East Coast Railway Infrastructure Protection Projects

Phase 3 Design Report

Whiterock Beach to South Killiney

COASTAL CELL AREA 2/3

DOCUMENT NO: 7694-CCA2_3-P3-ENG-CV-JAC-0005

11/08/25

Rialtas na hÉireann Government of Ireland | Tionscadal Éireann Project Ireland 2040 | NTA Údarás Náisiúnta Iompair National Transport Authority | Jacobs | ECRIPP Iarnród Éireann Irish Rail | Iarnród Éireann Irish Rail

Executive summary

This report presents the Phase 3 design for the Whiterock Beach to South Killiney section of the East Coast Railway Infrastructure Protection Projects (ECRIPP), commissioned by Iarnród Éireann (Irish Rail). The project aims to enhance the resilience of the coastal railway corridor against the impacts of climate change, particularly sea level rise, coastal erosion, and cliff instability.

The Whiterock to South Killiney frontage (Coastal Cell Area 2/3) is characterised by a mix of natural cliffs and engineered embankments. The railway line, perched above this dynamic coastline, is vulnerable to wave overtopping, cliff erosion, and structural undermining. The Phase 3 design addresses these risks with a 50-year design horizon, targeting a 1-in-200-year storm event standard of protection.

The preferred scheme includes:

- Rock revetments at Whiterock: Designed to protect the existing masonry wall and embankment from wave action and erosion. The revetment slope has been steepened to reduce its footprint and visual impact.
- Raised walkway and seawall at Killiney: A 3.0m wide elevated footpath with a rear wave wall and integrated beach access steps and ramps. The design ensures continued public access and future adaptability to beach level changes.

Public Consultation 1 (PC1) informed key design refinements, particularly at Whiterock, where revetment dimensions were reduced to address concerns about beach access and amenity. Alternatives such as vertical seawalls and beach nourishment were evaluated but ruled out due to technical, environmental, and safety constraints.

The project will proceed to Phase 4, focusing on statutory processes including Environmental Impact Assessment (EIA), Marine Area Consent (MAC), and planning applications. A second round of public consultation (PC2) is scheduled for late 2025.

Contents

Appendices

Tables

Figures

Acronyms and abbreviations

AEP	Annual Exceedance Probability
APIS	Authorisation to Place in Service
CCA	Coastal Cell Area
CFD	Computational Fluid Dynamics Modelling
CSM-RA	Common Safety Method Risk Assessment
DEHERR	Design Hazard Elimination and Risk Reduction Register
EIAR	Environmental Impact Assessment Report
GIR	Ground Investigation Report
HSA	Health and Safety Authority
IÉ	Iarnród Éireann
JPA	Joint Probability Analysis
MCA	Multi Criteria Analysis
MAC	Marine Area Consent
OHLE	Overhead Electrification Equipment
PC	Public Consultation
PSCS	Project Supervisors for the Construction Stage
PSDP	Project Supervisors for the Design Process
PPE	Personal Protection Equipment
RP	Return Period
SID	Strategic Infrastructure Development
SoP	Standard of Protection
UAV	Unmanned Aerial Vehicle
WL	Water Level

1. Introduction and scope

1.1 Project background

The East Coast Railway Infrastructure Protection Projects (ECRIPP) were established by Iarnród Éireann Irish Rail (IÉ) to provide improved coastal protection against predicted climate change effects of sea level rise and coastal erosion on the east coast railway corridor between Merrion Gates (Co. Dublin) and Wicklow Harbour (Co. Wicklow) (Figure 1-1)

ECRIPP aims to deliver improved coastal protection measures to the railway infrastructure, addressing vulnerabilities related to coastal erosion, wave overtopping and cliff instability that are projected to worsen due to climate change. To improve resilience, the project will be designed to withstand against a 1 in 200-year return period event, for a minimum of 50 years (i.e. to year 2075).

This report presents the Phase 3 designs for Whiterock Beach to South Killiney, Coastal Cell Area 2/3 (CCA2/3) (hereafter referred to as the 'Project').

Figure 1-1. ECRIPP Locations

1.2 Project location and description

The Project covers the frontage from Whiterock Beach to South Killiney (Figure 1-2). The frontage comprises a mix of natural hard cliffs to the north, engineered cliffs/embankments fronted by a mixed shingle-sand beach in the central section and natural steep cliffs ranging in height from roughly 6-12m fronted by beach to the south. For the purpose of design this Project is considered in two sections:

- Whiterock (CCA2/3-B), covering the area to the north and south of The Bluff where the existing masonry wall runs along the base of the embankment
- Killiney and South Killiney (CCA2/3-C and D), covering the area of steep cliffs from Killiney Beach car park south to the underpass leading to Seafield Road

This frontage is mainly a rocky outcrop, with a rocky cliffed frontage for the majority and softer cliffs to the south of the frontage. This frontage is typically non-urban with the railway perched at a high level above the coastline. The majority of the frontage are natural cliffs but there are intermittent man-made structures supporting the slopes and the perched railway.

Cliff instability, runoff of water, undermining of the cliffs/slopes by coastal erosion and shore platform lowering are the main hazards. There is evidence of past failures along both frontages, from toe erosion, failures in superficial material and undermining of structures.

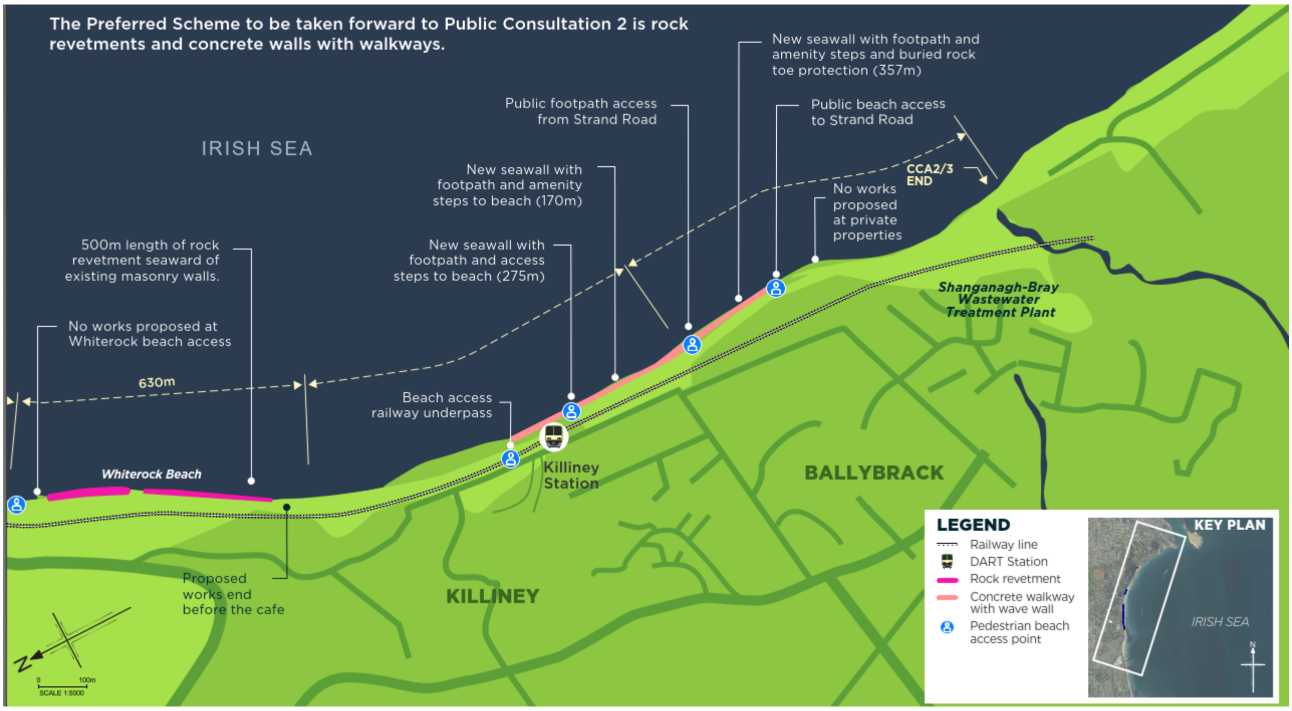

Figure 1-2. Whiterock Beach to South Killiney location plan

1.3 Project objectives

The objectives of engineering interventions for the Project are to manage the risk of cliff instability due to wave action eroding the base of the cliffs.

1.3.1 Transport benefits

The proposed works will ensure that the railway remains safe to operate over the next 50 years. Proposed works will reduce the risk of cliff erosion impacting the railway and preventing significant damage to railway infrastructure under large storms.

IE infrastructure at the site comprises a double-track railway with overhead electrification equipment (OHLE) that forms the electrified DART service that links Dublin and Greystones.

1.4 Project status

The project is currently in Phase 3, the Preliminary Design Stage. By integrating the proposed options (Options Selection Report) with the results of the Public Consultation 1 (Report PC1), a Phase 3 design has been developed, which aims to satisfy stakeholders whilst delivering the design requirements.

The design is likely to be recalibrated, based on further technical and environmental analysis, feasibility studies and stakeholder consultation.

1.5 Purpose of this report

This document provides the Phase 3 Design Report for Whiterock Beach to South Killiney. The report defines the design that will subsequently inform detailed design.

This report should be read in conjunction with associated appendices:

- Phase 3 coastal modelling report (Appendix A) – The coastal modelling report provides details of the coastal modelling analysis undertaken during Phase 3 to further understand potential future beach profile changes along Killiney and to assess any potential impacts of the scheme on waves, currents and sediment transport at Whiterock.
- Coastal processes report. The coastal processes (Appendix B) provides details of the baseline analysis of coastal process along Whiterock and Killiney and a review of potential impacts of the Project on the coastal processes.
- Geotechnical outputs (Appendix C). The ground investigation report (GIR) presents the results of desk studies and ground investigations in an engineering ground model. The document uses the ground model to undertake geotechnical calculations on the stability and settlement potential of the proposed structures. The GIR documents the geotechnical risks arising from the scheme that feed into the designers' risk assessment Appendix D)
- DEHERR, designers' risk assessment (Appendix D). A Design Hazard Elimination & Risk Reduction Register or DEHERR has been developed alongside the design of the preferred option at Phase 3 design. The DEHERR allows the designer to determine potential risks and design (where possible) against the risks presented. Where the risk is not possible to eliminate at this phase of design, further evaluation of the risk will occur at detailed design, before the risk is transferred to the contractor for them to consider when developing their safe system of works.

2. Design criteria and requirements

2.1 Design criteria

The design criteria applicable for all disciplines are summarised in Table 2-1.

Table 2-1. Design criteria

Criteria	Description	Reference
Design Life	▪ 50 years for new permanent works ▪ Variable for existing structures, beaches and soft solutions	Scope of Services
Proposed Standard of Protection – Damage to structures	0.5% AEP (1 in 200RP)	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Proposed Standard of Protection – Reduction of disruption to services	10% AEP (1 in 10RP) for damage to rolling stock / lineside assets 100% AEP (1 in 1RP) for temporary line speed restrictions	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Proposed Standard of Protection – Pedestrian Safety	100% AEP (1 in 1RP)	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Wave overtopping thresholds	Design protection measures to limit wave overtopping to: ▪ 20 l/s/m or 2000 l/m under a 0.5% AEP storm	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002 Note, the limit was increased from 20 to 50l/s/m given this is a cliffed section. Refer to Section 5.2.3.
Maintenance requirements	For new permanent works: zero heavy maintenance for up to 25 years.	Scope of Services

2.2 Design standards

The relevant design standards applicable to the Project are summarised in Table 2-2

Table 2-2. Relevant design standards and codes of practice

Discipline	Code/Standard	Application
Chief Civil Engineer (CCE), IE Requirements	PWY-1101 Requirements for Track and Structures Clearances	Geometrical constraints on proposed solutions, including installation and maintenance
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-389 Drawing Certification Process	All drawings produced on the project
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-399 Glossary of Civil and Permanent Way Engineering Term	All technical reporting relating to railway terminology
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-390 - Preparation of Drawings (Approval and Certification Process)	All drawings produced will follow the general guidelines in this standard. It is noted that as no track works are within scope, many of the specifics in this standard will not be applied.
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-410 - Civil Engineering and Structures Design Standard	Main IE standard for design (alongside Eurocode)
Chief Civil Engineer (CCE), IE Requirements	CCE-STR-PSD-005 - Technical Approval for Civil Engineering Structures	Main IE standard for design reporting
Electrification Manager, IE Requirements	I-ETR-4004 Iss1.0 Clearance Requirements for DC 1500V Electrified Lines	Assessing the hazards arising from the increased height of the sea boundary walls on the DART. Future proofing of DART extension to Wicklow should also be considered
Electrification Manager, IE Requirements	I-ETR-4009 Iss.2.0 Principles of Traction Bonding	Assessing the hazards arising from the increased height of the sea boundary walls on the DART. Future proofing of DART extension to Wicklow should also be considered
Electrification Manager, IE Requirements	I-ETR-4703 Iss1.0 Earthing and Bonding Guidelines	Assessing the hazards arising from the increased height of the sea boundary walls on the DART. Future proofing of DART extension to Wicklow should also be considered
Electrification Manager, IE Requirements	I-ETR-4021 Iss1.0 Maintenance Requirements for the DC 1500V DART Electric Traction System and its Interfaces	Assessing the hazards arising from the increased height of the sea boundary walls on the DART. Future proofing of DART extension to Wicklow should also be considered
Railway Electrification	EN 50162 :- Protection against corrosion by stray current from direct current systems	Electrical safety and installation of modified defences along the electrified railway (DART), including possible extension to Wicklow.
Railway Electrification	EN 50522:- Earthing of power installations exceeding 1 kV AC	Electrical safety and installation of modified defences along the electrified railway (DART), including possible extension to Wicklow.
Railway Electrification	EN 50562:- Railway applications. Fixed installations. Process, protective measures and demonstration of safety for electric traction systems	Electrical safety and installation of modified defences along the electrified railway (DART), including possible extension to Wicklow.

Discipline	Code/Standard	Application
Railway Electrification	EN 50122: Railway applications. Fixed installations. Electrical safety, earthing and the return circuit. Protective provisions against electric shock	Electrical safety and installation of modified defences along the electrified railway (DART), including possible extension to Wicklow.
Structural	EN 1990:2002 Eurocode - Basis of structural design	Principles and Requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability
Structural	EN 1991 Eurocode 1	Provides comprehensive information on all actions that should normally be considered in the design civil engineering works, including some geotechnical aspects.
Structural	EN 1992 Eurocode 2	Applies to the design of civil engineering works in concrete. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design in EN 1990.
Structural	EN 1996 Eurocode 6	Applies to the design of civil engineering works, or parts thereof, in masonry. The execution is covered to the extent that is necessary to indicate the quality of the construction materials and products that should be used and the standard of workmanship on site needed to comply with the assumptions made in the design rules
Structural	BS EN 206-1:2000 Concrete – Part 1: Specification, performance, production and conformity	Additional reference where Eurocode does not cover a specific topic adequately for the design of concrete structures
Geotechnical	Eurocode 7: Geotechnical Design	Default standard for geotechnical design, but may require other supporting documentation e.g. British Standards
Geotechnical	Engineers Ireland Specification and Related Documents for Ground Investigation in Ireland, 2016	For defining approach and content of the Ground Investigation Interpretive Report
Coastal	The Rock Manual: The use of rock in hydraulic engineering (Ciria/CUR/CETMEF, 2007)	Design of rock structures, including armour stability, scour, toe design
Coastal	BS6349 Maritime Works	Design of breakwaters, dredging, geotechnical design and materials used in maritime works
Coastal	Manual on wave overtopping of sea defences and related structures (EurOtop, 2018)	Wave overtopping performance assessment of defences
Coastal	The Coastal Engineering Manual (USACE, 2002)	Additional methods for scour, armour stability, hydrodynamic wave loading
Coastal	The Beach Management Manual (Ciria, 2010)	Design of beach nourishment and management

Discipline	Code/Standard	Application
Coastal	Revetment Systems against Wave Attack (McConnell, 1998)	Design of concrete blockwork and open stone asphalt
Coastal	The Use of Concrete in Maritime Engineering – a guide to good practice (Ciria, 2010)	Design of concrete structures
Coastal	Toe Structures Management Manual (Environment Agency, 2012)	Design of nearshore/offshore structures

2.3 Consideration of alternatives

During public consultation 1 (PC1), there was significant feedback relating to the impact of the Emerging Preferred Scheme on access and amenity use along the frontage, in particular at Whiterock where two large rock revetments were proposed. The main concerns were:

- Revetments preventing beach access around The Bluff
- Swimmer's safety (access, safe egress, emergency recovery, change of currents)
- Impact on surfing amenity, access and loss of sand
- Tourism, community physical and mental health/well-being impacts from visual/access impacts

During Phase 2 all feasible options were considered through a thorough option selection process (7694-CCA2_3-P2-ENG-CV-JAC-0001). At Whiterock the railway line is at risk of failure from the following three failure modes:

- Undermining of the existing masonry wall due to wave action and lower beach levels
- Wave action on the existing masonry wall leading to failure (collapse) of the wall and subsequent failure of the embankment supporting the railway line
- Wave overtopping onto the existing embankment leading to erosion of the slope and subsequent failure of the embankment supporting the railway line.

A solution is therefore required that addresses all three of the issues above.

Following PC1, options previously discounted were revisited to see if there was a feasible alternative that would be more palatable to the public. This took into consideration the following:

- The current wall is not currently providing adequate protection and cannot be repaired to provide future protection to the railway line.
- A rock revetment provides the required protection and is energy absorbing, so reduces further beach lowering and wave reflection.
- Other technically viable options are a new seawall, and various options to stabilize a beach, but the Multi Criteria Analysis (MCA) during Phase 2 ruled these out for various reasons as detailed in the Option Selection Report (7694-CCA2_3-P2-ENG-CV-JAC-0001).
- Alternative beach building options would all severely impact surfing, bring additional swimming risks and may not provide much improvement for access (beach control structures such as groynes on the beach would be required)
- A vertical seawall (existing or new) will increase wave reflections and will result in future beach lowering, both of which will have access impacts and increased beach user risk (walkers, swimmers, surfers etc).
- Due to the steep embankment slope, it is not technically viable to retreat the defence line to create a winder beach.

Rock revetments were therefore still considered to be the preferred solution. However, as discussed in more detail in this report, the size of the revetments has been reduced during Phase 3 to reduce the impacts on access and amenity. Furthermore, additional modelling has been undertaken to assess any potential changes to the sea conditions around Whiterock.

At Killiney, the feedback at PC1 was generally positive in relation to the raised footpath but there were some concerns around access to the beach from the footpath. Whilst the fundamentals of the option have remained, through CFD analysis it has been possible to remove the need for the seaward wall of the footpath and replace this with steps to the beach. Additional access steps and access ramps have also been included.

2.4 Design elements

2.4.1 Rock revetment

A rock revetment will be constructed on the beach in front of the existing structures at Whiterock. The rock revetment will comprise two layers of graded armour rock. The rock grading has been selected to provide stability over the scheme life using modelled wave conditions that allow for sea level rise. The rock grading is expected to be in the range of 6-10 tonnes grading. This rock will be of high quality to ensure that it meets and exceeds the design life. It will be placed over an underlayer of rock on a high-performance geotextile to minimise the risk of fines being lost through the existing structure.

The rock revetment will absorb wave energy, reduce wave run up and overtopping leading to erosion of the embankment.

2.4.2 Concrete seawall

At Killiney beach there are existing walkways along the rear of some sections of the beach. To provide the required Standard of Protection (a 1 in 200-year storm protection level), a new seawall will be constructed. This seawall will have a rear wave wall to protect the toe of the slopes/cliffs supporting the railway line. This seawall will incorporate a raised walkway 3m wide and will provide continuous access along the rear of the beach with access down onto the beach.

2.4.3 Rock toe protection

Along the southern most section of Killiney rock toe protection will be buried in-front of the new seawall to protect the seawall in the event of beach draw down in a storm. This rock armour will also help to reduce wave overtopping and wave action onto the seawall if beach levels lower in the future. This rock armour has been designed to be stable under the design conditions.

2.4.4 Access steps

Continuous access steps are provided from the footpath down to the beach for the northern 250m of the scheme at Killiney. These access steps continue to the predicted year 2075 beach level with additional allowance for scour and beach drawdown. This will ensure that safe access to the beach is maintained, even if the beach levels lower. It is currently assumed that these access steps will be precast units. Additional access steps are also provided in South Killiney where there are existing steps down to the beach. A sheet pile wall is provided at the base of the steps to provide additional support and to allow excavation in front of the steps if a rock toe is needed in the future if beach levels lower.

2.4.5 Amenity Steps

After the first 250m in Killiney the access steps are replaced with amenity steps that continue to the southern extent of the scheme. These amenity steps reduce wave reflections and wave run up leading to wave overtopping as well as providing an amenity function. The steps are 0.75m wide and 0.5m high and assumed to be precast units. These access steps continue to the predicted year 2075 beach level with additional allowance for scour and beach drawdown. A sheet pile wall is provided at the base of the steps to provide additional support and to allow excavation in front of the steps if a rock toe is needed in the future if beach levels lower.

2.4.6 Access ramps

Pedestrian access ramps are provided at the northern and southern extents of Killiney. These ramps extend from the raised footpath down to the year 2075 beach level at a maximum slope of 1 in 12 in accordance with Technical Guidance Document M (Access and Use) or equivalent standard.

2.5 Design assumptions and decisions

The main design assumptions made are related to future beach levels in Killiney and South Killiney. As discussed in Section 3 and Section 4.2, the future beach levels have a large impact on the design conditions and the geometry of the proposed structures but predicting future beach levels is complex and requires a number of assumptions. The design of the structures at Killiney is currently based on the results of the Phased 2 shoreline modelling. However, the Phase 3 coastal modelling indicates that the Phase 2 modelling may have overpredicted the amount of erosion at South Killiney. This means there may be some additional conservatism in the proposed design at South Killiney, specifically in the width of the buried rock toe protection. Additional analysis will be undertaken during detailed design to refine this.

The design assumes precast units are to be used where possible to limit the use of in-situ concrete required on site, however due to the size of units required it may be more feasible to use in-situ concrete. This will be considered further during reference design.

It is assumed that all existing buildings and structures will remain and be incorporated into the works. At the location of the two existing building the proposed works extend seaward to maintain the 3m walkway in front of the buildings.

During Phase 3, a representative cross-section has been analysed for each sub-cell; and therefore, in some locations a single cross-section is representing several hundred metres of frontage with variable wave exposure and existing beach profile. There are still opportunities to further refine the design through detailed design by introducing additional sub-sections to further tailor the design to local variations. In particular it may be possible to extend the section in sub-cell C4 further south and therefore reduce the extent of the larger cross section at sub-cell D1.

2.6 Safety certification and approval

2.6.1 Workplace safety: roles and responsibilities

Workplace safety in construction projects in Ireland follows the Safety, Health and Welfare at Work Act 2005 and the Safety, Health and Welfare at Work (Construction) Regulations 2013. The Safety, Health and Welfare at Work (Construction) Regulations 2013 aim to:

- Prevent accidents on construction sites.
- Define roles and responsibilities of key duty holders in a construction project.
- Ensure proper planning, coordination, and communication of health and safety throughout the construction process.

The 2013 Regulations ensure that health and safety is:

- Considered from the design stage through to completion.
- Managed by competent, clearly assigned roles.
- Proactively monitored and reviewed on all construction projects

Under these regulations, the responsibilities of duty holders are as follows:

Clients must:

- Appoint Project Supervisors for both the Design Process (PSDP) and Construction Stage (PSCS).
- Ensure that the PSDP and PSCS are competent and adequately resourced.
- Keep a copy of the Safety File at the end of the project.

Project Supervisor for the Design Process (PSDP) must:

- Identify hazards during the design stage.
- Coordinate designers to eliminate or reduce risks.
- Ensure early planning and coordination for safety.
- Prepare a Preliminary Health and Safety Plan.
- Maintain and update the Safety File.

Project Supervisor for the Construction Stage (PSCS) must:

- Coordinate health and safety during construction.
- Prepare and implement the Construction Stage Health and Safety Plan.
- Ensure compliance by all contractors.

Designers must:

- Eliminate hazards and reduce risk through design.
- Cooperate with the PSDP.
- Consider health and safety implications of their designs.

Contractors, including subcontractors, must:

- Comply with the Construction Stage Safety Plan.
- Provide relevant training and PPE to workers.
- Coordinate their activities with other contractors.
- Report incidents and cooperate with safety inspections.

2.6.2 Notification and training

Projects lasting more than 30 working days or 500 person-days must be notified to the Health and Safety Authority (HSA) before work begins. The AF1 form is used for this and is the reasonability of the client with the help of the PSDP

In relation to training and competence:

- All workers must have received Safe Pass training.
- Construction workers must be trained in manual handling, working at heights, etc., as applicable.
- Site-specific induction is required.

2.6.3 Iarnród Éireann safety standards

Due to the proximity to the railway line, the safety certification and approvals will be aligned with the process stated in IÉ standards and the general good practices of safety assurance and management.

Based on consultation with the stakeholders of IÉ, it has been confirmed that this project is considered non-significant in accordance with the Common Safety Method Risk Assessment (CSM-RA) and does not require Authorisation to Place in Service (APIS). In addition, the potential work will:

- Have minimal impact on the day-to-day operations and activities of Irish Rail
- Have minimal impact on the operations of trains and rail services.

With respect to this, the technical management standards CCE-TMS-391 (IÉ, 2020) will be generally followed for the safety certification and approvals, and the delivery process will be conducted through the engagement with stakeholders of IÉ.

- The objectives of the safety certification and approval are to ensure:
- The compliance with applicable legal and technical requirements;
- The credible hazards identified, and their impact assessed; and
- Safety associated with the work sufficiently controlled and managed.
- The following will be considered to support the safety certification and approval:
- The detailed definition of the change (i.e. scope of work and activities);
- Project team with the roles and responsibilities defined for project delivery and safety assurance;
- Identification of compliance requirements;
- Identification of potential affected stakeholders;
- Hazards identification and risk assessment to support the identification, assessment, control and management of safety hazards and risks; and
- Gathering evidence of demonstrating these requirements achieved.

3. Numerical modelling and coastal processes

Numerical wave modelling was undertaken during Phase 2 to determine the wave and water level conditions along the frontage and is summarised in Section 3.1. No additional wave modelling was required during Phase 3.

Shoreline evolution modelling undertaken in Phase 2 showed significant erosion in South Killiney which was driving the design in that location. However, this modelling did not account for the impact of the nearshore rock reefs which can impact wave breaking and amount of sediment transport. Additionally, PC1 feedback highlighted concerns of potential changes in the currents around Whiterock that might impact swimmers and surfers as well as concerns that the revetments might result in erosion of Whiterock beach due to loss of sediment into the Bay.

To further consider the above, a Coastal Area Model (CAM) was setup to undertake additional modelling to inform the design development during Phase 3. Further details are provided in Section 3.2 to 3.4 and Appendix A

Alongside the numerical modelling, Jacobs have undertaken an Expert Geomorphological Assessment (EGA) to assess the baseline coastal processes along Whiterock to South Killiney to inform the design of the structures and to assess any potential impacts of the Project on the coastal processes.

3.1 Wave modelling

A two-dimensional spectral wave model was used to derive multi-decadal hourly time series of nearshore wave data and extreme nearshore wave and water level conditions along the East Coast of Ireland. The model includes the effects of spatially varying water levels, wind forcing, spatially varying boundary data and climate change. The model was calibrated and validated using measured nearshore wave data in the Dublin Bay. The average RMSE (Root mean Square Error) for wave height (Hm0) over the storm conditions selected for calibration is about 0.2m with a bias of about 0.0m.

Hourly time series of nearshore wave data were extracted at regular intervals at approximately every 1 km. The nearshore wave roses show that waves are from ESE to SSE along this section of coast. More waves approach the coast from south of the shore normal than from north. To the south of CCA2-3 waves from the SE and ESE decrease in height. To the north of CCA2-3 waves turn clockwise to be dominantly from SE with increasing waves from the SSE towards the northern end. This change in wave direction is as a direct result of a change in orientation of the shoreline

The hourly wave height exceeded 1% of the year is approximately 1.8m (1.70m to 1.94m) and the median annual wave height is approximately 0.27m (0.23m to 0.32m) for present day wave conditions (wave climate simulated for the period Jan 1988 to Dec 2021). The modelled wave heights for future conditions (including climate change) are higher. The hourly wave height exceeded 1% of the year is about 1.88m (1.77m to 2.01m) for 2022-2055 and 2.00m (1.88m to 2.16m) for 2056-2100. The modelled hourly nearshore wave time series is used as input for the sediment transport and shoreline evolution modelling.

Joint probability analysis was carried out to determine extreme offshore wave and water level conditions for 22.5-degree wave direction sectors. Two joint probability analysis methods were used, namely: 1) Desk study method which uses correlation coefficients to determine the dependence of the two variables (wave height and water level); 2) The simplified method which considers astronomical tide are fully independent from the wave height while surge is considered fully dependent to wave height. The results that give the more conservative joint probability pairs are used as boundary conditions for the nearshore transformation modelling. The selected joint probability pairs were transformed to the nearshore using the wave model.

3.2 Shoreline evolution modelling (Killiney)

The Phase 3 CAM model used detailed simulations of local hydrodynamics and sediment transport processes to estimate future beach volume changes over time in the ‘baseline’ case, i.e. no changes to the coastal structures but accounting for future climate change projections.

The results indicated that for both current conditions and with future climate change, there will generally be a natural tendency for net beach erosion at South Killiney along the extent of Strand Road, which could result in approximately 10m landwards beach recession by 2075 within the Project area.

North of Strand Road to Killiney Station, the modelling indicated a net trend of beach accretion in the medium-long term, which could result in ~30-40m widening of the beach here by 2075 (Jacobs, 2025a). These modelling results suggest a tendency for the shoreline to straighten in future, with the 'ness' of sediment currently present near the southern end of Strand Road moving northwards to feed the beach further north.

These results have been used to inform the design of the proposed structures at Killiney (See Section 4.2.2).

3.3 Sediment transport modelling (Whiterock)

The proposed rock revetments will locally decrease the area of upper beach across which sediment can be transported alongshore a few hours either side of high tide. This has the potential to reduce the alongshore sediment feed to Whiterock Beach. Initial modelling of sediment transport in Phase 3 suggested that this reduction in northwards sediment transport caused by the revetment could be in the order of 20-30%. Although the new rock revetment would cover a relatively small proportion of the littoral zone for a limited duration a few hours either of high tide, the estimated magnitude of this reduction is because modelled longshore transport rates are highest on the upper beach where the new revetment would sit, due to the steeper beach slope and greater sediment depths landward of the natural rock platform.

However the magnitude of this reduction is considered likely to be an overestimation, and the effect of the revetment on sediment transport northwards to Whiterock Beach is considered likely to be smaller than this modelling suggests because:

- the modelling does not account for the presence of the existing rock revetment on the upper beach here, which will limit sediment availability and transport under the existing conditions of the baseline case, particularly under storm conditions when beach drawdown exposes more of this rock.
- the modelling does not account for the observed decrease in sediment size at Whiterock compared to North Killiney, which suggests that coarse gravel is not readily transported to Whiterock Beach, even under high energy south-easterly wave conditions. The sediment reaching Whiterock beach tends to be finer gravels and sands, which is more prevalent on the lower profile at North Killiney, which will be unaffected by the rock revetment. The proposed works indicate there would still be at least 10-30m width of mobile beach between the revetment toe and the natural rock platform across which longshore transport of these finer gravels can continue towards Whiterock beach.
- the modelling does not directly account for onshore sediment feed onto Whiterock Beach from the nearshore sand and/or gravel banks during ambient 'beach building' wave conditions, which would be unaffected by the new rock revetment.
- the modelling does not include cross-shore sediment transport processes however; this may mean that this modelling underestimates the influence of storm-driven drawdown of beach material to the lower beach.

Further modelling for detailed design will aim to address these factors to refine the estimated impact of the new revetment on sediment transport to Whiterock Beach, and any potential implications of this on beach widths.

3.4 Impact on waves and currents (Whiterock)

To the north of the proposed revetments, where there is an existing tendency for a rip current to form during certain wave conditions, the Phase 3 coastal modelling indicates that the new revetment may cause a ~20-30m northwards shift in the location of this current at high tide. However, it would not increase the maximum speed of the rip current generated under the worst-case wave and tide conditions. Instead, the modelling suggests there could be a minor decrease (<5%) in the maximum speed of this rip current from ~0.48m/s to 0.46m/s.

Regarding waves further seaward in the surf zone off Whiterock Beach, the Phase 3 coastal modelling indicates that any changes in sediment transport and deposition due to the change in rip current location discussed above would have a minor change (<0.1m) in wave heights in the surf zone and no significant impact on wave breaking location under all wave conditions modelled.

Regarding wave energy reaching the shoreline, the new revetment will locally enhance wave energy dissipation at the top of the foreshore when water reaches the structure at high tide (as it is designed to, to protect land/structures behind) due to the sloped angle of the revetment, high surface roughness of the rock and the gaps between the rock.

4. Design methodology and results

4.1 Design methodology and overall approach

All proposed structures are designed to a minimum of 1 in 200-year return period for the year 2075 (incorporating 50-yrs of predicted sea level rise). The overall design approach is summarised below and in Figure 4-1.

The waves were transformed to the proposed structure toe using the closest wave point to the structures, the bed level at the toe of the structure and nearshore slope (determined using a combination of UAV survey data, bathymetric data and recent LIDAR data). Offshore Joint Probability Analysis (JPA) used in the wave transformation was determined based on shoreline orientation and the wave direction. Shore-normal waves were used in all cases unless an obliquity either side of the shore-normal wave conditions provided a significantly larger wave.

The nearshore waves were used to inform empirical equations to determine the optimal rock sizing for revetment stability under the wave conditions, as well as to calculate potential scour depths. Computational Fluid Dynamics (CFD) modelling was used to analyse wave overtopping to determine crest level for the rock revetments at Whiterock and the footpath and wave wall heights at Killiney. CFD was also used to determine wave loading forces acting on the walls to inform the structural design of the wave walls. By combining these various analytical approaches, from empirical calculations to CFD we were able to develop a comprehensive understanding of the coastal dynamics at play. This holistic approach informed every aspect of the structure's geometry, resulting in a design that effectively balances coastal protection, structural integrity, and environmental considerations.

The cross-section was then analysed by the geotechnical team and any further changes to the geometry to satisfy the bearing or global stability checks were made.

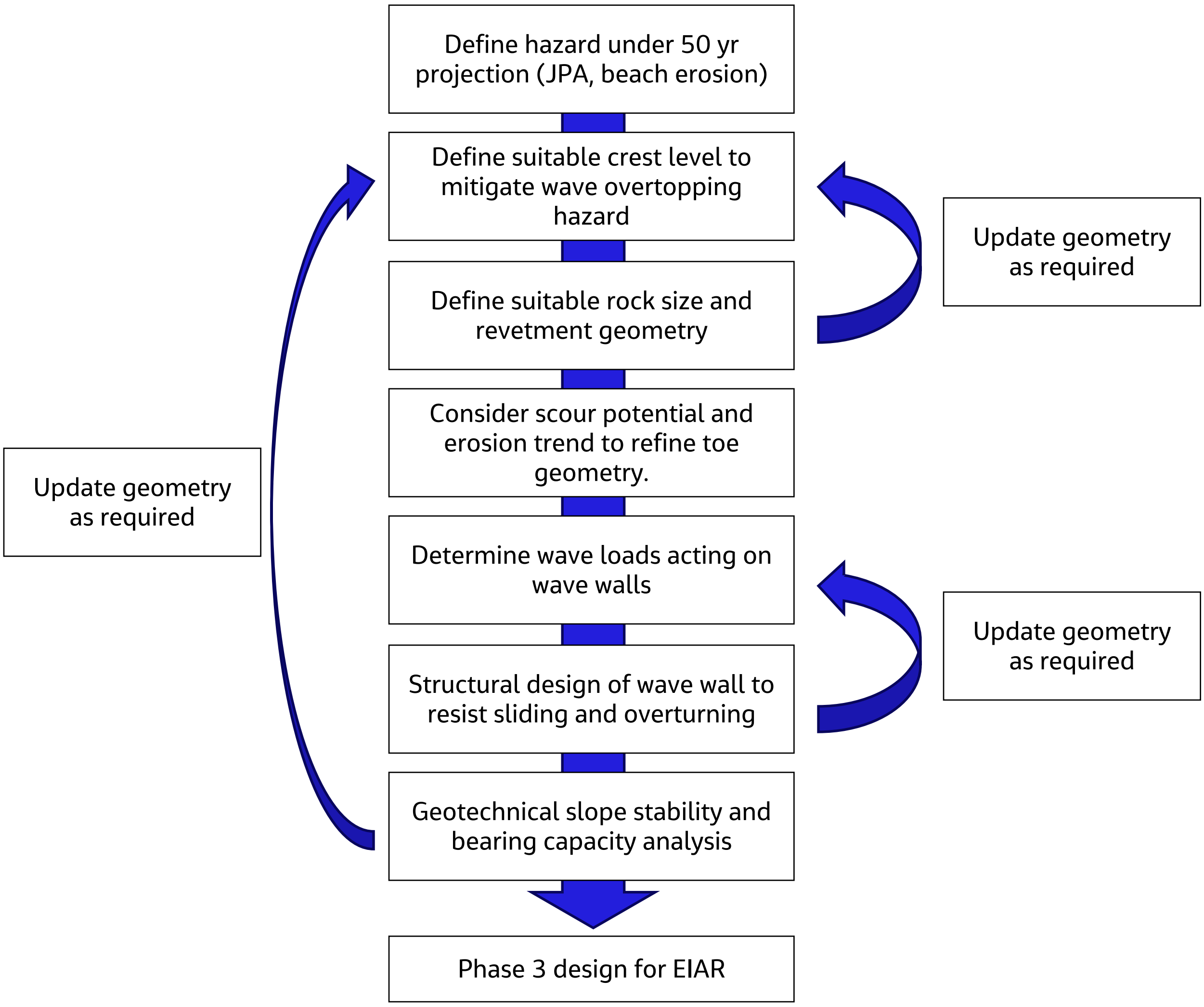

Figure 4-1 Design methodology

4.1.1 Key design parameters

All structures proposed shall be designed to recognised and proven current codes, standards, or regulations. Key design assumption used in the design of CCA2/3 are shown in Table 4-1.

Table 4-1. Key Design Parameters

Design Assumption	Value	Reference
Rock Density	2650kg/m^3	Typical values
Water Density	1025kg/m^3	Typical values
Storm Duration	6 Hours	Typical values
Coefficient of Gravity	9.81m/s^2	Typical values

Design Assumption	Value	Reference
Plunging Coefficient	6.2	CIRIA C683 (2007) Van Der Meer and Van Gent assessments
Surging Coefficient	1.0	CIRIA C683 (2007) Van Der Meer and Van Gent assessments
Nominal Permeability	0.1	CIRIA C683 (2007) Van Der Meer and Van Gent assessments
Nominal Permeability	0.3	Eldrup et al. (2019) for Van Der Meer Sensitivity
Wave Obliquity	0 degrees	Assumed based on selected wave conditions for worst case results
Damage Number (start of damage)	2	CIRIA C683 (2007) Van Der Meer and Van Gent assessments

4.1.2 Tide levels

Reference tide levels for Dublin North Wall and Wicklow are tabulated below.

Table 4-2. Reference tide levels

Reference level	Dublin North wall (mODM)	Wicklow (mODM)
Highest Astronomical Tide, HAT	1.99	
Mean High Water Springs, MHWS	1.59	0.19
Mean High Water Neaps, MHWN	0.89	-0.21
Mean Sea Level, MSL	-0.11	
Mean Low Water Neaps, MLWN	-1.01	-1.41
Mean Low Water Springs, MLWS	-1.81	-1.81
Lowest Astronomical Tide, LAT	-2.61	

4.2 Coastal engineering design

As noted above. the coastal design of the proposed defences at Whiterock and Killiney were undertaken using a combination of CFD analysis and empirical design equations.

CFD allows more detailed analysis as the actual structure geometry can be modelled whereas empirical equations rely on a number of assumptions to be made and for the structure to ‘fit’ with the assumptions used to develop the equations. Empirical equations were used to determine the required rock sizing and scour estimates as these cannot yet be undertaken with CFD.

4.2.1 Cross section locations

During Phase 2 the Project frontage was split into sub-cells based on the variation in physical characteristics, including the geomorphology, shoreline topography and orientation, environmental constraints, and existing defence type and the exposure due to different failure modes. The proposed works at Whiterock are split into two sub-cells whereas the works at Killiney are split into three sub-cells.

For Phase 3, one typical cross section has been designed for each sub-cell. The location of these cross sections was selected by analysing the existing beach profiles, predicted future shoreline position and existing structures along each sub-cell to select the most onerous location. The cross sections analysed were:

- B2 – Whiterock north of The Bluff
- B4 – Whiterock south of The Bluff
- C3 – Killiney, south of the existing steps and ramp
- C4 – Killiney – north of the Water Safety Ireland building
- D1 – South Killiney near the underpass to Seafield Road.

4.2.2 Structure toe levels

Understanding the beach levels in front of the proposed structures is important because it has a direct impact on the wave conditions and therefore the wave overtopping and wave loading onto the structures. Additionally, the structures need to be designed to ensure that they are not undermined in the future if the beach levels lower, similarly access from the structures to the beach in the future needs to be maintained. The following need to be considered to determine the level of the toe of the structures:

- Long term changes in beach levels due to sediment transport and shoreline evolution
- Changes in the beach profile due to cross shore losses during storm events
- Localised scour at the toe of the defences due to the interaction between waves and the defences

Each of the above were analysed to determine the beach levels at the toe of the structure to be used in the analysis and to inform the geometry of the defences.

4.2.2.1 Long term shoreline changes

As discussed in Section 3.2, the shoreline modelling has indicated erosion of approximately 10m over the 50-year design life in Southern Killiney. This has been translated to a change in beach level at the toe of the structure by ‘shifting’ the existing beach profile landward by 10m as shown in Figure 4-2.

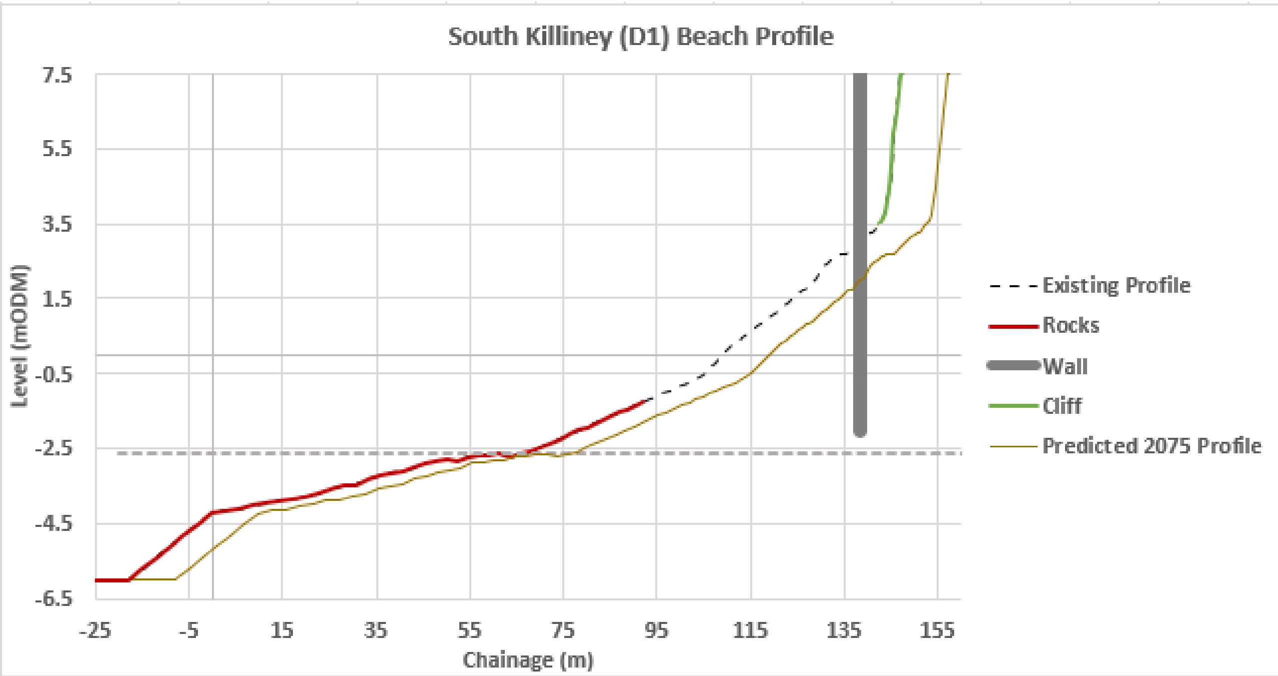

Figure 4-2. Present day and year 2075 beach profiles at South Killiney

4.2.2.2 Cross-shore modelling

The results of the CAM model for South Killiney do not take cross shore losses into account. Therefore, cross shore modelling was undertaken using open-source XBeach-G which is an extension of the main XBeach model and is used to simulate storm impacts on gravel beaches. This was undertaken for the 1 in 200-year storm event for present day, year 25 (2050) and year 50 (2075) to provide an estimate of the change in the beach profile following a storm. This was undertaken at three cross sections along the Killiney section.

The results of the analysis are provided in Table 4-3 and shown in Figure 4-3, Figure 4-4 and Figure 4-5. These results were used to inform the scour analysis.

Table 4-3. Maximum drawdown for a 1 in 200 yr RP for the Year 2075

Cross section	Maximum beach drawdown (m)
C3	0.95
C4	0.61
D1	0.75

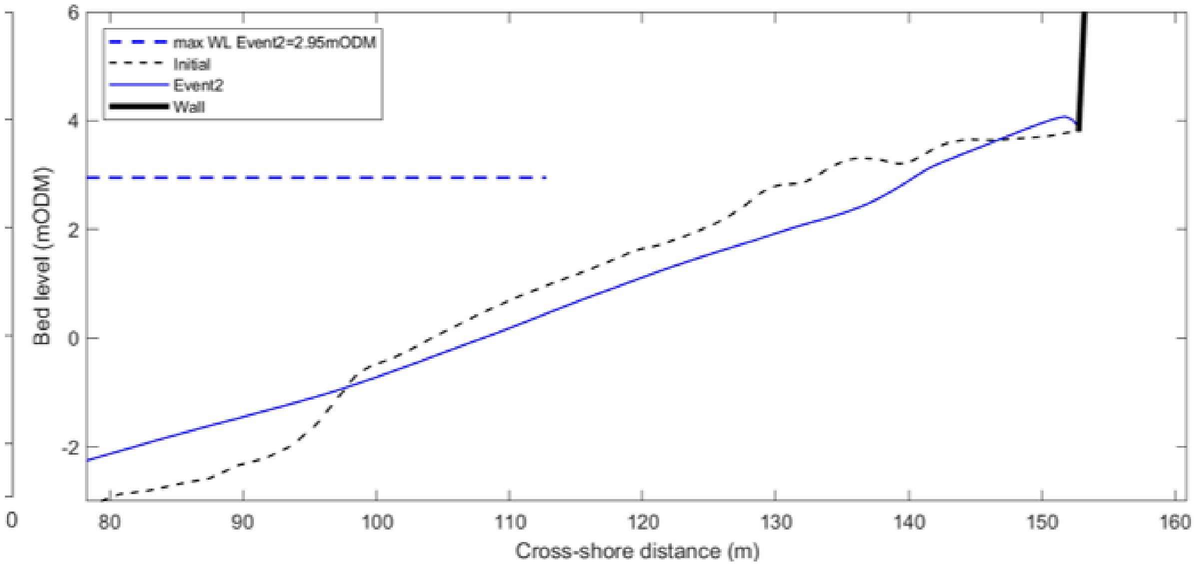

Figure 4-3. Beach Drawdown and shoreline evolution effect on the profile of C3

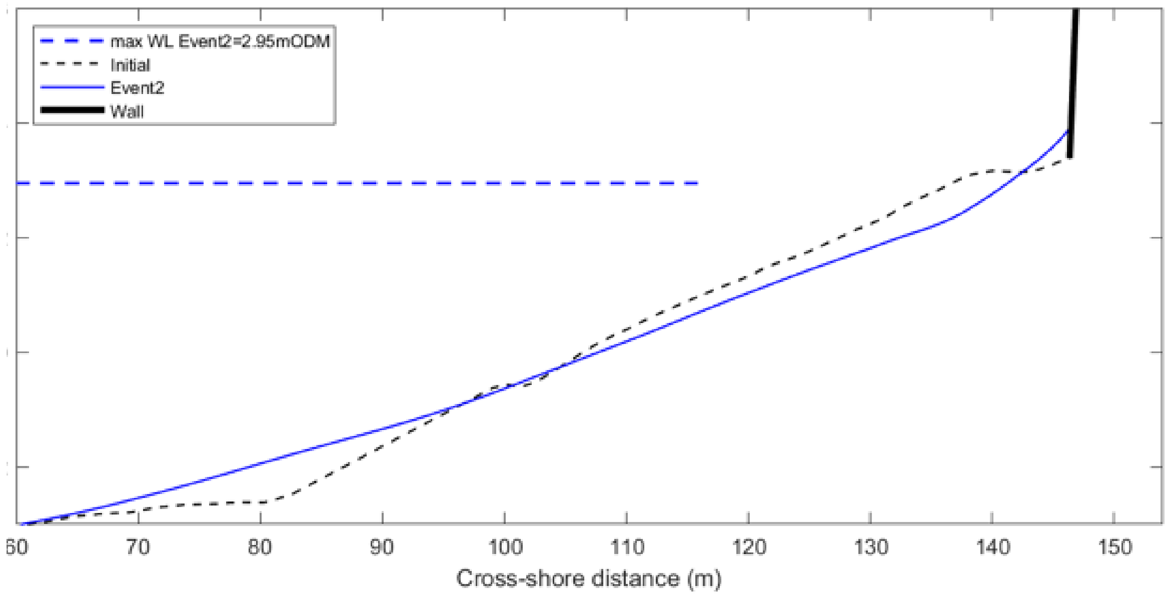

Figure 4-4. Beach Drawdown and shoreline evolution effect on the profile of C4

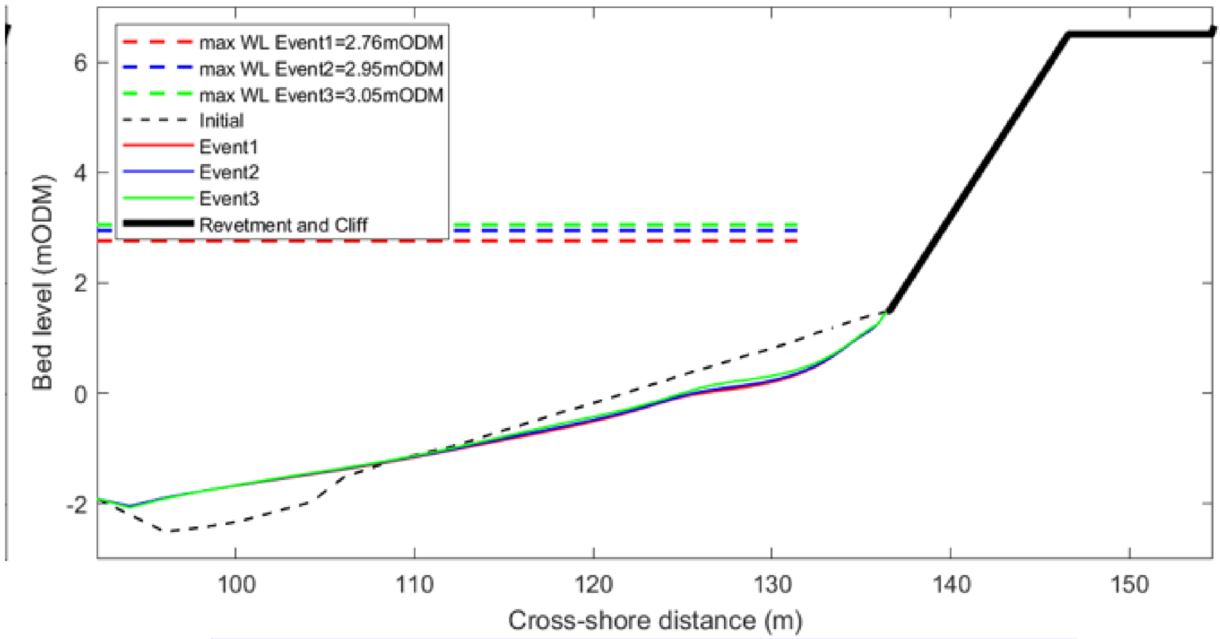

Figure 4-5. Beach Drawdown and shoreline evolution effect on the profile of D1

4.2.2.3 Scour analysis

Analysis was undertaken to estimate the potential scour depth at the toe of proposed structures. The calculated scour depths calculated are detailed in Table 4-4.

Table 4-4. Estimated scour depths

Location	Estimated Scour Depth (m)	Scour Width (m)
CCA2/3 C3	0.2	0.6
CCA2/3 C4	1.4	4.1
CCA2/3 D1	0.7	2.0

4.2.3 Rock armour sizing

The sizing of the armour has been based on the wave action of 200-year Return Period (RP) waves in accordance with the Van der Meer (1988) for non or marginally overtopped structures and Van Gent et al. (2004) as presented in The Rock Manual (CIRIA C638, 2007).

The rock revetment slope at Whiterock has been steepened from a 1 in 2 slope during Phase 2 to a 1 in 1.5 slope in Phase 3. The stability, and therefore required size of rock armour is influenced by the following factors in addition to the wave conditions:

- Structure slope – rock armour is more stable when placed at a flatter slope
- Structure permeability – rock armour is more stable if the structure is more permeable
- Damage number – the required rock size can be reduced if a higher damage number is accepted.

Sensitivity analysis was undertaken on the notional permeability value of the rock revetment, and review of the paper by Eldrup et al. (2019) indicated the permeability could be increased from 0.1 to 0.3 due to the inclusion of an underlayer. This allowed the slope to be steepened to 1 in 1.5 whilst maintaining a damage number of 2, in accordance with the design criteria, for 6-10t rock. If the foreshore in front of the revetment lowers significantly in the future, then the damage number would increase and additional maintenance may be required after storm events, however it is not expected that the foreshore will lower to these levels within the 50-year design life.

The rock armour for the buried rock toe at South Killiney has also been sized using the Van Der Meer shallow water equation. This rock will be placed on a very flat slope, for the purposes of the calculations a slope of 1 in 4 has been assumed.

Table 4-5. Rock armour sizing results

Location	Primary Armour Sizing (t)	Slope (1:X)
CCA2/3 B2 (Whiterock North)	6-10	1.5
CCA2/3 B4 (Whiterock South)	6-10	1.5
CCA2/3 D1 (south Killiney)	3-6	4

In order to improve the stability of the rock round heads of the revetment, the slopes are typically flattened. In the case of B2 and B4 where the interface occurs at the beach crest it has been deemed that they are at a sufficiently high level that the wave reaching the interface will be significantly smaller than that at the trunk of the revetment. It was therefore deemed unnecessary to change the cross section of the revetment for the roundhead.

4.2.4 Wave overtopping assessment

Wave overtopping analysis was undertaken using CFD to define the crest levels of the revetment and height of the footpath and wave walls.

4.2.4.1 Wave overtopping thresholds

Wave overtopping thresholds for The Project were reviewed and proposed in Phase 1 in technical note 7694-ZZ-P1-MMO-CV-JAC-0002, these thresholds are summarised in Table 4-6.

Table 4-6. Summary of wave overtopping thresholds (7694-ZZ-P1-MMO-CV-JAC-0002)

Threshold and Application	Justification
5 to 20 l/s/m depending on incident wave height or 2000 l/m under 0.5% AEP conditions for defence structural integrity	Table 3.3 from EurOtop; depending on wave height at the defence suggests that the railway can tolerate a reasonable discharge as long as waves are not too large.
1 to 2 l/s/m or 1000 l/m under 10% AEP conditions for damage to line-side assets, buildings or rolling stock	Table 3.2 from EurOtop; damage to equipment set back 5-10m and the Network Rail single-line operation (where the line closest to sea is closed) suggest that this is pragmatic choice.
0.5 l/s/m or 1000 l/m under 100% AEP conditions for areas beyond platforms, resulting in need for line speed restrictions.	Network Rail amber threshold resulting in line speed restrictions.
0.3 to 1 l/s/m depending on incident wave height or 600 l/m under 100% AEP conditions for pedestrian safety; we would propose to apply this along platforms and any other areas where public access is currently present.	Table 3.3 from EurOtop; depending on wave height at the defence suggests that these are appropriate thresholds for pedestrians. We consider that this would be suitable for platforms and ensure that a pedestrian would not be knocked off their feet and onto the tracks.

At Whiterock and Killiney, the railway runs along the top of the cliffs and therefore the overtopping risk is not directly onto the railway but from erosion of the Cliffs. Therefore, the proposed limit of 5 to 20 l/s/m for the 1 in 200 RP (0.5% AEP) was reviewed. Van Der Meer (2009) studied the resistance of inner slopes of dykes to overtopping, and it was concluded that a well-maintained grass revetment can sustain overtopping without large deterioration between 20 and 50l/s/m. The Van Der Meer paper does not provide any thresholds for Vmax. Therefore, an overtopping threshold of 50l/s/m was adopted for the 1 in 200 (0.5% AEP) event.

At Killiney, a limit of 1 l/s/m and 600l/m was adopted for the overtopping onto the walkway for a 1 in 1 event to ensure the safety of pedestrians using the walkway.

4.2.4.2 Cross sections analysed for wave overtopping

At Whiterock, the width and profile of the existing beach vary, which can influence wave overtopping. To account for this, two cross-section locations were selected to represent both the narrower and wider sections of the beach. Additionally, the height of the existing wall changes along the frontage, and the proposed design includes constructing a rock revetment in front of this wall. As a result, wall height becomes a key factor in the overtopping analysis. To reflect both the variation in beach profiles and wall elevations, overtopping assessments were carried out for two cross sections and two different wall heights.

At Killiney, two cross section locations were selected for the overtopping analysis. The northern part of Killiney Beach from C3 to C4 has a relatively wide and stable beach that is not predicted to vary significantly over the next 50 years. Therefore, for the overtopping analysis one cross section was considered sufficient to cover this area. The cross section selected was in C4 which is a slightly narrower beach than C3 and therefore conservative. At southern Killiney (D1) the beach becomes a lot narrower, and this location is predicted to erode over the next 50 years. The cross section location selected here was at the most southern part of this subcell where the most erosion is predicted to occur, providing a conservative location for the whole of the subcell. Due to the potential future changes in the beach level along Killiney, the overtopping analysis has been undertaken for the present-day beach profile as well as the predicted year 2075 beach profile.

Figure 4-6. Location of the section modelled for overtopping

4.2.4.3 Wave overtopping results

At Whiterock, wave overtopping results were obtained at the rear of the existing wall. Wave overtopping analysis was undertaken for the 1 in 200-year event in year 2075 for the proposed design. Figure 4-7 shows an extract of the CFD model setup for Whiterock.

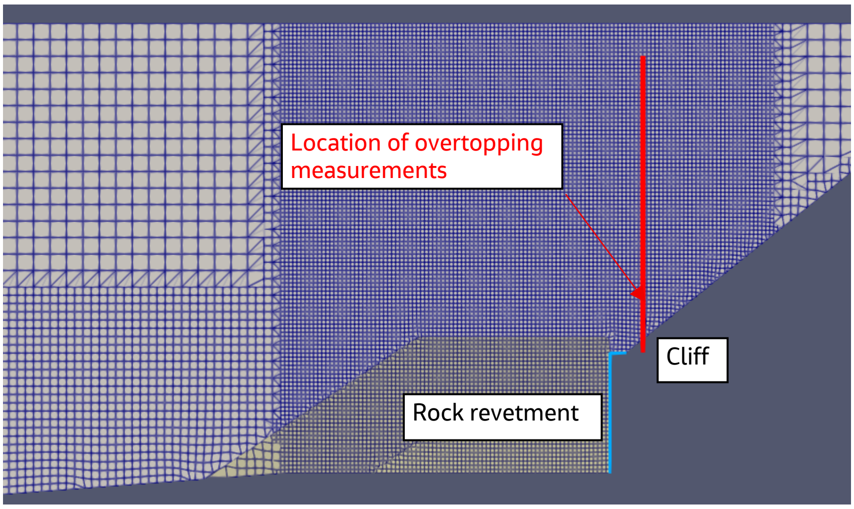

Figure 4-7. Example of CFD model setup up for wave overtopping at Whiterock

As mentioned, the existing wall height varies along Whiterock. The proposed design does not include any raising of the existing wall and therefore the height of the revetment rock armour should not be significantly higher than the existing wall as there would be no support provided to the rock armour. A typical rock revetment has a crest width of three rocks to provide stability in the crest. A wider rock crest can provide a reduction in overtopping as more wave energy is dissipated before reaching the back of the revetment. At the lower sections of the wall the overtopping thresholds were exceeded under the three-rock crest, therefore the crest width was increased to four rocks (approximately 1.5m extra). The revetment crest height was also set at +5.5mODM which is approximately half a rock above the lowest section of the existing wall. This additional height provides extra protection against overtopping without requiring any additional support behind the rock armour. Along the section where the existing wall is higher (up to +7.2mODM) the revetment crest level has been maintained at +5.5mODM but the crest width has been reduced to three rock wide to minimise the impact on the beach. Table 4-7 provides the results of the overtopping analysis at Whiterock.

Table 4-7. Summary of overtopping results at Whiterock

Cross Section	Existing wall crest Level	Proposed revetment crest width	Proposed revetment crest level	Overtopping at wall	
				Mean overtopping rate (Qmean) (l/s/m)	Maximum overtopping volume (Vmax) (l/m)
Narrow Beach	+5.0mODM	5.8m (4 rocks)	+5.5mODM	16	9,000
	+6.8mODM	4.3m (3 rocks)	+5.5mODM	2	3,500
	+7.2mODM	4.3m (3 rocks)	+5.5mODM	1	2,500
Wider Beach	+5.0mODM	5.8m (4 rocks)	+5.5mODM	18	11,800

From the results above it can be seen that the overtopping rates are comfortably within the threshold for Qmean. Reducing the revetment crest level to +5.0mODM resulted in a mean overtopping rate of 50l/s/m at the lower wall section. Whilst this is within the allowable tolerance, the maximum overtopping volume exceeded 15,000l/m and whilst no formal thresholds have been set for the maximum volumes, 15,000l/m was not considered acceptable, this would also result in increased wave loading on the existing wall. Therefore, a crest level of +5.5mODM is proposed.

At Killiney the overtopping was measured at the front of the walkway to determine the overtopping onto the walkway and at the back of the rear wall to understand the overtopping onto the cliffs. Due to the dynamic nature of the beach at Killiney and potential changes in the future beach levels the overtopping was assessed for present day and year 2075 conditions for the 1 in 1 and 1 in 200-year events.

The results of the overtopping analysis are presented in Table 4-8

Table 4-8. Summary of overtopping results at Killiney

Cross Section	Beach Profile	Return Period	Onto walkway		Over rear wall	
			Qmean (l/s/m)	Vmax (l/m)	Qmean (l/s/m)	Vmax (l/m)
C4 – Central Killiney	2075	200RP	63	9,700	5	3,400
	2075	1RP	1	240	0	0
D1 – South Killiney	2075	200RP	360	17,000	102	12,000
	Existing	200RP	133	13,300	13	6,300
	2075	1RP	64	3,000	4	1,000
	Existing	1RP	2	1,300	3	50

The wave overtopping analysis indicates that the threshold limits for overtopping are exceeded under the projected 2075 conditions at subcell D1. However, it is important to note that this assessment was based on the predicted 2075 beach profile derived from the Phase 2 modelling, which assumed a beach retreat of approximately 35 metres.

Subsequent Phase 3 modelling has refined these predictions, indicating that the extent of beach erosion by 2075 is likely to be significantly less, approximately 10 metres. As a result, the overtopping rates presented here are considered to be very conservative and likely overestimate the actual overtopping risk.

Given the revised erosion projections, the overtopping performance is expected to be more closely aligned with the ‘existing beach’ scenario. Further analysis will be undertaken during the detailed design stage to refine these estimates and ensure the design remains robust under future conditions.

4.2.5 Wave loading assessment

At Killiney, wave walls are included at the rear of the footpath to manage the risk of wave overtopping eroding the cliffs. Wave loads onto the walls have been determined using CFD.

4.3 Structural design

The proposed works at Killiney feature concrete structures positioned directly seaward of the existing cliffs. The structural design approach focuses on withstanding wave-induced pressures on these new elements, which serve to protect the cliffs. The design also aims to optimise material use, particularly by minimising the quantity of concrete and other construction materials. A structural assessment of the concrete revetment confirms its overall stability, including checks for sliding and overturning failure, as well as verification that it can be adequately reinforced.

The primary function of the concrete revetment is to protect the cliffs from ongoing erosion caused by storm wave action. A secondary function is to provide a pedestrian walkway along the beach, with occasional vehicle access for maintenance and other operational needs. At certain locations along the seaward side, access steps and ramps have been integrated to enable public access to and from the beach.

Wave forces were assessed using CFD analysis. The CFD modelling generated time series data for both horizontal and vertical wave forces under worst-case Joint Probability Analysis (JPA) conditions. To facilitate the structural design of individual elements and to better understand the critical loading location on the structure, the revetment was divided into multiple components. The individual force time series were then combined to produce an overall force time series for the entire structure, which was used to undertake stability checks, including sliding and overturning assessments. These analyses will be further refined during the detailed design phase.

To ensure compliance with Eurocode requirements and achieve efficient structural utilisation under the anticipated wave pressures, the preliminary design has been configured accordingly. In addition, the following factors were considered during the design process, where applicable:

- The design of access ramp is in accordance with Part M of the Building Regulations and BS 8300.
- Retaining wall analysis has been undertaken in accordance with Eurocodes and BS 6349-2:2010.
- Wave loading will need to be considered at access ramp points

Specific issues to be considered in the detailed design:

- Settlement induced by construction and future maintenance.
- Wave run up through access points.

A range of sequential construction scenarios will need to be evaluated to ensure that units can be cast, stored, lifted, and installed without compromising the stability of the slope or the integrity of individual units prior to completion. A summary of the various design conditions will be agreed upon before the commencement of detailed design and will be reviewed for constructability constraints, such as lifting limits and handling requirements.

The layout and geometry of the structure will be further optimised during the detailed design phase, informed by refined wave modelling and analysis.

4.3.1 Geometry and composition

4.3.1.1 Killiney

The central part of Killiney comprises a concrete revetment constructed on a layer of concrete or granular blinding, placed directly over the existing beach surface. This can be seen in Figure 4-8. The elevation of the top step and rear wall has been determined based on overtopping thresholds identified in the CFD study. A sheet-piled toe wall is positioned at the front of the revetment to enhance passive resistance, ensuring adequate sliding stability. Additionally, it acts as a cut-off to reduce uplift forces from wave action and provides protection against potential undermining due to beach erosion during storm events.

The stepped profile of the front face introduces a time lag in the impact of wave forces compared to a vertical wall. This staggered interaction reduces the peak global sliding load acting on the revetment. By incrementally dissipating wave energy across each step, the structure gains improved stability and places less demand on the sheet-piled toe.

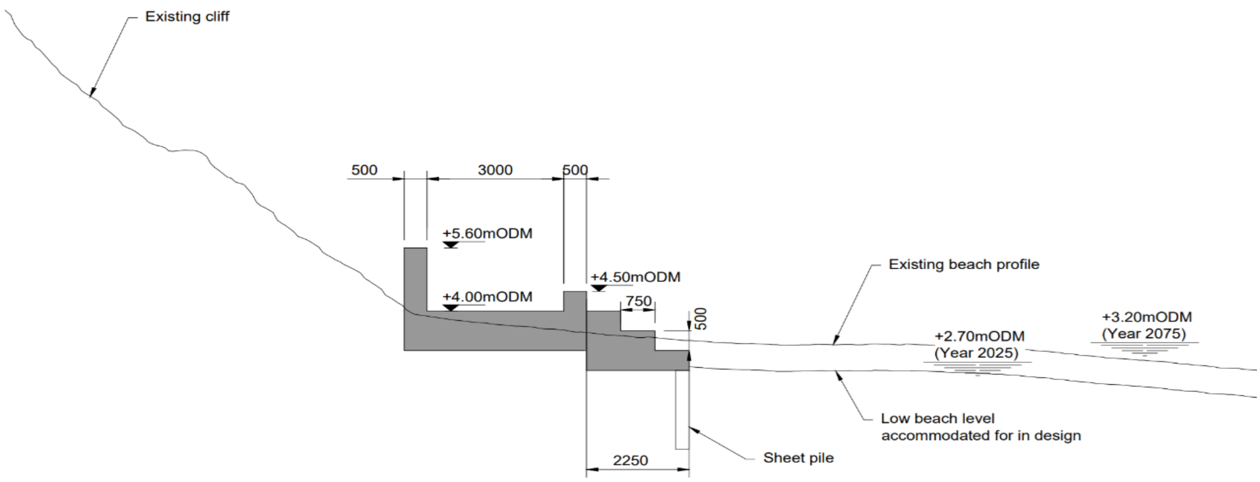

Figure 4-8. CCA2/3 C4 cross-section.

A 3.0 m wide footpath is included for both pedestrian and light vehicle access. The anticipated design vehicle for use of the footpath and ramps will be a small road going maintenance vehicles. Amenity steps are included in the layout to provide public access and seating. The access ramp will interface with the foot of the cliff which runs down to the beach immediately landward.

4.3.1.2 South Killiney

In southern Killiney the section has a similar geometry to central Killiney, however there is an addition of a buried rock toe in front of the sheet pile. This can be seen in Figure 4-9. The elevation of the top step and rear wall has been determined based on overtopping thresholds identified in the CFD study.

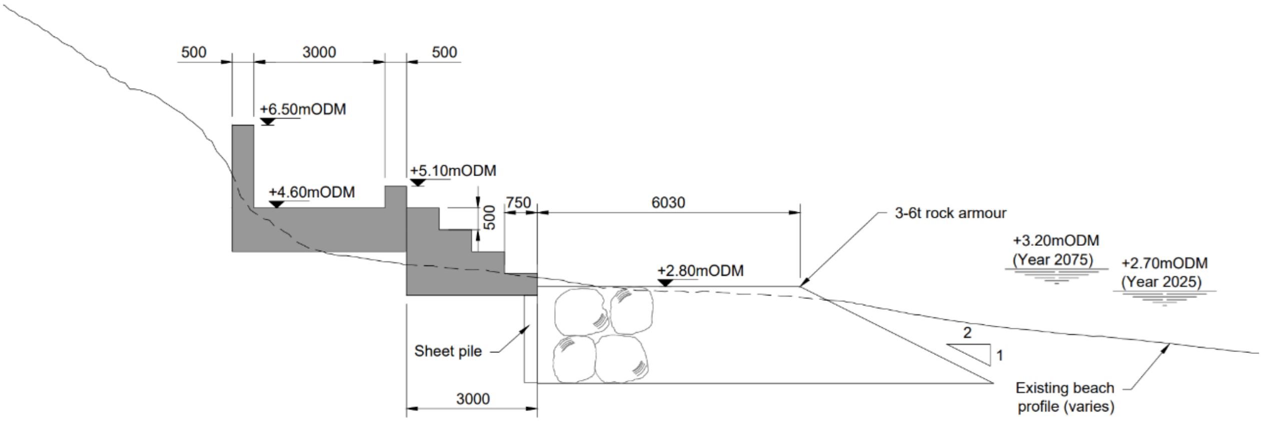

Figure 4-9. CCA2/3 D1 cross-section.

4.3.2 Buildability

Due to the location and limited working window, it is envisaged that the concrete structures will be constructed predominantly as precast concrete units. The size and length of the units will be agreed and set to lifting limits set by the Contractor. Therefore, precast units will be connected, either by in-situ concrete sections or other mechanical means, to minimise the risk of lateral movement due to wave action or vertical movement due to settlement. Movement and construction joints are to be placed at regular intervals to allow the structure to resist to effects of thermal actions and other sources of movement.

4.4 Geotechnical analysis

The site lies in Killiney Bay which is bound to the north by the granite hills of Killiney and Dalkey (Leinster granites), with superficial glacial material present along the entirety of the site to the southern extent at the Shanganagh-Bray Wastewater Treatment Plant. The foreshore consists of a shore platform cut in glacial material which is overlain by beach deposits that are highly variable in nature based on the seasonality of the wave and tide forces they are subjected to. The beach material consists of blown sands over a mix of sand and gravels. Coarse gravels and cobbles are deposited on the beach after storm periods, and this is the source of the coarser material found mixed with the beach deposits.

The geology to the north is characterised by superficial deposits primarily composed of till derived from limestones, with bedrock outcrops present both landward and seaward of the railway. Marine sands and gravels become increasingly common closer to the coast in Whiterock. The solid deposits predominantly consist of granite of Caledonian age, moving south transitioning to dark blue-grey slate, phyllite, and schist of Ordovician age. Additionally, an igneous intrusion boundary is present, dipping at an angle of 60 to 70 degrees.

The geology to the south is defined by superficial deposits mostly composed of Irish Sea till derived from limestones. Sandy alluvium is associated with local river channels, and thin strips of marine sands and gravels are present near the coast. The solid deposits are primarily dark blue-grey slate, phyllite, and schist of Ordovician age throughout. Furthermore, the outer limit of a high-grade aureole is present in the south.

Ground investigations from December 2023 to May 2024 gathered information on soil, rock, and groundwater for designing coastal defences. The works were supervised by Jacobs to ensure accurate data collection and reporting. A factual report (Causeway Geotech Ltd, 2025) summarized the methods and results.

An engineering ground model for the site has been developed in the Geotechnical Interpretive Report (GIR) for this Project. This is supported by boreholes, dynamic probes at track level and on the beach. For interpretation of ground conditions and soil/rock parameters refer to Report 7694-CCA2_3-P3-ENG-CV-JAC-0003 (Appendix C).

The geotechnical design comprises rock revetments, and concrete structures. The conducted analysis confirmed that the bearing capacity, sliding, settlements of the proposed structures are satisfied. Moreover, to satisfy stability requirements a sheet pile wall length was proposed. Section 4.4.1 to 4.4.5 below include summary of geotechnical results.

The geotechnical risks identified at this stage of the project have been included in Section 9 of the GIR. The major risks include scour in front of the proposed structures, variable or unforeseen ground conditions and risks associated with sheet piles (vibrations, driveability and corrosion).

The geoenvironmental risks associated with the construction phase are generally considered to be ‘moderate/low’ based on the information available. Risks to future site users are also considered to be generally ‘low’. Overall, with regard to arisings during works, all soils (except for those deemed to be suitable for reuse during the project) should be specifically tested and assessed prior to disposal and subsequently disposed of at a suitably licensed facility in line with the Waste Management Act 1996 (and 2001 Amendment), the Waste Management (licensing) Regulations 2014 and the European Union (Landfill) Regulations 2020. For details about assessment of potential site contamination refer to Section 7 of the GIR.

For geotechnical recommendations refer to Section 8 of the GIR.

4.4.1 Slope stability assessment

Internal slope stability check was undertaken for the rock revetment at Whiterock. For the assumed internal friction angle (40˚and 55˚) the internal slope stability is considered acceptable as Factor of Safety (FoS) is greater than or equal to 1.3 for the steepest slope with slope ratio of 1:1.5 (approx. 33.7˚). For details of the analysis refer to Section 6.1 of the GIR.

4.4.2 Sliding assessment

A sliding check was undertaken for the structures at Killiney for a cast in-situ reinforced concrete slab (3.25m wide) underlain by Marine Beach Sands. The stability of the proposed structures against sliding was acceptable. For details of the analysis refer to Section 6.2 and 6.3 of the GIR.

4.4.3 Bearing capacity

The bearing capacity was checked for rock armour and concrete structures underlain by founding stratum such as Marine Beach Sands, Glacial Till and Glacial Gravels.

Surcharge from rock armour self-weight was considered as unfavourable permanent loading. Vertical loading from wave action was considered as unfavourable variable loading.

The undertaken analysis carried out confirms that the bearing capacity of the founding stratum (Marine Beach Sand and Glacial Till) are acceptable. Maximum founding stratum utilisation of 60% was achieved for Marine Beach Sands in sub-cell D1. Gravel passes the bearing capacity check by inspection, as it exhibits superior strength parameters compared to Marine Beach Sands. For details of undertaken analysis refer to Section 6.1 to Section 6.3 of the GIR.

4.4.4 Settlements assessment

For all sub-cells total settlement was assessed. A proportion of this settlement is expected to occur immediately during construction. The total settlement at the landward edge of the proposed structures is insignificant and should not affect global stability of the cliff slope. No settlement is expected beneath the existing railway track.

No limit on total settlements of the rock armour was applied as over-build within the smaller sized material is adopted during design life to allow settlements and to maintain the defence crest level at or above the design level. Max total settlement of rock armour was 15mm.

Max total settlement of 10mm and 15mm was obtained for South Killiney (subcell D1) at the footpath and steps, respectively. It is recommended to undertake assessment of the differential settlements at detailed design. At subcells C3 and C4 the differential settlement is expected to be minimal, as the total estimated settlement was less than 5 mm.

For details of undertaken analysis refer to Section 6.1, 6.2 and 6.3 of the GIR.

4.4.5 Sheet pile assessment

The required toe level for the sheet piles was assessed, and internal stability was confirmed. A long-term case considering an 80% reduction of the flexural pile stiffness was checked to confirm the solution achieves the intended design life for the scheme. The wave action on the pile was included in the SLS case to determine if it impacts displacement in either direction.

The preliminary design has shown that a 5m long pile (AZ14 profile) is required to satisfy pile stability. Maximum sheet pile deflection was 17mm. It is expected that the final pile length will be determined by the temporary works requirements at detailed design stage.

For details of undertaken sheet pile analysis refer to Section 6.2 and 6.3 of the GIR.

4.5 Landscape design

Whiterock is an iconic landscape that blends both natural and manmade features, each contributing to its distinctive identity. Notable elements include the cliffs and the Bluff, a natural rocky outcrop within the bay, set against the backdrop of the tall, stone-built railway viaduct. The proposed revetments at Whiterock will consist of natural rock armour, designed to integrate with the existing coastal defences. Figure 4-10 presents an artistic impression of the proposed revetment at Whiterock.

Figure 4-10. Illustrative view of the proposed revetment at Whiterock

At Killiney, the proposed works continue the existing walkway with the addition of a rear wave wall and new steps at the front of the walkway leading to the beach. Existing concrete paths and structures form part of the current landscape. The proposed steps will provide both access and seating, while also reducing the need to construct a large vertical wave wall. During Phase 4 further design work will be undertaken by landscape architects, in collaboration with heritage consultants, to explore material choices and enhancements that will help integrate these coastal defences into the landscape at Killiney.

4.6 Access

Currently, access to Whiterock is via the access steps from Vico road in the north or via the beach from the south as shown in Figure 4-11. Access to Whiterock Beach from the south is currently tidally restricted due to the natural outcrop at The Bluff. The footprint of the proposed revetments is narrower than The Bluff. However, it is noted that The Bluff is approximately 35m long and at higher water levels it is possible to traverse around the base of The Bluff to access the beach to the north whereas the proposed revetments will result in a longer length of the beach being tidally restricted. Whilst every effort has been made to minimise impact to beach user the proposed revetments will limit access to the north by an additional 3 hours for each tide depending on the sea condition (i.e. in rougher sea states the access time will be reduced).

Access to Whiterock Beach from the northern access steps will not be impacted.

Figure 4-11. Whiterock Access location (shown in red)

At Killiney, public access is currently via three ramps and one set of access steps as shown in Figure 4-12. All of these access locations will be maintained and incorporated into the proposed scheme. In addition, access to the beach will be improved by providing continuous access from the new raised footpath to the beach. This access will be maintained if beach levels lower in the future as the new access steps and ramps will extend to the estimated year 2075 beach levels. In the short term it is likely that the majority of the steps will be buried by the beach and beach access will be directly from the footpath. New pedestrian access ramps will be included at the north of and south of Killiney.

Figure 4-12. Killiney Access locations (shown in red)

The access ramps have also been designed to allow access for small maintenance vehicles.

4.7 Utilities and services

A full utilities and services search will be undertaken to confirm the locations of any existing services that might be impacted by the Project. Initial information shows a wastewater pipe running along the crest of the beach from Killiney all the way to the underpass at Southern Killiney. It is currently assumed that this pipe will need to be re-routed.

4.8 Environmental enhancement/biodiversity design

The Phase 3 design will be further modified at detailed design having regard to the potential for environmental effects as identified by the Environmental Impact Assessment Report (EIAR) which will be produced in Phase 4 of the Project.

5. Preferred scheme

5.1 Description of preferred scheme design solution

The preferred scheme is rock revetments at Whiterock and a raised footpath with rear wall and beach access steps along Killiney. The Phase 3 design has further developed the initial designs from concept design stage (Phase 2) that were presented in the Option Selection Report (7694-CCA2_3-P2-ENG-CV-JAC-0001). Optimisations have included enhancing the access to these amenity areas compared to the design presented in Phase 2.

5.1.1 Design at Whiterock

The preferred design for Whiterock is a rock revetment along the front of the existing masonry wall to reduce the force on the wall and reduce the risk of erosion of the slope retained by the wall due to wave overtopping. The phase 3 design at Whiterock has primarily focussed on options to reduce the width of the revetment to minimise the impact on the beach in this location. This has been possible by steepening the slope from 1 in 2 to 1 in 1.5 and reducing the crest level of the rock revetment from +6.25mODM to +5.5mODM. This reduction in the required crest level has also removed the need for raising the existing wall. These refinements are a result of undertaking CFD. The rock revetments at Whiterock will comprise two layers of 6-10t rock armour over two layers of 0.3-1.0t underlayer with a geotextile underneath. Along the sections where the existing wall height is low (approx. +5.0mODM) the rock revetment has a crest width of 4 rocks (approximately 5.8m), this is reduced to 3 rocks (approximately 4.3m) where the wall is higher. Figure 5-1 provides an illustrative view of the proposed rock revetment at Whiterock.

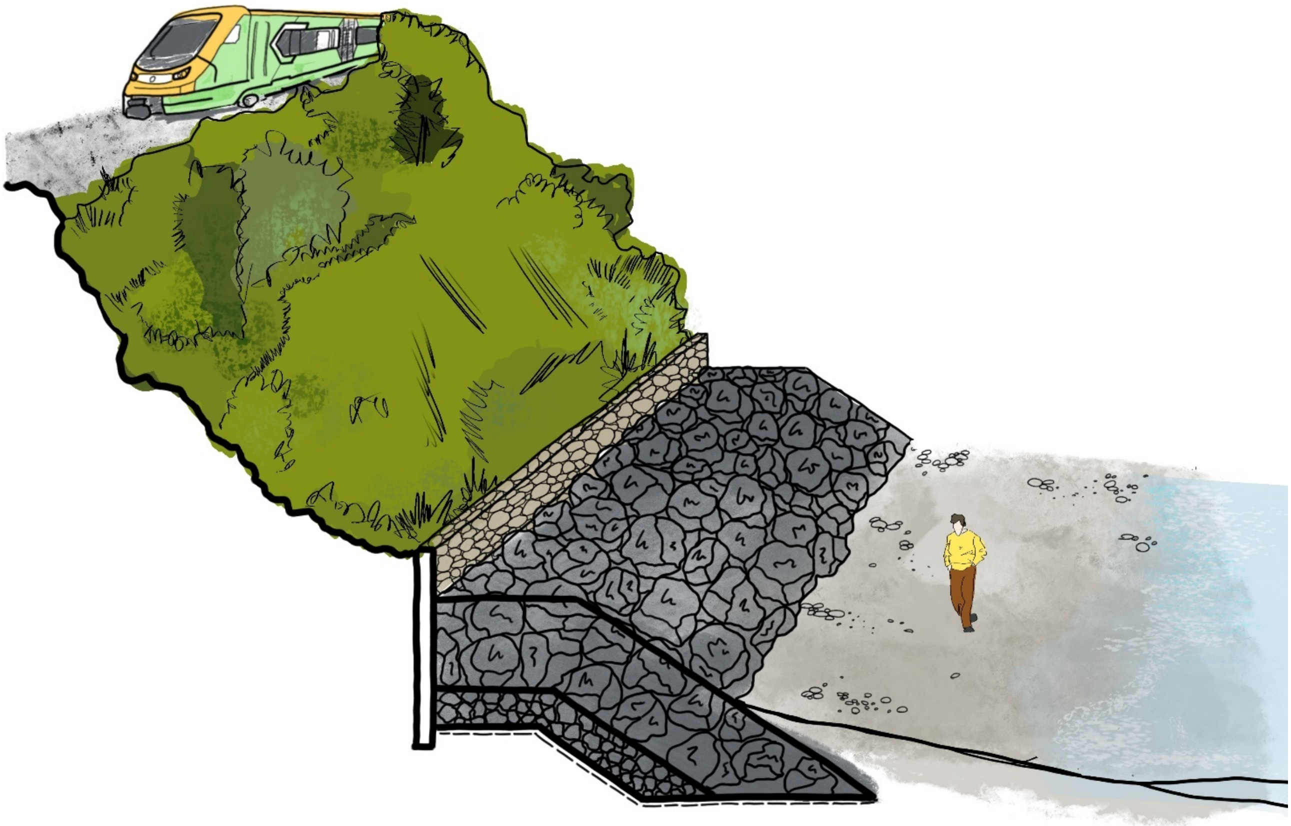

Figure 5-1. Illustrative view of rock revetment at Whiterock

5.1.2 Design at Killiney and South Killiney

The Phase 2 design presented at Public Consultation 1 (PC1) consisted of a raised walkway with a front and rear wave wall and rock toe protection along Killiney and South Killiney. Feedback from PC1 was generally positive but there were some concerns around beach access and the size of the wave wall at the back of the beach (up to 2.0m in some locations).

During Phase 3 it has been possible to remove the need for the front wall and instead incorporate steps from the footpath down to the beach. At the northern sections of Killiney it has also been possible to remove the need for the rock toe protection. At South Killiney, the beach levels are predicted to lower in the future and therefore the rock armour toe protection is still required here to prevent undermining of the defences and reduce wave overtopping rates in the future.

The northern section of Killiney (subcell C3) is approximately 80m long and will comprise a raised 3.0m wide footpath at +4.0mODM and a rear wall at +5.6mODM. The footpath will tie into the existing ramp at the same level. A new pedestrian access ramp will also be incorporated at the northern section of the scheme to provide accessible access to the beach. South of the ramp will be access steps from the raised footpath to the beach for approximately 50m. These access steps, as well as the ramp, extend below the existing beach level, to the predicted 2075 beach level, to ensure access to the beach is maintained in the future if the beach levels lower. In the short-term following construction, the majority of the steps may be buried as the beach levels are higher. Figure 5-2 shows an illustrative sketch of the proposed walkway and new access ramp tying into the existing footpath and ramp.

The existing access ramp south of the station car park will be maintained and will transition into the new footpath. In this location the footpath will extend seaward to maintain the 3.0m wide walkway in front of the existing building.

Figure 5-2. Illustrative sketch of the proposed works at Killiney looking north

Approximately 10m south of the existing building (subcell C4), where the existing footpath ends, the design changes from access steps to the beach to 'amenity steps'. These amenity steps will be wider and higher than the access steps in subcell C3. This section is approximately 170m long, continuing to the existing concrete steps close to Killiney station. The footpath and rear wall will continue at +4.0mODM and +5.6mODM along here. The existing building and existing concrete steps along this section will be maintained with the footpath extending seaward to maintain the 3.0m wide footpath in front of the structures.

South of the existing concrete steps subcell D1 starts. This subcell continues approximately 360m to the existing underpass to Seafield Road. Along this section the beach is narrower and the wave conditions are slightly higher. Due to this, the rear wave wall along here needs to be higher, at +6.5mODM. To minimise the exposed height of the rear wall (for structural and visual purposes) the footpath height is increased from +4.0mODM to +4.6mODM. This increase in the footpath height will be a gradual increase at a 1 in 25 slope to reduce impact on users. The amenity steps in subcell C4 continue along this section. A buried rock toe is included at the base of the steps to prevent undermining and reduce wave overtopping in the future if beach levels lower. At the southern extent of the frontage the footpath lowers to meet the existing concrete path at a level of +3.5mODM this transition will happen over approximately 20m at a slope of approximately 1 in 20. The rear wall will also reduce in height along the slope; this will be done in steps rather than a slope along the rear wall. To maintain access to the beach from the existing concrete path a pedestrian access ramp is included parallel to the new defences down to the year 2075 beach level.

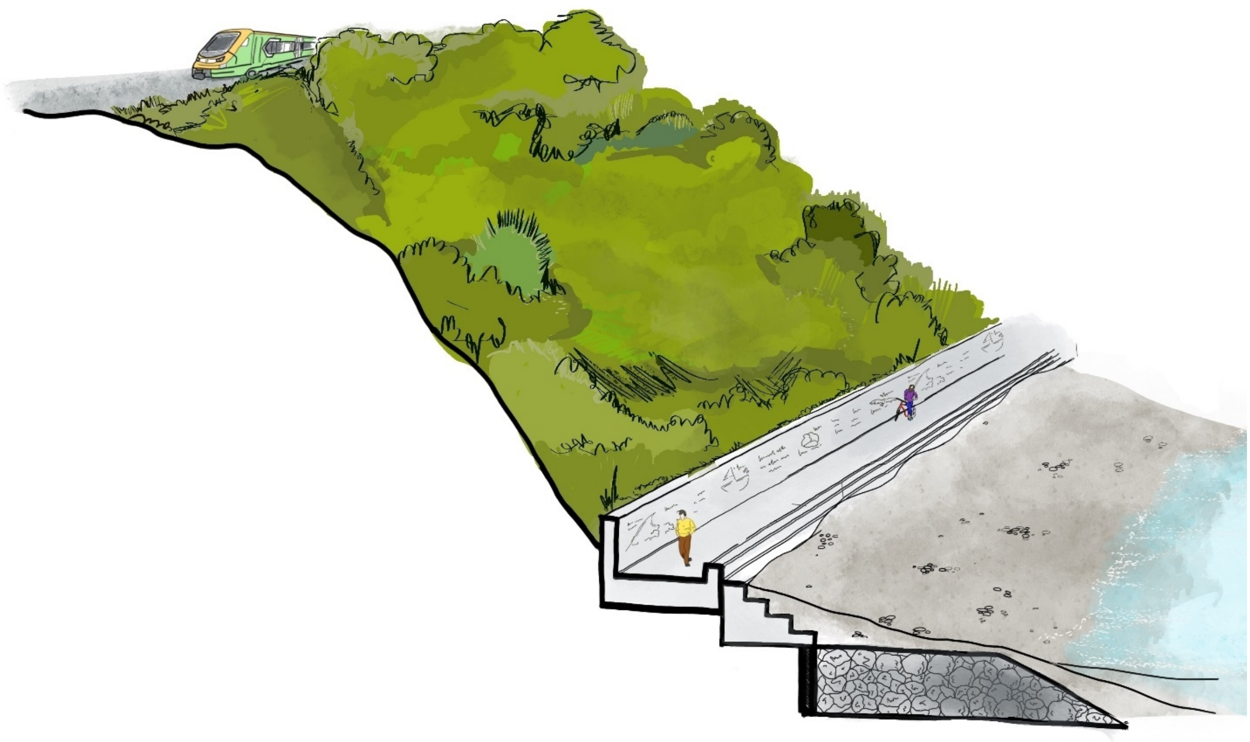

Figure 5-3. Illustrative section of the proposed works at South Killiney

Handrails and/or kerb edges will be provided along the edges of ramps and steps where there is a change in height leading to potential fall hazards.

5.2 Future adaptability of preferred scheme design solution

Future adaptability of the designs has been considered throughout the design process. At Whiterock additional rock armour can be added to the revetments if required in the future to reduce the wave overtopping and erosion risk of the embankments.

At Killiney and South Killiney future beach levels and shoreline changes are somewhat uncertain. Shoreline modelling suggests that the beach levels will erode in the future. Therefore, the structures have been designed with a sheet pile toe. This means that in the future rock toe protection can be added to the base of the structures, in front of the sheet pile. This will prevent undermining of the defences due to lowering of beach levels and will also help to manage future wave overtopping as the rock armour dissipates wave energy. In the longer-term further adaptation is possible by adding further rock armour to provide a rock revetment in front of the defences.

5.3 Interfaces between sub-cells and existing structures

There are a number of interfaces with existing structures that have been considered and will be further developed through the next project phases. The key interfaces are:

- Interface of rock revetment with existing wall at Whiterock (north and south of The Bluff)
- Transition of the rock revetment into the existing wall in Whiterock Bay
- Interface of the new raised walkway and rear wall with:
    - the existing path and ramp at Killiney
    - existing access ramps and steps along Killiney and South Killiney
    - existing buildings and structures along Killiney
    - existing access ramp at the underpass to Seafield Road

Along the length of the revetments at Whiterock, the rock armour will be placed against the seaward face existing wall. At the northern extent of revetement at Whiterock, as the revetment follows the curve of the existing wall, the revetment will reduce in size and taper in to meet the existing wall, minimising the impact on Whiterock Bay.

At Killiney and South Killiney the proposed defences have been designed to incorporate all existing access ramps and steps and buildings. At the two existing buildings, and the concrete steps south of the second building, the walkway will extend seaward to maintain the 3.0m wide footpath in front of the existing structures. The existing access steps to the bridge at Strand Road, will lead onto the new walkway with access steps from the walkway down to the beach being incorporated into the new structure thereby maintaining beach access at this location.

At South Killiney the new walkway will reduce in level to tie into the existing concrete path. The back wall level will also reduce to follow the reduction in level of the ramp.

5.4 Drawing list

Drawings prepared for the Project are summarised in Table 5-1.

Table 5-1. Drawing list for Whiterock to South Killiney Phase 3

Drawing No.	Title	Description
7694-CCA2_3-P3-DWG-CV-JAC-0010	SITE LOCATION PLAN	Overview of frontages between Whiterock and South Killiney
7694-CCA2_3-P3-DWG-CV-JAC-0100	LOCATION PLAN	Location of proposed works between Whiterock and South Killiney
7694-CCA2_3-P3-DWG-CV-JAC-0200	GENERAL ARRANGEMENT PLAN 1 OF 3	Location of proposed works in Whiterock

Drawing No.	Title	Description
7694-CCA2_3-P3-DWG-CV-JAC-0201	GENERAL ARRANGEMENT PLAN 2 OF 3	Location of proposed works in Killiney
7694-CCA2_3-P3-DWG-CV-JAC-0202	GENERAL ARRANGEMENT PLAN 3 OF 3	Location of proposed works in South Killiney
7694-CCA2_3-P3-DWG-CV-JAC-0300	GENERAL ARRANGEMENT CROSS SECTIONS 1 OF 5	Proposed cross-sections at Whiterock (north)
7694-CCA2_3-P3-DWG-CV-JAC-0301	GENERAL ARRANGEMENT CROSS SECTIONS 2 OF 5	Proposed cross-sections at Whiterock (south)
7694-CCA2_3-P3-DWG-CV-JAC-0302	GENERAL ARRANGEMENT CROSS SECTIONS 3 OF 5	Proposed cross-sections at Killiney (north)
7694-CCA2_3-P3-DWG-CV-JAC-0303	GENERAL ARRANGEMENT CROSS SECTIONS 4 OF 5	Proposed cross-sections at Killiney (central)
7694-CCA2_3-P3-DWG-CV-JAC-0304	GENERAL ARRANGEMENT CROSS SECTIONS 5 OF 5	Proposed cross-sections at Killiney (south)

5.5 Buildability / Constructability

The constructability considerations for the Project are very similar to those presented during the Option Selection Report; although the design has been developed further, the forms of construction and materials required are unchanged. The frontage has good accessibility for marine delivery of rock but has limited road access. It is expected that the rock armour will be delivered to site via sea whilst the plant and other materials will be delivered via road (likely via N11/M11 onto R837, then R119, then Shanganagh Rd onto Shanganagh Cliffs onto the beach front).

The expectation is that the construction phase will have at least one main site compound which will contain the site welfare, offices, laydown areas, car parking etc. There will also be smaller satellite compounds closer to the works for activities such as material load out points and welfare closer to the works.

Due to the large volumes of rock armour required there will be a requirement to stockpile rock in the water and on land during the construction phases. Handling of large rocks requires large construction equipment such as articulated dump trucks (ADT's) and large excavators. These pieces of equipment will require maintenance and an area for storage during times they are not used. The beach itself is not a suitable plant storage location and as such an area within the site boundary will need to be identified as large plant such as ADT's cannot use public roads.

A potential staging area can be setup at Killiney Beach adjacent to Shanganagh wastewater treatment works. This location would allow equipment to be brought into site, assembled and then access the beach for the full length of the project.

As the volumes of armour stone are significant the procurement of rock to the project is of key importance. Ultimately the contractor will decide where the rock is supplied from but there is a high probability that it will be sourced from overseas. Norwegian rock is known within the marine construction sector to be high quality, have good availability and can be a cost-effective way to source large volumes of rock. An allowance for some local supply of rock (such as the underlayer) should be allowed for and these smaller volumes could be brought to site via the road network. The marine approach would remove the requirement to bring rock in through the local road or rail network.

The assumption at this stage is that the rock arrives in large rock barges from overseas. The rock is then dropped into marine stockpiles at high tide via self-discharging rock barges. The rock is then recovered by land-based equipment at low tide by long reach excavators and can be 'daisy chained' up the beach and stockpiled local to the work area ready for placing. Depending on water depths and marine stockpile locations there may be a need to create a short causeway to provide a raised platform for the excavators to recover the rock.

The use of precast concrete is proposed for the raised walkway sections to reduce construction time and streamline the construction phases. Precast concrete offers many benefits such as improved quality, reduces health and safety risk and can provide efficiencies in construction. However, delivering precast units to remote working areas such as CCA2/3 can be challenging. Precast materials can be brought to site via the road network (or potentially from the railway) and offloaded at the staging area identified. From there the units could be loaded onto a tractor trailer for transporting up the beach.

5.6 Environmental assessment

The EIA screening and scoping documents are currently being prepared. The EIA screening report will determine whether the proposed project is of the nature and scale that requires an EIA. The EIA scoping report will outline the proposed assessment to be undertaken to generate an Environmental Impact Assessment Report (EIAR) for the proposed project including details of the environmental topics to be scoped in/out, the assessment methodology and the surveys, consultation and data required for the assessment.

The Phase 3 design will inform the environmental assessment under Phase 4 of the Project.

5.7 Health and safety

A Design Hazard Elimination & Risk Reduction Register or DEHERR, has been developed alongside the Phase 3 design. The DEHERR is presented in Appendix D and has been prepared following Jacobs' De5ign ('Five in Design') principals. The DEHERR allows the designer to determine potential risks and, where possible, design against the risks presented. Where the risk is not possible to eliminate at this stage of design, further evaluation of the risk will occur at detailed design, before the risk is transferred to the contractor to consider when developing their safe system of works. A table presenting the principal identified risks is provided Table 5-2. In addition, the following sections discuss in more detail some key hazard at Whiterock and Killiney.

Table 5-2. Top five risks identified in the DEHERR

Risk ID.	Activity	Potential Hazard	Design to Reduce Risk	Residual Risk	Action By	Comments
1	Existing Services	Damage to existing services during construction leading to death or injury to site personnel.	Full services survey to be undertaken during detailed design development.	Damage to existing services during construction leading to death or injury to site personnel.	Designer / Contractor	Full services search to be undertaken at detailed design stage. Contractor to survey location prior to excavation works, where reasonable.
2	Use of vehicles/plant on site – Staff / Public	Transportation over foreshore and access ramps, etc. Potential plant overturning leading to potential for injury/death to members of public with access to the foreshore.	Clear pedestrian routes within the site and fencing off of working areas to be considered during design development.	Contractor to put in sufficient safe system of works as well as sufficient temporary retaining structures to limit the chance of cliff slippages occurring when the revetment is in its most unstable (i.e. during construction).	Contractor	Contractor to put in sufficient safe system of works as well as sufficient temporary retaining structures to limit the chance of cliff slippages occurring when the revetment is in its most unstable (i.e. during construction).
4	Unstable ground conditions	Potential for site operatives or plant to become stuck in pockets of soft or lose ground. Instability of plant working in area of low soil strength. Risk of suffocation, crash injuries from sinking into ground/loss or damage to plant.	Inform contractor of risk of soft ground from GI and geotechnical analysis in detailed design.	Potential for site operatives or plant to become stuck in pockets of soft ground. Instability of plant working in area of low soil strength. Risk of suffocation, crash injuries from sinking into ground/loss or damage to plant.	Designer / Contractor	Contractor to prepare method statement and safe systems of work. Risk to be updated following completion GI and geotechnical analysis.
10	Managing public access to works	Potential for public to become injured if gaining access to site works while heavy plant etc are working.	At detailed design stage, contractor to address public access concerns as part of method statement.	Risk of injury to public due to access gained to site.	Designer / Contractor	Contractor to prepare method statement and safe systems of work. These will ensure that the chance of public access to the site is limited as much as practically possible.

Risk ID.	Activity	Potential Hazard	Design to Reduce Risk	Residual Risk	Action By	Comments
11	Public access to the beach restricted	The rock revetments will have a larger footprint on the beach than the existing structures, thereby reducing the useable area of the beach. This could lead to people becoming trapped during changing tides.	The footprint of the revetments has been minimised as much as possible at this stage, including burying the toe rather than an exposed toe.	People becoming trapped during changing tides.	Designer / Client	designer to review beach access points during detailed design development. Consider installing warning signs at access points to highlight risk to the public.

5.7.1 Whiterock beach

As discussed in Section 4.6, whilst every effort has been made to minimise impact to beach users, the proposed revetments will limit access to the north by approximately an additional hour either side of high tide depending on the sea condition (i.e. in rougher sea states the access time will be reduced). It is recommended that signage is installed highlighting the risk of being cut-off by the tide to beach users.

5.7.2 Cliff stability

The soft sediment cliffs along the Killiney Strand to Shanganagh shoreline are formed in unconsolidated glacial sediment with particle sizes ranging from clay to cobbles. The cliffs are fronted by a wide beach that limits the frequency of toe erosion by waves. The relative rarity of toe erosion events means the cliffs are degrading through the action of rainfall, surface water flows and wind that have resulted in accumulation of a debris apron ('talus') at their base. The three types of failure that are most likely in this type of cliff are:

- retreat of the cliff through toe erosion resulting from a sequence of storms that lower the beach, remove the talus and erode the cliff
- failure of the talus due to saturation by sustained wet weather
- localised instability of the cliff face as small amounts of material of a range of particle sizes is released through weathering and erosion and deposited on the talus slope or beach

Long term cliff behaviour has been assessed using historical Ordnance Survey Ireland maps and aerial imagery dating from 1830 to the present day. The assessment has been supplemented by Google Earth satellite imagery from the last c. 30 years to support classification of failure types. The assessment of historical maps and aerial photographs shows the long-term erosion rate is not detectable above the error in the input data. The maximum erosion rate recorded over shorter time periods between data epochs, is 0.08±0.05m/y for 1830 to 2023 and 0.06±0.03m/y for 1900 to 2023. It is therefore concluded that the proposed engineering will remove any possibility of toe erosion. However, the cliff will continue to freely degrade through the action of rainfall, surface water flows and wind meaning small amounts of material will continue to accumulate on the talus slope. It is possible that larger cobbles will bounce beyond the talus slope. These will be retained by the catch wall and are unlikely to affect pedestrians.

Assessment of available records provides evidence for one failure event. Between 2011 and 2013, a c. 23m long section of the talus slope failed 50m south of the beach access steps from Strand Road, south of Killiney station. The failure resulted in deposition of a 4.2 wide debris lobe onto the beach. There is no evidence that the cliff retreated. The debris lobe remains in place, indicating no significant wave erosion has subsequently occurred. This event resulted in the council constructing a wooden fence that prohibited access to a c. 60m long section of the cliff. These failures are considered possible, albeit infrequent over the next 50 years. The impact on the proposed engineering is negligible. The impact on pedestrians will also be negligible as the material will be retained by the structure and catch wall.

It has not been possible to document the frequency of material being released from the cliff face, but the evidence presented above suggests the process occurs infrequently. Site visits have shown the talus slope is generally well vegetated and that the cliff is sub-vertical with several gravel to cobble-sized clasts emerging from the face. It is therefore concluded that gravel to cobble-sized material may periodically fall from the cliff face, with larger material bouncing beyond the talus. There will be negligible impact on the proposed structures, but during detailed design the catch wall will need to be designed to ensure that material is retained and does not reach the pedestrian walkway. The area behind the catch wall will require periodic clearance to ensure sufficient freeboard is maintained.

5.7.3 Safety and maintenance plan

The safety and maintenance plan will be developed during detailed design.

As stated in Section 2.6, due to the proximity to the Irish Railway line to CCA2/3, the safety certification and approvals will be aligned with the process stated in Iarnród Éireann (IÉ) standards and the general good practices of safety assurance and management. However, based on the consultation with IÉ stakeholders, it has been confirmed that the scoped works are non-significant in accordance with the Common Safety Method Risk Assessment (CSM-RA) and does not require Authorisation to Place in Service (APIS).

Minimal maintenance of the rock revetements is anticipated during the design life of the scheme. Following significant storm events, it may be necessary to undertake some maintenance in the form of repositioning rocks within the revetment. It is assumed that the plant access to the foreshore for maintenance works would follow the same route as that suggested for construction of the revetments.

The concrete walkways and steps may require some patch repairs during the design life.

5.8 Recommendations for refinement at detailed design

As discussed, the Phase 3 modelling has shown a reduction in the expected shoreline erosion at South Killiney. Therefore, at detailed design further analysis of the proposed defences will be undertaken to determine the wave overtopping and wave loads under the estimated year 2075 beach profile. This might result in a reduction the width of rock toe protection required.

Additional numerical modelling is recommended to refine the sediment transport analysis at Whiterock to provide a better understanding of the potential impacts of the design,

The Phase 3 design assumes three typical cross sections along Killiney. At detailed design it is recommended that additional sections are considered to take into consideration the variation in the beach profile along this section. This may result in a shorter section of rock toe protection being required.

Other details to be refined at detailed design include:

- Interfaces between cross sections and existing structures;
- Health and Safety requirements such as the need for handrails and edging kerbs;
- Materials and finishes for the concrete structures;
- Reinforcement design of concrete elements; and,
- Prior to construction, further ground investigation will be undertaken to ensure that ground conditions at each site are fully understood, and that the location of any buried services is understood and accounted for in the design.

6. Conclusions and next steps

This Phase 3 Design Report is the principal deliverable at this phase. Future Project phases to deliver the Preferred Scheme are summarised below:

- Phase 1 – Project Scope and Approval (completed);
- Phase 2 – Concept, Feasibility and Options (completed);
- **Phase 3 – Preliminary Design (current phase);**
- Phase 4 – Statutory Process (next phase);
- Phase 5a- Detailed Design and Tender Issue (future phase);
- Phase 5b - Contract Award (future phase);
- Phase 6 – Construction; and,
- Phase 7 – Close out.

6.1 Design development

The next phase of design covers Statutory Process that is focussed on preparation of the environmental impact assessment report (EIAR) AA Screening reports, Natura Impact Statements and associated documentation required for a planning application.

6.2 Opportunities for consultation and engagement

The Phase 3 Design has been informed by Public Consultation 1 (PC1) undertaken in Nov/Dec 2024. The findings are summarised in the PC1 report (7694-CCA2_3-P2-PLA-EV-JAC-0010). A second round of consultation (PC2) will be undertaken in September 2025.

The Project will now undertake an environmental assessment which will be reported in the EIAR and other documentation in support of the statutory planning process for the Project. Stakeholders will be afforded the opportunity to engage on the Project again at this point through the statutory stakeholder engagement process. Outputs from this consultation process will be taken into consideration by the planning authority.

6.3 Consenting

The significant work streams undertaken during this phase of the project comprise the preparation of all documentation leading to a Marine Area Consent application and Planning Consent application to ACP.

An application(s) will be made to MARA for the Marine Area Consent (MAC). On receipt of a MAC a planning consent application will be made. At this stage it is considered that the application for planning will be made under the Seventh Schedule Strategic Infrastructure Development (SID) under the Planning and Development (Strategic Infrastructure) Act 2006 and Planning and Development Act, 2000 (as amended). However, the application will be made under the Planning and Development Act 2024 if the relevant sections are enacted at the time of the application.

6.4 Procurement and programme

The construction procurement will commence following the granting of the consents in Phase 5.

A high-level indicative programme of the next phases is as follows:

- Phase 3 programmed for summer 2025;
- Phase 3 completion autumn 2025; and
- Phase 4 programmed for winter 2025 and throughout 2026.

The programme for phases after planning submission (Phase 5 onwards) is subject to application durations.

7. Glossary

Term	Description
Annual exceedance probability	The probability that a given event will be equalled or exceeded in any one year
Antecedent rainfall	Cumulative rainfall totals over a given period
Beach lowering	Reduction in beach surface elevation over a timescale due to cross-shore and longshore sediment transport.
Beach nourishment	Supplementing the existing beach periodically with suitable material to increase beach volumes, reduce erosion and toe scour at flood defences and/or soft cliffs.
Breakwater	Offshore structure which dissipates wave energy due to their size, roughness and presence of voids. This reduces the wave heights at the shoreline defences
Caisson	A watertight retaining structure used as a foundation
Capital expenditure	Funds used to acquire, upgrade and maintain physical assets (e.g., construction costs)
Capping beam	Steel structures that join pile foundations together to increase their rigidity and reduce movement
Carbon management	An approach to mitigate or reduce carbon (or other greenhouse gas) emissions
Catch fence	A fence designed to catch falling debris and absorb impact
Circular economy	A system which reduces material use, redesigns materials, products, and services to be less resource intensive, and recaptures “waste” as a resource
Cliff recession	Landward retreat of the cliff profile (from cliff toe to cliff top) in response to cliff instability and erosion processes
Climate adaption plan	A plan which sets out measures that protect a community or ecosystem from the effects of climate change, while also building long-term resilience to evolving environmental conditions
Climate change	A change in global or regional climate patterns, in particular a change apparent from the mid to late 20th century onwards and attributed largely to the increased levels of atmospheric carbon dioxide
Climate resilience	Climate resilience is the capacity of social, economic and ecosystems to cope with a hazardous event or trend or disturbance caused by climate change
Coastal Cell Area	A spatial model which subdivides the coast based on the variation in physical characteristics, including the geology, geomorphology, shoreline topography and orientation, and existing defence type
Coastal erosion	Loss or displacement of land, or long-term removal of rocks and sediment along the coastline due natural impact of waves, wind, rain and tides
Coastal flooding	Submergence of normally dry and low-lying land by seawater
Coastal protection	Measures aimed at protecting the coast, assets and inhabitants from coastal flooding and erosion. Coastal protection may involve structural, non-structural or nature-based solutions
Coastal spit	A coastal landform, whereby a stretch of beach material projects out to the sea and is connected to the mainland at one end
Concept level design	Foundational phase of the design process which lays the groundwork for the entire project. The design work undertaken for the concept design is sufficient to confirm that the options will work from a technical perspective and will meet the Project objectives.
Concrete armour	Precast concrete units placed to form breakwaters or revetments to dissipate wave energy
Constructability	Also known as buildability. The extent to which a design facilitates the each and efficiency of construction
Design horizon	The period of time over which the scheme provides the required Standard of Protection (SoP) to the railway line.
Design life	The service life intended by the designer, which is the period of time after installation during which the structure meets or exceeds the performance requirements.

Dilapidation survey	A detailed survey that examines the existing state of the coastal structure
Dune regeneration	Stabilisation and enhancement of existing dune systems to deliver additional resilience
Embankment	Linear grassed earth structure providing flood protection; typically used along riverbanks
Emergency works	Works in response to an event that is unexpected and serious such that it presents a significant risk to human life, health and property or the natural environment and involves the need for immediate action to manage the risk
Feasibility study	An assessment of the practicality of a proposed project plan or method.
Flood proofing	Structural, and non-structural, solutions that can prevent or reduce flood damages to a property or its content.
Flood warning and preparedness	Measures undertaken to better prepare, respond and cope with the immediate aftermath of a flood event
Foreshore	The part of a shore between high- and low-water marks
Freeze-thaw weathering	Form of mechanical weathering whereby water enters cracks in rocks, freezes and expands, widening the cracks. Repetition of this cycle causes gradual break down of the rock.
Gabions	A basket or container filled with earth, stones, or other material
Geomorphology	The interaction between Earth's natural landforms, processes and materials
Geotextile	Permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain
Geotubes/ Geotextile Tubes	Tube shaped bags made of porous, weather-resistant geotextile and filled with sand slurry, to form artificial coastal structures such as breakwaters or levees
Groyne	Linear structure constructed perpendicular to the shoreline which helps retain beach material in place.
Hazard	A process or material that has the potential to cause harm.
High tide mark	A point that represents the maximum rise of a body of water over land
Hydrodynamic modelling	Used in the analysis of coastal hydrodynamic processes, it is employed to simulate major physical phenomena in the coastal region
Maintenance burden	The level of maintenance (repair, monitoring, rebuilding) required over the design life of the structure to retain the Standard of Protection of the coastal defence structure
Managed realignment	A coastal management strategy that involves setting back the line of actively maintained defences to a new line inland and creating inter-tidal habitat between the old and new defences
Mudslides	Mass of saturated sediment that moves downslope. Typically comprises distinct source, transport and debris accumulation zones
Multi criteria analysis (MCA)	A structured approach to determine overall preferences among alternative options, where the options should accomplish multiple objectives.
Nature-based solutions	The use of natural materials and processes to reduce erosion and flood risk to coastal infrastructure
Pore water pressure	The pressure of groundwater within voids between sediment particles. High pore water pressures push particles part, reducing the shear strength which may trigger slope failure.
Risk	The adverse consequence of a hazard event. Risk is typically described in financial terms, but may consider human harm, environment impact, programme delays or reputational damage.
Residual risk	The risk that cannot be completely eliminated by engineered mitigation measures. It is generally agreed to be at an acceptable level by the client.
Revetment	Sloping or stepped structure built parallel along the shoreline between the low lying beach and higher mainland to protect the coast from erosion and wave overtopping. The revetment may have a smooth or rough surface
Rock netting	A drapery system designed to control rockfall movement by guiding falling debris to a collection point at the toe of the slope

Saltmarsh	Coastal grassland that is regularly flooded by seawater
Sea level rise	An increase in the level of the oceans due to the effects of climate change and/or land-level change
Seagrass bed	Intertidal or sub-tidal beds of sea grass. Provides ecosystem benefits including carbon sequestration.
Seawall	Vertical or near-vertical impermeable structure designed to withstand high wave forces and protect the coast from erosion and/or flooding
Shellfish reefs	Sub-tidal or intertidal reefs formed of suitable material for settlement by oysters or mussels.
Sill	A low rock structure in front of existing eroding banks to retain sediment behind.
Standard of Protection	The expected frequency or chance of an event of a certain size occurring. Defined for this project as being a 0.5% Annual Exceedance Probability, also known as a 1 in 200-year storm protection level.
Storm surge	A temporary change in sea level that is caused by a storm event, which can lead to coastal flooding
Toe scour	Occurs when the toe (bottom) of the defence is worn away by the waves and can cause defences to fail.
Wave exposure	The degree to which a coast is exposed to wave energy
Wave overtopping	The average quantity of water that is discharged per linear meter by waves over a protection structure (e.g., breakwater) whose crest is higher than the still water level

8. References

CIRIA; CUR; CETMEF (2007). The Rock Manual. The use of rock in hydraulic engineering (2nd edition), C683, CIRIA, London (ISBN: 978-0-86017-683-1)

Eldrup, M.R., Andersen, T.L., & Burcharth, H.F. (2019). Stability of Rubble Mound Breakwaters—A Study of the Notional Permeability Factor, Based on Physical Model Tests

EurOtop, 2018. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com.

Government of Ireland. (2022), Building Regulations, Technical Guidance Document M, Access and Use

Irish Rail (2020) CCE Department Technical Management Standard, CCE-TMS-391, Safety Approval of Change in CCE owned plant, Equipment, Infrastructure and Operations (PEIO), version 1.0, Date: 17 Feb 2020.

McCall, R. T., Masselink, G., Poate, T. G., Roelvink, J. a., Almeida, L. P., Davidson, M., & Russell, P. E. (2014). Modelling storm hydrodynamics on gravel beaches with XBeach-G. Coastal Engineering, 91, 231–250. doi:10.1016/j.coastaleng.2014.06.007

Van Rijn, L. C. (2007). Unified View of Sediment Transport by Currents and Waves: part I and II. Journal of Hydraulic Engineering, (June), 649–667.

Zijlema, M., Stelling, G. S., & Smit, P. B. (2011). SWASH: An operational public domain code for simulating wave fields and rapidly varied flows in coastal waters. Coastal Engineering, 58(10), 992–1012. doi:10.1016/j.coastaleng.2011.05.015

Appendix A. Coastal modelling report

Document Number	Document Title
7694-CCA2_3-P3-REP-CV-JAC-0002	Phase 3 Coastal Modelling Report

Appendix B. Coastal processes technical note

Document Number	Document Title
7694-CCA2_3-P3-REP-CV-JAC-0001	Phase 3 Coastal Processes Report

Appendix C. Geotechnical outputs

Document Number	Document Title
7694-CCA2_3-P3-ENG-CV-JAC-0003	Geotechnical Interpretive Report

Appendix D. DEHERR – (designers risk assessment)

Document Number	Document Title
7694-CCA2_3-P3-REG-CV-JAC-0003	Design Hazard Elimination Risk Register