as part of the overall maintenance of the works. Revetment maintenance would include visual inspection either by divers or robotics and would be performed every year and after significant storm events. The inspection crew would check the revetment for damage to the toe, rock stability, lowering of nearshore bed levels or other damage. Suitable remediation works will be undertaken as required.

5.5 Coastal processes affected by the SWO

With regard to the SWO, given that it discharges at the shoreline (below MLWS), the construction and operation of the SWO, similar to that of the revetment, will not result in a significant effect in relation to coastal processes inside and/or outside the Area of Interest. Refer to Sections 5.4.1.1 and 5.4.1.2 for further detail on this assessment.

5.6 Coastal processes affected by the long sea outfall

5.6.1 Introduction

This section considers the likely significant effects on coastal processes as a result of the two open cut construction methodologies for the long sea outfall (i.e., construction by means of the float and flood or bottom pull method which requires a trench to be excavated) and the method for installation of the diffusers, for all three construction methods.

The likely significant effect due to dispersion of the excavated seabed material (which may be side casted along the edge of the trench during construction), dispersion of any sediment mobilised during the dredging process and the exposure of the outfall and/or scour protection during operation of the proposed development have been assessed.

The likely significant effects associated with the horizontal directional drilling method would not involve any change in the seabed geometry during construction or operation, therefore this option is not considered to result in any significant effects on coastal processes.

The assessment of likely significant effects associated with the potential presence of potential contaminants at the outfall location is outside the scope of this study, and is covered in Chapter 14 of the EIAR [1].

5.6.2 Construction phase

5.6.2.1 Potential effects within the area of interest

The construction of the outfall pipeline could result in an increased rate of sediment dispersion, with the dredged sediments being completely moved from their original position near the trench. Local currents could suspend the limited volume of sands causing its dispersion by waves and currents. This could have an effect on coastal processes, and in turn on sensitive ecological receptors, such as marine species and Natura 2000 sites.

However, there are a number of factors which will reduce the significance of effect in this regard, as follows:

- This material given its limited volume and the wave conditions necessary for the dredging operation (low wave conditions generally required) would be mostly deposited within the coastal Area of Interest, and if any, it would slightly increase the overall volume of seabed material in the seabed and submerged beach of the adjoining area.

- The seabed material is expected to be non-cohesive and with a low content of fines (Section 8.2), therefore significant suspension of fines is not anticipated. The CIRIA Report 159 [9] states that tide-induced seabed velocities alone are rarely sufficient to initiate motion of non-cohesive sediments at coastal sites, and hence significant movement of sand and gravel tends to be associated with periods of high wave activity only. In this regard, the most significant movement may happen in or close to the surf zone.

  The bed sediment has been characterised as non cohesive and reasonably coarse, therefore it can be inferred that any sediment mobilised from the works is likely to be deposited in the vicinity of the outfall and will not impact on the wider marine environment.

- Works are envisaged to be undertaken in the summer season to facilitate the necessary calm wave conditions to operate the plant and equipment required for excavation of the trench.

A maximum wave height of 0.5m is a typical operational limit for the dredging operations. Under these wave conditions, the surf zone is estimated to be nearshore. Due to the vicinity of the outfall with the river's breakwaters, this area is sheltered from the south. Where the outfall falls outside the nearshore surf area and Arklow Harbour entrance, the suspended sediment transport is expected to be very limited (some minor local sediment movements may occur on the seabed) as shown by the estimation of the break area and also historical evidence as shown in the historical comparison of the profiles for the offshore area.

- The marine environment is dynamic and there is a continuous process of sedimentation/deposition which naturally occurs. Against this background, the impact of the sedimentation due to the engineering works will not be significant.
- The volumes of excavated material are considered relatively low are and expected to be partially re-used as described in section 2.4.2.2, if deemed suitable by the contractor.

In summary therefore, it is considered that the reasonable worst case will not affect the overall coastal patterns. No erosion or accretion of adjoining areas is expected to result from the outfall construction.

Thus, the likely effect of local sediment movement is considered to be not significant in relation to coastal processes within the Area of Interest during construction of the long sea outfall. Effects on other environmental receptors, such as Natura 2000 sites and marine species are addressed in the EIAR and NIS.

5.6.2.2 Effects outside the area of interest

The risk of the sediment transport having an impact on the areas outside of the Area of Interest is considered very low due to the following:

- those factors presented above in Section 5.6.2.1;
- There is a very limited potential for dredged material to be dispersed within a larger area outside the Area of Interest; and
- The presence of the Arklow Harbour piers provides shelter to the adjacent area to the south in the area where most of sediment transport is expected (nearshore area).

No negative impacts on receptors from sediment transport (such as emerged beach or dunes) are anticipated given the very limited increase of the material that could be transported.

Thus, the likely effect of local sediment movement is considered to be not significant in relation to coastal processes outside the Area of Interest during construction of the long sea outfall. Effects on other environmental receptors, such as Natura 2000 sites and marine species are addressed in the EIAR and NIS.

5.6.3 Operational phase

5.6.3.1 Effects within the area of interest

Scour protection will be installed to ensure the structural integrity of the outfall during operation. The scour protection will consist of a layer of concrete mattresses embedded in the existing seabed. This scour protection will be designed to be stable and prevent any scour of the seabed against nearshore wave action and currents. The scour protection will be designed to match the seabed level to avoid the creation of a sediment transport barrier. The scour protection will also stabilise and prevent the movement of seabed material in the local area of the outfall.

In the event that seabed levels in the area close to the scour protection reduce, the concrete mattresses would accommodate to the new geometry. It is important to note that the outfall and associated scour protection will be designed against this outcome, but it is assessed as a reasonable worst case scenario. This potential lowering of seabed will not impose a barrier to sediment transport based on the following:

- Longshore sediment transport occurs within the break area.
- The break area of the outfall is mostly sheltered by the entrance of Arklow Harbour.
- Bed load represents a small fraction of longshore transport compared to suspended load transport and therefore any local new feature of the seabed would not change any existing longshore sediment transport patterns.

On this basis, no change in the existing coastal processes involving erosion or accretion of the adjoining coastal areas is expected due to the presence of this outfall and therefore no significant effects are likely during operation of the proposed development.

Thus, the likely medium to long term effect of the outfall is considered to be not significant with respect to coastal processes within the Area of Interest during operation of the proposed development. Effects on other environmental receptors, such as Natura 2000 sites and marine species are addressed in the EIAR and NIS.

5.6.3.2 Effects outside the area of interest

Given that there are no significant effects within the Area of Interest, it can be concluded that are no significant effects are likely to arise outside of the Area of Interest from the existence of the outfall for the same reasons as described above.

Thus, the likely effect of the outfall is considered to be not significant in relation to coastal processes outside the Area of Interest during operation of the proposed development. Effects on other environmental receptors, such as Natura 2000 sites and marine species are addressed in the EIAR and NIS.

5.6.4 Mitigation measures

Construction of the long sea outfall will generally be restricted to the period May – September, with the period between November-February generally avoided. In this manner, the months with likely worst wave and wind conditions, which lead to higher levels of sediment suspension and transport, are avoided.

5.6.5 Monitoring

As for all such infrastructure, the scour protection shall be monitored to ensure its performance and avoid any potential risk derived from the potential future exposure of the pipe. Scour protection will be monitored by Irish Water as part of the overall long outfall maintenance. Outfall monitoring would include visual inspection either by divers or robotics and would be performed every 5 years and after significant storm events as part of the overall operational management regime. The inspection crew would check the pipeline for scour protection damage, slide, anchor, or other damage. Scour protection shall be reinstated and/ or repaired if any damage is observed.

5.7 Residual effects

It is considered that, with the implementation of the proposed mitigation and monitoring measures, that there are no significant residual effects from the proposed development on coastal processes including sediment dispersion and local scour/siltation effects. Residual effects on other environmental receptors, such as Natura 2000 sites and marine species are addressed in the EIAR and NIS.

6 References

[1] Arup (2018) ‘Environmental Impact Assessment Report’, ref. 247825/EIA/EIAR/1

[2] Drawing M-R-1001 Arklow WWTP Revetment Upgrade Existing Layout Plan

[3] Drawing M-R-1002 Arklow WWTP Revetment Upgrade Proposed Layout Plan

[4] Drawing M-R-2001 Arklow WWTP Revetment Upgrade Cross Sections – Sheet 1 of 3

[5] Drawing M-R-2002 Arklow WWTP Revetment Upgrade Cross Sections – Sheet 2 of 3

[6] Drawing M-R-2003 Arklow WWTP Revetment Upgrade Cross Sections – Sheet 3 of 3

[7] Stevens (2015) ‘Trenchless solutions for sewer networks and sea outfalls’

[8] WRC (1990) ‘Design guide for marine treatment schemes: Volumes I – IV’

[9] CIRIA 159 Report (1996) ‘Sea outfalls – Construction, inspection and repair. An engineering guide’.

[10] ICPSS report Irish Coastal Protection Strategy Study Phase 2 - South East Coast

[11] Irish Hydrodata Limited report (2018), ‘Arklow WWTP Investigation of the Impact of Treated Wastewater discharges to the Irish Sea’

[12] J.P Byrne & Partners (1990) ‘Coastal Protection Works at North Beach Arklow’

[13] Byrne, J.P. and Motherway, F.K. (1990) ‘Design and construction of coastal protection works at north beach, Arklow, Co Wicklow’, Paper presented to a seminar on ‘Engineering for Coastal protection’ at the Institution of Engineers of Ireland

[14] Ordnance Survey Map, 1980. Sheet 40

[15] Irish Hydrodata Limited report, ‘Arklow WWTP Comparison of marine bathymetric data sets, Irish Hyrdodata 1985 & 1996 vs GSI Informar 2016’

[16] Reeve, D., Chadwick, A. and Fleming, C. (2004), ‘Coastal engineering: Processes, theory and design practice’.

[17] Robert B. Nairn, Howard N. Southgate, “Deterministic profile modelling of nearshore processes. Part 2. Sediment transport and beach profile development” Coastal Engineering Volume 19, Issues 1–2, February 1993, Pages 57-96

[18] U.S. Army Corps of Engineers (2002) 'Coastal Engineering Manual'.

[19] U.S. Army Corps of Engineers (1984) 'Shore Protection Manual'

Appendix A

Wave modelling report

Irish Water

Arklow Waste Water Treatment Plant

Wave modelling report

247825-00

Issue | 28 August 2018

Job number 247825-00

Ove Arup & Partners International Ltd

Arup
50 Ringsend Road
Dublin 4
D04 T6X0
Ireland
www.arup.com

ARUP

Contents

1 Introduction

Arup has been commissioned by Irish Water to provide Engineering services for the Arklow Waste Water Treatment Plant (WWTP) including specimen design. The scope includes the design of the upgraded section of the revetment at the eastern site boundary (see Figure 1), which requires an accurate estimation of the design wave height. This estimation has been carried out by undertaking numerical wave modelling.

This includes:

- Acquiring offshore wave data
- Setting up and applying a spectral wave model to model wave propagation from offshore to the near shore

Figure 1: Arklow waste water treatment plant preferred location including the existing revetment.

2 Methodology

The following steps were undertaken as part of the wave modelling study:

1. Analysis of the offshore wave climate;
2. Setup of the bathymetry, computational mesh and boundary conditions and application of the model;
3. Extraction of the model results at the location of the revetment;

2.1 Software

The model used for wave propagation from offshore to the proposed location is MIKE21-SW developed by DHI. This software is a 3rd generation spectral wind-wave model that simulates the growth, decay and transformation of wind-generated waves and swell in offshore and coastal areas. The software includes the following physical phenomena:

- Wave growth by wind action;
- Non-linear wave-wave interaction;
- Dissipation by white-capping;
- Dissipation by wave breaking;
- Dissipation due to bottom friction;
- Refraction due to depth variations:
- Wave-current interaction;
- Diffraction;
- Reflection;

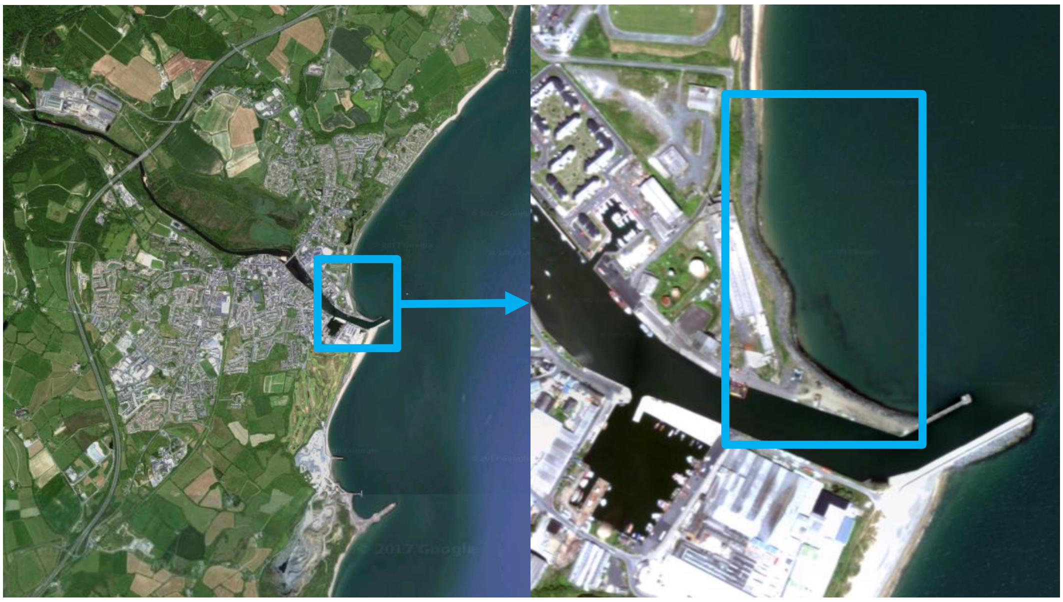
A major application area for this model is the design of nearshore structures where accurate assessment of wave loads is of utmost importance for a safe and economic design.

3 Site condition analysis

3.1 Site location

The location of the proposed revetment at the site of the Arklow WWTP is located to the north of the entrance to Arklow port (see Figure 2). The site is fully exposed to waves from the NE and E but is partially protected from waves from the SE due to the presence of the piers.

Figure 2: Location of upgraded revetment at Arklow WWTP – Source: Google Maps - ©2014 Google.



3.2 Design water level

The design water level for the proposed revetment is based on the following sources:

- Tidal levels in Arklow Harbour [1]
- Irish Coast Protection Strategy Study [4]

Tidal levels were taken from Admiralty Chart 1468 as shown in Table 1 below. LAT is not provided by the tide tables, but is taken as 0m Chart Datum.

Table 1: Tidal levels in Arklow Harbour.

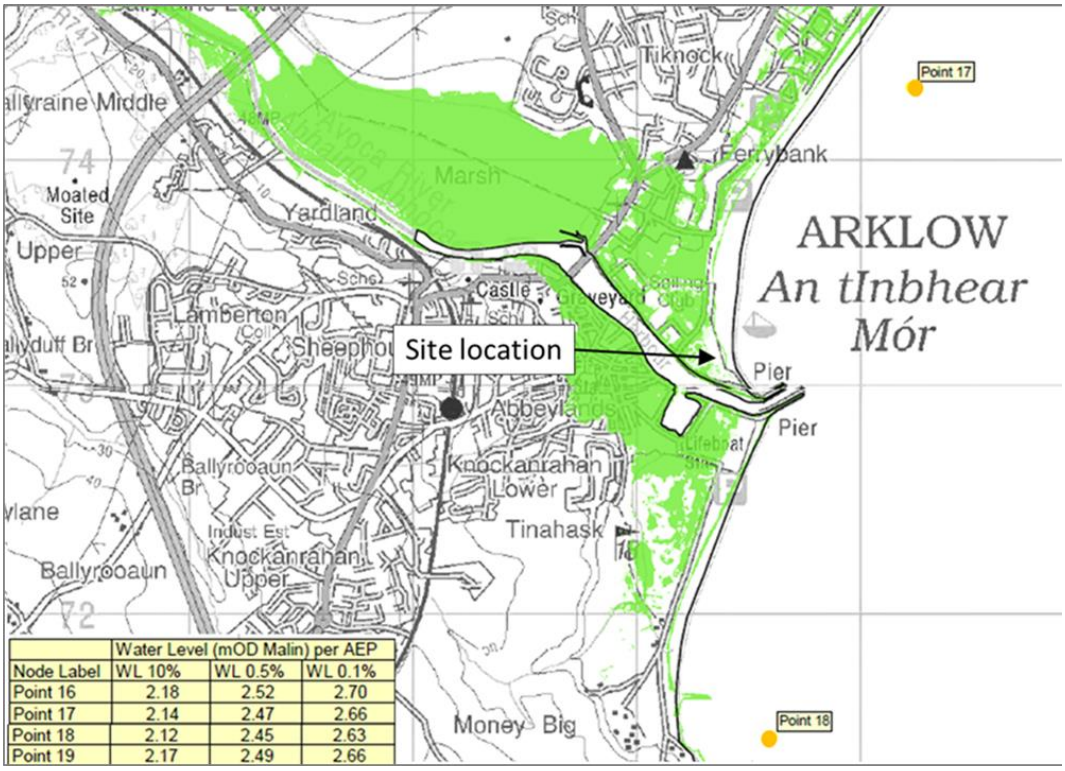
Tidal Level	Chart Datum	OD Malin
MHWS	1.4 m	0.28m
MHWN	1.2 m	0.08m
MLWN	0.9 m	-0.22m
MLWS	0.6 m	-0.52m
LAT	0 m	-1.12m

The Irish Coastal Protection Strategy Study (ICPSS) is a national study that maps extreme water levels along the Irish coast. Package 9A of this study, which includes the future scenario assessments of extreme coastal water levels, was used to obtain predicted extreme water levels for the design of the revetment. Since the WWTP is deemed critical infrastructure, the high end future scenario (HEFS) water levels were adopted. These predicted water levels include a combination of storm surge and extreme tidal levels, based in both numerical modelling and statistical analysis of historic tide gauge data. The HEFS levels also allow for land movement and +1.00m sea level rise by the year 2100.

