East Coast Railway Infrastructure Protection Projects

Phase 3 Design Report

Kilcoole to Newcastle

COASTAL CELL AREA 6.1

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Executive summary

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 railway corridor. This project extends between Kilcoole and Newcastle (the 'Project').

The Kilcoole to Newcastle frontage (Coastal Cell Area 6.1) is approximately 4km long. The trainline runs along a natural embankment at the back of the beach and is locally protected by long sections of rock revetments. The Project is located within several designated sites, most notably The Murrough SPA (Code 004186), the Murrough Wetland SAC (Code 002249) and the Murrough pNHA (Code 000730).

The main hazards here are coastal erosion, wave overtopping and steepening and narrowing of the beach due to long-shore transport and drawdown during storm conditions. 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:

- A new rock revetment south of Kilcoole
- Concrete revetments with a wave wall and rock toe protection at the Breaches
- New setback wave wall at Leamore Lower
- A new rock revetment with a wave wall at Newcastle

Future adaptability has been considered during the design process. Although it is unclear what the drivers of design might be beyond 2075, most of the structures proposed can be adapted by amending their geometry. The defences adjacent to the Breaches will be more complex to adapt due to the environmental and physical constraints at this location. However, additional rock and or beach recharge may be able to extend the life of the proposed defences.

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.

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1. Introduction and scope

1.1 Project background

The East Coast Railway Infrastructure Protection Projects (ECRIPP) were established by Iarnród Éireann (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).

ECRIPP aims to deliver improved coastal protection measures to the railway infrastructure, addressing vulnerabilities such as coastal erosion, wave overtopping, and cliff instability, that are projected to worsen due to climate change. To improve resilience, the proposed works will be designed to withstand a1 in 200-year return period event, for a minimum of 50 years (Year 2075).

This report presents the Phase 3 design for Kilcoole to Newcastle, within Coastal Cell Area 6.1 (CCA6.1) (hereafter referred to as the 'Project').

Figure 1.1. ECRIPP locations

1.2 Project location and description

The Project is situated between Kilcoole and Newcastle and is approximately 4km long with works proposed for 2.5km of this frontage. The location of key features within the Project are shown in Figure 1-2. The trainline runs along a natural embankment at the back of the beach; this is a barrier beach feature and is soft; underlain by hard geology. The railway is locally protected by long sections of rock revetment. The Project includes the Breaches, an area of land that is reclaimed intertidal land, the channel of which is crossed by the railway at UBR159. The Project is located within several designated sites, most notably The Murrough SPA (Code 004186), the Murrough Wetland SAC (Code 002249) and the Murrough pNHA (Code 000730).

The main hazards here are coastal erosion (shoreline recession), wave overtopping (the railway is very low-lying and the beach is generally narrow) and steepening and narrowing of the beach due to long-shore transport and drawdown during storm conditions. The latter hazard may lead to undermining of the rock structures and the railway in the long-term.

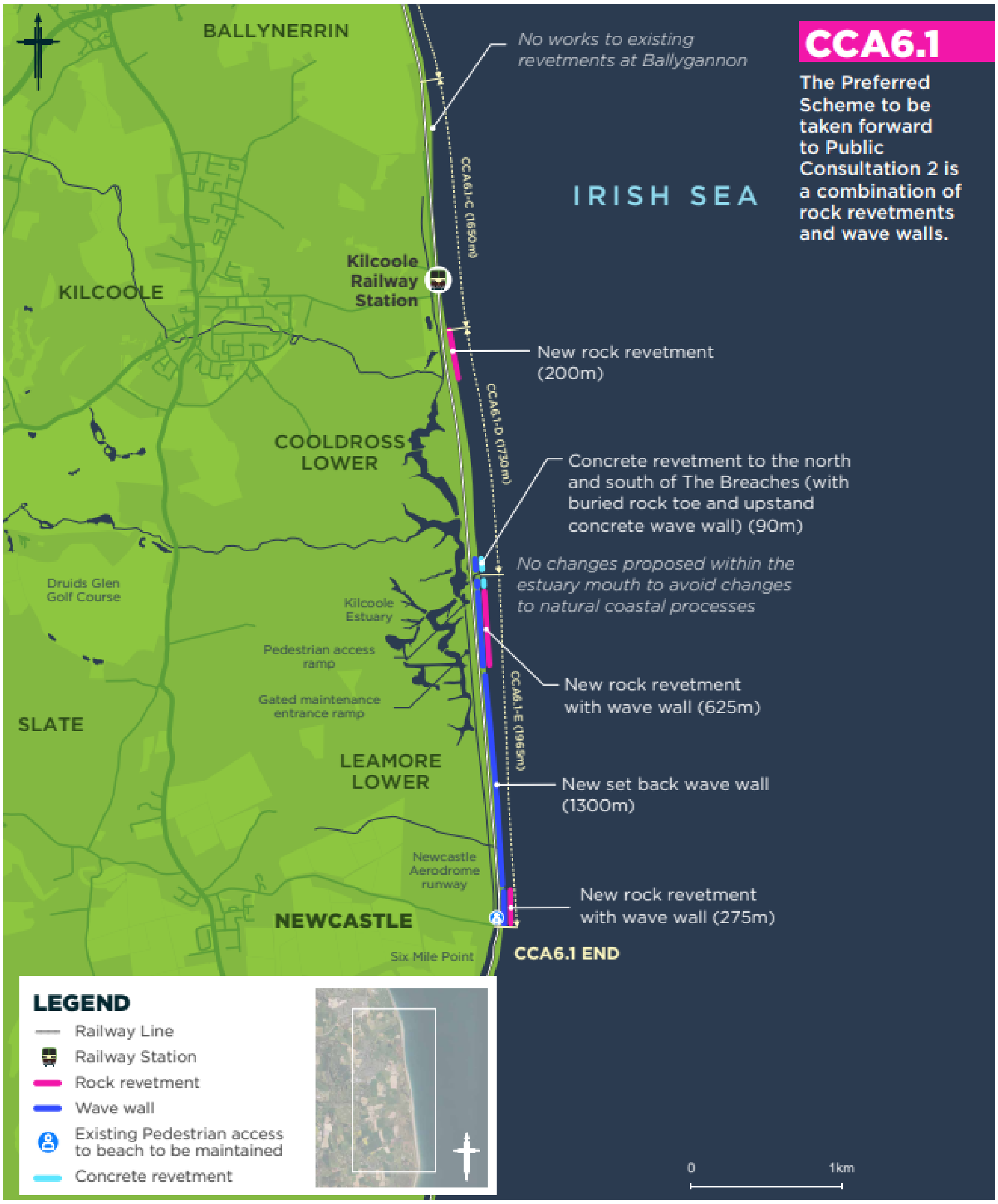


Figure 1-2. Project location plan

1.3 Project objectives

The objectives of engineering interventions between Kilcoole and Newcastle (the Project) are two-fold:

- To reduce the impacts of wave overtopping discharges on railway infrastructure and operations,
- To reduce the potential for coastal erosion undermining the railway

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 wave overtopping impacts to the railway, increasing service reliability under minor storm conditions and preventing significant damage to railway infrastructure under large storms. The works will also reduce the potential for undermining of the track due to coastal erosion.

IÉ infrastructure at the site comprises a single-track railway with no overhead electrification equipment (OHLE). A diesel service that links Dublin with Wicklow and the Rosslare Europort uses the railway.

The proposed design works take into consideration potential future expansion of the rail services in this area.

1.4 Project status

The Project is currently in Phase 3 Design (preliminary level of design). By integrating the proposed options (Options Selection Report) with the results of the Public Consultation (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 and feasibility studies.

1.5 Purpose of this report

This document provides the Phase 3 Design Report for CCA6.1 - Kilcoole to Newcastle. The report defines the design which will subsequently inform the detailed design phase.

This report should be read in full in conjunction with the associated appendices:

- Modelling outputs (Appendix A). This describes the numerical modelling of waves and water levels that support overtopping calculations and revetment design.
- Geotechnical outputs (Appendix B). 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 C)
- DEHERR, designers' risk assessment (Appendix C). 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 stage 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

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

Table 2-1. Key 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 30l/s/m in line with reducing erosion behind the structure (Van der Meer et al., 2009). 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

Key design standards are summarised in Table 2-2 below.

Table 2-2. Key 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.

Discipline	Code/Standard	Application
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.
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.

Discipline	Code/Standard	Application
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, 2016)	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
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

Consideration of alternatives has been undertaken throughout the design process to try to maximise efficiency of the design while reducing the impact on the landscape. Under Phase 2, a broad range of solutions were considered; many of these were discounted due to their inability to provide protection against the eroding nature of the shoreline (e.g. vertical walls) or due to their low resilience against large storms (e.g. nature-based solutions).

The proposed defences have been designed to maintain as much useable public area as possible. Where possible, the need for the rock revetment has been eliminated, this includes areas where the railway corridor is not considered to be at risk from wave overtopping or erosion in the next 50 years and in locations where a wave wall alone can provide sufficient protection to the railway line. Rock revetments may be required in the future at these locations.

2.4 Design elements

2.4.1 Rock revetment

A rock revetment will be constructed for some frontages to prevent erosion and reduce overtopping onto the railway corridor.

The rock revetment will comprise two layers of graded armour rock overlaying a geotextile. The rock grading has been selected to provide stability over the design life using modelled wave conditions that allow for sea level rise. This rock will be of high quality to ensure that it meets and exceeds the design life.

The geometry of the rock revetments is determined through design calculations to limit the wave overtopping to acceptable thresholds to prevent disruption and damage to the railway corridor. This is a combination of the slope of the revetment, the height and width of the revetment and the height of a crown wall (where present).

2.4.2 Concrete revetment

In the location around the Breaches underbridge (Section D5 and E1) a concrete revetment is proposed rather than a rock revetment to reduce the risk of predators close to the Little Tern nesting site. The concrete revetment will comprise a concrete slope with a sheet pile at the toe and rock toe protection. The sheet pile is a vertical structure and can lead to increased scour potential at the toe of the revetment which could result in undermining of the revetment. Due to the impermeable, rigid nature of the concrete revetment, the structure cannot respond to undermining; therefore, rock toe protection will be provided at the base of the revetment. The length of the sheet pile (and need for) and the size of the rock toe protection are determined during the Phase 3 design following initial ground investigation works and additional assessments into scour depth and predicted long term foreshore levels. A concrete revetment does not dissipate wave energy in the same way as a rock revetment due to it being impermeable. Therefore, the wave run-up and overtopping discharges will be higher than for an equivalent rock revetment. The wave wall is expected to be larger than in adjacent sections where a rock revetment is proposed. The geometry of the revetment and wave wall will be determined through wave overtopping and wave loading calculations.

2.4.3 Wave wall

Many of the rock revetments require a wave wall at the rear of the crest to provide an impermeable barrier at the back of the permeable rock revetments. During Phase 2 concept design, it was assumed that these would be precast reinforced concrete. The size of the walls will be determined through overtopping and wave loading calculations.

In some locations (E3 and E4), the beach is considered to be sufficiently stable to construct a wave wall at the back of the beach to protect the railway and no revetment is included.

2.4.4 Pedestrian access steps

To facilitate safe pedestrian access/egress from the beach pedestrian access steps are included within the Phase 3 design. A new set of pre-cast concrete pedestrian access steps is proposed in section E2 to reduce the possibility of pedestrians getting cut-off by the tide between the Breaches and the southern end of E2. No access steps are proposed in section E3 and E4 as full access to the beach remains. The existing set of steps in E5 near Newcastle will be replaced under the Project to provide continued beach access at this location.

2.4.5 Maintenance access ramp

To facilitate safe maintenance access for continued clearing of blockages at the Breaches, a concrete access ramp cast in situ is included within section E2. Currently an excavator crosses the railway at level crossing XR014A, tracks onto and along the beach to access the south of the Breaches. Due to space constraints between the railway boundary fence and the proposed crest wall in E2, an access ramp at XR014A has been included in the Project. The gap in the crest wall will be closed by a set of stop logs; these will be lifted out of position by the excavator when access is required and replaced once maintenance activities are complete.

2.5 Design assumptions and decisions

The design assumptions principally reflect the absence of historical monitoring data that provide information on long-term trends or patterns in beach behaviour and storm response. The primary dataset for the ground surface is the 0.1m resolution digital elevation model derived by photogrammetry from high resolution drone imagery flown in 2023 under ‘normal’ conditions. This survey extends to around mean low water. The bathymetric survey data used starts significantly further offshore than the mean low water and as such extrapolation between the two surveys has taken place.

The design assumes precast units are to be used where possible to limit the use of in-situ concrete required on site. For the concrete revetment in D5/E1, in situ concrete is preferable, but given the proximity to the rail line, it is assumed this can easily be pumped for rail running vehicles.

Due to the limitations of access to the site via land all materials will need to be delivered from the sea or via rail. At the southerly extent of the Project there is some limited land access but a temporary level crossing may be required. Where possible it is assumed that plant will operate from the railway but temporary safe storage of excavators can be achieved at the back of the beach in the areas where there is a wider buffer.

The storm duration has been reduced from 12 hours to 6 hours during Phase 3 design. This reflects the semi diurnal tides seen at the coastline. It reduces the conservativism set at concept design to allow for a more optimised design.

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.

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 of the proposed works to the railway corridor between Kilcoole and Newcastle, 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.

Based on the consultation with the stakeholders of IÉ, it has been confirmed that the scoped work including between Kilcoole and Newcastle 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, IÉ's technical management standards CCE-TMS-391 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
- 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.
- Gathering evidence of demonstrating these requirements achieved

3. Modelling results

3.1 Wave modelling

A two-dimensional spectral wave model has been 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 0.2 m with a bias of about 0.0 m.

Hourly time series of nearshore wave data are extracted at regular intervals at approximately every 1 km. The seabed level at the nearshore extraction points along the Project is approximately -6 m Mean Sea Level. The nearshore wave roses show that waves along this frontage are from northeast clockwise to southeast. The largest waves are from the northeast. The hourly wave height exceeded 1% of the year is about 2 m (1.90m to 2.10 m) and the median annual wave height is about 0.35 m (0.31m to 0.36m) 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 2.10 m (1.97m to 2.19m) for 2022-2055 and 2.25m (2.11m to 2.35m). 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 deg 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.

Extreme nearshore wave and water level data are extracted at regular intervals at approximately every 1 km. The nearshore wave extraction points, sample nearshore wave roses and joint probability curves are shown in 9. The modelled extreme wave and water level conditions are used as input for design of the coastal structures.

3.2 Cross-shore modelling

No cross-shore modelling has been conducted up to this phase of design. Modelling carried out between Newcastle and Wicklow Murrough (south of this Project) predict that beach levels could draw down by up to 1.8m and up to 10m of beach retreat could occur under the back-to-back storm scenario. The coastline along this frontage is more exposed than further south and may therefore experience more storm-induced erosion. However, as defences are proposed along most of this frontage, the impacts of this on the railway are mitigated (Figure 3-1).

It is recommended that cross-shore modelling is carried out at detailed design to quantify the impact of storm induced cross-shore sediment transport and associated shoreline changes. This will help to improve accuracy and reduce uncertainty.

3.3 Shoreline modelling

Shoreline change modelling in LITLINE has been undertaken in Phase 3 to estimate the impact of the Project on the shoreline between Kilcoole and Newcastle. The modelling exercise aimed to identify any differences in the shoreline before and after the implementation of the proposed defences. The shoreline in the “with Project” model is mostly defended, with only the area between D1 and D5 undefended.

The baseline model set-up for Phase 2 including extent and wave conditions was adopted as the proposed interventions would not be expected to change these inputs. The model was run for 50 years (2025 to 2075).

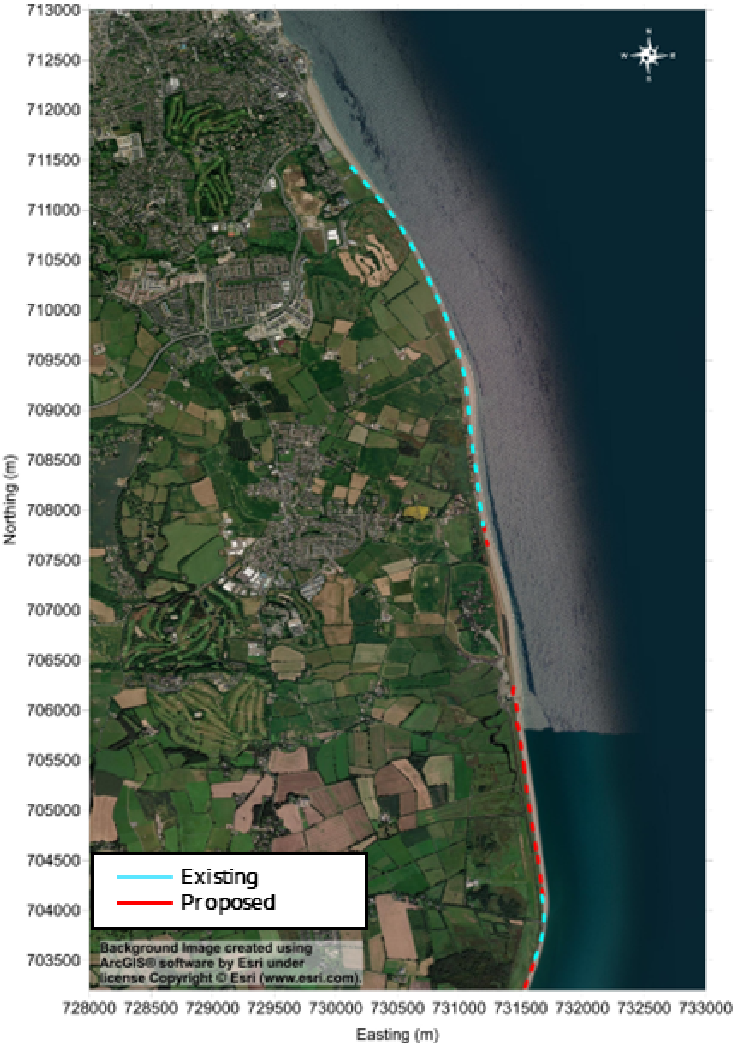


Figure 3-1 Extent of proposed defences (red dashed line) within the model area

The shoreline modelling shows that the Project produces similar trends to the baseline model (Figure 3-2):

- Accretion to the north (Greystones South Beach) where no works are proposed under the Project,
- Continued erosion trends in the areas that are already protected from Kilcoole to Newcastle; and
- Reduced erosion south of the Breaches when compared against the baseline modelling from Phase 2 (a reduction in shoreline retreat of up to 50m over the 50 year period.

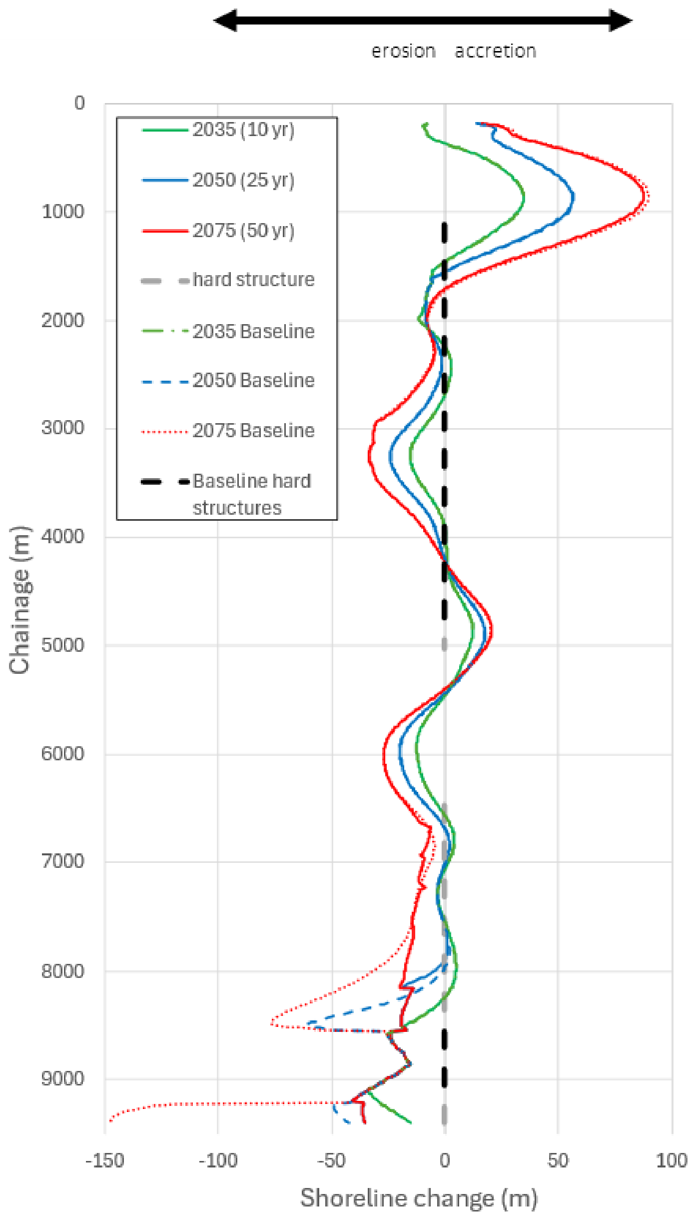


Figure 3-2 Predicted shoreline change for baseline and with Project model scenarios

3.4 CFD (if applicable)

No CFD modelling was undertaken for Kilcoole to Newcastle under Phase 3 of the Project. During detailed design, it is recommended that CFD modelling of the proposed defences be undertaken to further refine the wall and concrete revetment design adjacent to the Breaches (sections D5 and E1).

3.5 Justification of areas where no works are proposed

Approximately 1.5km of frontage between sections D1 and D5 has no works proposed under the Project and will remain undefended. This area was identified in the Option Selection Report (7694-CCA6_1-P2-ENG-CV-JAC-0001) as being relatively stable and/or having a reasonably large buffer back to the railway corridor and therefore, not requiring interventions under ECRIPP.

The shoreline modelling predicts that the northern 500m of this area will accrete over the next 50 years so this area is not expected to need defences during this timeframe. The remaining 1km north of the Breaches could erode by up to 25m by 2075; however, there is generally a wide area of vegetation at the back of the beach here (in the region of 30-40m) and therefore no intervention is required at this time. Monitoring of the beach is recommended to inform when interventions might be needed in the future.

4. Coastal processes assessment

4.1 Introduction

The Kilcoole to Newcastle coastline comprises two distinct morphological units. The coastline from Greystones to Kilcoole faces East-Northeast and is formed in low cliffs of glacial sediment that are fronted by a narrow beach. Erosion has been a persistent problem here, leading to realignment of the railway a short distance inland in recent decades and protection by a continuous rock revetment. From Kilcoole to Newcastle, the coastline faces East and forms a gravel barrier with wide beach that fronts an area of low-elevation intertidal marsh and estuarine lagoons. The railway embankment is constructed on the gravel barrier and is locally defended with rock revetements at Kilcoole station and at the mouth of the estuary, known as The Breaches. Sediment movement along the coastline regularly causes blockage of the Breaches intertidal inlet, causing flooding of intertidal marshes. The channel is therefore subject to periodic maintenance dredging by the landowner.

The Phase 2 sediment transport modelling (report 7694-CCA6_1-P2-ENG-CM-JAC-0001) indicates the northern section of the coastline within the Project area is relatively stable but erosion is predicted along the majority of the shoreline from the Breaches to the southern extent over the next 50 years. Net annual transport is to the north except for a small area with low southward net transport. This change in net direction is related to the change in the shoreline orientation and the influence of tidal currents. However, when climate change is considered this small area of southward transport switches to northwards.

5. Design methodology and results

5.1 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 5-1.

The waves were transformed to the proposed structure toe using the closest wave point to the structures, the initial toe 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.

From the nearshore wave determined, a suitable rock size has been calculated based on its stability in relation to the wave energy, the crest level determined based on EurOtop II (2018) and the toe detail based on scour. Where the revetment slope has been updated to provide greater rock stability, a further iteration of the wave overtopping assessment is needed to revise the proposed defence crest level. Wave loads on wave walls have been estimated where required for application in the structural design of the walls. The combined revetment and wave wall 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.

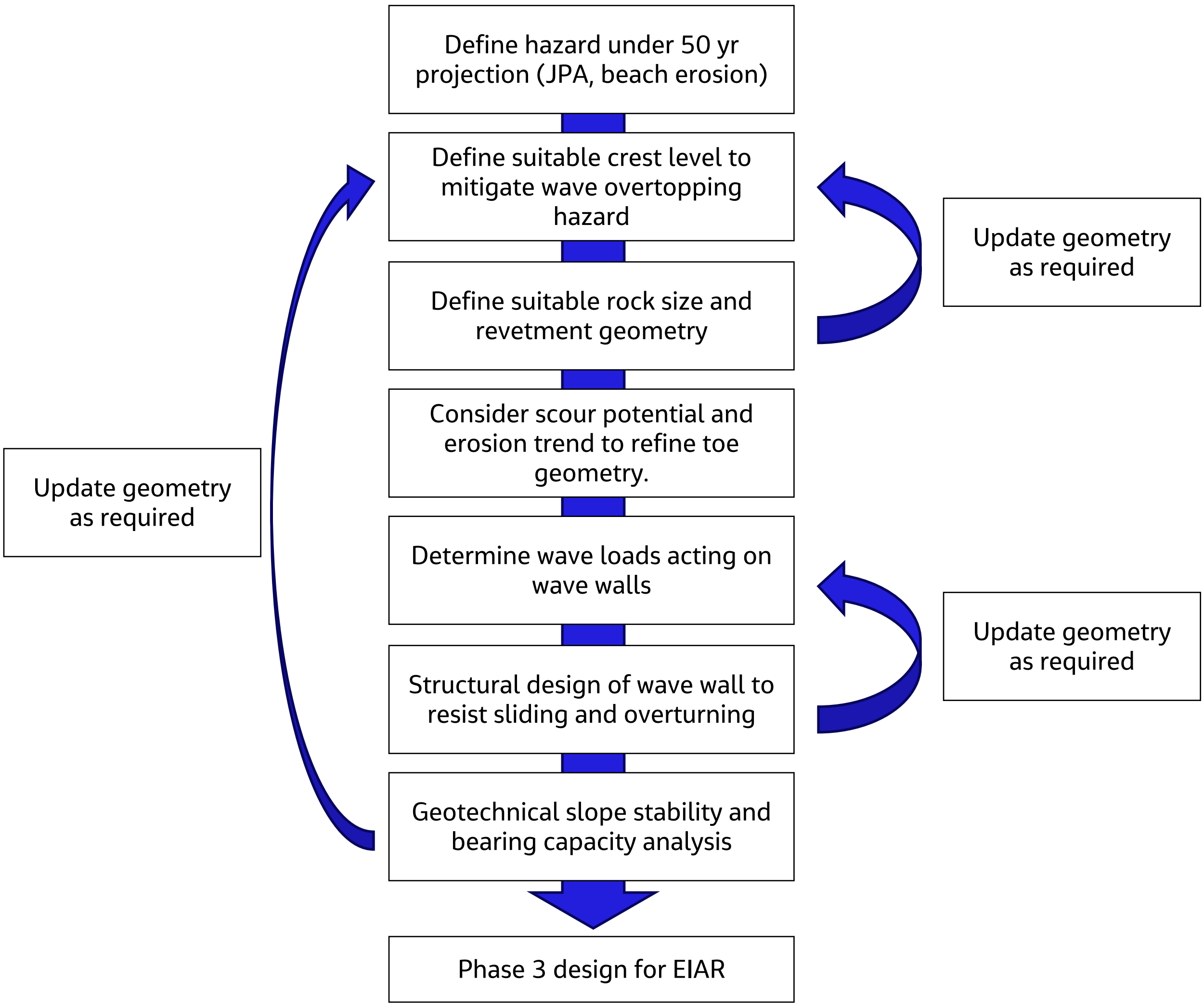


Figure 5-1 Design methodology

5.2 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 the Project are shown in Table 5-1.

Table 5-1. Key design parameters

Design Assumption	Value	Reference
Rock Density	2650kg/m^3	Typical value
Water Density	1025kg/m^3	Typical value
Concrete unit weight	25 kN/m^3	Typical value
Storm Duration	6 Hours	Semi-Diurnal Tide
Coefficient of Gravity	9.81m/s^2	Typical Value
Rock armour angle of friction	40°	
Bedding material angle of friction	30°	
Plunging Coefficient	6.2	CIRIA C683 (2007)
Surging Coefficient	1.0	CIRIA C683 (2007)
Nominal Permeability	0.1	CIRIA C683 (2007)
Nominal Permeability	0.3	Eldrup et al. (2019) for Van der Meer Sensitivity
Wave Obliquity	0 degrees	Assumed for conservativism
Damage Number (start of damage)	2	CIRIA C683 (2007)

5.2.1 Tide levels

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

Table 5-2. 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	

5.3 Coastal engineering design

5.3.1 Assessment of scour

Scour assessments have been undertaken for D5, E1 and E3 where the structure does not have the ability to dynamically vary due to beach levels. Where rock has been used in the cross-section slope, a conservative assumption regarding the beach drawdown has been considered; the beach level at each cross-section has been lowered by 1.5m based on the application of the Mean Sea Level contour retreating onshore due to shoreline erosion. It is assumed that maximum scour has already occurred within this beach lowering. A three-rock wide toe formed of two layers of rock armour is adopted to allow the revetment to respond to this change over time.

For the sections including a concrete revetment or wave wall with no revetment (D5, E1, E3 and E4), the Carpenter and Powell (1998) assessment of scour in sand fronting a vertical wall has been undertaken; the results are shown in Table 5-3. The Carpenter and Powell (1998) scour method provides a scour estimation based on experimentally derived scour plots. The estimation in the graph is determined based on wave height, wavelength and water depth at the toe of the structure. In the case of this calculation the toe of the structure has been assumed from the current LiDAR data available. Wave conditions have been transformed to the toe using Goda (2000) and the wave parameters determined are based on the Battjes and Groenendijk (2000) formulae.

The predicted scour values are small (in the region of 0.5m) and have been incorporated into the structure geometry adopted in the Phase 3 design drawings.

Table 5-3. Scour assessment results

Location	Maximum Scour Depth (m)	Maximum Scour Width (m)
D5 and E1	0.44	1.31
E3	0.49	1.46

5.3.2 Rock armour sizing

The sizing of the armour has been based on the wave action of 200-year RP waves in accordance with the Van der Meer deep water equations (1988) for non or marginally overtopped structures and Van Gent et al. shallow water equations (2004) as presented in The Rock Manual (CIRIA/CUR/CETMEF, 2007). The results of this analysis are shown in Table 5-4. Although the change in rock slope from 1 in 1.5 to 1 in 2 would permit a smaller rock grading, the 3-6t from concept design has been adopted to provide greater longevity and robustness beyond the initial 50 year design period.

Table 5-4. Rock armour sizing results

Location	Required Rock Grading	Rock Grading Adopted
D1	1-3t	3-6t
D5 and E1	0.3-1t	3-6t
E2	1-3t	3-6t
E5	1-3t	3-6t

5.3.3 Wave overtopping assessment

Wave overtopping discharges and volumes were calculated at the initial crest level adopted from Phase 2 at each revetment structure, following EurOtop II (2018) guidance. The wave overtopping discharge limit (q) considered in this analysis is 20l/s/m for structural damage. Maximum wave volume (Vmax) has also been considered, with a limit of 2000l/m. Where EurOtop does not provide a single method to accurately parametrise the defence, multiple methods have been applied. This is recommended to be confirmed at detailed design using CFD.

Table 5-5 shows the range of minimum required crest level determined in the analysis (range spans required crest level for low or high toe and q and Vmax) in addition to the adopted crest levels for design.

Table 5-5. Wave overtopping results

Location	Minimum crest level required (mODM)	Adopted Revetment Crest Level (mODM)	Adopted Wave Wall Crest Level (mODM)
D1	3.90 to 4.15	4.50	N/A
D5 and E1	5.50 to 6.24	4.50	6.50
E2	4.24 to 4.63	4.75	4.75
E3	3.84 to 5.47	N/A	5.50
E4	3.71 to 3.91	N/A	5.00
E5	4.69	4.50	5.00

In the case of E4 initial calculations were based on the concept design rock revetment. Based on the low crest level detailed above the rock revetment would be almost completely buried in the short term until the beach levels drop. This provides an operational risk with the public walk over the revetment. It is known that a coastal defence is needed in this location, so a change in the structure type has been adopted. The E4 defence is now a wave wall similar to that proposed for E3. Since run up results from concept design showed a mild requirement for protection, a wall shall be erected so that the beach levels are 1.m below the crest of the wall. This is to discourage the public from climbing on the wall. It is likely that the crest level of the wall could be lower but for the operational public requirements this has not been considered at this stage. This also reduces the volume of beach material to be excavated during construction.

5.3.4 Wave loading assessment

The crown walls are included to reduce wave overtopping impacts on the railway and therefore resistance to wave loading is provided by the passive resistance of the railway embankment on the landside. Wave loading on the wall has been considered as follows.

The wave loading acting on the crown walls was calculated using Pedersen method for crown walls protected by rock armour (1996), Shoreline Protection Manual method for walls at the back of a beach (1984) and Goda method for vertical structures with and without fronting rock armour (1990 and 1994) looking at impulsive and non-impulsive wave loads. Limitations of empirical formulae development means that for scenarios, such as those demonstrated along this coastline, a number of equations are required to be used as all equations are not fully valid. By considering multiple methods the result deemed to be most accurate based on the validity of the equations can be supported by similar results from less valid formulae.

Dimensions for the walls were sized to give a height which would limit overtopping and a base size to satisfy stability checks through overturning, uplift and sliding. The crown wall will be constructed of prefabricated mass concrete units with appropriate dimensions to suit each location were designed. Typical 'L' shaped units (with a shear key where required) were designed with appropriate dimensions to suit each location based on the results of the wave loading analysis, the results are shown in Table 5-6.

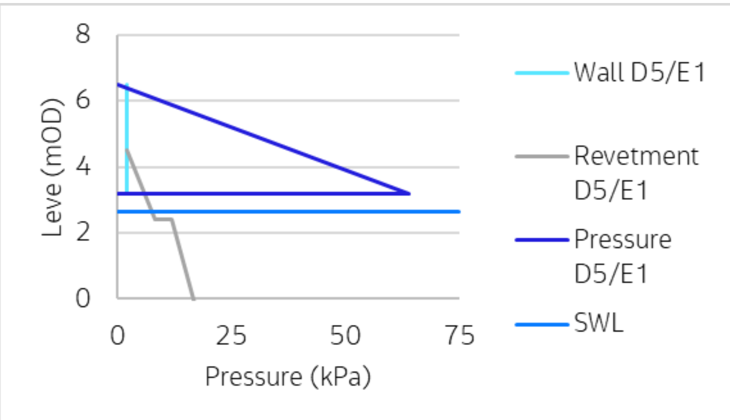
Table 5-6. Wave loading results

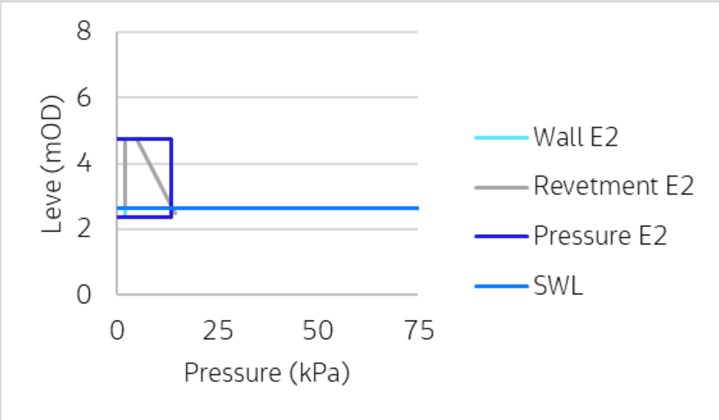
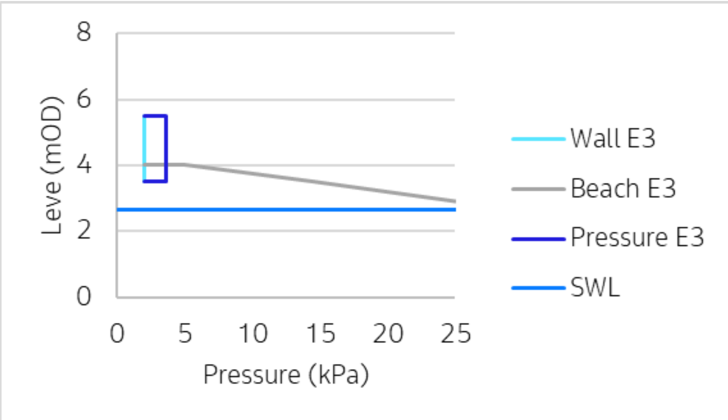
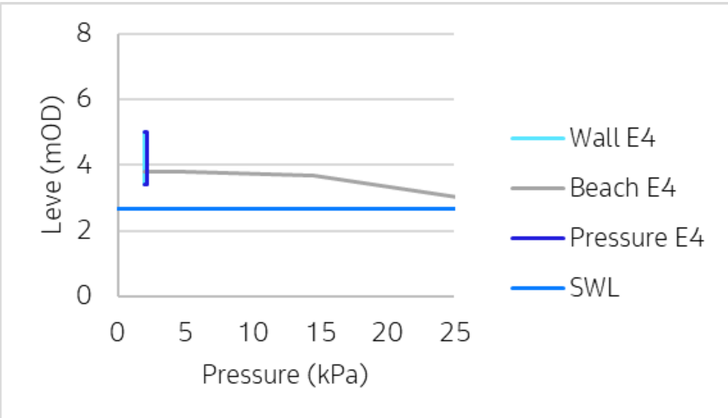
Location	Formulae	Force Calculated	Units	Value
D5 and E1	Goda Non-Impulsive (1990)	Wave induced Horizontal Force	kN/m	59.0
		Maximum uplift pressure at base	kN/m²	17.6
		Wave Induced Force / wall height	kN/m²	10.7
	Goda Impulsive (1994)	Wave induced Horizontal Force	kN/m	83.9
		Maximum uplift pressure at base	kN/m²	22.0
		Wave Induced Force / wall height	kN/m²	27.5
	Pederson (1996)	Maximum uplift pressure at base	kN/m²	0.0
		Exposed pressure	kN/m²	14.9
		Covered pressure	kN/m²	7.5
	SMP (1984)	Dynamic Pressure acting over area h'	kN/m²	4.7
		Hydrostatic Pressure at base of wall	kN/m²	12.5
E2	Goda Non-Impulsive (1990)	Wave induced Horizontal Force	kN/m	15.3
		Maximum uplift pressure at base	kN/m²	17.6
		Wave Induced Force / wall height	kN/m²	7.7
	Goda Impulsive (1994)	Wave induced Horizontal Force	kN/m	24.6
		Maximum uplift pressure at base	kN/m²	21.9
		Wave Induced Force / wall height	kN/m²	12.3
	Pederson (1996)	Maximum uplift pressure at base	kN/m²	14.9
		Exposed pressure	kN/m²	16.5
		Covered pressure	kN/m²	8.3
E3	Goda Non-Impulsive (1990)	Wave induced Horizontal Force	kN/m	17.4
		Maximum uplift pressure at base	kN/m²	23.1
		Wave Induced Force / wall height	kN/m²	11.6
	Goda Impulsive (1994)	Wave induced Horizontal Force	kN/m	28.6
		Maximum uplift pressure at base	kN/m²	29.1
		Wave Induced Force / wall height	kN/m²	19.1
	SMP (1984)	Dynamic Pressure acting over area h'	kN/m²	1.6

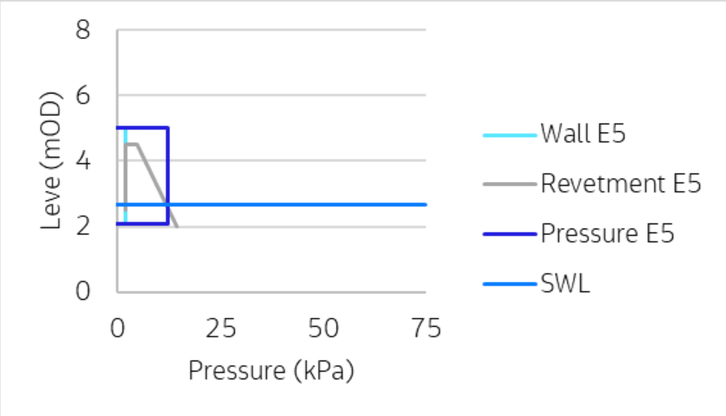
Location	Formulae	Force Calculated	Units	Value
		Hydrostatic Pressure at base of wall	kN/m²	10.2
E4	SMP (1984)	Dynamic Pressure acting over area h'	kN/m²	0.2
		Hydrostatic Pressure at base of wall	kN/m²	2.0
E5	Goda Non-Impulsive (1990)	Wave induced Horizontal Force	kN/m	30.1
		Maximum uplift pressure at base	kN/m²	17.5
		Wave Induced Force / wall height	kN/m²	16.7
	Goda Impulsive (1994)	Wave induced Horizontal Force	kN/m	28.5
		Maximum uplift pressure at base	kN/m²	22.0
		Wave Induced Force / wall height	kN/m²	15.8
	Pederson (1996)	Maximum uplift pressure at base	kN/m²	24.8
		Exposed pressure	kN/m²	24.8
		Covered pressure	kN/m²	12.4

The wave loading above shows the results of all the empirical formulae considered for each sub cell. As can be seen in the table the results vary significantly depending on the calculation undertaken and as such, the formulae derivation has been considered to determine a suitable pressure diagram. The methods selected and the pressure distribution applied in the structural design for each cross-section are summarised in Table 5-7.

Table 5-7. Wave pressure distributions

Pressure distribution and schematised defence	Wave Method Applied
	For D5 and E1 the Goda Impulsive (1994) formulae was considered with the wall length increased to the bottom of the concrete ramp. This has been done to try to reduce the effect of the limitation of its validity. Goda does not take into account a slope without roughness, as such by increasing the length of the wall and assuming no slope and then taking the force at the point of exposure for the wall allows for a conservative estimate of the pressure.

Pressure distribution and schematised defence	Wave Method Applied
	For the cross sections where the wall is protected by rock armour units (E2 and E5), Pederson (1996) formulae has been used for the development of the pressure diagrams. Pederson considers wave loading in terms of run up and provides a different result for the wave loading is covered by rock, which some of the other formulae are unable to do.
	Where the wall at the back of the beach sits higher than the highest water level it is not possible to consider Pederson and Goda formulae. In these cases (E3 and E4), the SPM formulae has been used.
	Where the wall at the back of the beach sits higher than the highest water level it is not possible to consider Pederson and Goda formulae. In these cases (E3 and E4), the SPM formulae has been used.

Pressure distribution and schematised defence	Wave Method Applied
	For the cross sections where the wall is protected by rock armour units (E2 and E5), Pederson (1996) formulae has been used for the development of the pressure diagrams. Pederson considers wave loading in terms of run up and provides a different result for the wave loading is covered by rock, which some of the other formulae are unable to do.

5.4 Structural design

5.4.1 Wave wall design

Design of the wave walls has been undertaken following methods for sliding and overturning checks as described in BS6349-1-2. Set B and Set C load factors have been adopted and resistance factors from UK national annex for BS EN 1997 have been applied. The minimum required wave wall geometry to withstand the wave pressures described in Section 5.3.4 and achieve utilisation less than 1 for both sliding and overturning has been determined.

Where rock armour is present in front of the wall, it is assumed this acts as a soil layer. Due to the marine exposure, 100mm equivalent self-weight of the concrete wall is assumed to be lost due to abrasion.

Following a review of pedestrian access along the frontage, amendments to the wall geometry were made to better align with the ground levels and provision of a safe wall height (1.2m) for all upstands. The structural calculations were updated to reflect these changes to ensure the sliding and overturning checks were still met.

5.5 Geotechnical analysis

The railway is constructed on an embankment built upon low-lying land underlain by glacial sediment in the north and a natural barrier beach system in the south. It is fronted by a narrow intertidal beach and intermittently protected by rock armour, concrete blocks and sea walls that are often in poor condition. The hinterland comprises of low land formed in glacial sediment and freshwater lagoons with.

Bedrock does not outcrop on site and ground investigations undertaken for this project have encountered rock or boulder in one position, potentially representing the Bray Head Formation (principally hard greywacke slate and quartzite lithologies) is present at elevations below -14m ODM. Bedrock is overlain by a sequence of glacial sediments, including tills and sand and gravel, that crop out to form low cliffs near Ballygannon and reach a thickness of at least 14m. Elsewhere the glacial sediments are obscured by Holocene beach, intertidal estuarine and river deposits. On the seaward side of the railway, glacial sediments are obscured by sand and gravel of the beach and barrier complex. On the landward side, glacial sediments are mantled by estuarine and fluvial deposits that comprise soft sandy silt alluvium (to around 4.5m thick) and an upper unit of peat (to around 1m thick).

The water table is close to 0m ODM and is strongly influenced by the tide. Sandier units in the alluvium and glacial gravels are particularly permeable.

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

For interpretation of ground conditions and soil/rock parameters refer to CCA6.1- Geotechnical Interpretive Report, Doc No 7694-CCA6_1-P3-ENG-CV-JAC-0002 (Appendix B).

5.5.1 Rock revetment in Sections D1, E2 and E5

Due to the similarities in the design geometries of these three sections they have been grouped together for the purpose of Phase 3 design.

For more details about coastal area CCA6.1 geotechnical assessment refer to Sections 6.1, 8 and 9 of the GIR (Appendix B).

5.5.2 Concrete revetment and L-shaped wall with a sheet pile and rock armour toe in Sections D5 and E1

Section D5 is located to the north of the Kilcoole Estuary (the Breaches) and has unfavourable ground conditions consisting of softer alluvial clay and some fen peat associated with fluvial processes present. Structure assessment within D5 and E1 consisted of: Ultimate Limit State (ULS) bearing check, sliding check, sheet pile toe assessment, Serviceability Limit State (SLS) 3D consolidation analysis to determine likely ground settlements. The ULS and SLS check were undertaken when founding soils of the structure are surcharged by self-weight of the structure (unfavourable permanent loading) and foot traffic (unfavourable variable loading). Overtopping maximum wave force was considered as accidental load case for sliding check.

For more details about coastal area CCA6.1 geotechnical assessment refer to Sections 6.2, 8 and 9 of the GIR (Appendix B).

5.5.3 Precast L-shaped concrete wall in Sections E3 and E4

Due to similarities in the geometry and ground conditions of E3 and E4 the preliminary geotechnical design of these structures has been grouped together. Structure assessment consisted of: Ultimate Limit State (ULS) bearing and sliding check, Serviceability Limit State (SLS) 1D consolidation analysis to determine likely ground settlements. The ULS and SLS check were undertaken when founding soils of the structure are surcharged by self-weight of the wall (unfavourable permanent loading), foot traffic and maintenance load (unfavourable variable loading). Overtopping maximum wave force was considered as accidental load for sliding check.

Maximum utilisation of 94% was achieved for bearing check of marine beach sands. Therefore, bearing capacity for marine beach sands is considered satisfied because the degree of utilisation was less than 100%.

The settlement assessment considered the effects of immediate settlement and primary consolidation. Settlement in fully saturated granular materials was defined by immediate settlement, whereas settlement in cohesive material was defined by primary consolidation settlement. Maximum obtained total settlement was less than10mm.

For more details about coastal area CCA6.1 geotechnical assessment refer to sections 6.3, 8 and 9 of the GIR (Appendix B).

5.6 Landscape design

Landscape design is not needed at this stage of the project given the more rural setting of the proposed works. The proposed defences make use of natural materials as far as possible, similar to existing structures along the Project frontage. The reinforced concrete wall in E3 and E4, is proposed to be located close to the railway boundary fence reducing visual intrusion. At detailed design some landscape design around the Breaches area may be beneficial once the structural design is further refined using CFD modelling.

5.7 Access

Permanent pedestrian and maintenance access through the proposed works has been considered during Phase 3 of the Project. Between Kilcoole and Newcastle, pedestrian access to the frontage is either via the pedestrian level crossing at Kilcoole Station (XR014) or via the pedestrian level crossing at Newcastle (XR015). Currently, pedestrian access is possible along the frontage either at beach level or at the back of beach area and a footway is present along the bridge at the Breaches.

Through the construction of rock revetments, safe access/egress to the beach becomes limited and therefore, where there are long sections of revetment, additional access points are proposed as part of the Project. Pedestrian access steps are proposed to be formed of pre-cast marine grade concrete step units with wing walls and handrails. These will be cast as two-step units to enable easier lifting.

In addition, maintenance access to clear blockages from the Breaches is needed. Currently access is via the level crossing XR014A to the south of the Breaches. An access ramp through the proposed wall and revetment in section E2 is proposed to provide continued maintenance access for a 30t long-reach excavator. The ramp will be formed from reinforced marine-grade concrete cast in situ and will have a 1 in 8 slope, extending through the full revetment footprint. To prevent access by pedestrians, stop logs are proposed to close the gap across the ramp.

5.8 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.

6. Preferred scheme

6.1 Description of preferred scheme design solution

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-CCA6_1-P2-ENG-CV-JAC-0001). Optimisations considered include changing the structure type, flattening the slope, altering the rock grading and optimising the shape of the wave wall to include a shear key. Iterations of design variations, coupled with the public consultation requirements, have defined the approach taken in optimisation for each section.

6.1.1 Section D1

A rock revetment shall be placed offset from the trainline along the length. The rock crest shall be +4.50mODM with a 1 in2 slope down to a toe crest level of +2.50mODM. The rock shall be two layers of 3-6t placed directly on a geotextile. The cross-section is provided in drawing 7694-CCA6_1-P3-DWG-CV-JAC-0300 and reproduced below.

The shallower slope increased the volumes of rock armour required for this section by approximately 5% but removes the underlayer, resulting in a net reduction in rock volume.

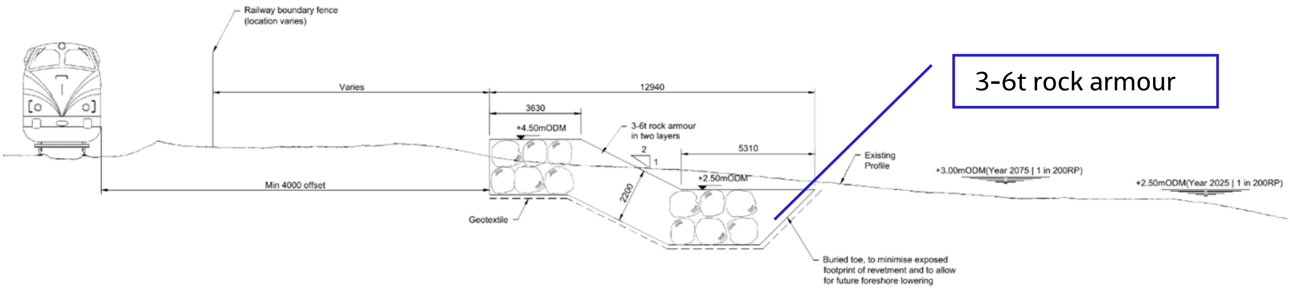


Figure 6-1. Proposed Phase 3 cross-section at D1

6.1.2 Section D5 and E1

A concrete revetment with a crown wall and a rock protected sheet pile toe shall be constructed either side of the Breaches underbridge (UBR159). The slope shall be 1 in 4 with the beach profiled to accommodate this. The concrete has been selected to minimise the rock in this section which will provide less habitat for predators to nesting little terns. The cross-section is provided in drawings 7694-CCA6_1-P3-DWG-CV-JAC-0300 and 7694-CCA6_1-P3-DWG-CV-JAC-0301 and reproduced below.

The rock volume has increased by 30% due to the addition of a tie-in detail around the Breaches bridge. The concrete volume has decreased by a similar percentage.

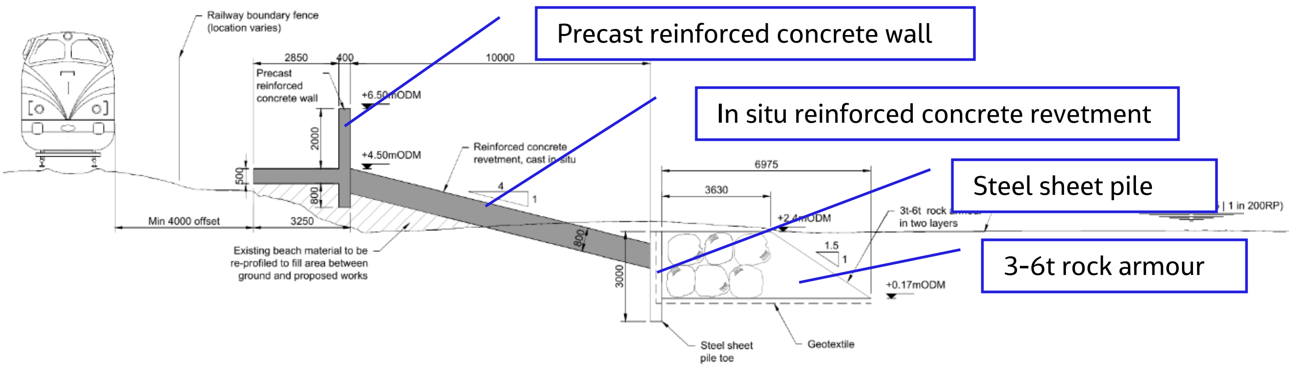


Figure 6-2. Proposed Phase 3 cross-section at D5/E1

6.1.3 Section E2

A rock revetment has been proposed with a precast reinforced concrete crown wall flush with the crest at +4.75mODM. The slope of the revetment is a 1 in 2 down to a top of toe level of +2.5mODM. The offset from the railway varies along its length but is its smallest in the north. The rock shall be 2 layers thick of 3-6t rock with a 3-rock width crest. The cross-section is provided in drawing 7694-CCA6_1-P3-DWG-CV-JAC-0301 and reproduced below.

Concrete volume has decreased by approximately 25% following refinement of the wall during Phase 3. The rock armour volume has decreased by approximately 5% and the underlayer has been excluded, removing a further 20% of the total volume from Phase 2.

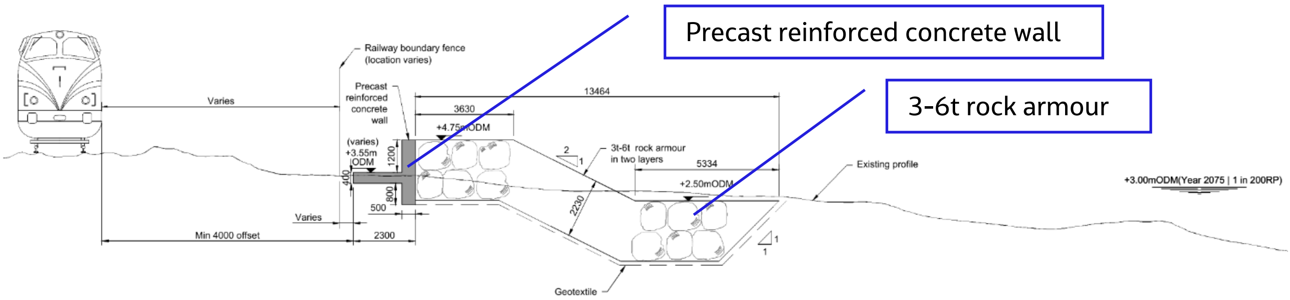


Figure 6-3. Proposed Phase 3 cross-section at E2

6.1.4 Section E3

A L-shaped precast reinforced concrete wall is proposed along this section. The proposed wall alignment follows the railway boundary fence and the wall height upstand (located at the back of the wall) is 1.2m to discourage the public from climbing on it. The alignment at the back of the beach reduces the wave exposure of the wall and improves public access along the beach. The beach is currently stable; however, a rock berm and/or toe protection may be required in the long-term if beach erosion trends increase. The wall crest level shall be +5.50mODM. The cross-section is provided in drawing 7694-CCA6_1-P3-DWG-CV-JAC-0302 and reproduced below.

The wall section has been refined during Phase 3 design and the indicative steel sheet pile excluded; concrete volume is unchanged despite the cross-section refinement.

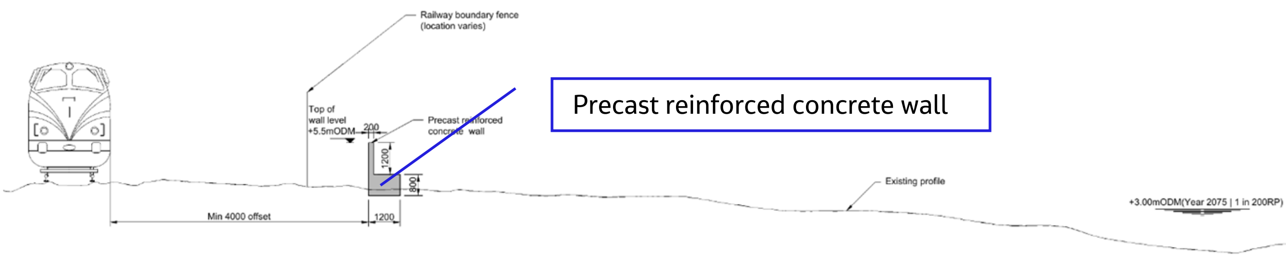


Figure 6-4. Proposed Phase 3 cross-section at E3

6.1.5 Section E4

A L-shaped precast reinforced concrete wall is proposed along this section. The proposed wall alignment follows the railway boundary fence and the wall height upstand (located at the back of the wall) is 1.2m to discourage the public from climbing on it. The alignment at the back of the beach reduces the wave exposure of the wall and improves public access along the beach. The beach is currently stable; however, a rock berm and/or toe protection may be required in the long-term if beach erosion trends increase. The wall crest level shall be +5.00mODM. The cross-section is provided in drawing 7694-CCA6_1-P3-DWG-CV-JAC-0302 and reproduced below.

The wall section has been refined during Phase 3 design and the volume of concrete reduced by approximately 70%. The rock armour revetment has been excluded, removing approximately 13,000m³ of rock from the scheme.

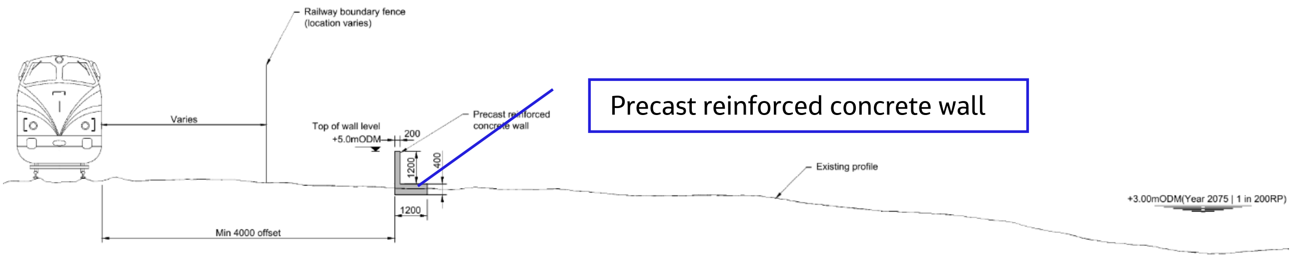


Figure 6-5. Proposed Phase 3 cross-section at E4

6.1.6 Section E5

A rock revetment with reinforced concrete crest wall shall be constructed approximately 8-10m from the railway corridor. The rock crest shall be +4.50mODM with a 1 in 2 slope down to a toe crest level of +2.00mODM. The rock shall be two layers of 3-6t placed directly on a geotextile. The crest wall shall have a crest elevation of +5.00mODM. The cross-section is provided in drawing 7694-CCA6_1-P3-DWG-CV-JAC-0303 and reproduced below.

The material volumes have increased following refinements during Phase 3 design; the concrete and rock armour volumes have increased by 20% and 4%, respectively. The underlayer has been excluded, reducing the total rock volume by approximately 30%.

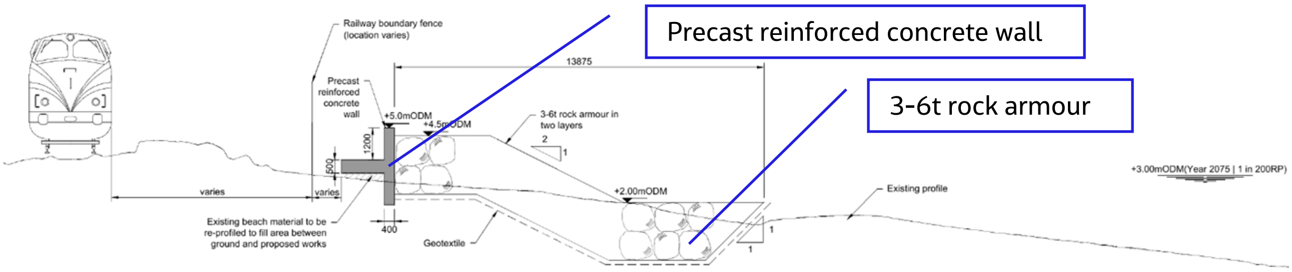


Figure 6-6. Proposed Phase 3 cross-section at E5

6.2 Future adaptability of the design

All defence structures have been designed with future long-term adaptability in mind. It is unclear what the drivers of design will be by the end of the design life and is therefore not possible to determine what design route to precisely go down. For wave walls it would make logical sense for the next stage of design to require the installation of rock in front of the wall. The rock revetment can be either be built up to increase the crest height or more rock can be added over the top of the existing rock. The slope can be flattened which will reduce overtopping but increase the plan of the structure.

For D5 and E1 future adaptability at the end of its design life is more complex due to the environmental and physical constraints in this area. The concrete will need to be reviewed for structural stability and environmental constraints will need to be reviewed in line with beach evolution data. It may be possible for rock to be added to the slope of the revetment in the future. It might be more prudent in this area to consider beach design features such as recharge to improve the design life of the existing structure rather than necessarily add to the existing cross section.

6.3 Interfaces between sub-cells and existing structures

Interfaces with existing structures exist at the northern end of D1, at the Breaches underbridge (UBR159) and along E5. Where there is no existing structure to tie into, the proposed rock revetments will taper around the crest. To the north of D1 there is an existing rock revetment with a small viewing platform; the proposed rock revetment will tie in with this structure, blending the rock armour into the existing rock.

At D5 to the north of the Breaches underbridge, the concrete revetment will have a row of sheet piles installed under the end of the concrete structure and the rock toe protection will curve round to provide additional support to this corner should the beach levels erode in the future. The interface between the north bridge pier and the concrete revetment is proposed to be composed of a rock armour tie-in detail where the rock tapers around the crest.

The same rock armour tie-in detail will be adopted for the south bridge pier for E1. The proposed transition from E1 to E2 involves locally amending the rock armour revetment and toe detail for E2 to transition to the slopes and levels of the E1 cross-section.

The southern tie-in for E2 will include the rock armour tapering around the crest and an overlap of 10m with the wave wall proposed for E3.

The transition between E3 and E4 involves a slight change in wall crest level. To reduce the visual impact, a gradual transition over several wall units is proposed.

The transition between E4 and E5 will also include an overlap in the wall alignments with E5 sitting in front of E4 for 10m. The rock armour on E5 will taper around the crest. E5 involves reconstructing the existing revetment adjacent to Kilcoole station (CDR137D)

6.4 Drawing list

Drawings prepared for the Project are summarised in Table 6-1.

Table 6-1. Drawing list for Kilcoole to Newcastle Phase 3 Design

Drawing No.	Title	Description
7694-CCA6_1-P3-DWG-CV-JAC-0010	Site location plan	Overview of frontages between Greystones to Wicklow Harbour
7694-CCA6_1-P3-DWG-CV-JAC-0100	Location plan	Location of proposed works between Greystones to Wicklow Harbour
7694-CCA6_1-P3-DWG-CV-JAC-0200	General arrangement plan 1 of 6	Location of proposed works in D1, D5 and E1
7694-CCA6_1-P3-DWG-CV-JAC-0201	General arrangement plan 2 of 6	Location of proposed works in E2
7694-CCA6_1-P3-DWG-CV-JAC-0202	General arrangement plan 3 of 6	Location of proposed works in E3
7694-CCA6_1-P3-DWG-CV-JAC-0203	General arrangement plan 4 of 6	Location of proposed works in E3
7694-CCA6_1-P3-DWG-CV-JAC-0204	General arrangement plan 5 of 6	Location of proposed works in E4 and E5
7694-CCA6_1-P3-DWG-CV-JAC-0205	General arrangement plan 6 of 6	Location of proposed works in E5
7694-CCA6_1-P3-DWG-CV-JAC-0300	General arrangement cross sections 1 of 4	Proposed cross-sections at D1 and D5
7694-CCA6_1-P3-DWG-CV-JAC-0301	General arrangement cross sections 2 of 4	Proposed cross-sections at E1 and E2

Drawing No.	Title	Description
7694-CCA6_1-P3-DWG-CV-JAC-0302	General arrangement cross sections 3 of 4	Proposed cross-sections at E3 and E4
7694-CCA6_1-P3-DWG-CV-JAC-0303	General arrangement cross sections 4 of 4	Proposed cross-sections at E5
7694-CCA6_1-P3-DWG-CV-JAC-0400	Typical access details	Proposed pedestrian access steps and maintenance access ramp

6.5 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 extremely limited road access. Therefore, it is expected that all materials and plant will need to be transported to site via rail or sea. The Project area is long (2.5km of proposed works within a 4km frontage) but due to the nature and locations of the proposed works, several work frontages could proceed at the same time, independently of each other.

The volumes of rock armour required for the scheme between Kilcoole and Newcastle are significant and therefore, the procurement of rock is of key importance to the success of the Project. Ultimately the contractor will decide where the rock is supplied from but there is a high probability that it may 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 and low carbon way to source large volumes of rock.

The expectation at the Phase 3 design stage is that rock will arrive at the Project site via barge, will be discharged at low tide and moved up the beach for construction of the revetments by land-based plant.

Precast reinforced concrete wave walls will be manufactured off-site and transported via RRVs to the installation area. The majority of precast elements (e.g. D5, E1, E3 and E4) are to be placed close to the railway corridor so installation from the rail line will be possible in these sub-cells. At E2 and E5, the proposed wall alignment is typically 10m from the rail line; this may therefore require a crane to sit between the rail and the wall alignment working along the footprint of the proposed defence. For approximately 100m of E2, the proposed wall alignment is 20m from the railway; it is therefore assumed that a crane will sit between the railway and defence alignment and could lift wall units directly from the RRVs into final position.

The rock revetments at E2 and E5 will be constructed to the underside of the rock armour crest, the walls will be installed and the final two-layer rock crest placed. At D5/E1, the concrete revetment is proposed to be cast in situ as the size and weight of precast units would be difficult to transport to site and lift into position. Rail mounted concrete mixing plant and concrete pumps can be used to place the concrete. The steel sheet toe piles will be push-piled first, then the beach will be reprofiled behind the piles and the insitu revetment slab with integral capping beam cast. The rock toe protection will then be placed and finally the wave wall installed.

6.6 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. Due to the proximity of the proposed works between Kilcoole and Newcastle to several designated sites (The Murrough SPA (Code 004186), the Murrough Wetland SAC (Code 002249) and the Murrough pNHA (Code 000730)), it is anticipated that a full EIAR will be required for the Project.

6.7 Health and Safety

A Design Hazard Elimination & Risk Reduction Register or DEHERR, has been developed alongside the design of the preferred option during Phase 3 design. The DEHERR is presented in Appendix C and has been prepared following Jacobs' De5ign ('Five in Design') principles. 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 for them to consider when developing their safe system of works. A table presenting the principal identified risks is provided in Table 6-2. Top five risks identified in the DEHERR.

Table 6-2. Top five risks identified in the DEHERR

Risk ID.	Activity	Potential Hazard	Design to Reduce Risk	Residual Risk	Action By	Comments
32	Transportation of precast units	Striking of live services overhead rail cables causing electrocution, and/or explosion.	No overhead rail cables are present at this location. Known services identified on drawings	Striking of live services overhead rail cables causing electrocution, and/or explosion is still possible beyond the immediate site area.	Designer / Contractor	Further consideration by design team at detailed design. Client to agree procedures for cable isolation. Contractor to provide method statement and safe system of work.
52	Public accessing beach areas during storm conditions	Risk of drowning	Beach access points included to reduce the likelihood of becoming cut off by the tide. Designer to advise Client that warning signs should be installed at the access points through the defence on to the beach	Risk of drowning	Designer / Client	Client to ensure signage is installed at visible locations along the access points. Signs should also be provided to warn pedestrians of presence of maintenance vehicles
1 / 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).
7	Unstable ground conditions	Potential for site operatives or plant to become stuck in pockets of soft or loose ground. Instability of plant working in area of low soil strength. Risk of suffocation, crush injuries from sinking into ground	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.
16	Handling and placement of rock armour	Death/injury to site personnel from loss of control of rocks (movement due to soft ground conditions/dropped by construction plant).	Early design of the rock structures & grading to allow delivery rock delivery to commence early in programme.	Death/injury to site personnel from loss of control of rocks (movement due to soft ground conditions/dropped by construction plant). Risk of injury to eye as a result of rock splinters.	Contractor	Contractor to prepare method statement and safe system of work. Experienced Contractor and subcontractors to be appointed.

6.7.1 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 between Kilcoole and Newcastle, 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 work are non-significant in accordance with the Common Safety Method Risk Assessment (CSM-RA) and does not require Authorisation to Place in Service (APIS).

Limited maintenance of the designed engineering is proposed because the revetments are designed to respond to beach movement and toe scour. Inspection and renewal of seals between adjacent concrete wall units will be required; however, this can be undertaken at a safe distance from the railway corridor. It is possible that re-profiling of the rock revetments may be necessary if a storm exceeding the design conditions occurs within the revetments design life.

6.8 Recommendations for refinement at detailed design

It is noted that the Phase 3 design has only considered one location along each section for the development of the design and that the existing beach levels, slopes and exposure varies along each section. At detailed design, each section of works should be looked at in greater detail, with coastal analyses (scour calculations, rock armour stability, wave loading and wave overtopping assessments), structural and geotechnical analyses further refined to allow the Phase 3 proposals to be tailored to local topography, ground conditions and wave exposure.

Further cross-shore modelling to understand beach response to storms is also recommended. In addition, modelling of the structures in CFD could enable further refinements in the rock gradings, cross-sections and required volumes of rock armour, enabling value engineering of the proposed design to be undertaken. Detailed analysis of sections D5/E1 in CFD are recommended to more accurately represent the combined defence proposed in the coastal analyses.

The principal construction risks identified relate to the interaction of plant on site with construction workers and the public, construction in an exposed marine environment and unforeseen ground conditions.

The use of plant will be carefully planned and managed during construction to ensure the safety of workers. Working zones will be clearly marked and fenced to prevent members of the public from providing access to the works and/or areas where beach access/egress may be temporarily reduced.

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.

7. Conclusions and Next Steps

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

- Phase 1 – