Facility Centre for Water Based Activities at Killiney Beach,
Flood Risk Assessment

Technical Report

June 24

2023s0468

Dún Laoghaire Rathdown County Council (DLRCC)

JBA Project Manager

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Revision History

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S3-P01 25/04/2023	Initial issue	Sarah Clifford
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S3-C02 30/11/203	Revised issue following site plan revision	Sarah Clifford
A3-C01 13/12/2023	Final Issue	Sarah Clifford
A3-C02 02/02/2024	Final Issue including wave overtopping results	Sarah Clifford
A3-C03 28/05/2024	Final issue for planning with design layout change included	Sarah Clifford
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Contract

This report describes work commissioned by Dún Laoghaire Rathdown County Council (DLRCC), by an appointment letter dated 10/03/2023. DLRCC's representative for the contract was Sarah Clifford. Hannah Chisnall and Ross Bryant of JBA Consulting carried out this work.

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Purpose

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Contents

List of Figures

Abbreviations

AEP................................ Annual Exceedance Probability
CFRAM .... Catchment Flood Risk Assessment and Management
DoEHLG.......................... Department of the Environment, Heritage and Local Government
FB .... Freeboard
FFL................................. Finish Floor Levels
FRA................................ Flood Risk Assessment
GSI................................. Geological Survey of Ireland
OPW .... Office of Public Works
PFRA .... Preliminary Flood Risk Assessment
SFRA .... Strategic Flood Risk Assessment

1 Introduction

Under the Planning System and Flood Risk Management Guidelines for Planning Authorities (DoEHLG & OPW, 2009) the proposed development must undergo a Flood Risk Assessment to ensure sustainability and effective management of flood risk.

1.1 Terms of Reference and Scope

JBA Consulting was appointed by Dún Laoghaire Rathdown County Council (DLRCC) to prepare a Stage 1 Flood Risk Assessment (FRA) that reviews the flood risk of a site located in Killiney, Co Dublin.

1.2 Flood Risk Assessment; Aims and Objectives

This study is being completed to inform the future development of the site as it relates to flood risk. It aims to identify, quantify, and communicate to Planning Authority officials and other stakeholders the risk of flooding to land, property and people and the measures that would be recommended to manage the risk.

The objectives of this FRA are to:

- Identify potential sources of flood risk;
- Confirm the level of flood risk and identify key hydraulic features;
- Assess the impact that development has on flood risk;
- Develop appropriate flood risk mitigation and management measures which will allow for the long-term development of the site.

Recommendations for development have been provided in the context of the OPW / DoECLG planning guidance, "The Planning System and Flood Risk Management". A review of the likely effects of climate change, and the long-term impacts this may have on any development has also been undertaken.

For general information on flooding, the definition of flood risk, flood zones and other terms see 'Understanding Flood Risk' in Appendix A.

1.3 Proposed Development

Refer to Figure 1-1 for proposed site layout. The proposed development is a set of sports facilities including changing rooms located along Killiney beach.

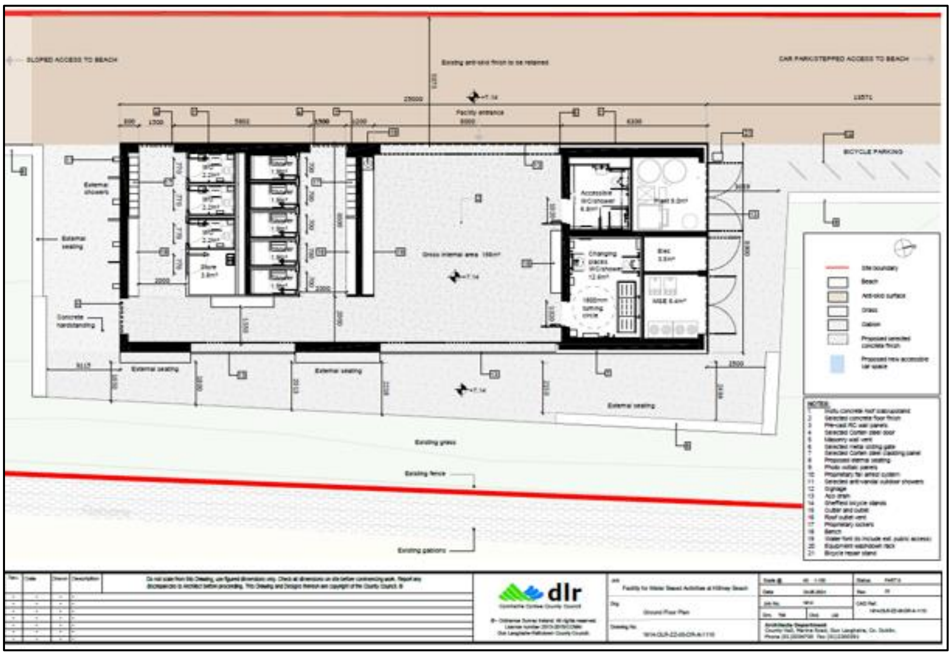


Figure 1-1: Proposed site layout (provided by client)

2 Site Background

2.1 Site Location and Watercourses

Refer to Figure 2-1 for the site location. The site is bounded by Killiney Beach to the north, south and east and the DART line to the west. The main water source proximal to the site is the Irish Sea. The site is located approximately 30m away from the High-Water Mark. The nearest fluvial watercourse, the Deansgrange River, is located 1.25km south of the site location.

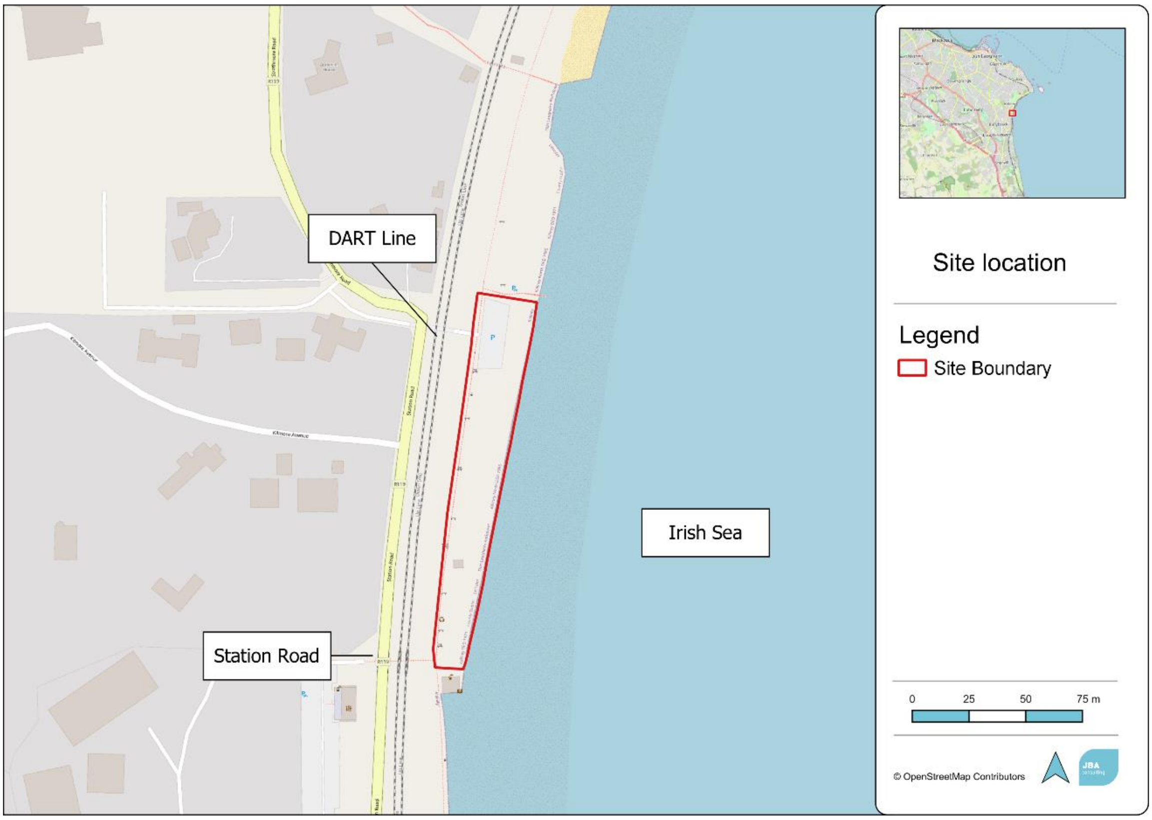


Figure 2-1: Site location and watercourses

2.2 Site Topography

The topography of the site and surrounding area is shown in Figure 2-2. The land generally slopes from west to east towards the Irish Sea. The pre-development elevations at the site range from approximately 6.06 - 6.98mOD.

Figure 2-2: Site topography

2.3 Site Geology

The Geological Survey of Ireland (GSI) groundwater and geological maps of the site were reviewed. The underlying bedrock at the site location is the Maulin Formation which consists of dark blue-grey slate and schist. Figure 2-3 shows the quaternary sediments at the site and surrounding area which consist of various Tills and beach sands.

The associated groundwater vulnerability, which is a measure of the likelihood of groundwater contamination and is an indicator of groundwater interaction is classified as 'Extreme'. The 'Extreme' classification indicates bedrock is exposed at surface which is expected due to the presence of cliff faces and bedrock along the beach. There are no karst features, also frequently linked to groundwater interaction, at the site or in the surrounding area.

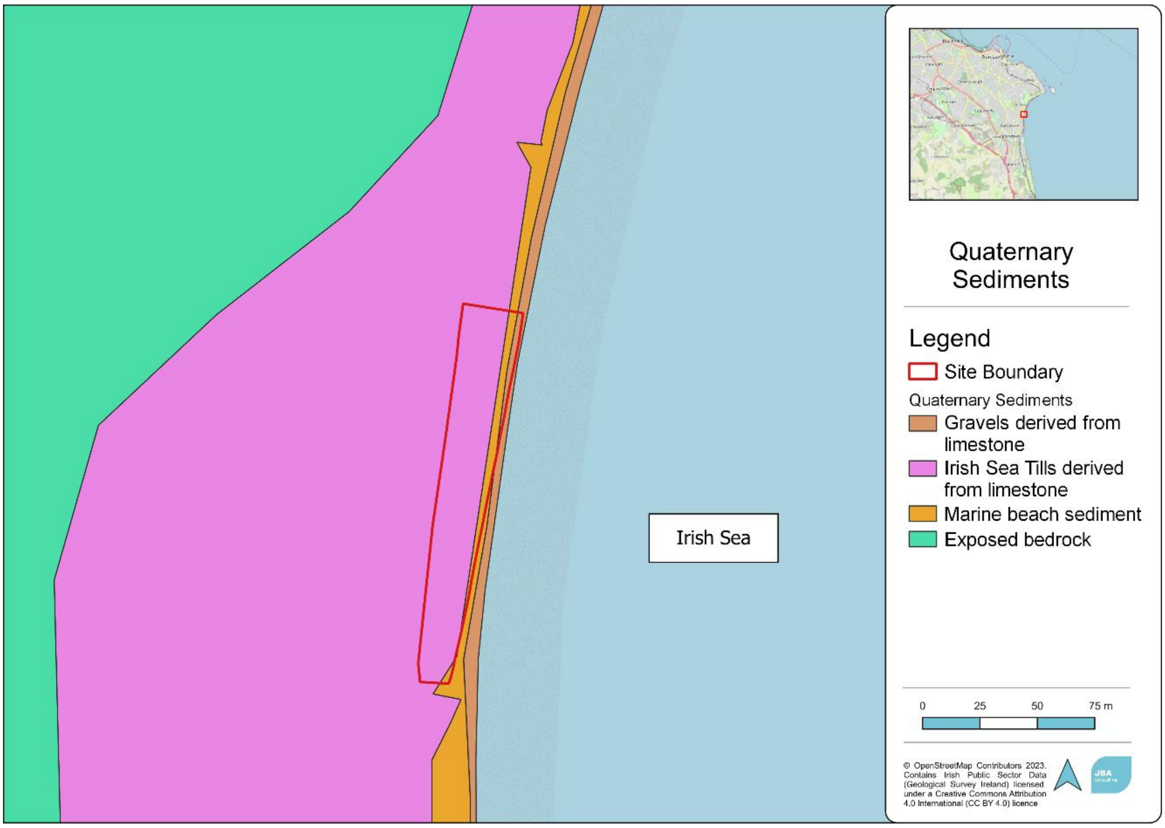


Figure 2-3: Quaternary Sediments

3 Flood Risk Identification

An assessment of the potential for and scale of flood risk at the site is conducted using historical and predictive information. This identifies any sources of potential flood risk to the site and reviews historic flood information. The findings from the flood risk identification stage of the assessment are provided in the following sections.

3.1 Flood History

Several sources of flood information were reviewed to establish any recorded flood history at, or near the site. This includes the OPW's website, www.floodinfo.ie and general internet searches.

3.1.1 Floodmaps.ie

The OPW host a National Flood hazard mapping website, www.floodinfo.ie, which highlights areas at risk of flooding through the collection of recorded data and observed flood events. The following past flood events in the surrounding area are shown in Figure 3-1.

Figure 3-1: Past Flood Event Locations (Source: floodinfo.ie)

Review of the historic flooding record indicates no recorded flood events at the site location. Flood events are recorded to the south of the site location around the Deansgrange and Shanganagh Rivers, but no records are found for the beach location. The lack of flood event records could be because there are limited risk receptors along the beach.

3.1.2 Internet Searches

An internet search was conducted to gather information about whether the site was affected by flooding previously. No additional information on flooding at or around the site was identified.

3.2 Predicative Flooding

The area has been a subject of one predicative flood mapping study and two other related studies and plans:

- Dún Laoghaire Rathdown County Council County Development Plan 2022-2028 Strategic Flood Risk Assessment (SFRA)
- Irish Coastal Protection Strategy Study (ICPSS) (2003)
- Irish Coastal Wave and Water Level Modelling Study (ICWWS) (2018)

3.2.1 Dún Laoghaire Rathdown County Council County Development Plan 2022-2028 Strategic Flood Risk Assessment (SFRA)

Section 10 of the Planning and Development Act 2000 requires that development plans comprise objectives for the zoning of lands for particular purposes, in the interest of proper planning and sustainable development. Effective zoning promotes orderly development by integrating land use and transportation, providing a high quality of life for the county's population, eliminating potential conflicts between incompatible land uses, and establishing an efficient basis for investment in public infrastructure and facilities. In the 2022-2028 County Development Plan the site has no identified land use zoning but is under Specific Local Objective 18 which states:

"To promote the development of the Sutton to Sandycove Promenade and Cycleway as a component part of the National East Coast Trail Cycle Route and also the Dublin Bay trail from the boundary with Dublin City up to the boundary with Co. Wicklow. Any development proposal will protect and enhance public access to the coast where feasible. Any development proposals shall be subject to Appropriate Assessment Screening in accordance with the requirements of the EU Habitats Directive to ensure the protection and preservation of all designated SACs, SPAs, and pNHA(s) in Dublin Bay and the surrounding area."

In addition, it is important that the Development Plan fulfils the requirements of the document "The Planning System and Flood Risk Management Guidelines for Planning Authorities" (OPW/DoEHLG, 2009), which states that flood risk management should be integrated into spatial planning policies at all levels to enhance certainty and clarity in the overall planning process.

As part of the Dún Laoghaire Rathdown County Development Plan 2022-2028, a Strategic Flood Risk Assessment (SFRA) was undertaken to inform the zoning of settlements. In the SFRA coastal flood risk was assessed using water levels from the Irish Coastal Protection Strategy Study (ICPSS) and an assessment of potential wave overtopping. Figure 3-2 shows the SFRA coastal flood mapping. From the Figure the site is located in an area at low flood risk (Flood Zone C) at present day but at risk of wave overtopping. The wave overtopping risk area is based on the peak 0.1% AEP High End Future Scenario (HEFS) extreme scenario (H++FES) level with an additional 2.00m-3.00m buffer.

Figure 3-2: Flood Zone Map Killiney to Loughlinstown (Source: DLRCC CDP 2022-2028 SFRA Appendix B)

3.2.2 Irish Coastal Protection Strategy Study (ICPSS)

The ICPSS was completed in 2013 and examined coastal flood risk around Ireland. Peak water levels for extreme events were derived using numerical modelling and storm surge estimates along with gauge data, these values were then applied to hydraulic models to develop coastal flood risk maps. Figure 3-3 shows the ICPSS flood risk map for the Killiney Beach coastline with the water levels from estimation points 3 and 4 shown in Table 3-1. From the figure the site is located next to an area at risk in the 0.5% AEP event, from Table 3-1 however the current site elevation places it above the peak levels for the extreme events for present day levels.

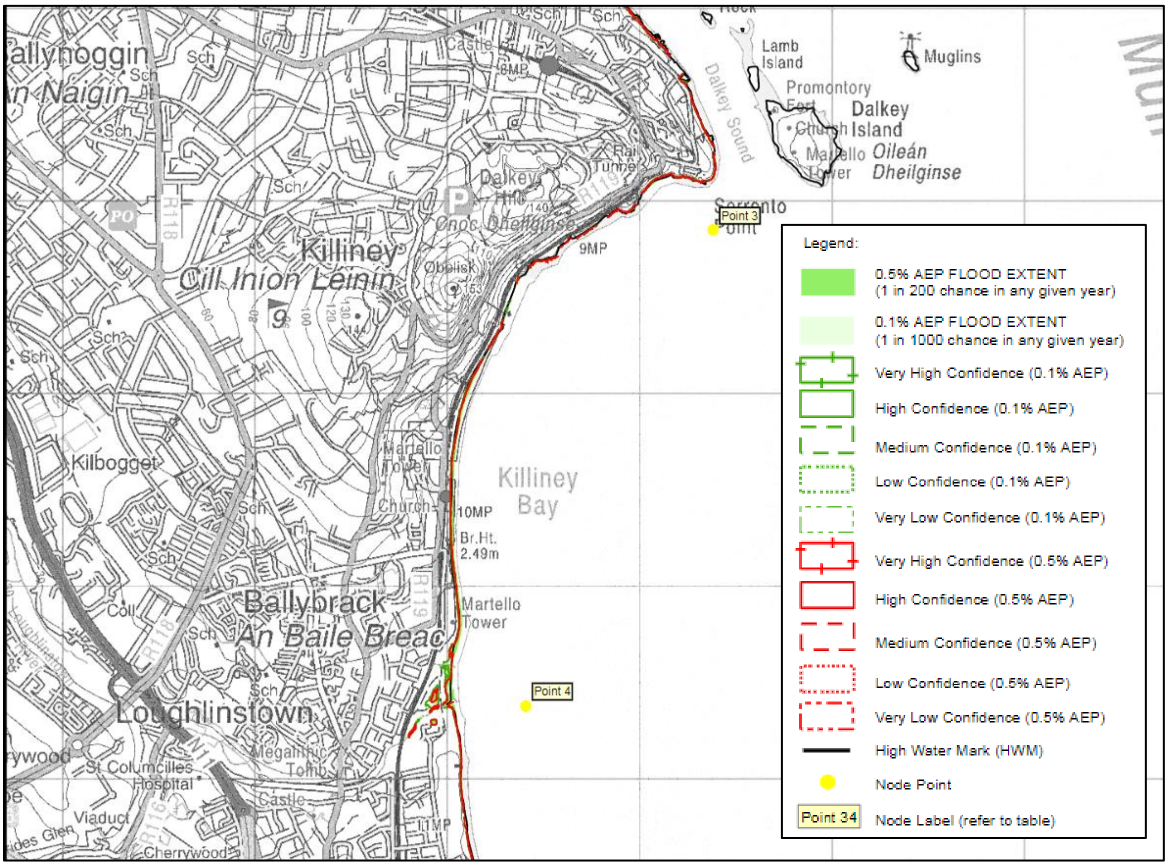


Figure 3-3: Extract of ICPSS Southeast coast flood extent map (Source: ICPSS Phase 2 appendix 7

Table 3-1: ICPSS peak water levels for point 3 and 4

AEP %	Point 3	Point 4
10	2.47mOD	2.45mOD
1	2.80mOD	2.78mOD
0.5	2.90mOD	2.88mOD
0.1	3.14mOD	3.11mOD

3.2.3 Irish Coastal Wave and Water Level Study (ICWWS)

The ICWWS was completed in 2018 and builds upon and supersedes the work carried out in the ICPSS to estimate peak water levels for extreme events. Table 3-2 presents the present day peak levels estimated for Points 3 and 4 (refer to Figure 3-3 for locations). From the table the levels are higher compared to the ICPPS values in Table 3-1 however the site elevations are still above these values.

Table 3-2: ICWWS peak water levels for point 3 and 4

AEP %	Point 3	Point 4
10	2.85mOD	2.81mOD
1	3.17mOD	3.12mOD
0.5	3.26mOD	3.21mOD
0.1	3.48mOD	3.42mOD

3.3 Flood Sources

The initial stage of a Flood Risk Assessment requires the identification and consideration of probable sources of flooding. Following the initial phase of this Flood Risk Assessment, it is possible to summarise the level of potential risk posed by each source of flooding. The flood sources are described below.

3.3.1 Fluvial

The nearest watercourse to the site is the Deansgrange River located to the south. The watercourse does not impact the site therefore it is within Flood Zone C and at low risk of fluvial flooding. Flood risk from fluvial sources is therefore screened out at this stage.

3.3.2 Tidal

The development site is located along Killiney Beach. Review of the SFRA mapping shows that while the site is at low risk of coastal flooding, but it is at risk of wave overtopping which should be accounted for in the design of the site to mitigate impacts.

3.3.3 Pluvial/ Surface Water

Pluvial flooding is the result of rainfall-generated overland flows that arise before run-off can enter a watercourse or sewer. It is particularly sensitive to increases in hard-standing ground/urbanised areas and is usually associated with rainfall events of high intensity. Several sources have been researched such as floodmaps.ie. Based on review of the available information, the site is not at risk of pluvial flooding. Appropriate stormwater design for the site is recommended to mitigate any potential risk of increased run off the proposed site may cause.

3.3.4 Groundwater

Groundwater flooding results from high sub-surface water levels that impact upper levels of the soil strata and overland areas that are usually dry. Ground water vulnerability which is an indicator of both groundwater flood risk and risk of contamination is shown as 'Extreme'. This is due to the presence of exposed bedrock. As there are no records of groundwater interaction and no karst features in the area the overall risk of groundwater flooding is considered low and is screened out at this stage.

4 Flood Risk Management

This section of the report will assess the likelihood of flooding at the site and any additional considerations regarding flood risk.

4.1 Flood Risk

Section 3 of this report confirms that the site is at risk of wave over topping from coastal flooding. Table 4-1 presents the present-day peak water levels for point 3 estimated in the ICWWS and approximate values for wave heights. From the table the site FFL is above the estimated overtopping values however it is recognised that the wave heights quoted are assumed values and may be higher or lower than quoted. Given this identification of risk a formal wave overtopping study was commissioned. The overtopping study is provided in Appendix B of this report but the key points from the study are presented below:

- A model of Killiney beach at the site location was developed using the beach slope and the slope of the gabion cages was developed and run to assess whether overtopping could be a potential issue.
- Based on the modelling there will be some overtopping experienced during a large storm event however the rate and volumes at present day are low and do not pose a risk to vehicles or humans.
- It is noted that the proposed development itself does not increase the risk of overtopping at this location. It is also recommended that suitable materials are used for the outside of the building to protect against debris etc being thrown against it.
- When climate change is considered the overtopping rates and volumes at the site increase to a point where they exceed the safe limits for vehicles and humans therefore in the future additional work will be required to protect the site and its users.
- The severity of wave overtopping at this location is very sensitive to changes in the beach profile (slope) and monitoring of coastal erosion is recommended to ensure the level of protection and safety is maintained.

Table 4-1: Present day peak sea levels with wave overtopping estimates

AEP (%)	Still water level (point 3)	Approximate wave height	Combined level	FFL of site
0.5	3.26mOD	+3.00m	6.26mOD	7.15mOD
0.1	3.48mOD	+3.00m	6.48mOD	7.15mOD

4.1.1 Climate Change Flood Risk

Table 4-2 presents the ICWWS values for Point 3 for the Medium Range and High-End Future Scenarios (MRFS and HEFS) as well as approximate values for wave heights. A wave overtopping study was carried out to assess the risks due to climate change is found in Appendix B with the key points summarised in Section 4.1. It was found that there is increased risk from wave overtopping when climate change is considered and potential defence options were explored to provide protection to the site in the future.

Table 4-2: Climate change peak sea levels with wave overtopping estimates

Climate Scenario	Still water level (point 3)	Approximate wave height	Combined level	FFL of site
0.5% AEP event				
MRFS	3.76mOD	+3.00m	6.76mOD	7.15mOD
HEFS	4.26mOD	+3.00m	7.26mOD	7.15mOD
0.1% AEP event				
MRFS	3.98mOD	+3.00m	6.98mOD	7.15mOD
HEFS	4.48mOD	+3.00m	7.48mOD	7.15mOD

4.2 Proposed development considerations

4.2.1 Surface Water Management

The system utilises the existing infrastructure already present. Refer to the drainage report for further details of the proposed system. The proposed surface water management system should also undergo a third-party stormwater audit to assess its suitability. Typical management measures that are recommended include the provision of non return valves and sealed manholes for the surface water and foul system.

To address the risk from overtopping and the residual risk of stormwater system failure there are two potential approaches for the site. The first is to raise the FFL of the building 150mm above the external hardstanding area, the other is to retain level access and install ACO drains with the aim of minimising surface water from entering the building.

4.2.2 Finished Floor Levels & Building Resilience

The proposed finished floor level (FFL) is approximately 7.15mOD. As stated in Section 4.1 this is above extreme still water tidal levels and in terms of wave overtopping in the current scenario whilst there is some wave overtopping the rate/volume is within safety limits for people and cars at the crest of the nearby defence.

As stated in the Surface Water Management section above, there is a chance of exceedance flow entering the building during a costal overtopping event if the level access option is chosen. This impact would be acceptable given the intended use of the building and on the condition that the construction is flood resilient and incorporates measures outlined in Section 4.6 of the Technical Appendices to the Planning System and Flood Risk Management Guidelines:

- Flood resilient construction accepts that floodwater will enter buildings and provides for this in the design and specification of internal building services and finishes.
- Such measures limit damage caused by floodwater and allow relatively quick recovery.
- This can be achieved by using;
    - wall and floor materials such as ceramic tiling that can be cleaned and dried relatively easily,
    - Use of resilient substrate materials (e.g. blockwork)
    - Electrics, appliances and kitchen fittings may also be raised above floor level, by 500mm, and;
    - Non return valves and sealed manholes on stormwater and foul systems
    - The risk of gravel/beach material damaging the exterior of the building and windows should also be considered.

Regardless of whether level access option were chosen it would be prudent to incorporate the full range of resilient construction methods.

4.2.3 Climate Change

In the climate scenarios for wave overtopping the safe thresholds for vehicles and people at the crest are exceeded and additional measures should be considered in the future with regard to the defence crest adjacent to the site.

4.2.4 Access/Egress

Access to the site is via the car park, and a road leading from the west, under the DART line. In the current scenario there is some overtopping volume that would impact the car park and access route, although this is not predicted to be above safe thresholds there would be excess surface water impacting the car park. The risk could be managed with signage and warning users of the risk under storm conditions. Options are available for active warning on site using Triton or Tidewatch.

4.2.5 Coastal Erosion

As the proposed development is located along Killiney beach coastal erosion is important to consider for the longevity of the development. Coastal erosion was examined in the ICPSS for the coastline of Ireland by extrapolating the rate of observed erosion along the coast into the future. Figure 4-1 shows the location of the predicted coastline for the 2050 scenario. From the figure the site is not located in an area where significant erosion is expected and still on the land side of the predicted coastline for 2050. From the available data the site is at low risk of issues regarding coastal erosion however monitoring overtime is recommended as the predictions are subject to uncertainty and as shown in the wave overtopping study the severity of inundation is sensitive to changes in the beach profile.

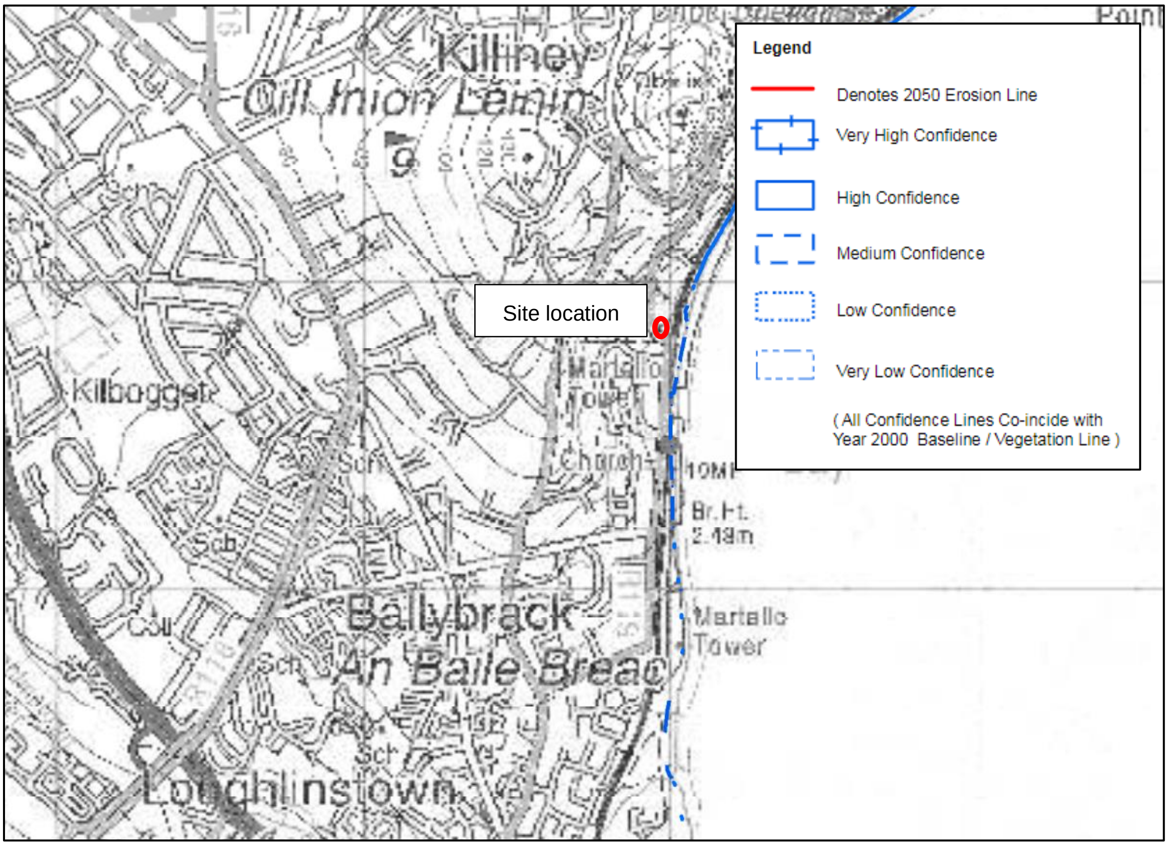


Figure 4-1: Extract from ICPSS Coastal Erosion 2050 map for Killiney Bay (source: ICPSS Phase 2 - South East Coast Work Packages 2, 3 & 4A - Appendix 8 - Erosion Mapping IBE0104, /June 2010)

5 Conclusion

JBA Consulting has undertaken a Flood Risk Assessment for a proposed sports facility in Killiney Co Dublin. The development will include changing rooms, bathrooms, and equipment storage areas.

The main source of flood risk to the site is from wave overtopping during extreme coastal events. The site is above the estimated peak water levels for the 0.5% and 0.1% AEP events meaning it is in Flood Zone C however potential risks from wave overtopping particularly in the climate change scenarios were identified. To fully understand the potential risks to the site and mitigation measures a formal wave height and wave overtopping assessment was carried out. This assessment showed that at present day some overtopping will occur at the site location but the total volumes are within risk thresholds for vehicles or humans (minimal overtopping). It was found that overtopping does become a greater risk to potential users of the site in the climate scenarios and potential mitigation measures were suggested in the future. The severity of wave overtopping is also sensitive to changes in the beach profile and monitoring is recommended. It is noted that the inclusion of the building itself does not increase the risk of overtopping at this location.

Regarding pluvial flood risk, review of the available information does not indicate that the site is at risk of pluvial flooding. To protect the building from the risk of inundation from the wave overtopping and potential stormwater system failure two solutions are considered appropriate. The first is raising the FFL of the building 150mm above the external hardstanding area, and the second is the inclusion of ACO drains which direct water away from the proposed building (sloping away from the entrances etc), but this may not be effective when overtopping is occurring. Regardless of the approach on level access, resilient building finishes are also recommended these would mitigate and direct inundation of the building and also any damage to the exterior of the building from beach material. Access is not anticipated to be significantly disrupted by current day wave overtopping, but in stormy conditions there would be additional surface water on the access road/car park. Warning signage or use of the formal Triton/Tidewatch system could be incorporated into a formal plan, if the council deemed it necessary.

This Flood Risk Assessment was undertaken in accordance with 'The Planning System and Flood Risk Management Guidelines'.

Appendices

A Appendix - Understanding Flood Risk

Flood Risk is generally accepted to be a combination of the likelihood (or probability) of flooding and the potential consequences arising. Flood Risk can be expressed in terms of the following relationship:

Flood Risk = Probability of Flooding x Consequences of Flooding

A.1 Probability of Flooding

The likelihood or probability of a flood event (whether tidal or fluvial) is classified by its Annual Exceedance Probability (AEP) or return period years, a 1% AEP flood 1 in 100 chance of occurring in any given year. In this report, flood frequency will primarily be expressed in terms of AEP, which is the inverse of the return period, as shown in the table below and explained above. This can helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval and is the terminology which will be used throughout this report.

Table: Conversion between return periods and annual exceedance probabilities

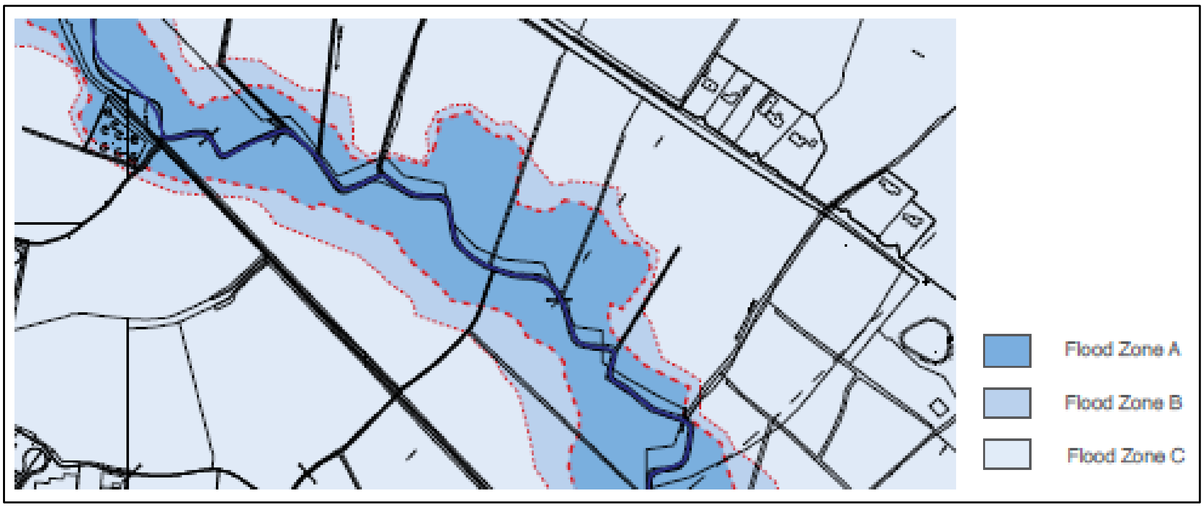
Return period (years)	Annual exceedance probability (%)
2	50
10	10
50	2
100	1
200	0.5
1000	0.1

A.2 Flood Zones

Flood Zones are geographical areas illustrating the probability of flooding. For the purpose of the Planning Guidelines, there are 3 types of levels of flood zones, A, B and C.

Zone	Description
Flood Zone A	Where the probability of flooding is highest, greater than 1% (1 in 100) from river flooding or 0.5% (1 in 200) for coastal/ tidal Flooding
Flood Zone B	Moderate probability of flooding, between 1% and 0.1% from rivers and between 0.5% and 0.1% from coastal/ tidal.
Flood Zone C	Lowest probability of flooding, les than 0.1% from both rivers and coastal/ tidal.

It is important to note that the definition of the flood zones is based on an undefended scenario and does not take into account the presence of flood protection structures such as flood walls or embankments. This is to allow for the fact that there is a residual risk of flooding behind the defences will be maintained in perpetuity.



A.3 Consequences of Flooding

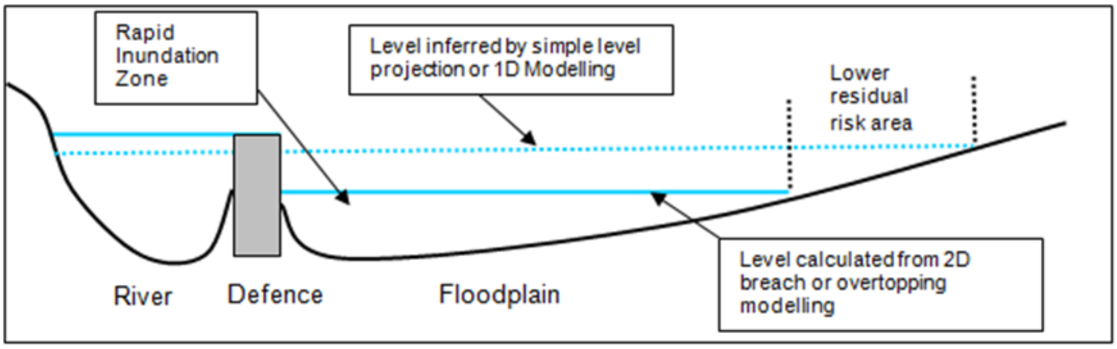
Consequences of flooding depend on the Hazards caused by flooding (depth of water, speed of flow. Rate of onset, duration, wave-action effects, water quality) and the vulnerability of receptors (type of development, nature, e.g. age-structure of the population, presence and reliability of mitigation measures etc.)

The 'Planning System and Flood Risk Management' provides three vulnerability categories, based on type of development, nature, which are detailed in Table 3.1 of the Guidelines, and are summarised as:

- **Highly vulnerable**, including residential properties, essential infrastructure and emergency service facilities
- **Less vulnerable**, such as retail and commercial and local transport infrastructure, such as changing rooms.
- **Water compatible**, including open space, outdoor recreation and associated essential infrastructure, such as changing rooms.

A.4 Residual Risk

The presence of flood defences, by their very nature, hinder the movement of flood water across the floodplain and prevent flooding unless river levels rise above the defence crest level or a breach occurs. This known as residual risk:



B Wave Overtopping Report

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Killiney Sports Facility Coastal Wave Overtopping Assessment

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Prepared for:

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Contract

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This report describes work commissioned by Dun Laoghaire Rathdown County Council, by an instruction dated 1st November 2023. The Client's representative for the contract was Sarah Clifford of Dun Laoghaire Rathdown County Council. Florian Bellafont and Ian Gaskell of JBA Consulting carried out this work.

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The methodology adopted and the sources of information used by JBA in providing its services are outlined in this Report. The work described in this Report was undertaken between (01 December and 22 December 2023) and is based on the conditions encountered and the information available during the said period. The scope of this Report and the services are accordingly factually limited by these circumstances.

Contents

List of Figures

List of Tables

Abbreviations

AEP	Annual Exceedance Probability
CAPO	Coastal Areas Potentially Vulnerable to Wave Overtopping
DTM	Digital Terrain Model
ICWWS	Irish Coastal Wave and Water Level Modelling Study 2018
mOD	Metres Above Ordnance Datum (OD Malin 15)
MRFS	Medium Range Future Scenario
HEFS	High-End Future Scenario

Executive Summary

The proposed development is a set of sports facilities located along Killiney Beach. The site is located in an area at low flood risk (Flood Zone C) from present-day extreme water levels but at risk of wave overtopping. The development is identified as 'Less vulnerable'; based on the Dún Laoghaire-Rathdown Strategic Flood Risk Assessment guidance document (Appendix 15), the design event for tidal flood risk assessment is the 0.5% Annual Exceedance Probability (AEP) event under the Medium Range Future Scenario (MRFS).

The present report details the wave overtopping assessment results. It proposes concept flood defence designs to mitigate future wave overtopping risk, considering two usages of the defence crest during storm conditions: people or vehicles.

The wave conditions used in wave overtopping calculations come from the ICWWS (2020) phase 2 report at the Bray Coastal Areas Potentially Vulnerable to Wave Overtopping (CAPO). Bray CAPO, located about 6km south of the proposed development, was judged suited for the present study. Wave overtopping calculations are primarily based on the EurOtop II empirical equations. The Artificial Neural Network 2 tool was used to cross-validate the empirical calculations and study the sensitivity of the defence to beach erosion.

Based on the 2m 2011 LiDAR dataset, the defence profile comprises a lower slope of 1 in 10 corresponding to the shingle/pebble beach up to an elevation of 3mOD, and a 1 in 2 upper slope of gabion baskets. The defence's toe and crest levels are estimated to be 0 and 7mOD, respectively.

For present-day wave climate, the current configuration satisfies the recommended overtopping limits for people at the crest. Under MRFS conditions, the current configuration does not provide the level of protection required for people or vehicles on the crest.

Concept defences were proposed to reduce the overtopping risk and bring it within tolerable limits for people or vehicles on the crest under the design event (0.5% AEP event MRFS epoch). The defence's crest level was increased until the tolerable limits were met.

Only minor modifications to the current configuration are required for vehicles on the crest during overtopping events. The defence crest level should be between 7 and 7.19mOD under the design event. Considering the minor modifications, gabion baskets may be used in the flood defence design. However, replacing gabion baskets with rock armour units after failure or at the end of their design life is recommended due to their vulnerability to vandalism, erosion, and corrosion. The design flood defence should cover, at least, the proposed development, the Killiney Beach Parking to the North (main access/egress of the public to the site), and some additional length to account for the angle of wave attack to the South. The defence height increase represents about 100m of length in total. The lower sloping ground around the proposed development site is expected to act as a flood storage area, and a drainage system through the defence should be included to allow the overtopping water to drain back to the sea. With such a design, people are still highly exposed and vulnerable during overtopping events. Therefore, public access to the beach should be forbidden and controlled by local authorities during storm conditions. Use of a

flood forecasting system such as Triton or Tidewatch to identify storm conditions is critical in such a strategy. Moreover, shingle or pebble debris in overtopping flows can exacerbate the risk to people in case of overtopping and damage to the proposed building (windows). Putting wooden panels or metal shutters on the windows before overtopping events may reduce the damage to the building.

Substantial modifications to the current configuration are required for people on the crest during overtopping events. If gabion baskets are used, the defence crest level should be higher than 8.28mOD under the design event. Replacing gabion baskets with rock armour units in the concept design is recommended. If one layer of rock armour units with an impermeable core is used, the design crest level is reduced to 7.66mOD. Such a design raises the question of the spatial extent of the flood defence. Only protecting the section in front of the proposed development means that the remaining part of the promenade is unsafe for people during overtopping events. It is, then, recommended to protect the whole promenade, including the two main accesses to the beach and promenade, representing about 300m of length in total. Using a flood forecasting system such as Triton or Tidewatch to identify storm conditions could also be beneficial in a local flood risk management strategy.

Information on the risk of overtopping could be provided to the public by signage on site, located at the two main beach access points.

The study highlights that overtopping rates are susceptible to the shingle/pebble beach slope. An eroded profile (beach slope of 1 in 7) does not provide overtopping protection to vehicles or people at the crest for present-day wave climate. Monitoring the beach profile with regular topographic surveys is recommended. The flood risk should be re-assessed if the beach profile substantially and durably becomes steeper after storm events or under future climate change.

1 Introduction

The proposed development site is located along the beach frontage at Killiney in County Dublin and is part of the Dún Laoghaire–Rathdown administrative region (Figure 1-1). Coastal flood risk to the site was identified within the Dún Laoghaire Rathdown County Council County Development Plan 2022-2028 Strategic Flood Risk Assessment (SFRA). The site was identified as being located in an area at low flood risk (Flood Zone C) from present-day extreme water levels but at risk of wave overtopping.

This report aims to assess the risk to the proposed development site from wave overtopping and the results used to inform the requirement for flood mitigation measures.

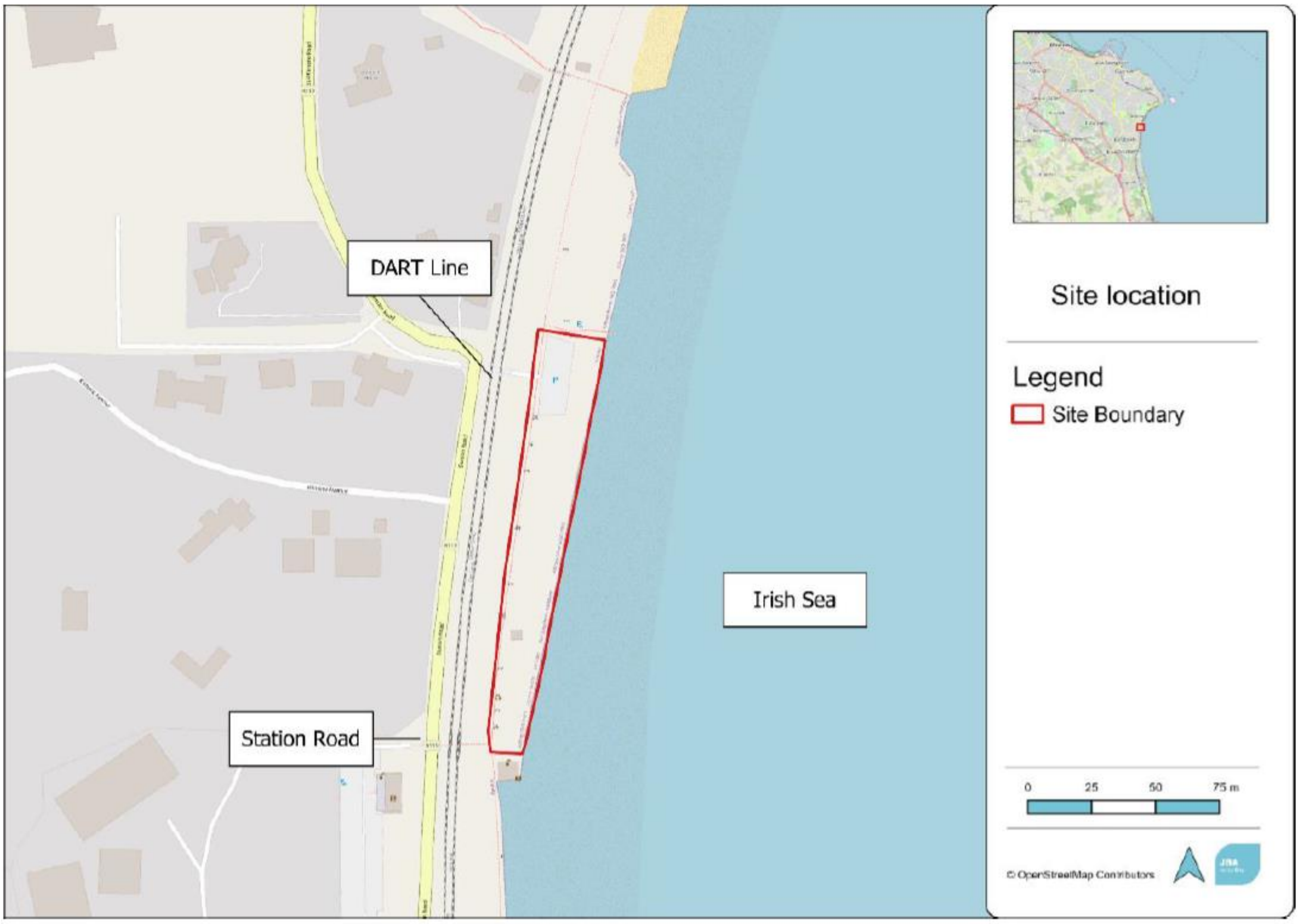

Figure 1-1: Site location

1.1 Proposed development

The proposed development is a set of sports facilities, including changing rooms located along Killiney beach (Figure 1-2).

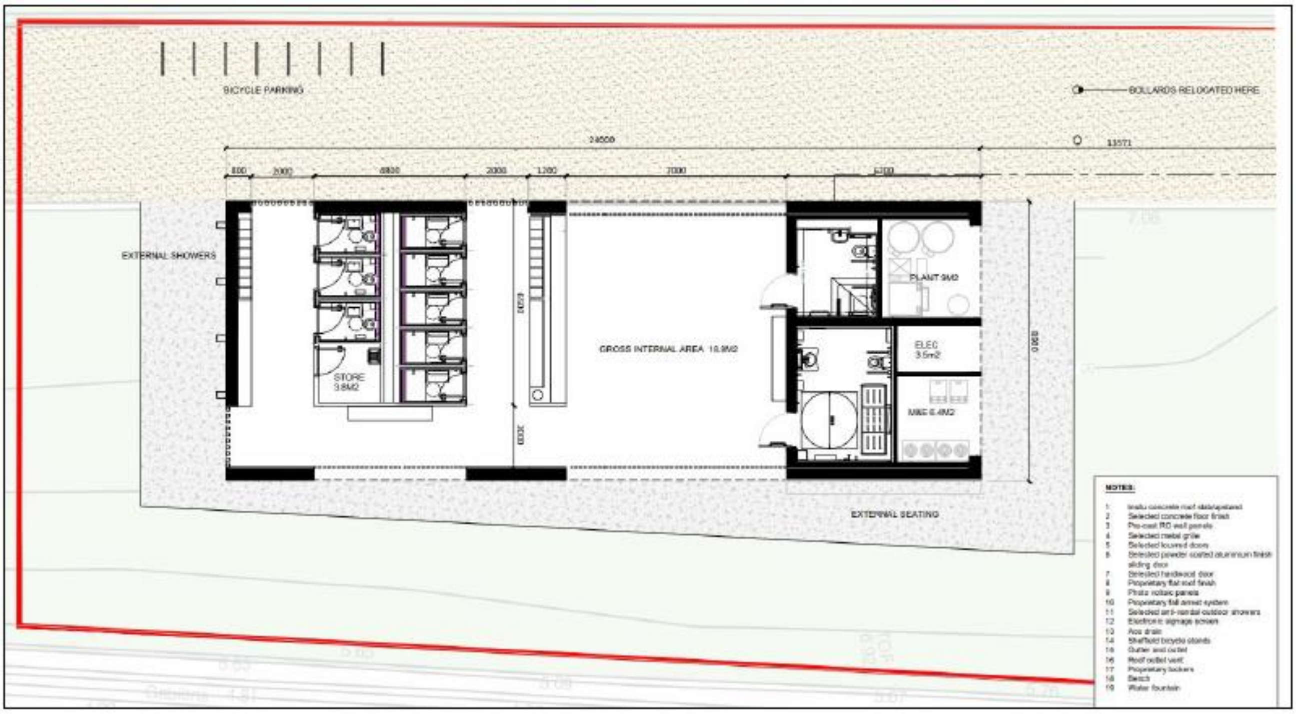

Figure 1-2: Proposed site layout (provided by client)

1.2 Development vulnerability classification

The 'Planning System and Flood Risk Management' provides three vulnerability categories, based on the type of development nature, which are detailed in Table 3.1 of the Guidelines, and are summarised as:

- Highly vulnerable, including residential properties, essential infrastructure and emergency service facilities.
- Less vulnerable, such as retail and commercial and local transport infrastructure, such as changing rooms.
- Water compatible, including open space, outdoor recreation and associated essential infrastructure, such as changing rooms.

The development classification falls under the 'Less vulnerable' category as it includes bathrooms, changing facilities and doesn't need to be operational during storms. OPW guidance[1] recommends a suite of climate change allowances within development proposals (Table 1-1). For a 'Less vulnerable' development, the guidance recommends using the 0.5% Annual Exceedance Probability (AEP) event under the Medium Range Future Scenario (MRFS) for development proposals.

1 OPW Assessment of Potential Future Scenarios, Flood Risk Management Draft Guidance, 2009

Table 1-1: Climate change allowances by vulnerability and flood source

Development vulnerability	Fluvial climate change allowance (increase in flows)	Tidal climate change allowance (increase in sea level)	Storm water / surface water
Less vulnerable	20%	0.5m (MRFS)	Refer to the Stormwater Management Policy in Appendix 7.1 for details of climate change allowances
Highly vulnerable	20%	1.0m (HEFS)	
Critical or extremely vulnerable (e.g. hospitals, major sub-stations, blue light services)	30%	1.2m (and test up to 2m)*	
Note: there will be no discounting of climate change allowances for shorter lifespan developments. * From OPW Sectoral Climate Change Adaptation Plan (2019) where a 2m rise in sea level is plausible under certain scenarios.			

2 Wave overtopping modelling

2.1 Boundary data

2.1.1 ICWWS 2018 Phase 2 - Bray CAPO (from appendices)

The Irish Coastal Wave and Water Level Modelling Study (ICWWS) (2018) - Phase 2 generated combined nearshore wave climate and water level conditions for the Coastal Areas Potentially Vulnerable to Wave Overtopping (CAPOs), as identified in Phase 2 of the ICWWS 2018. Following the joint probability analysis (Figure 2-1), offshore wave climate/water level pairings were transformed to inshore using the spectral wave model MIKE21 SW, and wave conditions were extracted near the shoreline/defence line at each CAPO. This provided six joint combinations of water levels and wave climate for multiple AEPs from 50% to 0.1% for present-day, MRFS and High-End Future Scenario (HEFS) scenarios.

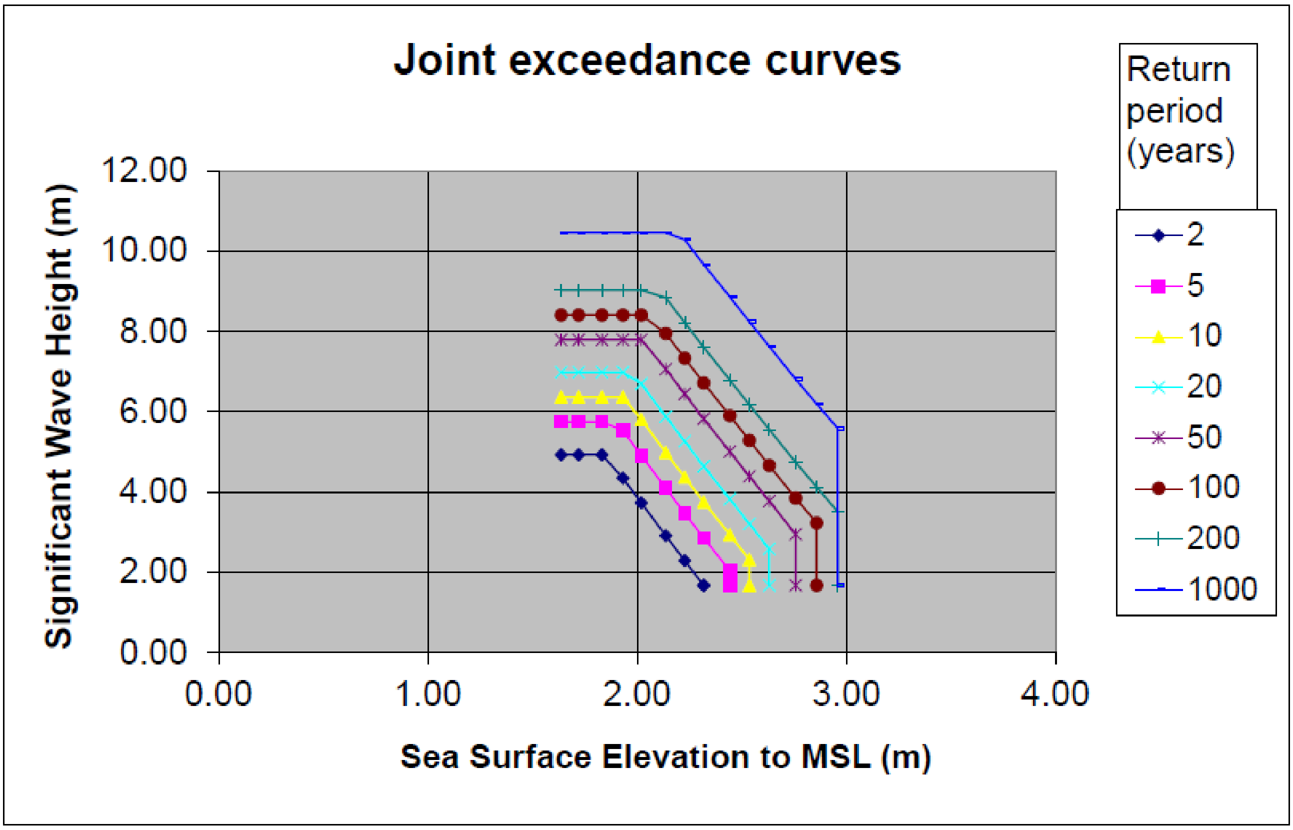

Figure 2-1: Offshore joint wave (Bray 135 degrees) and water level (Bray) exceedance curve (figure C 1.20 of Appendices)

The water level and wave climate conditions at Bray CAPO were used to assess wave overtopping risk at the study site. Indeed, Bray CAPO is located roughly 6km south of Killiney, and based on assessment of bathymetric data was deemed appropriate for use in the wave overtopping calculations (Figure 2-2).

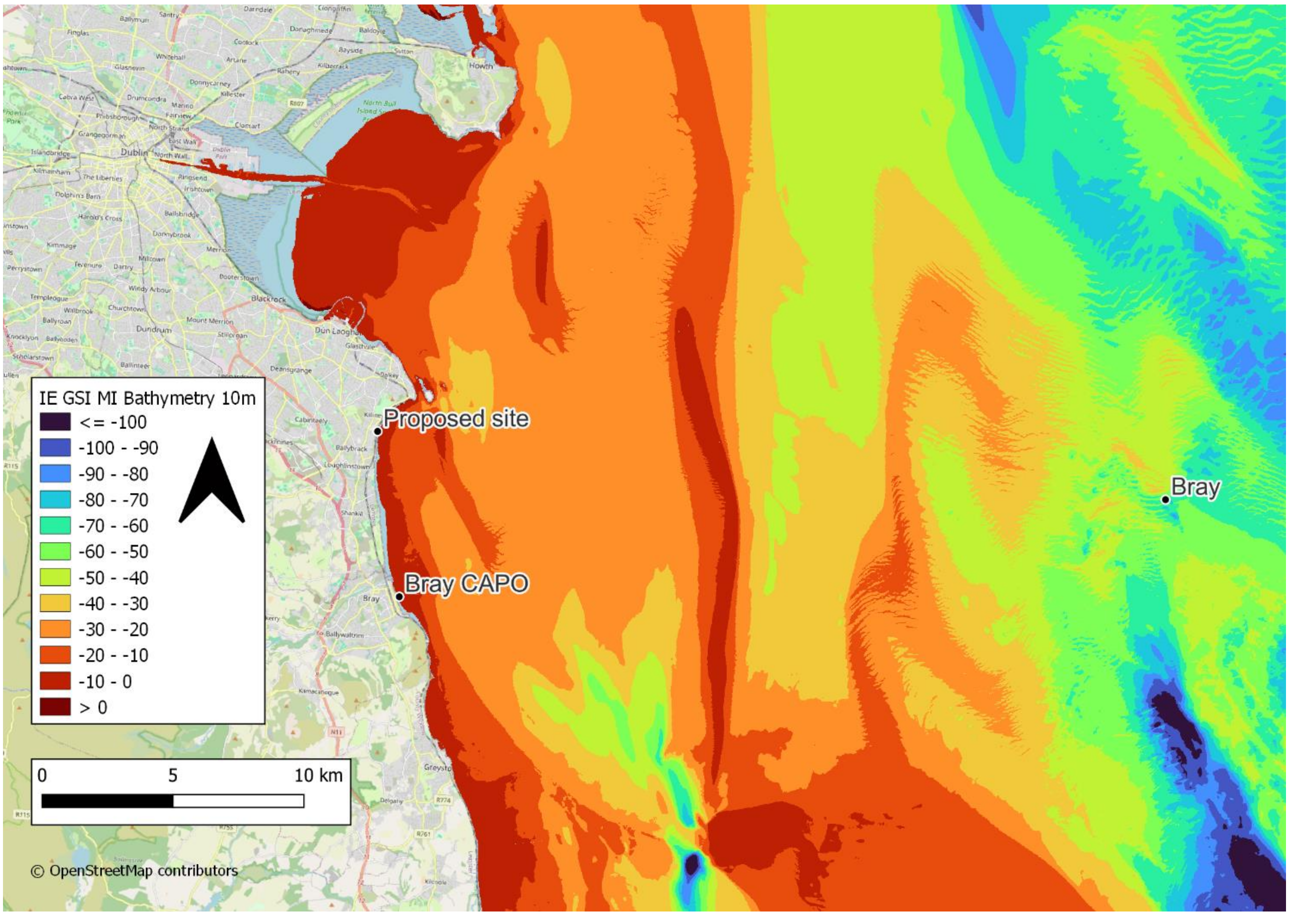

Figure 2-2: Bathymetry data[2] - proposed site location, Bray CAPO and Bray Wave data point

The water level (WL, OD Malin OSGM15), spectral significant wave height (Hm0) and spectral peak wave period (Tp) at Bray CAPO (bed level = 0.042mOD) are presented in Table 2-1 for the 0.5% AEP. As the orientation of the coast is different at Bray CAPO and the proposed study site, the output wave direction was not used. Instead, an angle of wave attack (β) of 10° was assumed in the wave overtopping calculation.

Table 2-1: Bray CAPO water level and wave conditions for the 0.5% AEP

Present-day			MRFS			HEFS		
WL (mOD)	Hm0 (m)	Tp (s)	WL (mOD)	Hm0 (m)	Tp (s)	WL (mOD)	Hm0 (m)	Tp (s)
2.25	1.92	11.10	2.75	2.22	11.23	3.25	2.51	11.34
2.46	2.04	10.92	2.96	2.32	10.91	3.46	2.60	11.01
2.67	2.10	10.00	3.17	2.38	10.13	3.67	2.64	10.20
2.86	2.14	9.29	3.36	2.40	9.36	3.86	2.64	9.42
3.09	2.15	8.30	3.59	2.36	8.37	4.09	2.54	8.43
3.19	2.11	7.88	3.69	2.28	7.94	4.19	2.39	8.00

2 https://www.infomar.ie/maps/interactive-maps/dynamic-bathymetric-viewer

2.1.2 Fetch limited calculation check

The fetch-limited method proposed by Goda[3] was used to sense-check the offshore wave height in the joint probability analysis. Based on British Standards (figure 6, BS 6399-2:1997) basic wind speed map, the wind speed at the elevation of 10m above the sea surface (U10) is 24m/s in the Irish Sea with an AEP of 2% (50-year return period).

The wind fetch is the length of water over which the wind can blow without obstruction. The fetch was estimated to be 200km between the proposed site location and the opposite British coast (North-Est direction). The acceleration due to gravity was set to 9.81m/s^2.

It was assumed a fetch-limited wave growth, meaning that the wave growth is limited by the fetch length, not the wind duration. The minimum wind duration (t_{min}, equation 3.4 in Goda) is 11.1 hours to reach such a condition.

Based on a fetch of 200km, a wind duration of 11.1 hours and a wind speed of 24m/s, the significant wave height is 6.0m (equation 3.1 of Goda), and the significant wave period is 9.1s (equation 3.2 of Goda). The results align with the joint probability analysis (Figure 2-1).

2.2 Defence schematisation

Bottom elevation data was extracted from the 2011 2m LiDAR (Light Detection and Ranging) Coverage Office of Public Works (OPW) National Aerial Survey Contract (NASC) Ireland (ROI) ITM dataset[4] - block ids 3337 and 3357. The bottom elevation was extracted at five transects placed at 25m intervals (Figure 2-4).

Figure 2-3: Foreshore of Killiney beach

3 Goda, Y., 2010. Random seas and design of maritime structures (Vol. 33). World Scientific Publishing Company

4 https://gsi.geodata.gov.ie/server/rest/services/Lidar/IE_GSI_LiDAR_Coverage_OPW_NASC_IE26_ITM/MapServer

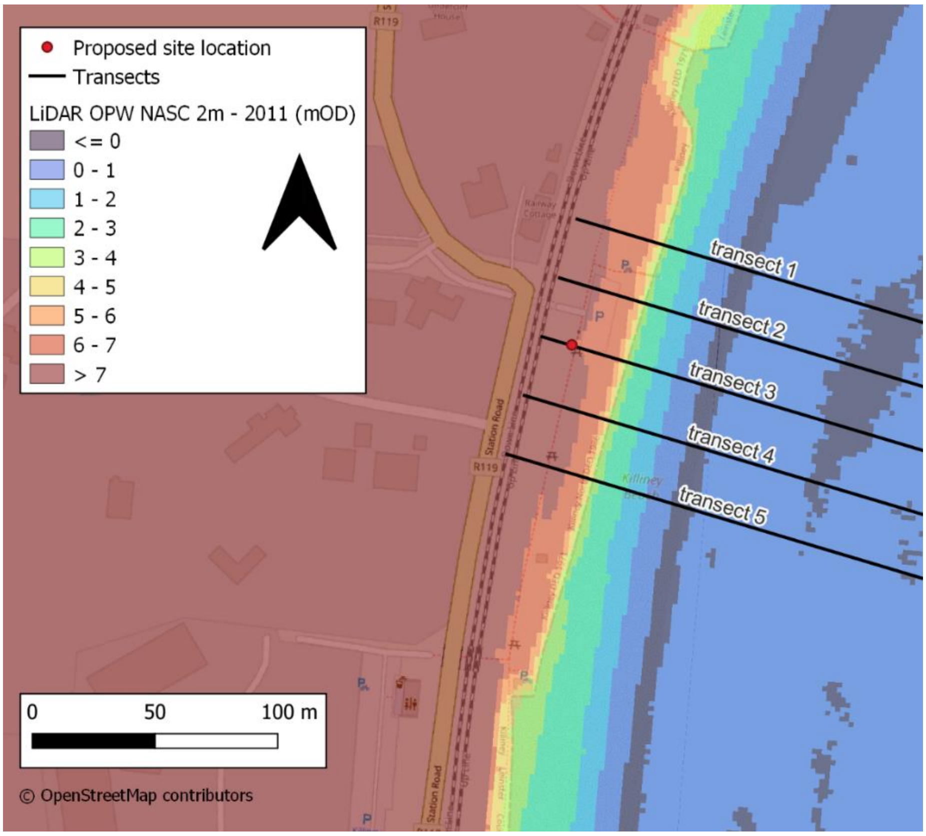

Figure 2-4: LiDAR data - transects locations

Figure 2-5 shows the bottom elevation at the transects and the profile schematisation used in the wave overtopping calculation. The profile has two distinctive slopes:

- Elevation between 0 and 3mOD: shingle/pebble beach characterised by a gentle slope of 1 in 10.
- Elevation between 3 and 7mOD: gabions characterised by a steeper slope of 1 in 2.

The foreshore is defined as the gentle sloping part located seaward of the toe of a coastal structure. It is characterised by depth-induced wave processes (shoaling, wave breaking). A foreshore steeper than 1:10 directly in front of a defence can be better considered part of the structure, as recommended by the EurOtop II manual[5]. Therefore, the shingle/pebble beach was considered part of the defence (refer to Figure 2-3).

[5] http://www.overtopping-manual.com/assets/downloads/EurOtop_II_2018_Final_version.pdf

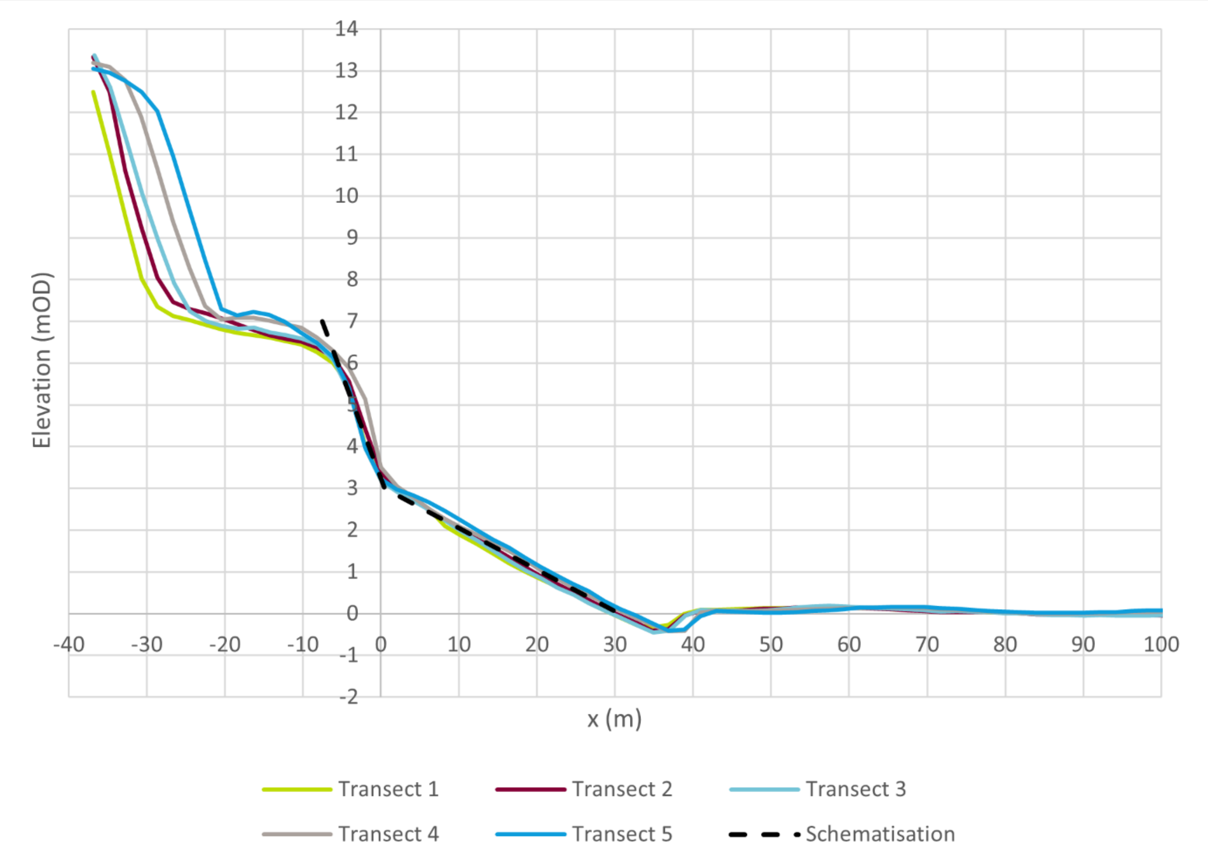

Figure 2-5: Transects elevation and structure schematisation (x = 0m corresponds to the base of the gabions)

The defence schematisation parameters are detailed below and in Table 2-2:

- Crest level: 7mOD
- Toe level: 0mOD
- Normal angle of defence (degrees from N): 103 degrees

Table 2-2: Defence schematisation parameters

Slope	Downward	Upward
Elevation	0 to 3mOD	3 to 7mOD
Type	Shingle/Pebble	Gabions
Slope $(\cot \alpha)$	10	2
Roughness factor (γ_f)	0.8	0.7

As the defence toe level (0mOD) is very close to the elevation at which the wave conditions were extracted at Bray CAPO (0.042mOD), the wave conditions in Table 2-1 are used in the wave overtopping calculations.

2.3 Calculation method

The wave overtopping discharge rates were estimated using the EurOtop II empirical equations[5]. The use of the EurOtop II is twofold:

- Simple defence geometry (two slopes, no berm, no crest width, no toe width)
- Downard slope of the schematisation (1 in 10) outside of the range of applicability of Artificial Neural Network (ANN 2[6])

The ANN 2 tool was used to confirm the empirical calculation and study the sensitivity to downward slope erosion by adjusting the downward slope to 1 in 7 (section 2.4.1).

The following information is used in the empirical equations:

- Water level and wave conditions at the defence's toe (section 2.1)
- Defence geometry (section 2.2)

2.3.1 Mean value and Design/Assessment approaches

The EurOtop II manual describes the reliability of the empirical formula by considering the coefficients as stochastic parameters with a given mean value and standard deviation. Based on this, two approaches are proposed in the manual:

- Mean value approach: use the mean values of the empirical coefficients in the prediction formula.
- Design or assessment approach: add one standard deviation to the empirical coefficients in the prediction formula.

The mean value approach formula should be used to predict or compare with test data. This approach is used to compare against ANN 2 tool's rates (section 2.4.1).

The Design/Assessment approach is semi-probabilistic and includes a partial safety factor. This approach is used to assess wave overtopping rates and design optioneering.

The following sections detail the equations from the EurOtop II manual used in the wave overtopping assessment.

2.3.2 Wave overtopping discharges

The general formula for the average overtopping discharge on a slope (dike, levee, embankment) are:

- Equations (5.10) and (5.11) - Mean value approach
- Equations (5.12) and (5.13) - Design/Assessment approach

2.3.3 Run-up

The wave run-up $R_{u2\%}$ is calculated using:

- Equations (5.1) and (5.2) - Mean value approach
- Equations (5.4) and (5.5) - Design or assessment approach

6 http://overtopping.ing.unibo.it/overtopping

$R_{u2\%}$ is the wave run-up height exceeded by 2% of the incoming waves.

2.3.4 Influence factors

The following formula are used to determine the effect of oblique waves:

- Equation (5.28) - Wave run-up
- Equation (5.29) - Wave overtopping

2.3.5 Composite slope

As the profile consists of a composite slope, a characteristic (average) slope should be determined for each input condition (water level and wave conditions). The method detailed in section 5.4.6 of EurOtop II was applied (Figure 2-6) to identify the run-up/run-down area used to calculate the average slope.

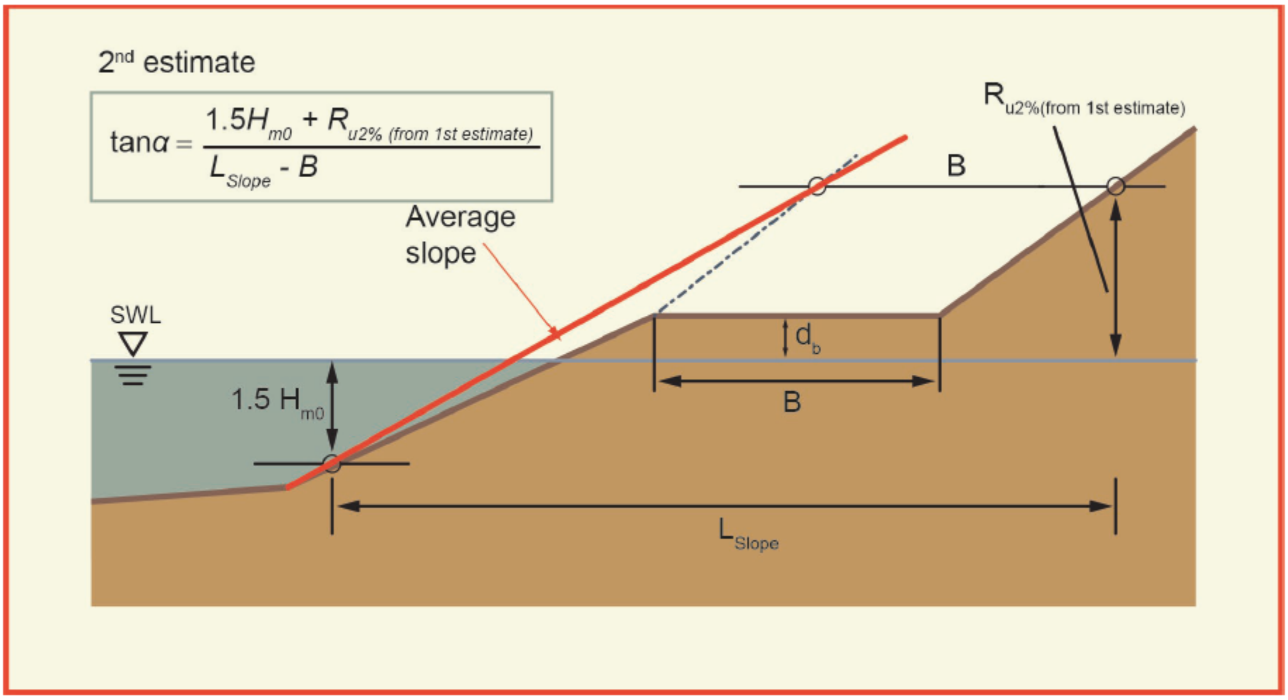

Figure 2-6: Determination of the average slope (EurOtop II)

The iterative method consists of identifying a lower and upper point located on the defence profile as follows:

- Vertical distance below the water level: $h_l = \min\,(1.5H_{m0};\ h)$
- Vertical distance above the water level: $h_u = \min\,(R_{u2\%};\ R_c)$

The toe water depth (h) is equal to the still water level (SWL) minus the toe level, and the crest freeboard (R_c) is equal to the crest level minus SWL.

The average slope is used in the calculation of the following variables:

- Breaker parameter (Iribarren number)
- Average roughness
- Wave overtopping rates

- 2% run-up level

2.3.6 Vmax calculation

The Vmax calculation is based on the following:

- 2% run-up level $R_{u2\%}$: see section 2.3.3
- equations (5.53 to 5.57)

A storm duration of 2 hours was assumed to calculate the Vmax.

2.4 Sensitivity testing

2.4.1 Beach slope and Artificial Neural Network 2 (ANN 2)

The experimental dataset used to train the ANN 2 tool only covers cases with defence slopes steeper than 1 in 7. Therefore, the wave overtopping prediction is extrapolated for defence slopes between 1:7 and 1:10, limiting the reliability of the wave overtopping calculation. The downward slope was adjusted to 1 in 7 in the ANN 2 input data to

- Cross-validate and confirm the empirical calculations - the downward slope is also adjusted to 1 in 7 in the empirical calculations
- Study the sensitivity to downward slope erosion leading to the loss of material and a steeper shingle/pebble beach slope

2.5 Tolerable wave overtopping limits for people and vehicles

Tolerable wave overtopping limits are defined by:

- mean overtopping discharge, q
- maximum overtopping wave volume, V_{max}

The wave overtopping rate (q) represents an average discharge per linear meter of width. However, there is no constant discharge over the crest of a structure during overtopping. The overtopping process is random in time, space and volume, and a mean overtopping discharge does not describe how many waves will overtop and how much water will be overtopped in each wave. That is why the overtopping severity is characterised by the average overtopping discharge and the maximum overtopping wave volume, V_{max}.

Table 2-3 shows the tolerable overtopping for people and vehicles on defence's crest.

Table 2-3: Limits for overtopping for people and vehicles (from EurOtop II)

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

2.6 Wave overtopping results

2.6.1 Baseline scenario (existing situation)

The wave overtopping calculations were undertaken for each of the six joint water level and wave climate combinations for the 0.5% AEP (Table 2-1). The worst-case overtopping rate from the six joint combinations was identified as the design overtopping rate. The results for the 0.5% AEP present-day, MRFS and HEFS are detailed in Table 2-4. It is noted that the OPW climate change guidance is over 10 years old; therefore, wave overtopping results were also calculated under HEFS conditions.

The water level and wave conditions leading to the highest overtopping rates are detailed in Table 2-5.

Table 2-4: Baseline overtopping results 0.5% AEP present day, MRFS and HEFS epochs - design events in bold

		Present-day	MRFS	HEFS
Mean value approach	q (l/s per m)	0.05	0.99	5.27
	Vmax (l per m)	60	1315	3299
Design/Assessment approach	q (l/s per m)	**0.21**	**2.15**	9.81
	Vmax (l per m)	**481**	**2240**	5041

Table 2-5: Design water level and wave conditions

Epoch	Present-day	MRFS	HEFS
Water level (mOD)	2.46	2.96	3.46
Hm0 (m)	2.04	2.32	2.60
Tp (s)	10.92	10.91	11.01

Based on the development proposal, it is considered that vehicles and people may be at the crest during a storm event. For present-day wave climate, the current configuration satisfies the recommended overtopping limits for people at the crest (bold values in Table 2-4).

As the significant wave height at the toe is between 2 and 3m under MRFS conditions (Table 2-1), the mean overtopping discharge should be lower than 5 l/s per m for vehicles on the crest. This limit is respected (q = 2.15 l/s per m), but the V_{max} of 2240 l per m just exceeds the limit of 2000 l per m recommended by EurOtop II (Table 2-3). Therefore, the current configuration does not provide the level of protection required for vehicles at the crest based on the V_{max} value. It should be noted that there is no clear guidance on the selection of the storm duration, that being the length of time over which high sea-level and high wave conditions persist. In this case two hours was selected for the storm duration. The V_{max} calculation is highly sensitive to the storm duration: V_{max} changes from 2240 to 1987 l per m if the 2-hour storm duration is reduced to a 1.5-hour storm duration.

Table 2-6 shows the wave overtopping calculated for the six joint probability events. The second most extreme overtopping satisfies the overtopping limits for cars at the crest.

Under MRFS conditions, people are at risk during all 6 joint probability overtopping events.

Table 2-6: Baseline overtopping results 0.5% AEP event MRFS epoch - 6 joint probability events

Event		1	2	3	4	5	6
Mean value approach	q (l/s per m)	0.99	0.88	0.61	0.66	0.30	0.23
	Vmax (l per m)	1315	1157	1006	916	519	416
Design/Assessment approach	q (l/s per m)	**2.15**	**1.92**	**1.47**	**1.47**	**0.85**	**0.68**
	Vmax (l per m)	**2240**	**1976**	**1864**	**1591**	**1061**	**885**

2.6.2 Cross-validation and sensitivity to beach erosion

Overtopping calculations were completed with a downward slope of 1 in 7 using EurOtop II empirical equations and ANN 2 tool to study and test the sensitivity to beach erosion. The wave conditions were kept unchanged.

The mean overtopping discharges estimated from the empirical equations are in good agreement with the Neural Network tool's results (Table 2-7): the mean overtopping discharges using empirical equations (ANN 2 tool) are 1.31 l/s per m (1.22) for present-day, and 7.72 l/s per m (5.53) under MRFS conditions. The good agreement adds confidence to the empirical calculations.

Overtopping rates are highly sensitive to the shingle/pebble beach slope. The present-day design overtopping rate increases from 0.21 to 2.93 l/s per m, and under MRFS conditions, q increases from 2.15 to 13.83 l/s per m. An eroded profile does not provide overtopping protection to vehicles or people at the crest for present-day wave climate.

This sensitivity study highlights the need to monitor the beach profile with regular topographic surveys and to re-assess the flood risk if the beach profile substantially and durably becomes steeper under future climate change or after storm events.

Table 2-7: Sensitivity to beach erosion - downward slope adjusted to 1 in 7

		Present-day	MRFS	HEFS
Mean value approach	q (l/s per m)	1.31	7.72	25.90
	Vmax (l per m)	1405	4055	8012
Design/Assessment approach	q (l/s per m)	**2.93**	**13.83**	45.73
	Vmax (l per m)	**2869**	**5817**	12502
ANN 2 - mean value	q (l/s per m)	1.22	5.53	17.1

3 Defence optioneering

Based on the baseline wave overtopping discharges, EurOtop tolerable thresholds were exceeded during a 0.5% AEP MRFS event for vehicles or people at the crest. Concept defences were modelled in an iterative process to identify potential flood mitigation options to reduce the overtopping risk and bring it within tolerable limits.

3.1 Concept defence

Only raising the defence's crest level was considered during the optioneering. The slope angles were unchanged. It is assumed that the upper part of the defence, above the beach, will be made of porous material with the same roughness as the gabions - a roughness factor of 0.7. The design crest level will be lower if a material with a higher roughness than the gabions, like rock armour units, is used in the upper slope.

The extreme sea level under MRFS conditions is 3.76mOD, meaning the proposed site location, with an elevation of about 7mOD, is not exposed to still water flooding but only to wave overtopping. The lower sloping ground around the proposed development site is expected to behave as a flood storage zone, and a drainage system through the upper part of the defence should be included to allow the overtopping water to drain back to the sea.

3.1.1 Limitations of gabion baskets in flood defence

Gabion baskets can suffer severe damage during storm conditions and in addition, gabion baskets are vulnerable to vandalism, erosion, and cage corrosion.

Supposing gabion baskets are maintained in the defence flood design, it is recommended to develop a regular inspection strategy of the state of the gabion baskets, at least every winter, before the storm season. A rapid intervention plan to allow a quick reparation of the gabion baskets in case of failure is also recommended.

Gabion baskets can be substituted for another porous material, like rock armour units, that offers the same benefit in overtopping mitigation due to their high porosity but is less vulnerable to vandalism and erosion.

3.2 Design crest levels

The defence crest level was increased, from the current height of 7mOD, until the tolerable limits (q and V_{max}) were met for people or vehicles on the crest. The minimum crest levels are presented in Table 3-1 according to the level of protection required.

Under the design event (0.5% AEP event MRFS epoch), the overtopping limits for vehicles on the crest are met with the following defence crest levels:

- 7mOD - storm duration of 1.5 hours (section 2.6.1)
- 7.19mOD - storm duration of 2 hours (Table 3-1)

Under the design event, a defence crest level higher than 8.28mOD is required to reach the overtopping limits for people at the crest with a clear view of the sea.

Table 3-1: Defence optioneering - 0.5% AEP event MRFS epoch

Hazard type		Vehicles on crest	People on crest
Design crest level (mOD)		**7.19**	**8.28**
Mean value approach	q (l/s per m)	0.76	0.07
	Vmax (l per m)	1137	90
Design/Assessment approach	q (l/s per m)	**1.69**	**0.26**
	Vmax (l per m)	**1989**	**594**

3.3 Cross-validation and sensitivity to beach erosion

In the same way as section 2.6.2, overtopping calculations with the concept crest levels were completed with a downward slope of 1 in 7 using EurOtop II empirical equations and ANN 2 tool to study the sensitivity to beach erosion. The wave conditions were kept unchanged.

The mean overtopping discharges estimated from the empirical equations are in good agreement with the Neural Network tool's results (Table 3-2): the mean overtopping discharges using empirical equations (ANN 2 tool) are 5.95 l/s per m (4.38) for the vehicles on crest hazard, and 1.16 l/s per m (1.29) for the people on crest hazard. The good agreement adds confidence to the empirical calculations.

Overtopping rates are susceptible to the shingle/pebble beach slope. The design overtopping rate increases from 1.69 to 11.28 l/s per m for the vehicles on crest hazard, and from 0.26 to 2.49 l/s per m for the people on crest hazard. An eroded profile does not provide the overtopping protection for which it was designed, and both designs are no longer safe for people and vehicles on the crest.

This sensitivity study highlights the need to monitor the beach profile with regular topographic surveys and to re-assess the flood risk if the beach profile substantially and durably becomes steeper under future climate change or after storm events.

Table 3-2: Sensitivity to beach erosion - design event - downward slope 1 in 7

Hazard type		Vehicles on crest	People on crest
Crest level (mOD)		7.19	8.28
Mean value approach	q (l/s per m)	5.95	1.16
	Vmax (l per m)	3677	1528
Design/Assessment approach	q (l/s per m)	**11.28**	**2.49**
	Vmax (l per m)	**5505**	**2829**
ANN 2 - mean value	q (l/s per m)	4.34	1.29

3.4 Hazard type - Vehicles on the crest

EurOtop II overtopping limits for vehicles on the crest during storm events are

- $q < 5$ l/s per m (Hm0 ~ 3m)
- $V_{max} < 2000$ l per m

To respect these limits, the flood defence's crest level should be between 7 and 7.19mOD under the design event (0.5% AEP event MRFS epoch) - Table 3-3. Figure 3-1 presents the concept design and the location of the defence height increase. The extent can be split into three parts:

- Extent of the proposed development building
- North (right in Figure 3-1) - car park (main access/egress of the public to the site)
- South (left in Figure 3-1) - additional length to account for the angle of wave attack

The defence height increase represents about 100m of length in total.

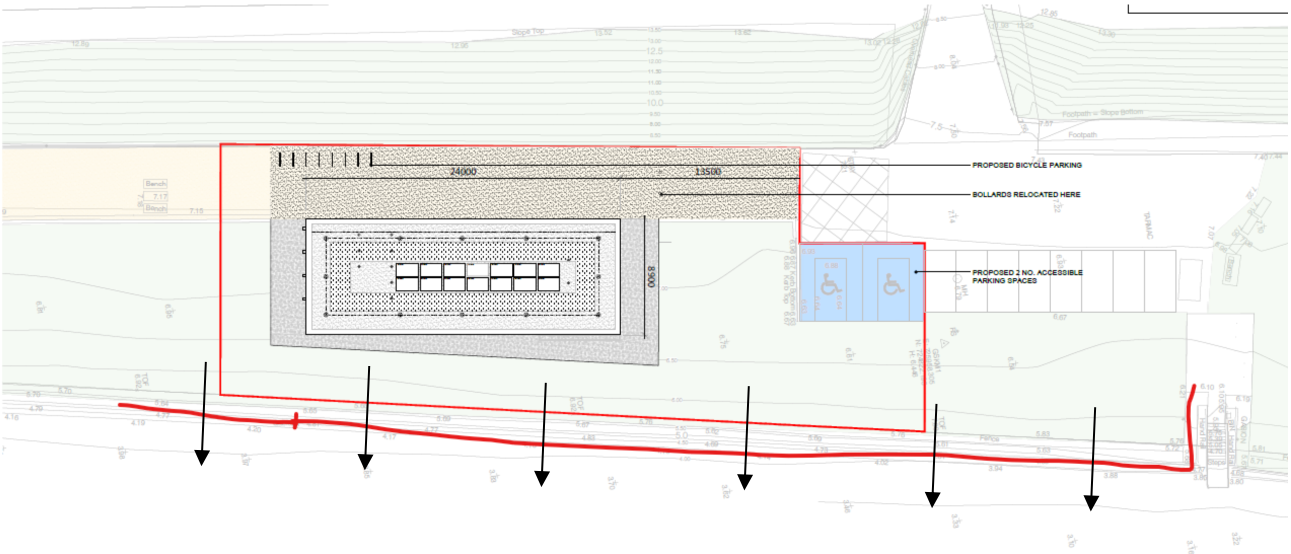

Figure 3-1: Concept design: elevated defence crest level (red), drainage system (black arrows)

Designing the defence to overtopping limits safe for vehicles means that people are still highly exposed and vulnerable during overtopping events. Therefore, public access to the beach should be forbidden and controlled by local authorities during storm conditions. Using a flood forecasting system such as Triton or Tidewatch to identify storm conditions is critical in such a strategy. Moreover, shingle or pebble debris in overtopping flows can exacerbate the risk to people in case of overtopping and damage to the proposed building (windows). Putting wooden panels on the windows prior to overtopping events may reduce the damage to the building.

These design overtopping limits allow emergency vehicles to safely access the site if needed.

The lower design crest level required for vehicles on the crest has the following benefits:

- reduce the environmental impact of the construction - less material is needed
- improve the user experience of the coast during non-extreme conditions - better view of the open sea

As the modifications of the current profile - an increase of the crest level by 0.2m maximum - are minor, gabion baskets may be used in the flood defence design, considering the caveats highlighted in section 3.1.1. However, replacing gabion baskets with rock armour units after failure or at the end of their design life is recommended. If one layer of rock armour units with an impermeable core (roughness factor of 0.6) is used in the upper slope of the structure, the design crest level is reduced to 6.7mOD.

The overtopping rates significantly increase if the beach slope becomes steeper (e.g. erosion). Thus, it is recommended monitoring the beach geometry through regular topographic profile surveys and re-assess the flood risk if the average beach slope (elevation between 0 and 3mOD) becomes steeper than 1 in 8.5 (mean value approach, q = 2.33 l/s per m, Vmax = 2124 l per m).

If a shorter storm duration of 1.5-hours is used instead of 2-hours, there is no need to change the crest level to satisfy the safety requirements for vehicles, as detailed in Table 3-3.

Table 3-3: Impact of the storm duration on the design crest level - vehicles on the crest

Storm duration		1.5 hours	2 hours
Design crest level (mOD)		**7.00**	**7.19**
Mean value approach	q (l/s per m)	0.99	0.76
	Vmax (l per m)	1146	1137
Design/Assessment approach	q (l/s per m)	**2.15**	**1.69**
	Vmax (l per m)	**1987**	**1989**

3.5 Hazard type - People on the crest (clear view on the sea)

EurOtop II overtopping limits for people on the crest with a clear view on the sea during storm events are:

- $q < 0.3$ l/s per m (Hm0 ~ 3m)
- $V_{max} < 600$ l per m

To respect these limits, the flood defence's crest level should be higher than 8.28mOD under the design event (0.5% AEP event MRFS epoch) - Table 3-2.

This study illustrates the non-linear impact of sea level rise on wave overtopping. Although the MRFS scenario corresponds to an increase of sea level by 0.5m, the crest level should increase by approximately 1.3m to keep the same level of protection. Indeed, the reduction in freeboard is associated with an increase of the wave height at the defence toe by about 0.3m, as shown in Table 2-5.

As the modifications of the current profile - an increase of the crest level by about 1.3m - are significant, gabion baskets are not recommended in the flood defence design due to their high vulnerability (section 3.1.1). Replacing gabion baskets with rock armour units in the concept design is recommended. If one layer of rock armour units with an impermeable core (roughness factor of 0.6) is used in the upper slope of the structure, the design crest level is reduced to 7.66mOD (Table 3-4).

Table 3-4: Impact of the upper slope roughness on the design crest level - people on the crest

Upper slope - elevation > 3mOD		Gabion baskets*	Rock armour units
Roughness factor		0.7	0.6
Crest level (mOD)		8.28	7.66
Mean value approach	q (l/s per m)	0.07	0.07
	Vmax (l per m)	90	115
Design/Assessment approach	q (l/s per m)	**0.26**	**0.26**
	Vmax (l per m)	**594**	**596**

* not recommended solution

Designing the flood defence for people's safety raises the question of the spatial extent of such defence. Only protecting the section in front of the proposed development, as shown in Figure 3-1, means that the remaining part of the promenade is unsafe for people during overtopping events. Therefore, in contrast to the vehicles on crest design, it is recommended to protect the whole promenade, including the two main accesses to the beach and promenade, as shown in Figure 3-2. The defence height increase represents about 300m of length in total.

Using a flood forecasting system such as Triton or Tidewatch to identify storm conditions could also be beneficial in a local flood risk management strategy. These systems are in place and presumably already used by DLRCC, advance warning can be provided up to 2 days before a storm. Information on the risk of overtopping could be provided to the public by signage on site, located at the two main beach access points.

The overtopping rates significantly increase if the beach slope becomes steeper (e.g. erosion). Thus, it is recommended that monitoring the beach geometry through regular topographic profile surveys and re-assess the flood risk if the average beach slope (elevation between 0 and 3mOD) becomes steeper than 1 in 9 (mean value approach, q = 0.27 l/s per m, Vmax = 597 l per m).

Figure 3-2: Extent of the flood defence designed for people on the crest during overtopping events: defence extent to protect the whole promenade (black line), principal access to the beach (red double arrows)

3.6 Limitations

The main limitations of the present study are:

- The joint-probability approach to determine water level/wave condition pairings is a basic approach that does not take account of the associated wind speed and direction or wave period and direction. A more detailed multi-variate probability approach allows the creation of a synthetic 10,000-year storm events dataset, that takes account of all wind and wave variables and gives more reliable results.
- The wave conditions in the present overtopping study come from the Bray CAPO, neglecting the potential variability at the study site (e.g. bathymetric features).
- Infragravity (IG) waves (periods between 30s and 5-10min) are not included in the MIKE21 SW model. IG waves could increase the wave height at the toe, potentially leading to higher overtopping. Wave-induced run-up and set-up can also be influenced by IG waves (surf beat).
- Wave- and wind-induced set-up is not included in the MIKE21 SW modelling, as the water level is the same at the toe and offshore.
- The topographic data used to determine the profile geometry dates to 2011.
- A storm duration of 2 hours is used in V_{max} calculations. Further investigation may be required to adapt this value to the Irish Sea.

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