A return period of 500 years was calculated using BS 6349-1:2000, for a 50 year design life and a 10% occurrence probability, which is considered appropriate for the significance of the infrastructure. As the ICPSS maps do not provide HEFS water levels for a 500 year return period, interpolation between the 200 and 1000 year return periods was used to obtain this value. As shown in Figure 3, the site is located approximately half way between two data points on the ICPSS maps. Therefore, water levels at the site are found by interpolation between these two points.

The resultant extreme water level at the site is 2.56mOD Malin or 3.68m Chart Datum.

Figure 3: Extract from ICPSS drawing no. SE / RA / EXT / HEFS / 9 and 10.



Node Label	Water Level (mOD Malin) per AEP		
	WL 10%	WL 0.5%	WL 0.1%
Point 16	2.18	2.52	2.70
Point 17	2.14	2.47	2.66
Point 18	2.12	2.45	2.63
Point 19	2.17	2.49	2.66

3.3 Offshore wave and wind data

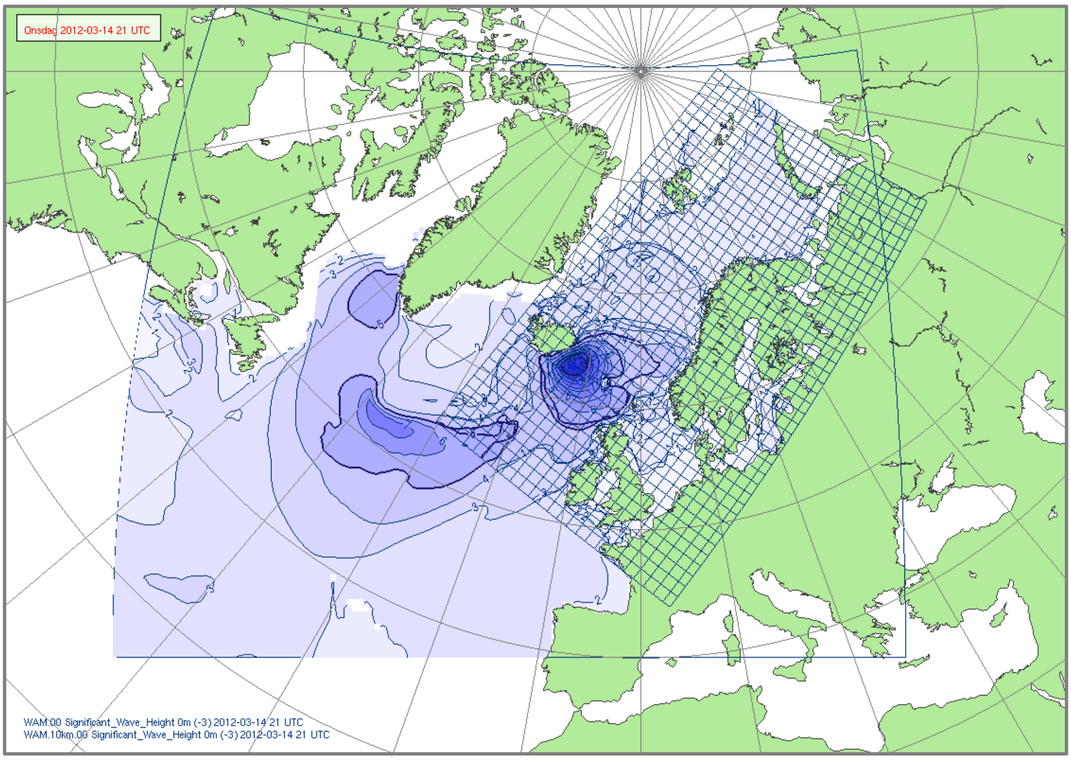
To approximate the design wave height for the new revetment design process, a computational model was used that simulates wave propagation from offshore to nearshore. The quality of the offshore data used to define the boundary conditions of the model is a key factor in setting up a reliable wave model.

3.3.1 Data source

For this site, Arup used the Norwegian ReAnalysis 10km database (NORA10) [1] This database consists of wave and wind data records for the period September 1957 to September 2017.

The NORA10 hindcast model was developed by the Norwegian Meteorological Institute[2]. It is a regional atmospheric and wave hindcast model (HIRLAM and WAM Cycle 4) covering the Northern European waters. The regional model uses wind and wave boundary conditions from the ERA-40 reanalysis (based on wind data from1958 to 2002) and is extended using the ERA-Interim reanalysis from 2002 to 2011. NORA10 produces three-hourly wave and wind fields at 10km spatial resolution, see Figure 4.

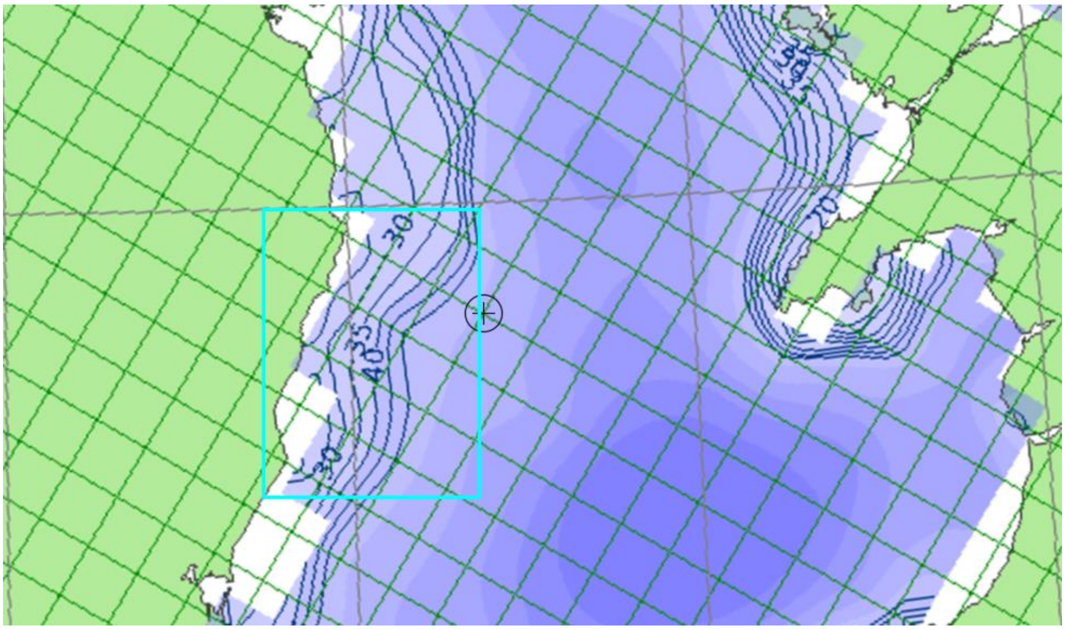
Figure 4: NORA10 (Norwegian ReAnalysis 10km) database: Area coverage.



For this wave study, the data has been obtained from the NORA10 grid point at location 52.80N, 5.62W (see Figure 5) as the offshore position on the East coast of Ireland allows for the estimation of the wave climate in the offshore area adjacent to Arklow.

This 60 year dataset provides sufficiently long record for a reliable statistical extreme value analysis.

Figure 5: NORA10 wave buoy location (52.80N, 5.62W) and computational boundaries used.



Directional analysis of the offshore wind and wave climate was carried out to assess the metocean conditions at the proposed model boundary. Only waves originating from the first and second quadrants (from North to South directions clockwise) propagate to the proposed location. For this reason, only offshore wind and wave conditions from the first and second quadrant were considered.

Wave data

Figure 6 presents the directional wave distribution of the offshore wave climate; this corresponds to the NORA10 offshore node. From Figure 6, it is shown that the predominant wave direction is SSW with an associated frequency of approximately 36%.

Figure 6: Offshore Wave Rose – All directions.

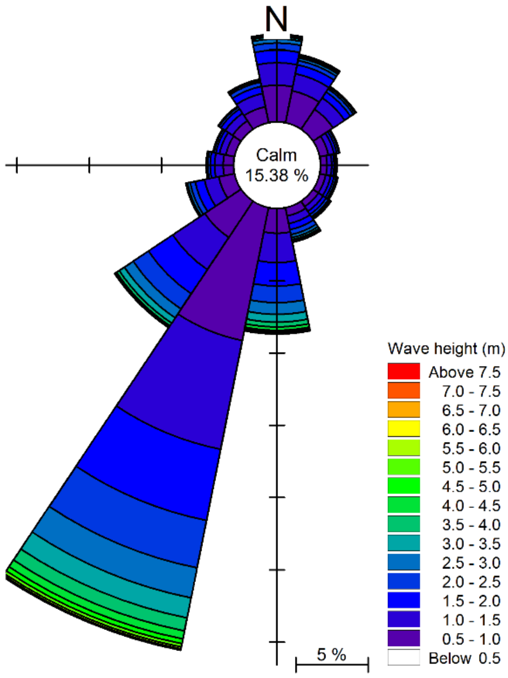
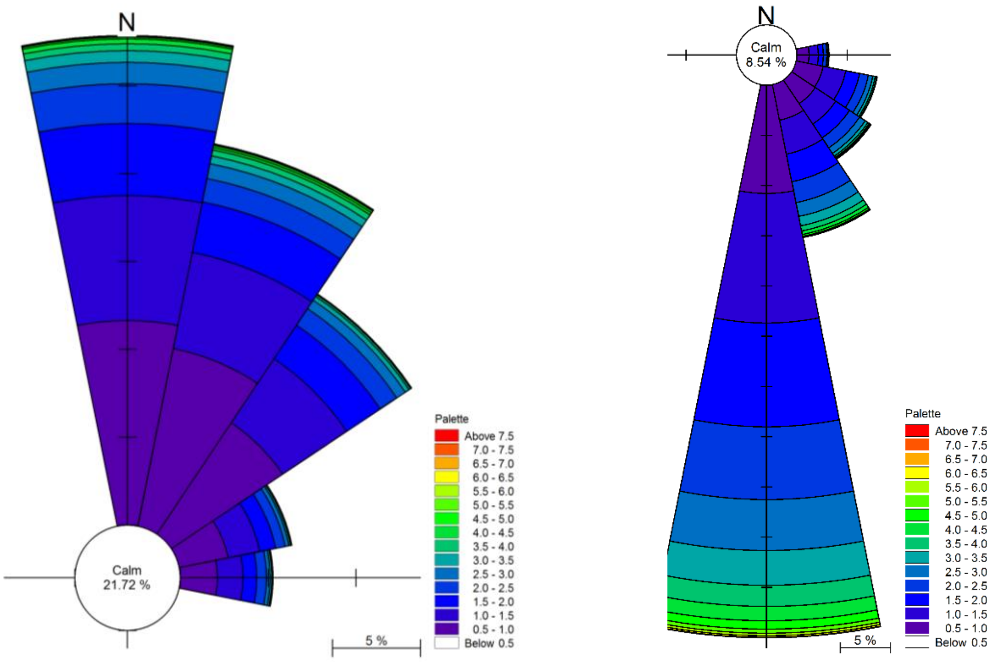


Figure 7 shows the directional wave distribution in the first and second quadrant respectively. Within the first quadrant the most frequent directions are N, NNE and NE with decreasing frequency towards the east.

Within the second quadrant the most frequent direction is S. The northern and southern directions also have higher offshore significant wave height values, H_s, with a maximum of approximately 7.8m.

Figure 7: Offshore wave roses first and second quadrant.

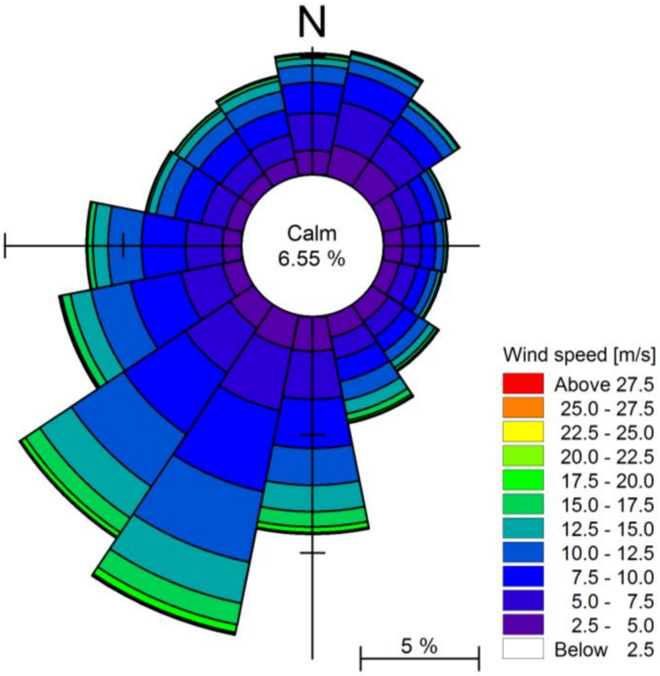


As previously stated, the proposed location is only exposed to waves from both the first and second quadrants. Therefore, based on the bathymetry and the directions likely to affect the location, waves coming from north to south will be assessed.

Wind data

Figure 8 shows the directional wind distribution of the full wind rose at the NORA10 grid point. The predominant wind directions are SSW and SW. For the first and second quarter, the predominant directions are NNE and NE. Maximum wind speed values for the first and second quarter are approximately 25m/s for the south easterly directions.

Figure 8: Offshore Wind Rose – All directions



3.3.2 Extreme value analysis

The offshore data has been analysed statistically using the EVA module of MIKE software, which is also used for the wave modelling. Specific analysis of all directions from North to South has been carried out to obtain the best fit to the function of extreme values of H_S.

Annual maximum values have been analysed to select the extreme value of H_S for each of the directions. This analysis was based on both, the Gumbel extreme value function and the Generalised Extreme Value (GEV) function, which are given by Equation 1 and Equation 2 respectively:

$$T_R = \frac{1}{1-F(Hs)}, \text{ where } Hs(x) = e^{-e^{\left(-\frac{(x-\mu)}{\Psi}\right)}} \quad (1)$$

$$T_R = \frac{1}{1-F(Hs)}, \text{ where } F(Hs) = e^{-\left(1+\xi\frac{Hs-\mu}{\psi}\right)^{-\left(\frac{1}{\xi}\right)}} \quad (2)$$

In which T_R is the return period, H_s is the significant wave height, μ (location), ψ (scale) and ξ (shape) represent the statistical parameters for the functions chosen.

Wave analysis

This analysis was carried out in order to determine the distribution of significant wave height (H_s [m]) over the return period [y]. For each of the directions assessed the software provides a best fit curve through the data for all requested distributions. Generally, the GEV distribution fits better the significant wave height dataset. The chosen distribution curves for the 5 most important directions for the site (NE, ENE, E, ESE and SE clockwise) are shown in Figure 9 to Figure 13 respectively. The distribution curves for the other directions can be found in Appendix A.

In each of the diagrams, the solid line represents the best fit curve whereas the dashed lines on either side represent the confidence limits which indicate the 90% confidence around the predicted function.

The best fitting formula was used to calculate the significant wave height for the required return period. The wave peak period (T_p) is matched with the significant wave height (H_s) using the Jonswap-formula based on Holthuijsen, 2007 [7]. The result is shown along with an extrapolation of the NORA10 hindcast data in Figure 14.

Figure 9: NE: Extreme value analysis for the significant wave height (H_s).

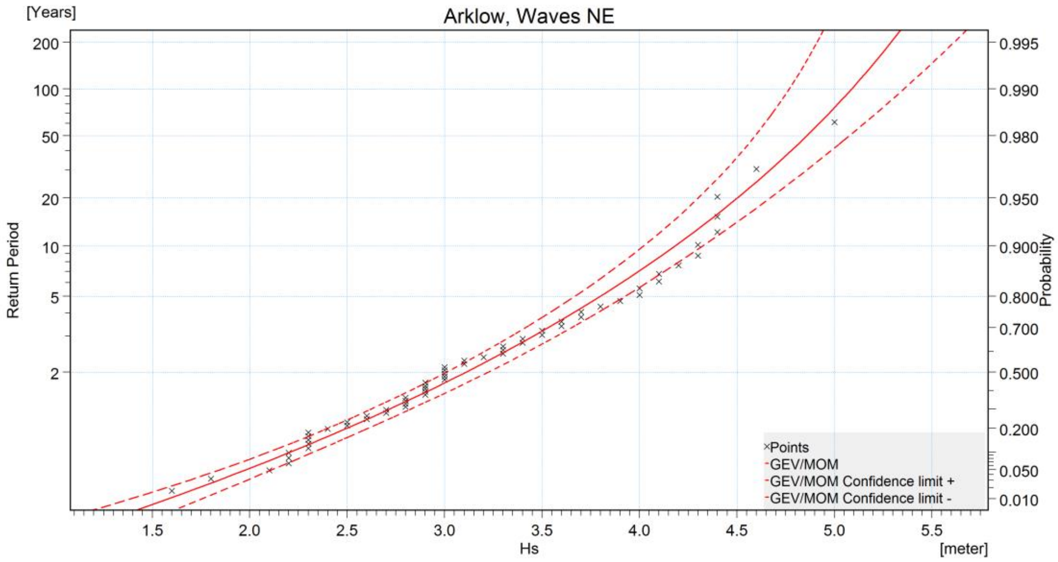


Figure 10: ENE: Extreme value analysis for the significant wave height (H_s).

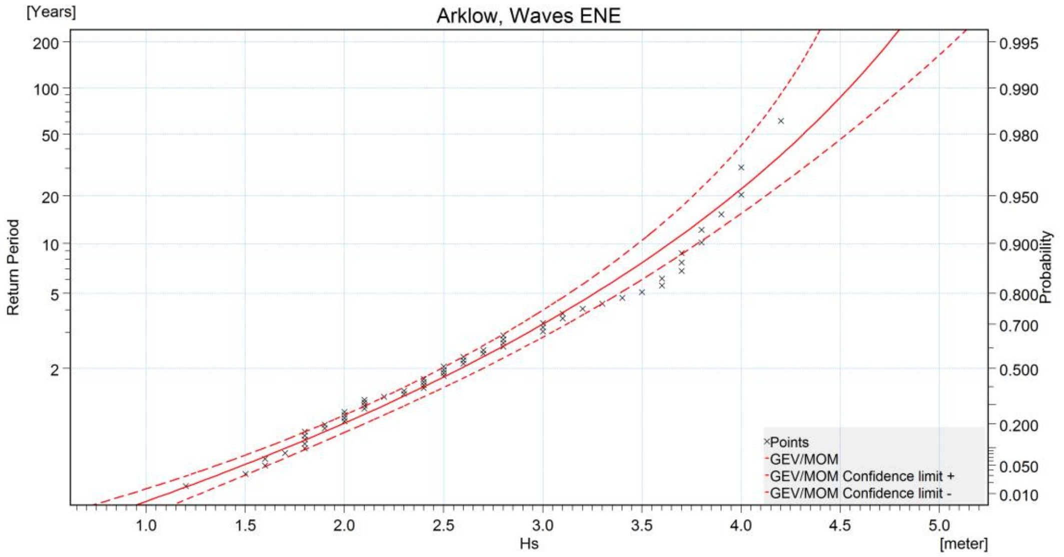


Figure 11: E: Extreme value analysis for the significant wave height (H_s).

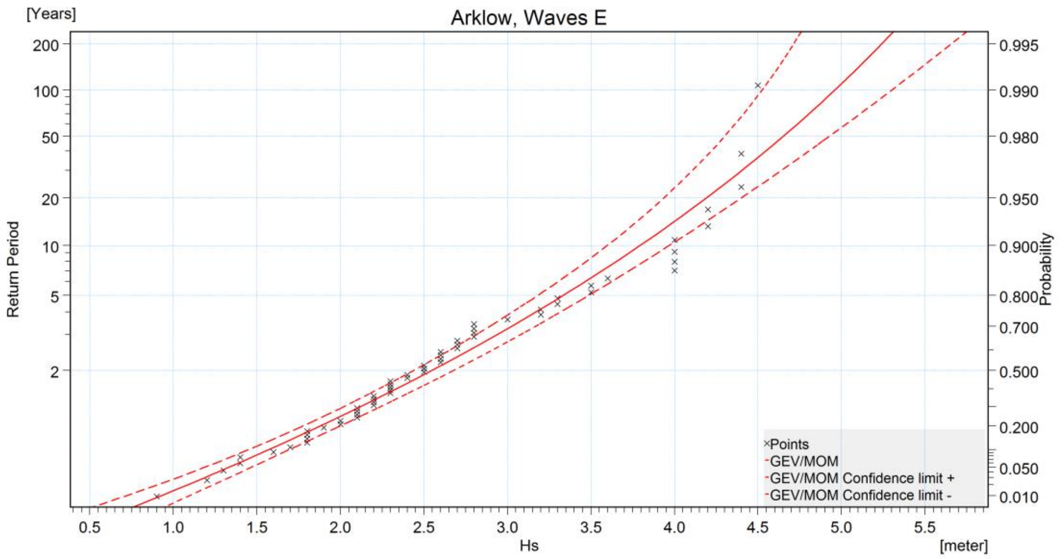


Figure 12: ESE: Extreme value analysis for the significant wave height (H_s).

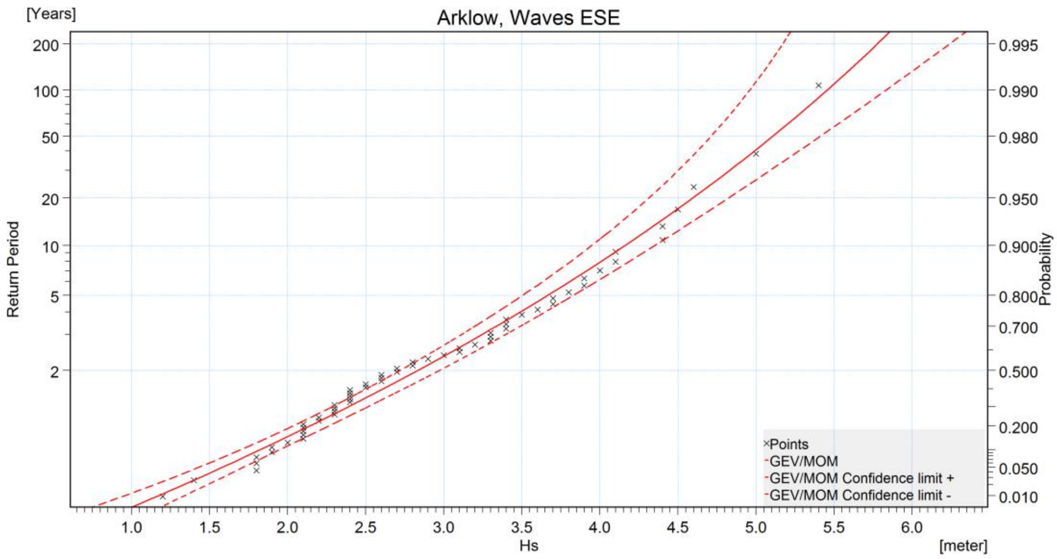


Figure 13: SE: Extreme value analysis for the significant wave height (H_s).

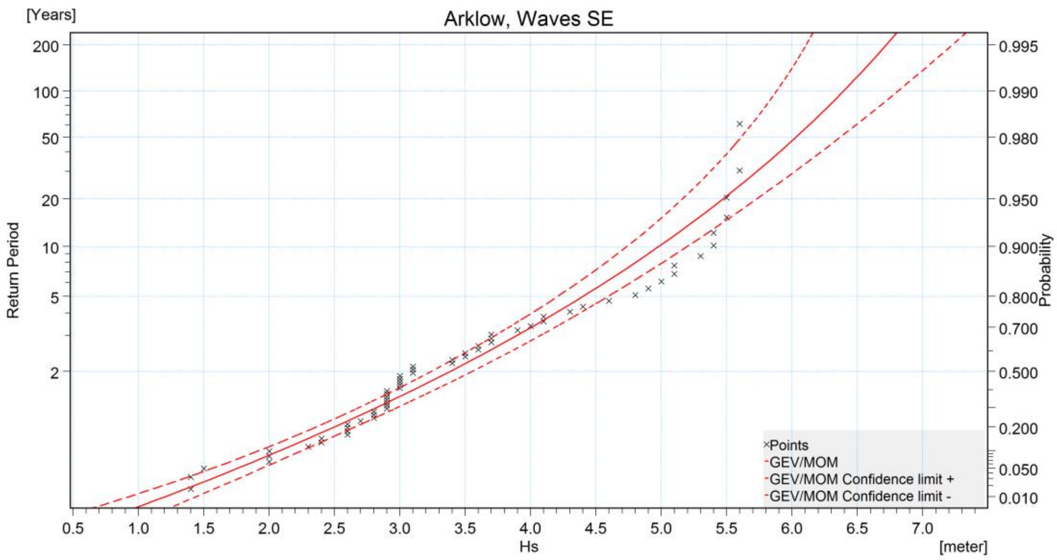
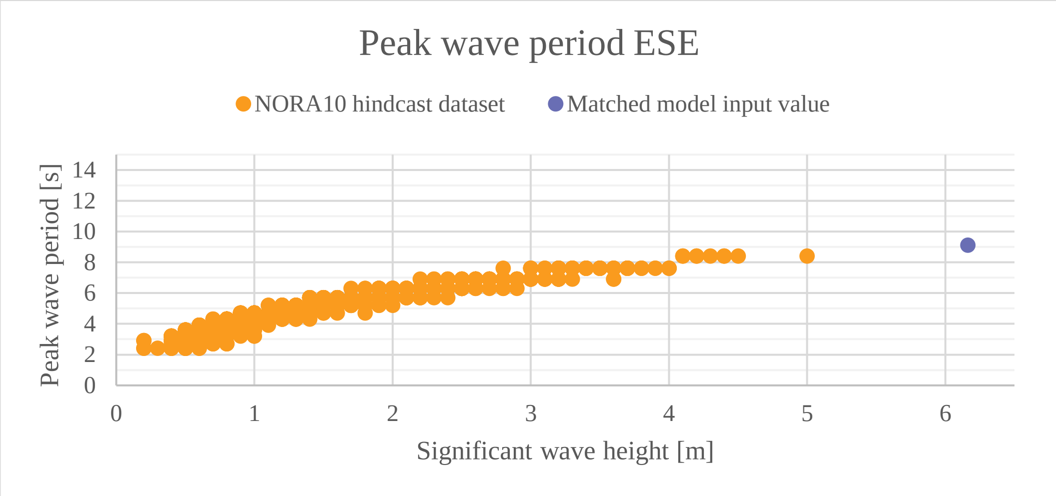


Figure 14: ESE: Comparison of cloud plot from NORA10 hindcast dataset with the matched peak wave period (Tp) based on Jonswap formula.



Wind analysis

This analysis was carried out in order to determine the distribution of average wind speed (W_{sp} [m/s]) over the return period [y]. For each of the directions assessed, the software provides a best fit curve through the data for all requested distributions as has been done for the significant wave height analysis. Generally, the Gumbel distribution fits better the wind speed dataset. The plots can be found in Appendix A.

3.4 Summary of model input data

Table 2 below summarises the results of the assessments of design water level and offshore wind and wave data as described in the previous sections. This data will be used as boundary input data in the numerical wave modelling and will be used to define a number of model runs as appropriate.

Table 2: Water level, wave and wind input summary.

			Extreme sea level		**Offshore wind and wave data**			
Wave direction		**Scenario**	**Return Period**	**Water Level**	**Return Period**	**H_S**	**T_p**	**V_w**
	^{0}N		**[year]**	**[m CD]**	**[year]**	**[m]**	**[s]**	**[m/s]**
NE	45	HEFS	500	3.68	500	5.5	8.6	24.3
ENE	67.5	HEFS	500	3.68	500	5.0	8.3	20.7
E	90	HEFS	500	3.68	500	5.6	8.7	24.4
ESE	112.5	HEFS	500	3.68	500	6.2	9.1	25.0
SE	135	HEFS	500	3.68	500	7.1	9.7	23.0

4 Wave modelling analysis

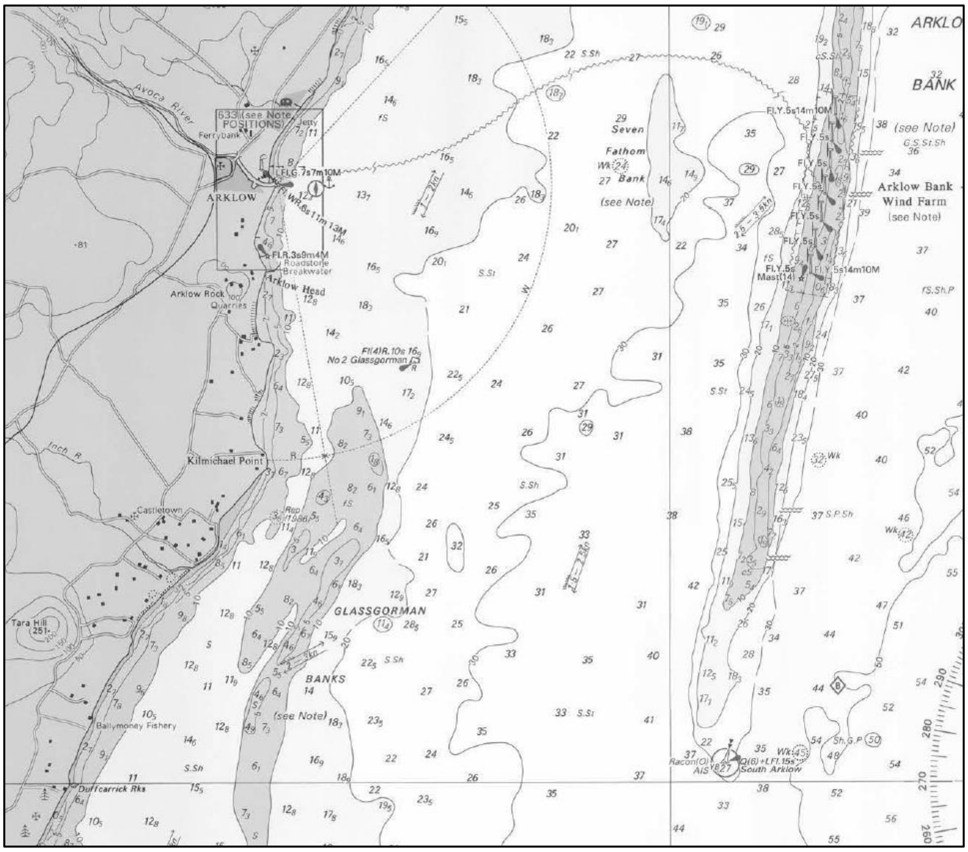
When waves approach the coastline, they undergo a number of changes caused by refraction, diffraction, breaking and shoaling, which affect their characteristics; steepness, wave height, propagation velocity and direction. In this study, a wave transformation model was used to propagate the waves from offshore to the nearshore at the site location. As described in the “Methodology” section the model used for wave propagation from offshore to the proposed location is Mike21-SW developed by DHI.

4.1 Bathymetric data

In order to assess the changes in wave characteristics it is necessary to gather all available bathymetric data for the proposed site location. The sources used are:

- UK Hydrographic Office Admiralty Chart number 1787 (high level): Carnsore Point to Wicklow Bay, scale 1:100,000, depths in metres reduced to Chart Datum (see Figure 15 below).
- UK Hydrographic Office Admiralty Chart number 633 (nearshore): Arklow, scale 1:10,000, depths in metres reduced to Chart Datum.
- GSI Infomar 2016 bathymetric survey (nearshore): 20m grid, main source of nearshore bathymetry, depths in metres reduced to Chart Datum.
- Murphy Surveys topographic survey, March 2016 (governing nearshore): revetment contour lines, main source of nearshore coastline location, depths in metres reduced to Chart Datum.

Figure 15: Snapshot of Admiralty Chart 1787.

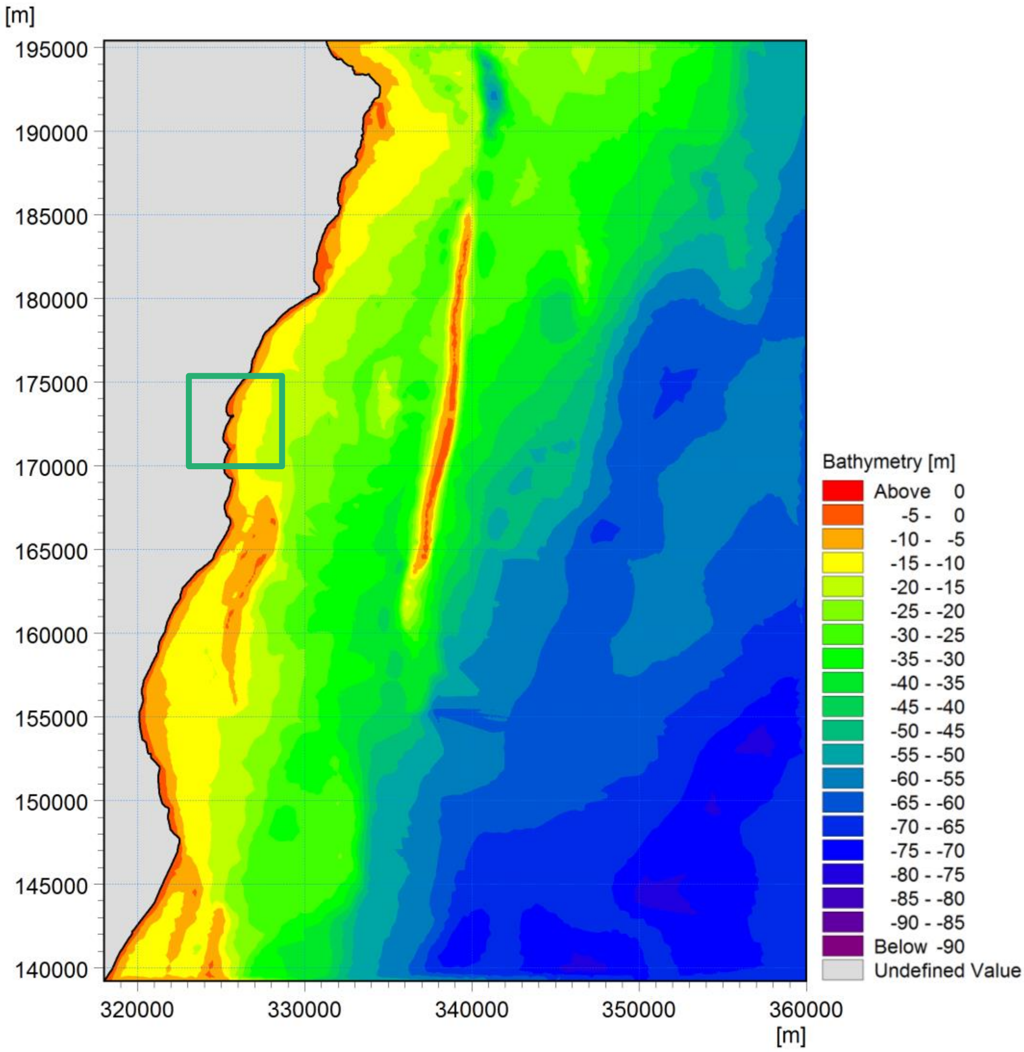


4.2 Model

4.2.1 Model bathymetry

The admiralty chart was digitized and combined with the topographic and bathymetric surveys in order to extract the bathymetric data in xyz format relevant to Chart Datum for use with the MIKE21-SW model, see Figure 16. The model bathymetry varies from approximately -70m CD to 0m CD and takes into account bathymetric features such as the Arklow Bank.

Figure 16: Bathymetric model derived from the Admiralty Charts and bathymetric survey. The box shows approximately the revetment location.



4.2.2 Computational Mesh

The MIKE21-SW model uses a flexible mesh to calculate wave parameters within the computational domain. This mesh can be manipulated to be finer in area of interest and coarser in less significant areas. Three different areas have been defined within the model for mesh generation. Each area has different mesh sizing with a finer mesh for the area of interest and coarser grid elsewhere. Figure 16 shows the bathymetry used in the model whereas the size of the mesh in the various model areas is shown in Figure 18, and Figure 19.

Table 3 gives the mesh sizing adopted for the different areas.

The initial conditions of wave height, Hs, peak wave period, Tp, and direction of the offshore waves are provided within the model boundaries.

Table 3: Mesh size for various areas in the computational domain.

Section	Mesh Size
Offshore (course density)	80,000 m^2
Intermediate	10,000 m^2
Nearshore (high density)	1000 m^2

Figure 17: Bathymetric computational mesh showing coarse and intermediate densities. Blue square shows the boundaries of Figure 18.

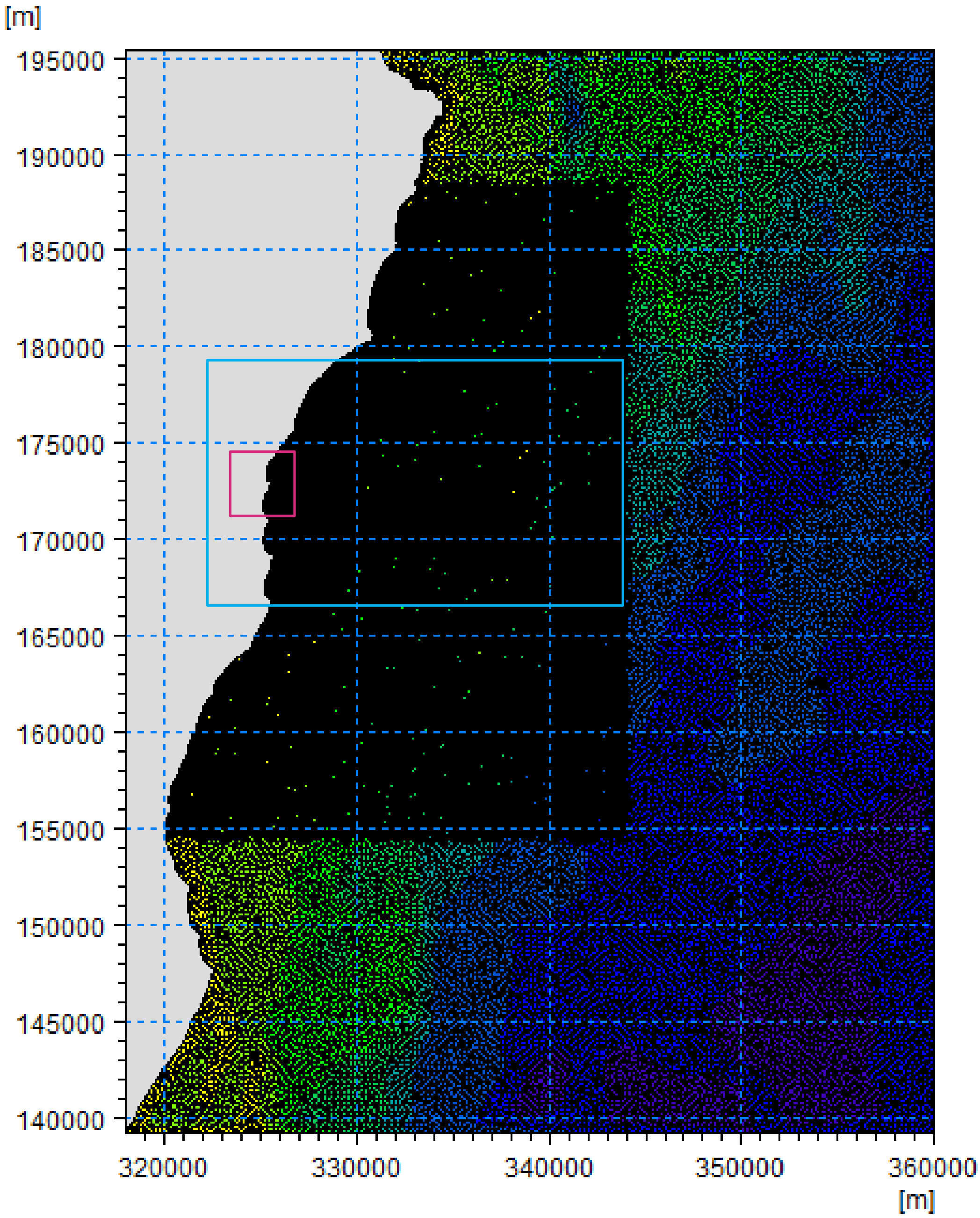


Figure 18: MIKE21-SW mesh (intermediate and high density). Blue square shows the boundaries of Figure 19.

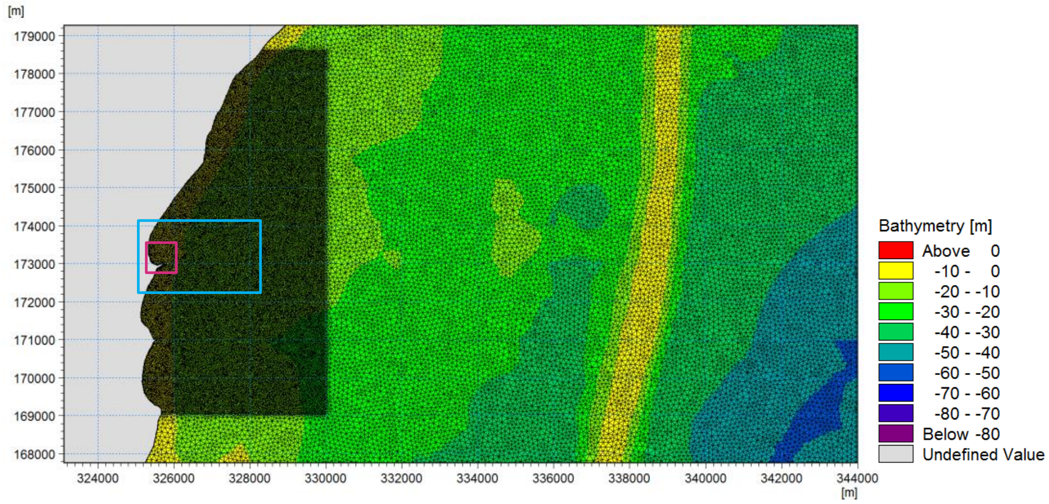
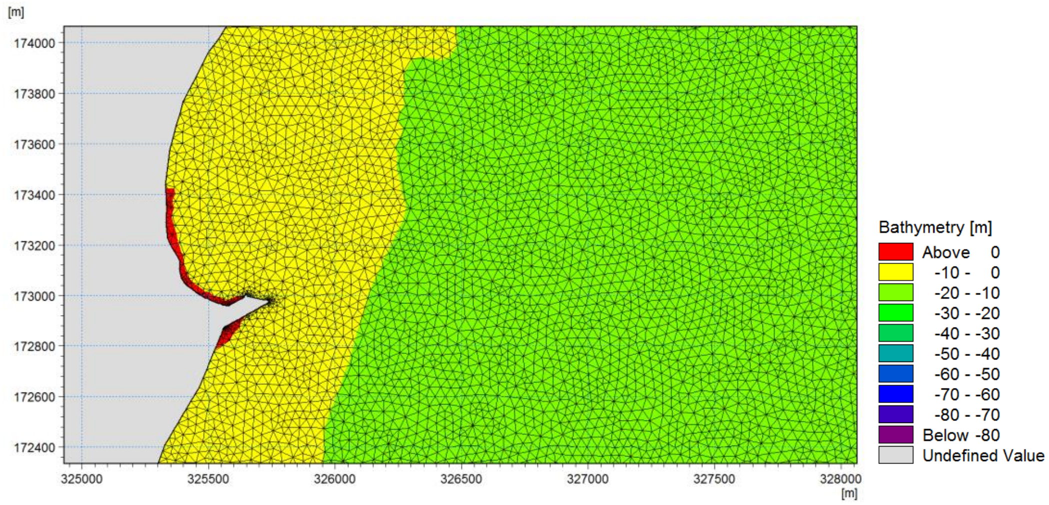


Figure 19: MIKE21-SW (high density) mesh at revetment location.

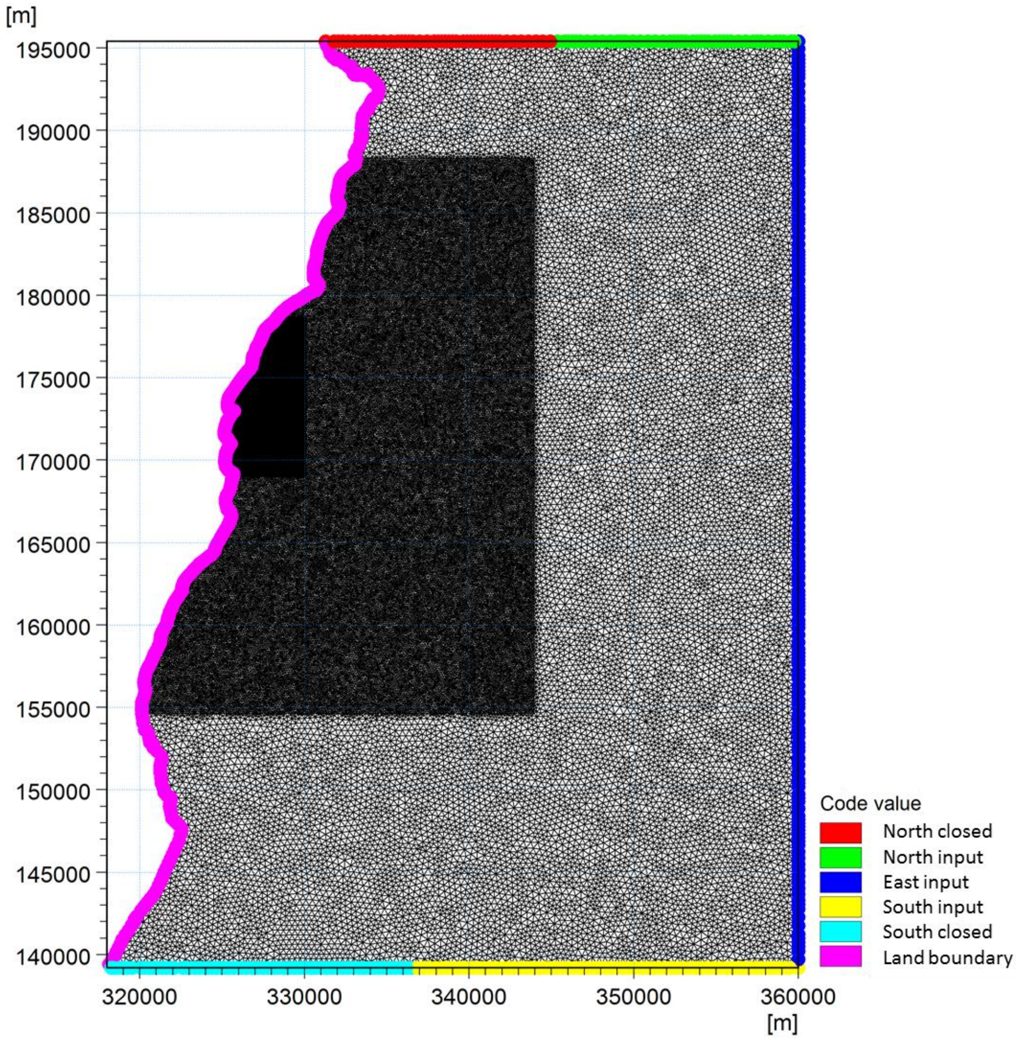


4.2.3 Model boundaries

The offshore waves were transformed over the domain, assuming conditions at the boundary to be the same as the NORA10 point (deep water conditions). In addition to this, the wind at NORA10 point was assumed to be acting along the entire computational domain (constant in time and space).

For this study, the input boundaries were extended along the northern and southern boundaries in order to generate a more accurate representation of the NE and SE wave directions. The boundaries are shown in Figure 20.

Figure 20: Boundaries used in the MIKE21-SW model. The green, blue and yellow boundaries are input boundaries using the NORA10 deep water conditions.



4.3 Results

The model was run in a fully coupled mode for wind and waves, hence it took into account both the waves entering the model domain through the offshore boundary as well as the waves generated within the model domain due to wind action. Figure 21 and Figure 22 show the wave height distribution in the offshore and revetment site location respectively for the ESE wave and wind direction.

Table 4 shows the model results for the various combinations of wind/ wave data and direction for the 500 year return period events for all the directions assessed. The model results were derived at distances of 20m and 40m from the point of contact of the revetment with the water level used for the model. The results shown are the maximum values for the points located 20m offshore of the existing revetment.

Table 4: MIKE21-SW modelling results. Displayed are the return period (T_r), water level (h), wave height (H_s), wave period (T_p) and Wind speed (W_{sp}).

			Offshore		**20m from existing revetment**		
Direction	**T_r [y]**	**h [m CD]**	**H_s [m]**	**T_p [s]**	**W_{sp} [m/s]**	**H_s [m]**	**T_p [s]**
NE	500	3.68	5.5	8.6	24.3	2.9	9.5
ENE	500	3.68	5.0	8.3	20.7	2.8	9.5
E	500	3.68	5.6	8.7	24.4	3.0	9.9
ESE	500	3.68	6.2	9.1	25.0	3.0	10.5
SE	500	3.68	7.1	9.7	23.0	3.0	11.4

From Table 4, it can be seen that storms from all directions give similar nearshore wave heights despite having significantly higher input offshore conditions from the south easterly directions. This is partly due to the presence of the Arklow Bank located parallel to the coast on which waves likely break and lose energy (visible in Figure 21 at x=339000), and partly due to the shallow waters adjacent to the site. The latter phenomenon can also be described as 'depth limited wave conditions' in which the limited water depth limits the possible wave height. As the area affected by diffraction is not governing for the determination of the design wave height of the new rock revetment, the results presented do not include diffraction.

Refer to Appendix B for graphical output from all the model runs. For storms coming from SE and E, the significant wave height at the site is in the region of 3.0m for the 500 year return period. The proposed design condition is the ESE wave and wind direction. As this direction has the highest significant wave height for the largest area of the revetment.

On the basis of the hydraulic modelling results, the proposed design conditions for the revetment design phase are:

- H_s=3.0 meters
- T_p=10.5 seconds

Figure 21: Offshore wave height distribution with waves from the ESE.

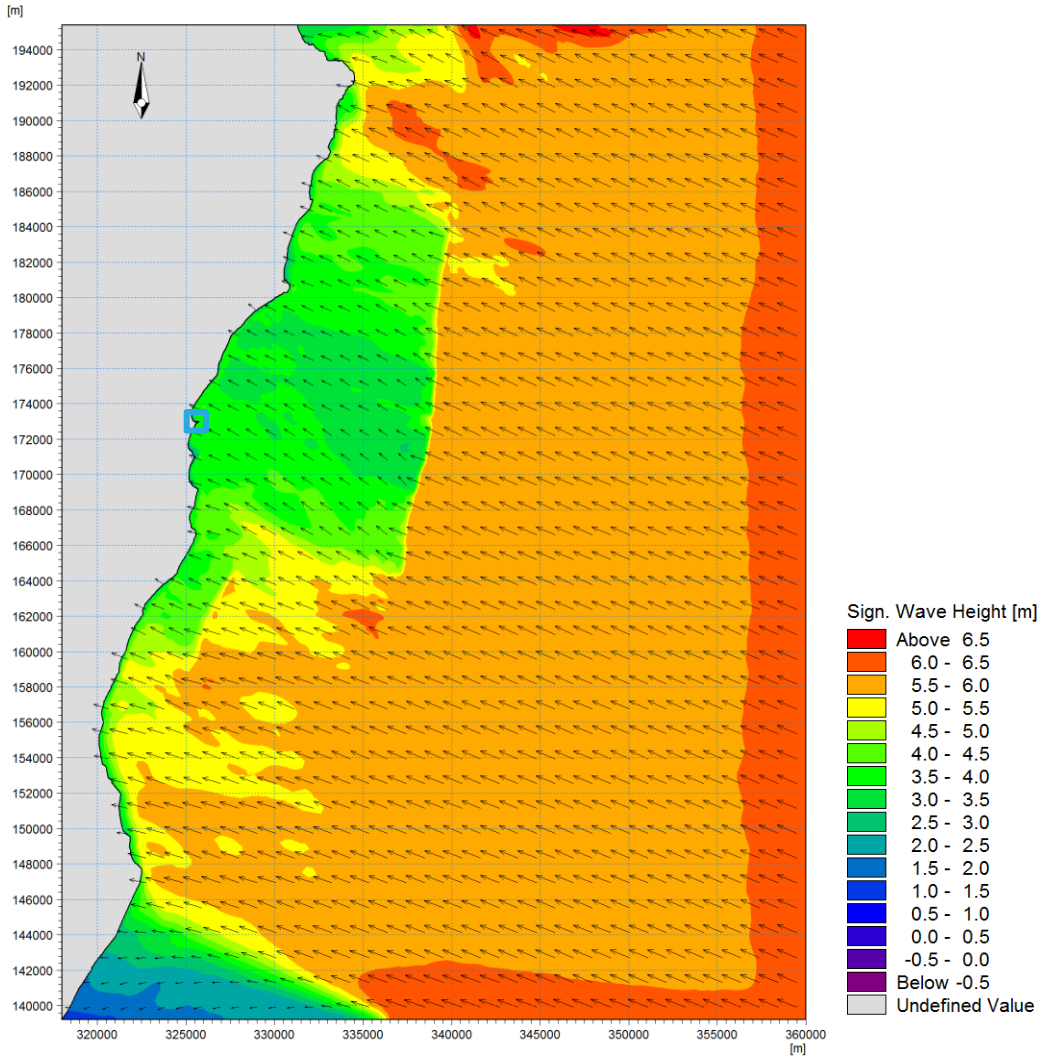
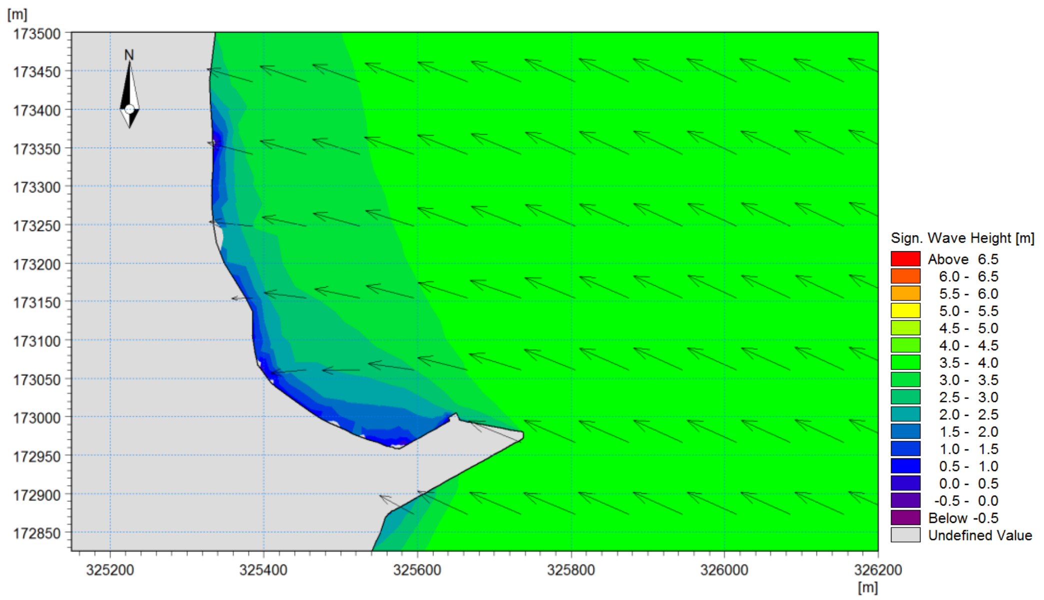


Figure 22: Nearshore wave height distribution with waves from the ESE.



5 Summary and conclusions

A wave modelling study was carried out in order to estimate nearshore wave heights adjacent to the proposed site for the Arklow waste water treatment plant. The wave modelling results are used to determine the design wave height for the design of the upgraded revetment adjacent to the proposed site. The model used for this study was the spectral transformation model MIKE21-SW.

The results of the wave modelling study indicate that storms approaching from the east to south east cause the largest wave conditions at the proposed site. Variations in the maximum wave heights between the different directions are small due to the presence of the Arklow Bank and the local bathymetry that is limiting wave heights adjacent to the site. The wave modelling results are represented in graphical format for all test cases in Appendix B.

The design wave conditions to be considered for the design of the revetment are significant wave height of 3.0 meters with a peak wave period of 10.5 seconds.

References

[1] NORA10 (2011) Norwegian ReAnalysis 10 km

[2] Reistad M., Breivik O., Haakenstad H., Aarnes O.J. and Furevik B.R., 2011. A high-resolution hindcast of wind and waves for the North Sea, the Norwegian Sea and the Barents Sea. J. Geophys. Res., 116, C05019, doi:10.1029/2010JC006402

[3] Admiralty Tide Tables, 2010, Volume 1, United Kingdom and Ireland

[4] Irish Coastal Protection Strategy Study Phase 2 - South East Coast. Work Package 9A -Appendix 2 - Flood Mapping for High End Future Scenario. IBE0104/November 2013, OPW

[5] UK Hydrographic Office Admiralty Chart number 1787: Carnsore Point to Wicklow Bay, scale 1:100,000, depths in metres reduced to Chart Datum

[6] UK Hydrographic Office Admiralty Chart number 633: Arklow, scale 1:10,000, depths in metres reduced to Chart Datum

[7] Holthuijsen L.H. 2007. Waves in Oceanic and Coastal Waters, p16-19, ISBN-13: 978-0521129954

Appendix A

Extreme value analysis results

A1 Waves

Figure 23: Extreme value analysis for the significant wave height (H_s) and N.

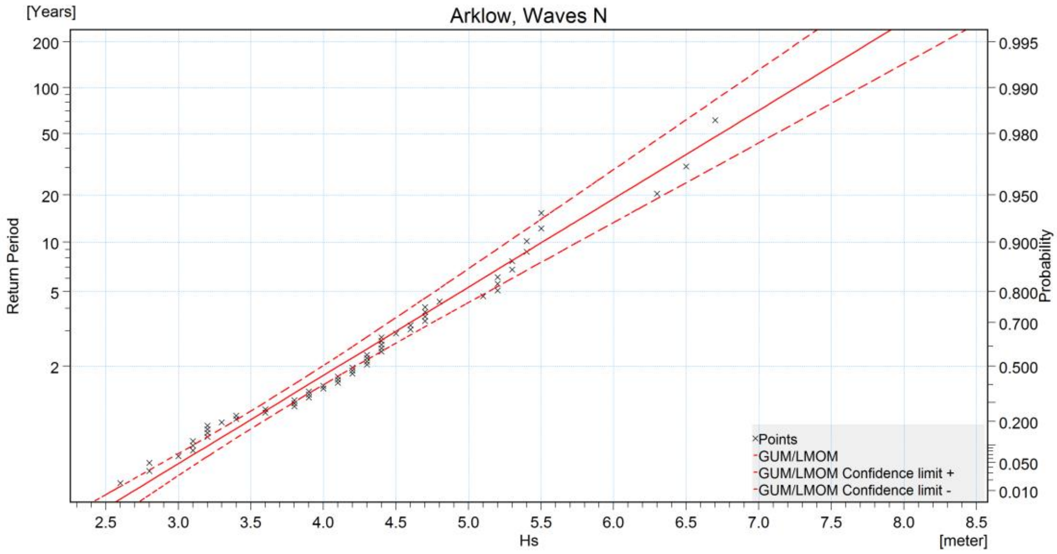


Figure 24: Extreme value analysis for the significant wave height (H_s) and NNE.

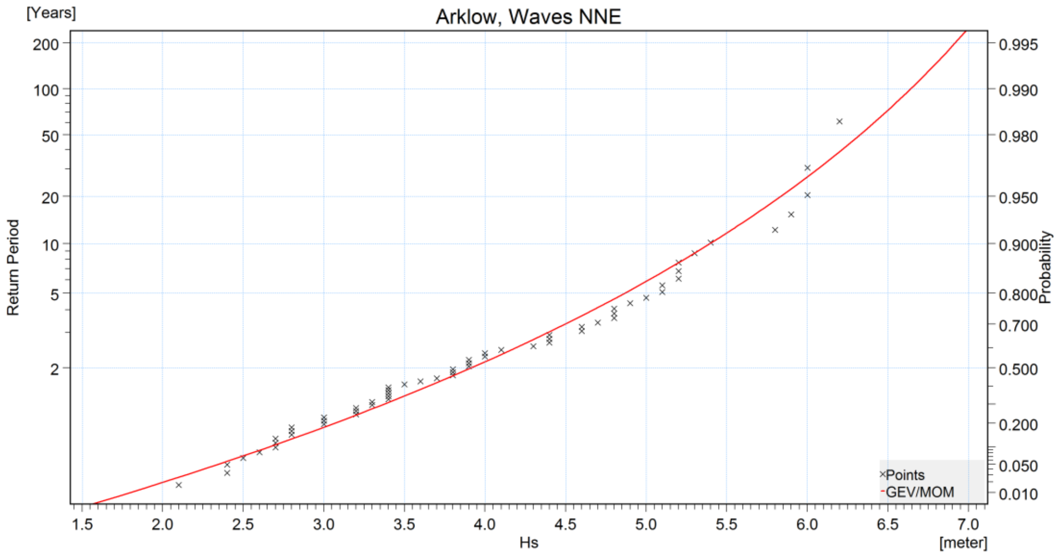


Figure 25: Extreme value analysis for the significant wave height (H_s) and NE.

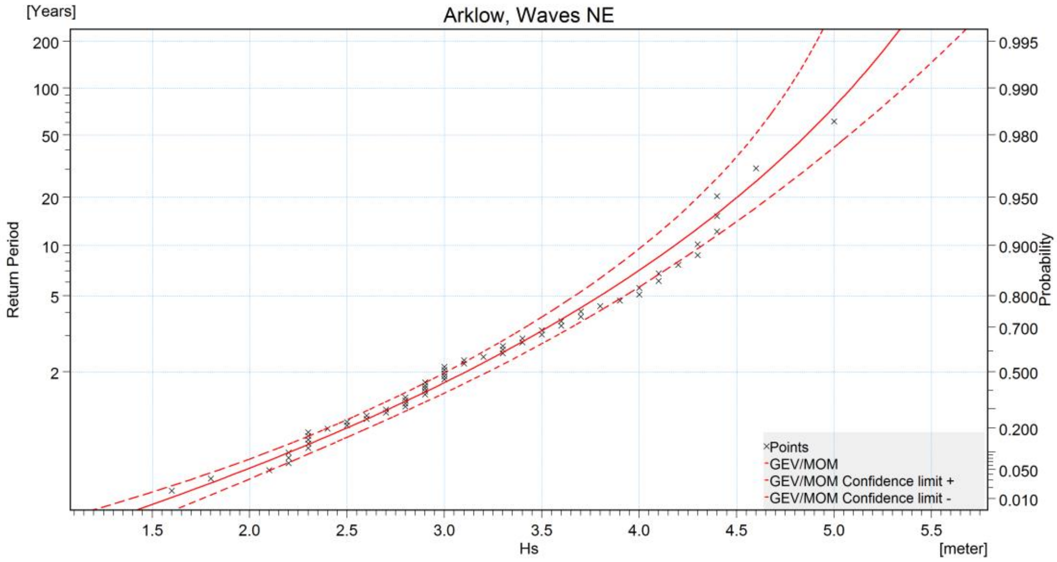


Figure 26: Extreme value analysis for the significant wave height (H_s) and ENE.

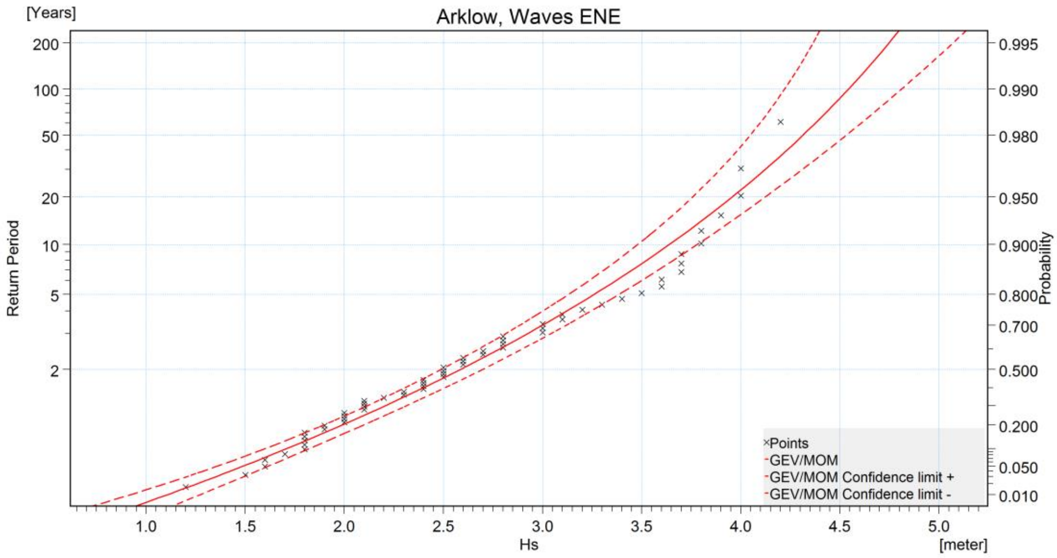


Figure 27: Extreme value analysis for the significant wave height (H_s) and E.

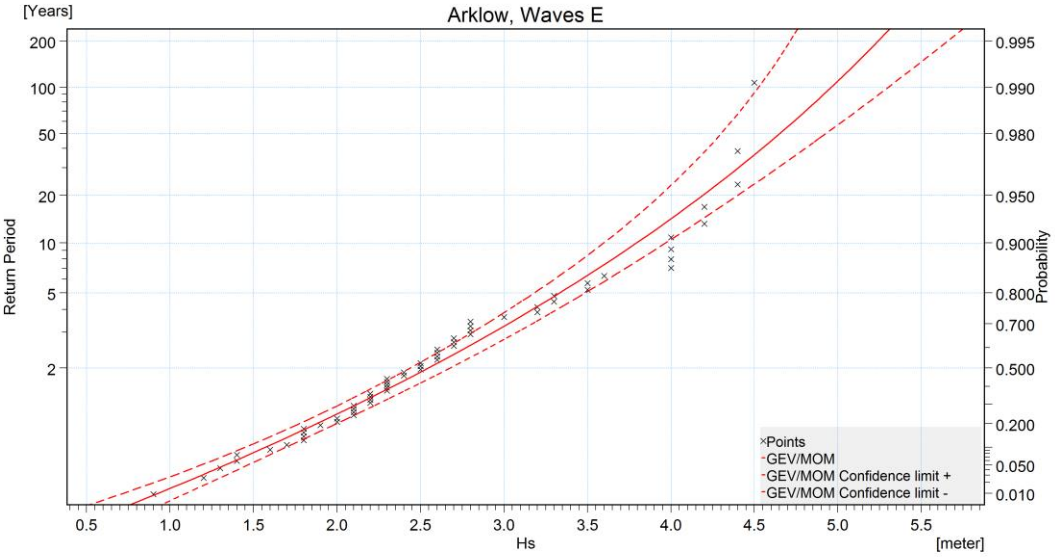


Figure 28: Extreme value analysis for the significant wave height (H_s) and ESE.

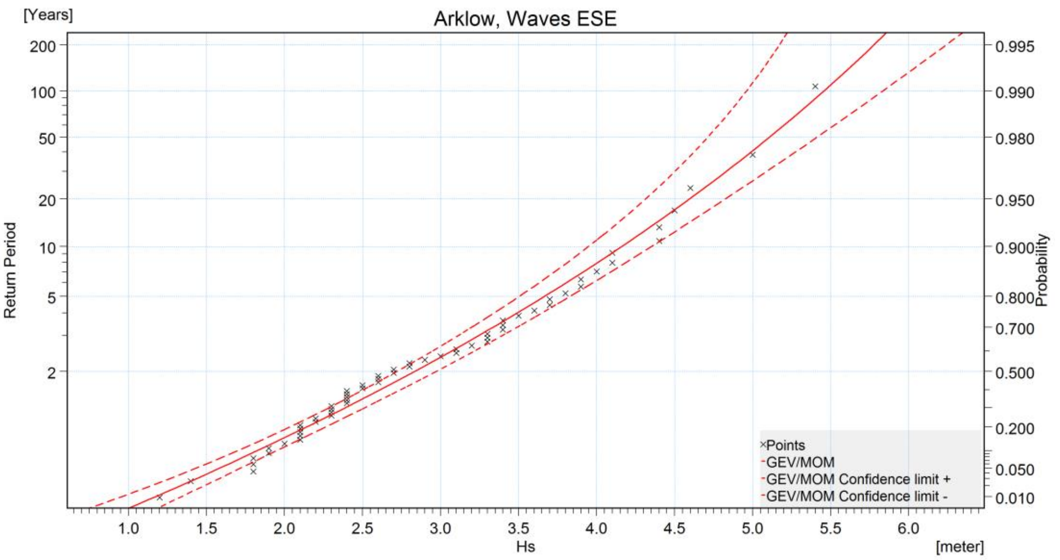


Figure 29: Extreme value analysis for the significant wave height (H_s) and SE.

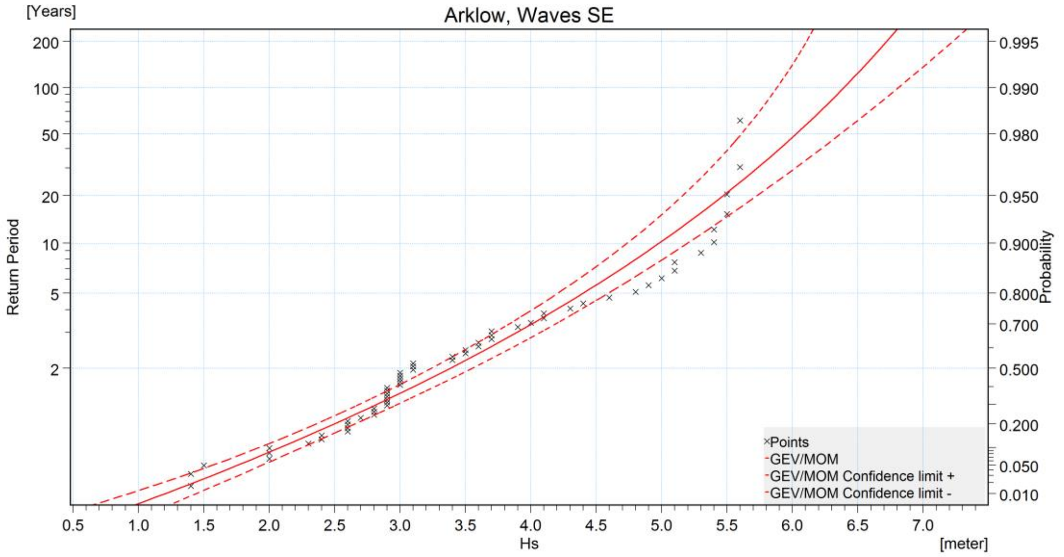


Figure 30: Extreme value analysis for the significant wave height (H_s) and SSE.

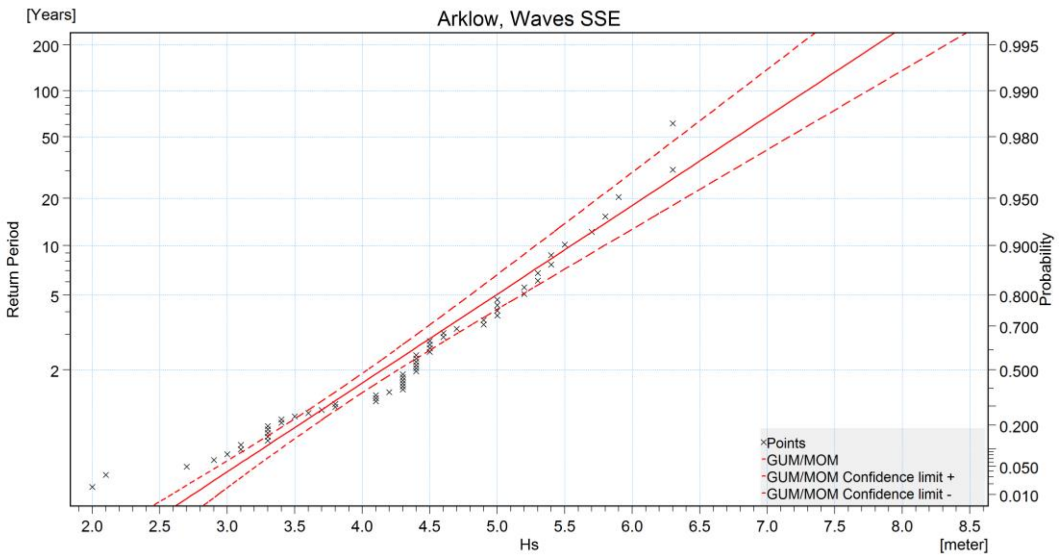
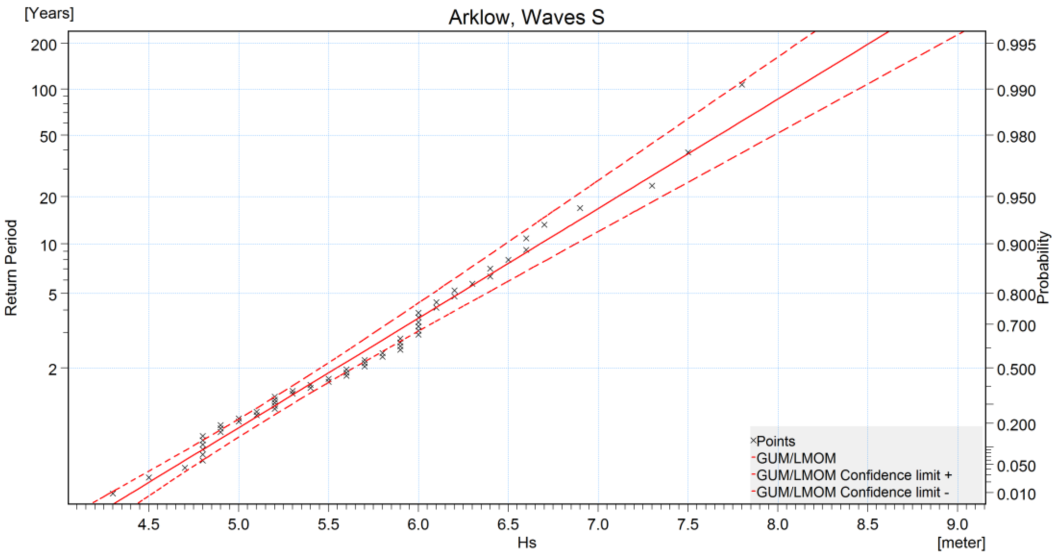


Figure 31: Extreme value analysis for the significant wave height (H_s) and S.



A2 Wind

In addition to the distributions used in the wave height extreme value analysis, the Weibull distribution is also considered.

Figure 32: Extreme value analysis for the wind speed (W_{sp}) and N.

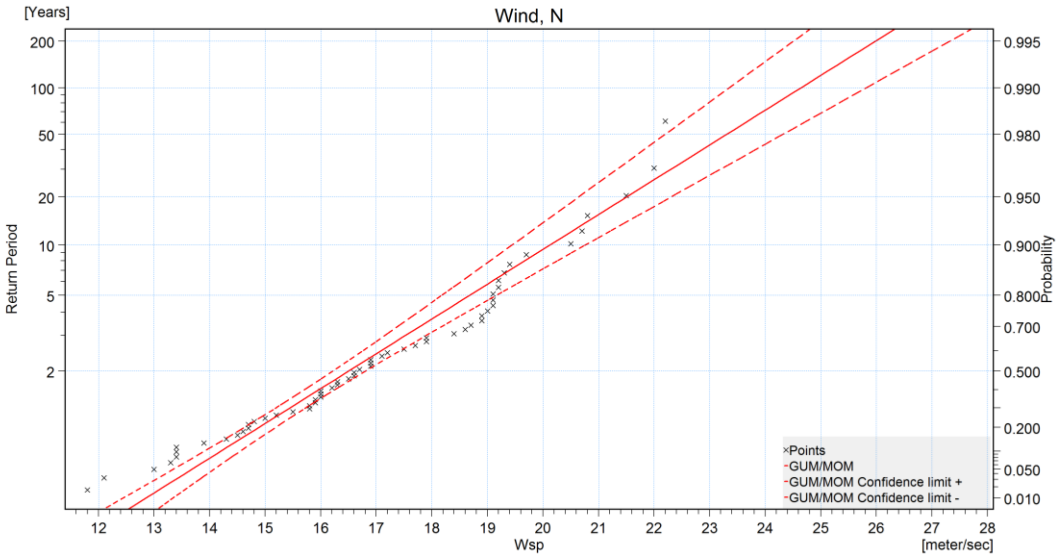


Figure 33: Extreme value analysis for the wind speed (W_{sp}) and NNE.

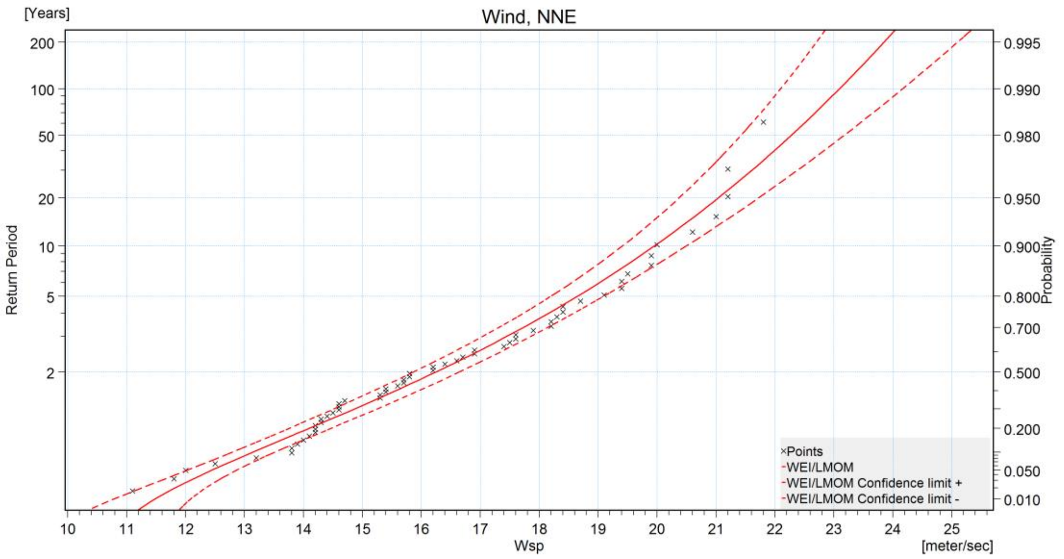


Figure 34: Extreme value analysis for the wind speed (W_{sp}) and NE.

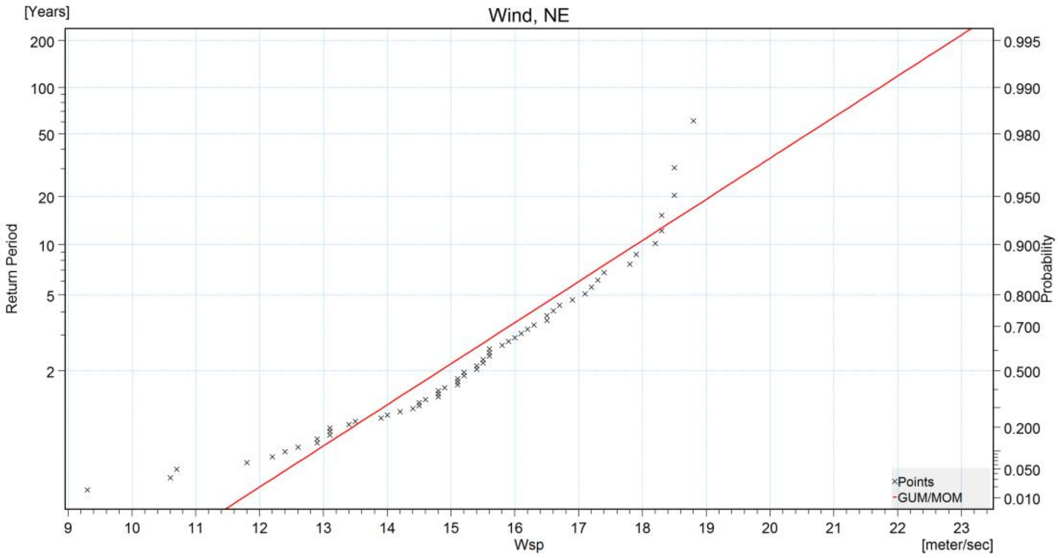


Figure 35: Extreme value analysis for the wind speed (W_{sp}) and ENE.

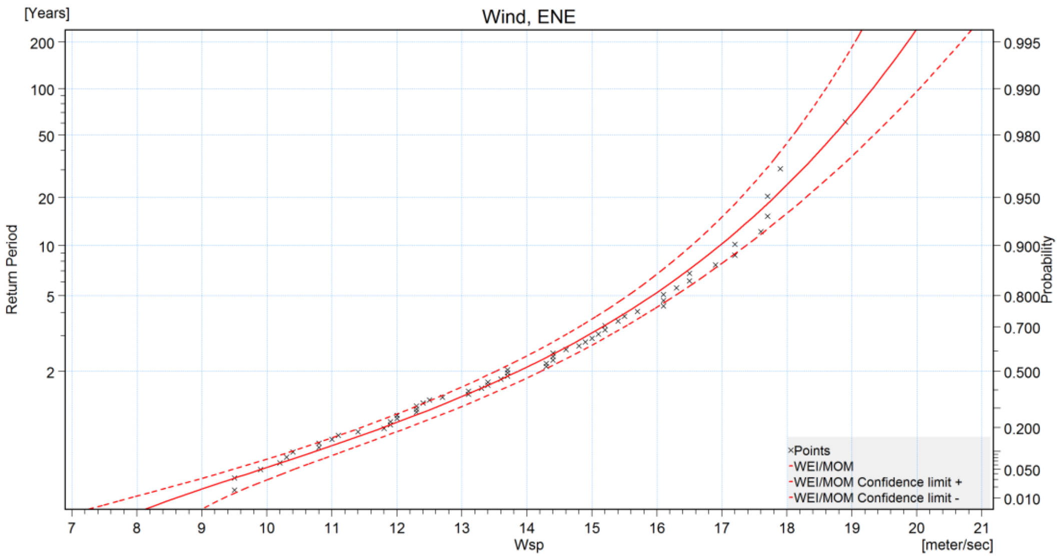


Figure 36: Extreme value analysis for the wind speed (W_{sp}) and E.

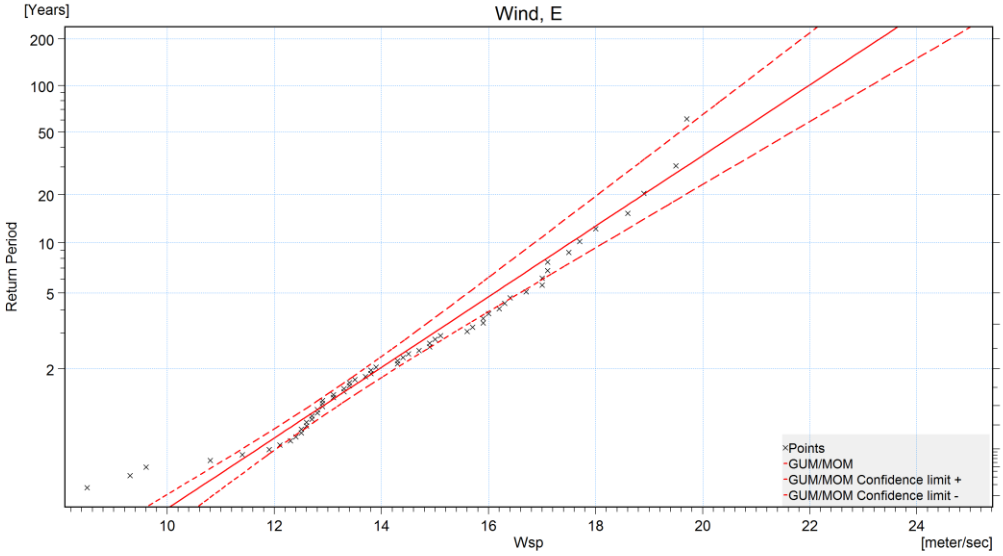


Figure 37: Extreme value analysis for the wind speed (W_{sp}) and ESE.

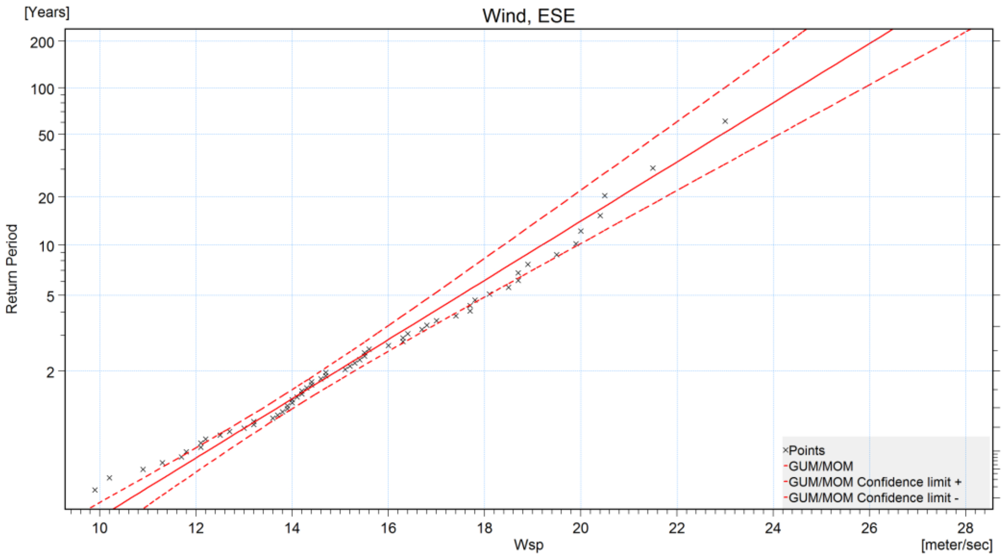


Figure 38: Extreme value analysis for the wind speed (W_{sp}) and SE.

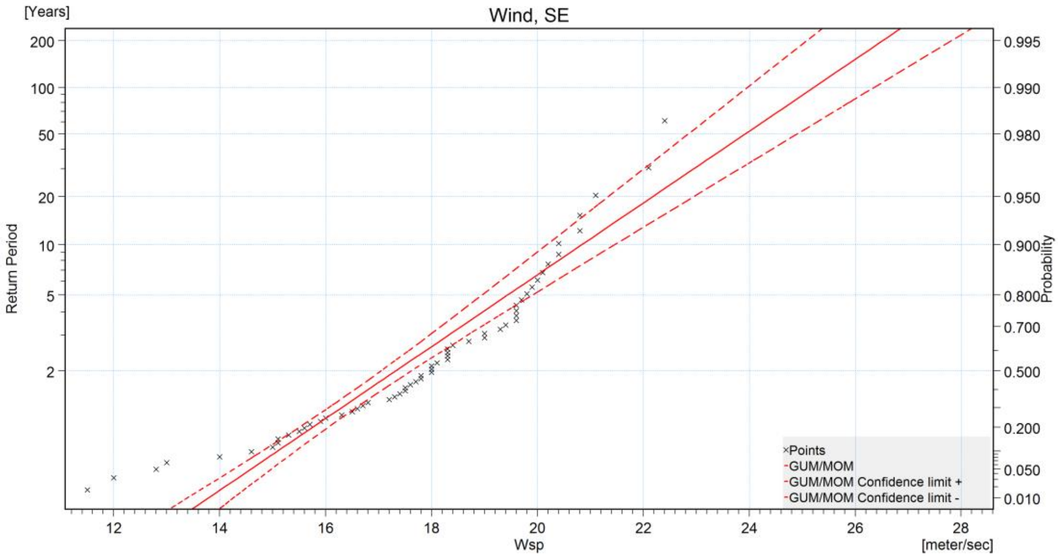


Figure 39: Extreme value analysis for the wind speed (W_{sp}) and SSE.

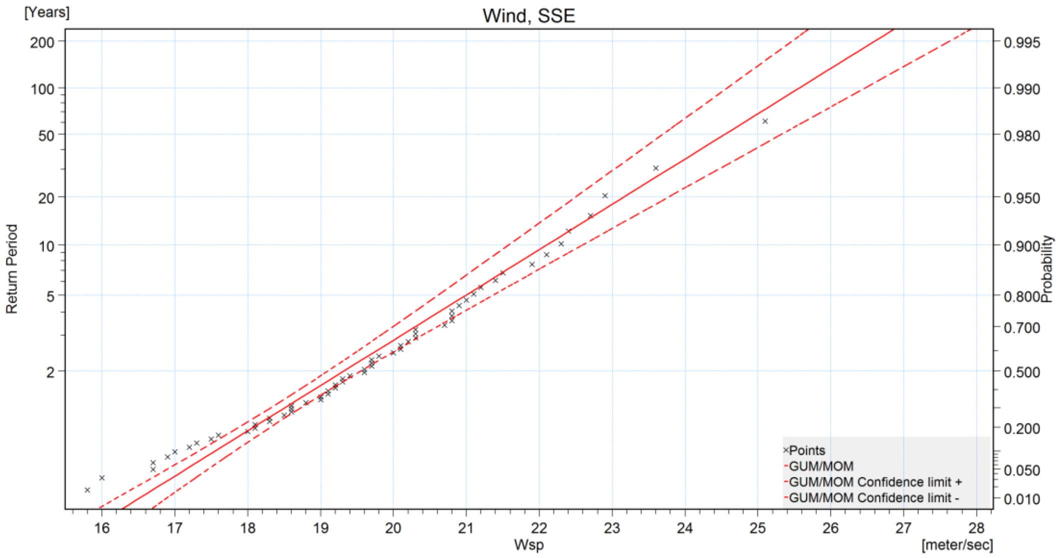
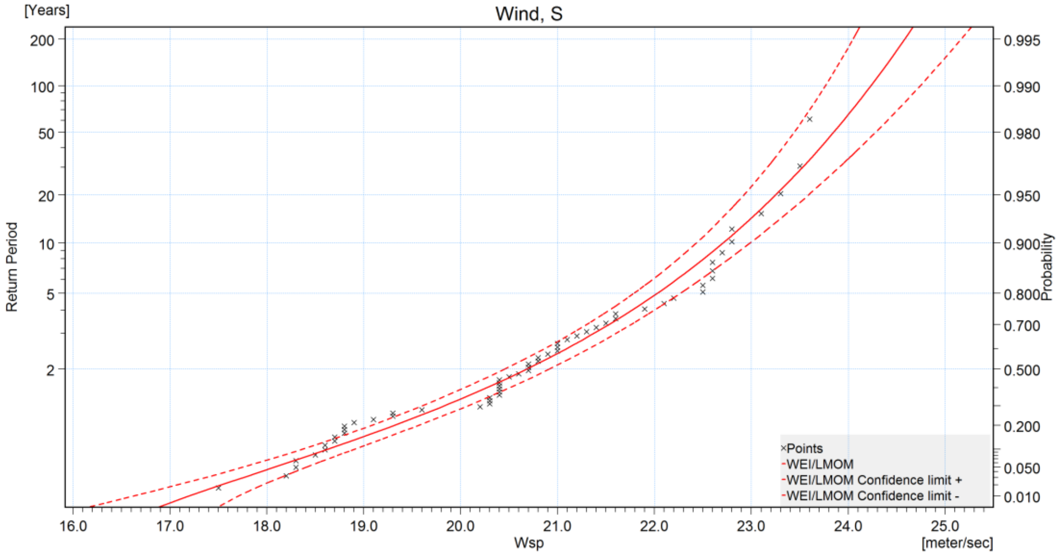


Figure 40: Extreme value analysis for the wind speed (W_{sp}) and S.



Appendix B

Wave height distribution results

B1 Wave height distribution results

Figure 41: Offshore wave height distribution with waves from the NE. Total computational domain.

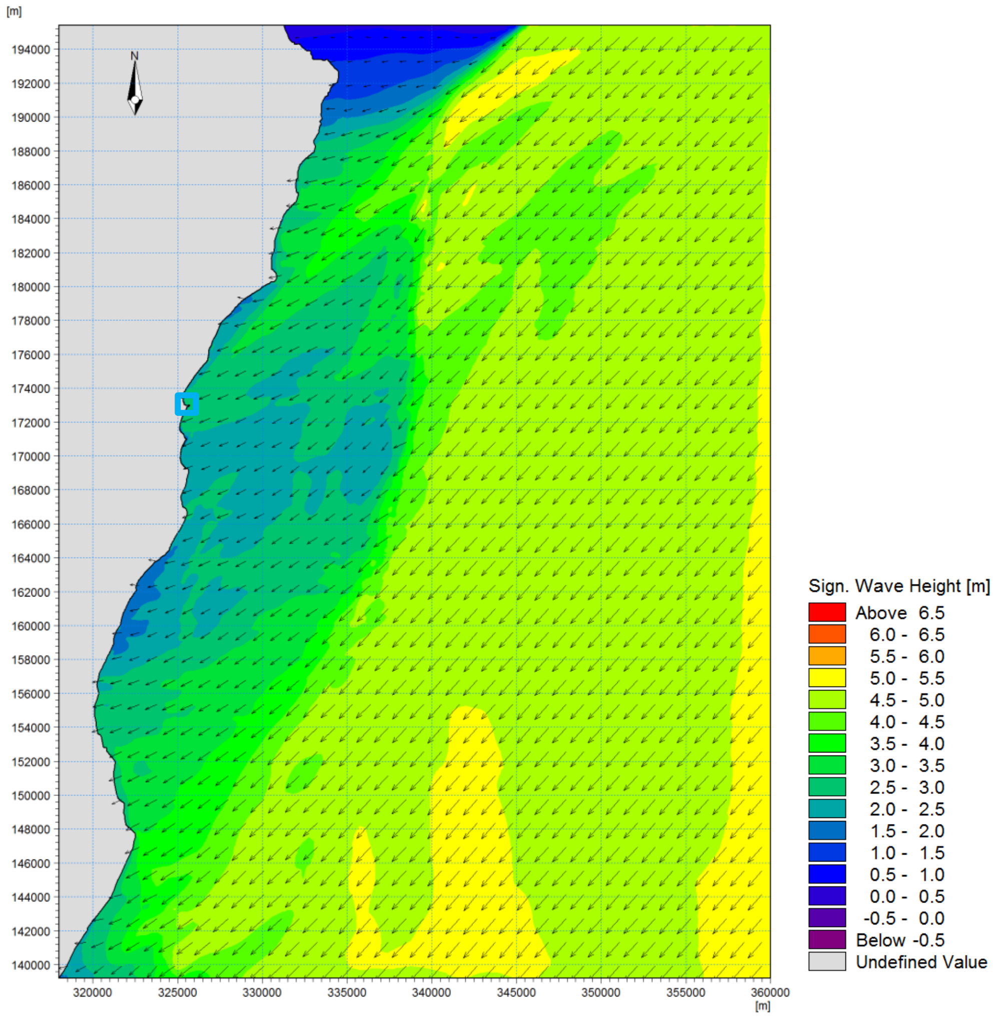


Figure 42: Wave height distribution with waves from the NE. Zoomed in on Arklow revetment site location.

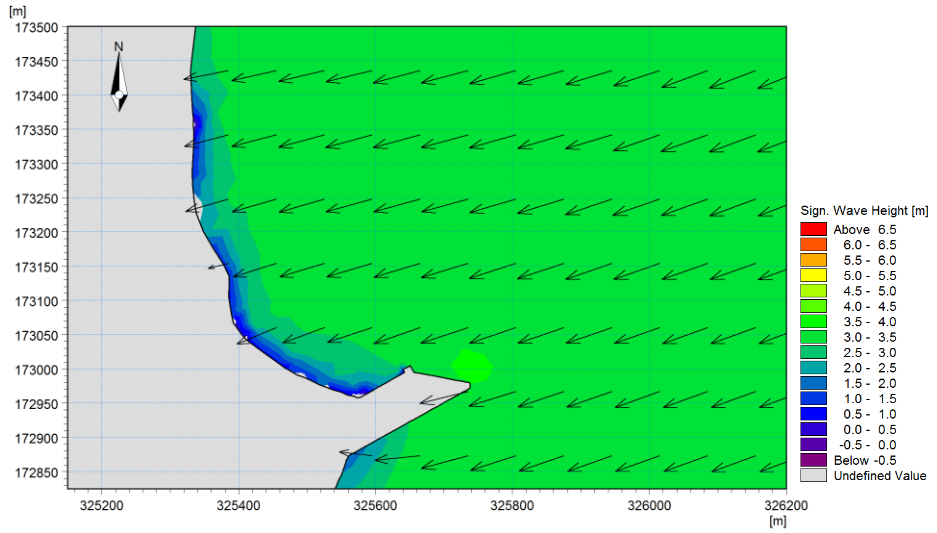


Figure 43: Offshore wave height distribution with waves from the ENE. Total computational domain.

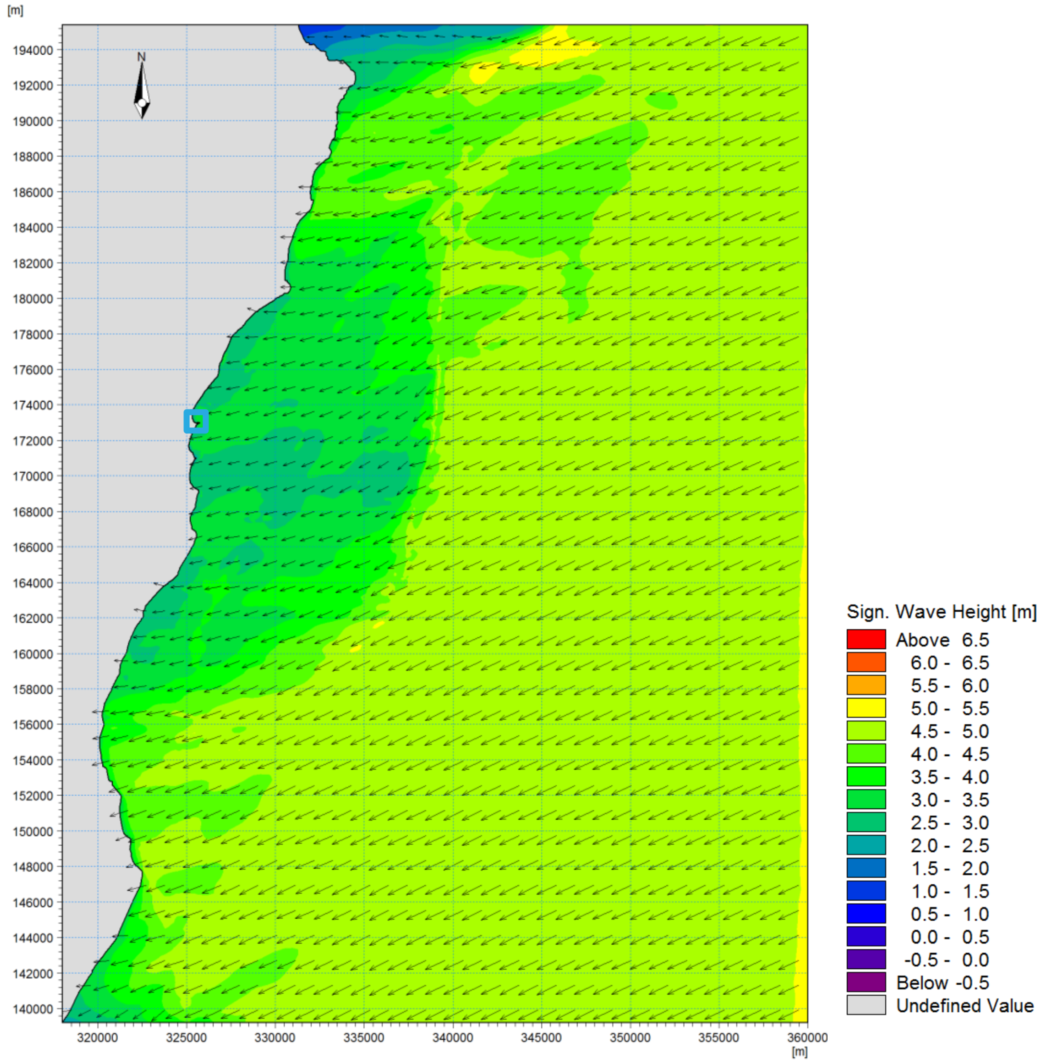


Figure 44: Wave height distribution with waves from the ENE. Zoomed in on Arklow revetment site location

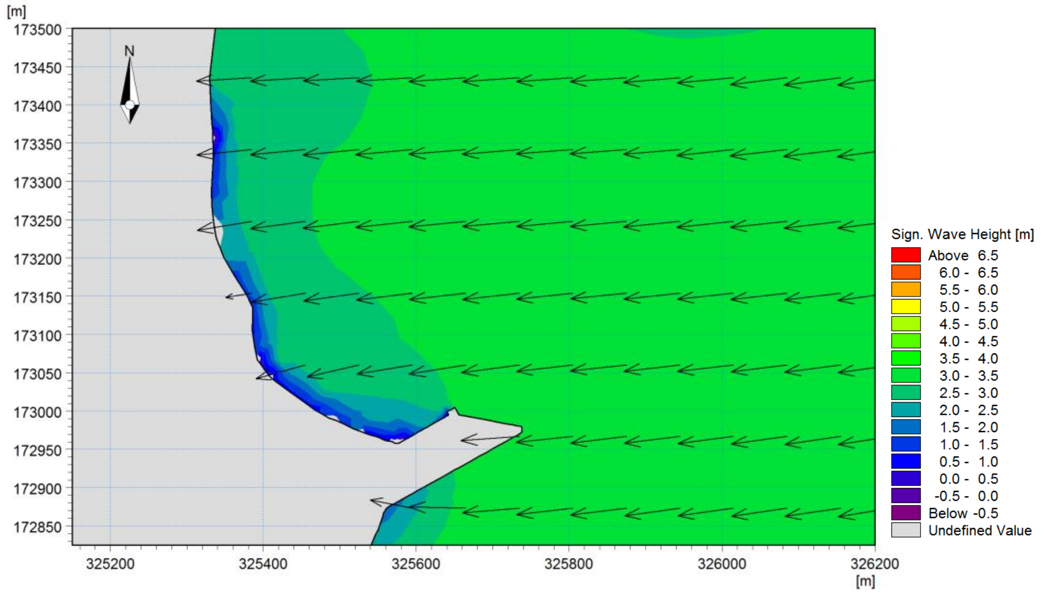


Figure 45: Offshore wave height distribution with waves from the E. Total computational domain.

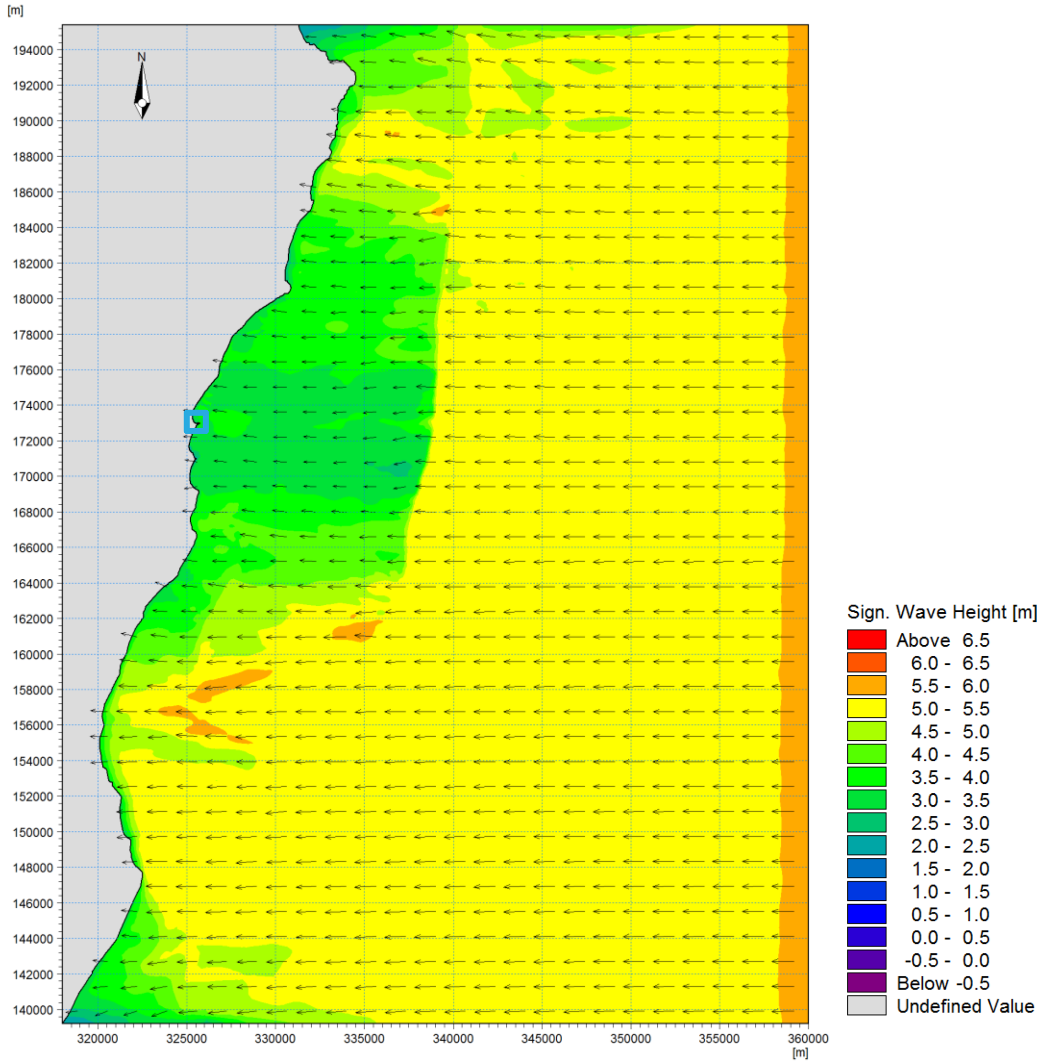


Figure 46: Wave height distribution with waves from the E. Zoomed in on Arklow revetment site location.

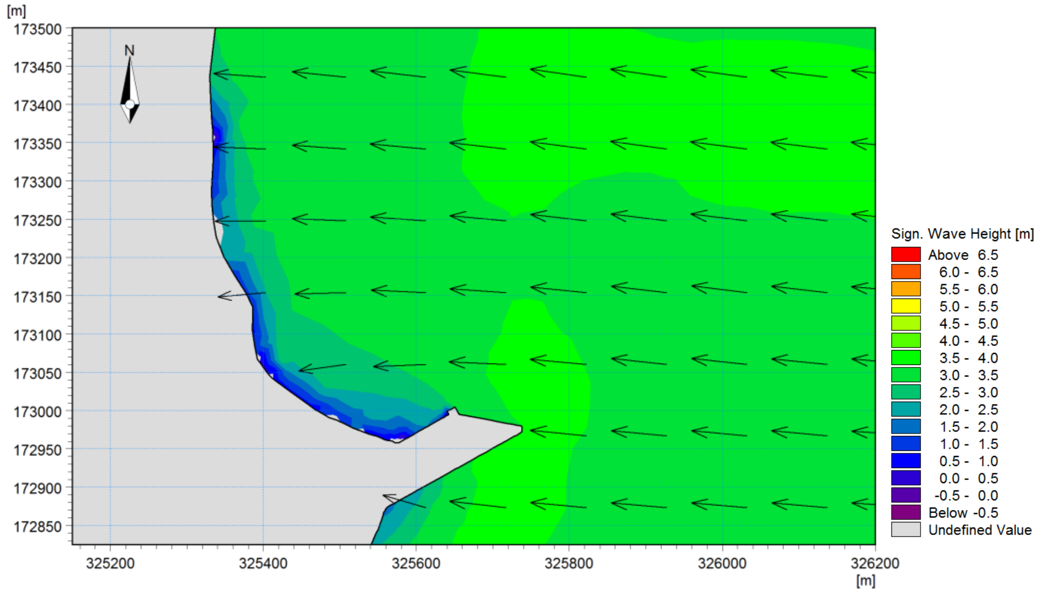


Figure 47: Offshore wave height distribution with waves from the ESE. Total computational domain.

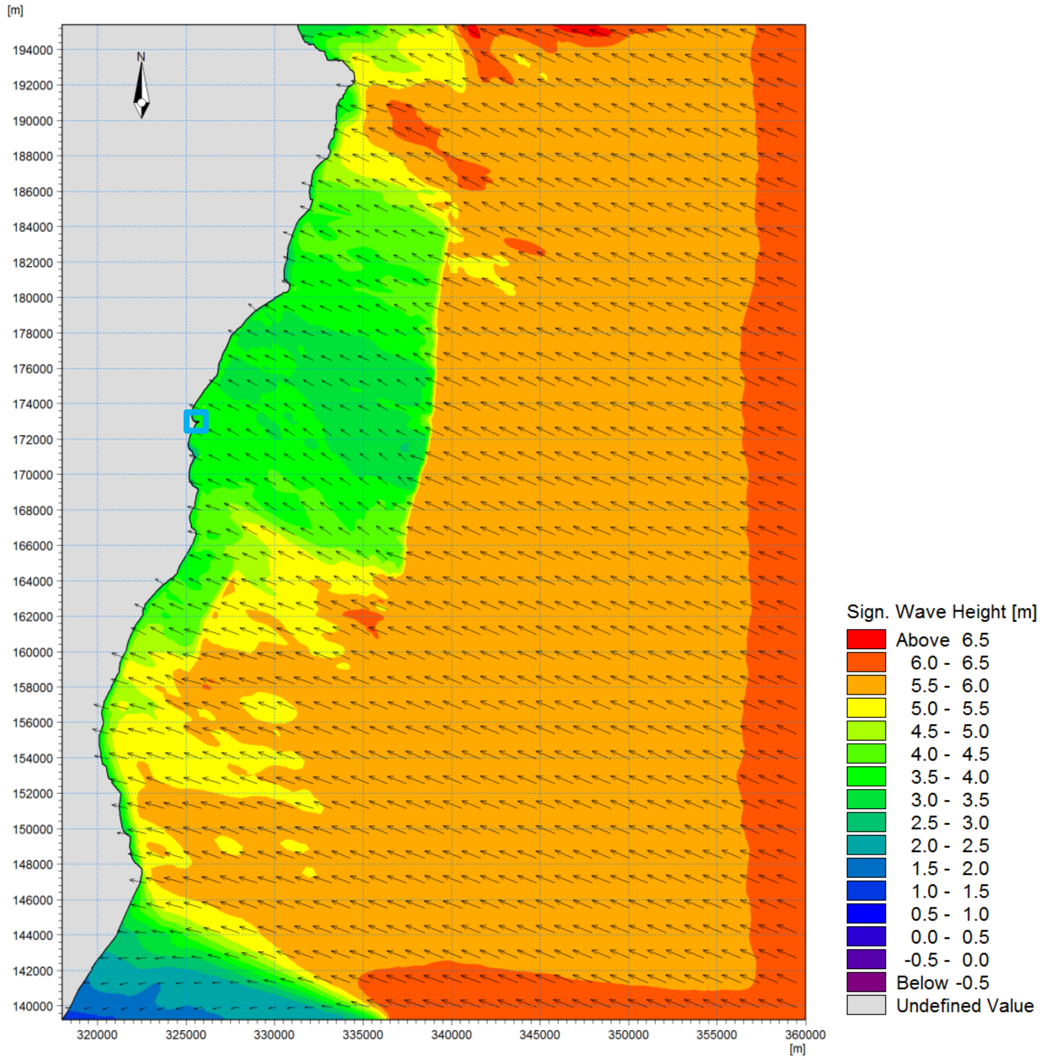


Figure 48: Wave height distribution with waves from the ESE. Zoomed in on Arklow revetment site location.

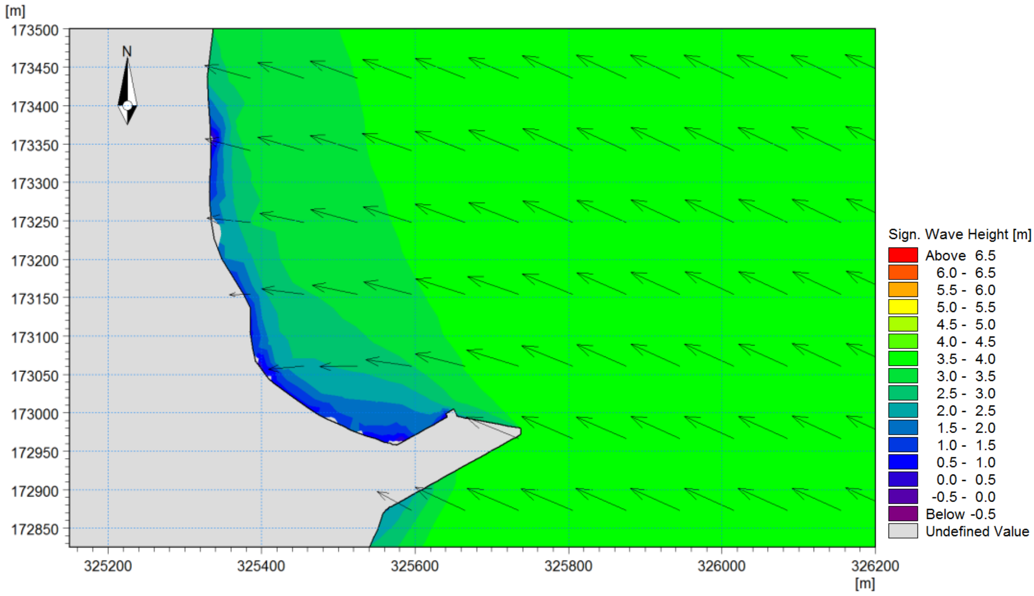


Figure 49: Offshore wave height distribution with waves from the SE. Total computational domain.

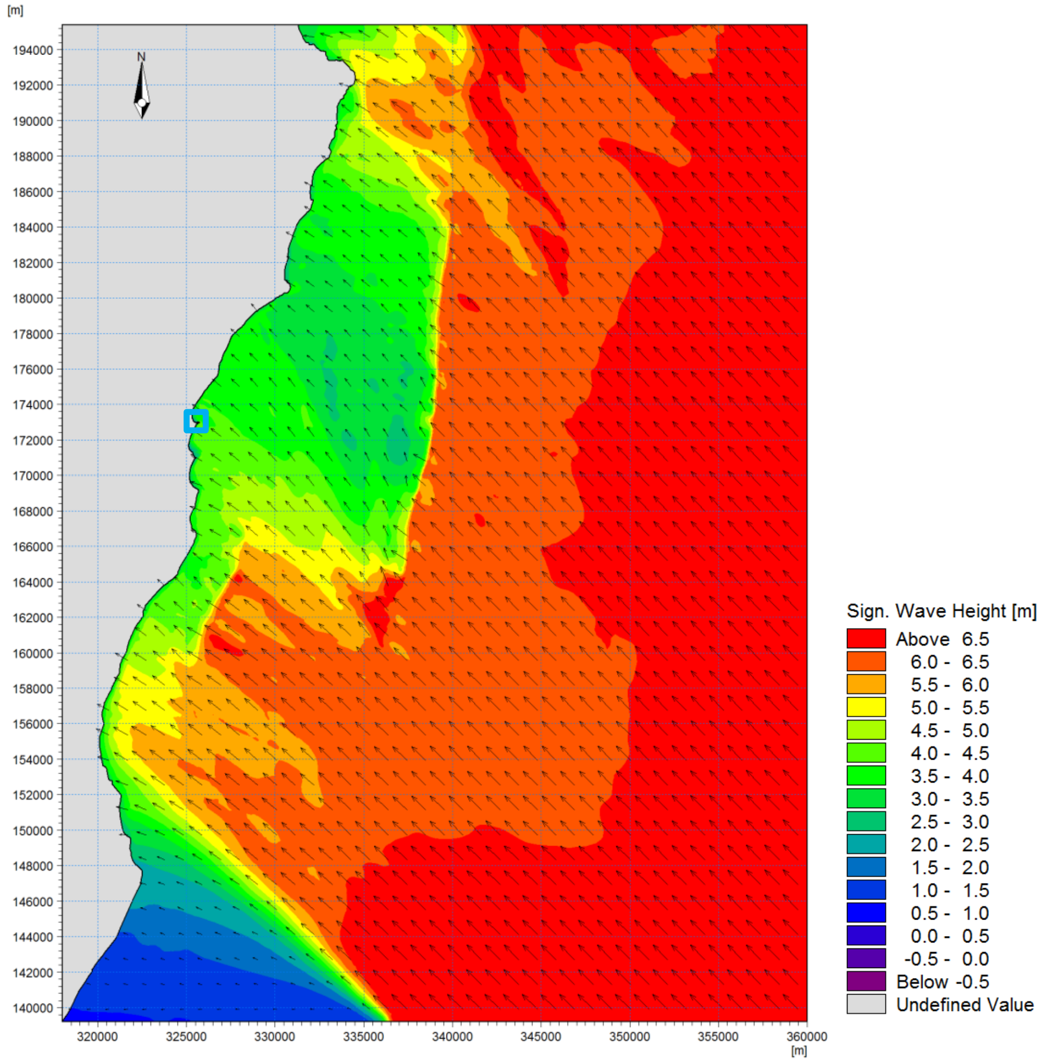


Figure 50: Wave height distribution with waves from the SE. Zoomed in on Arklow revetment site location.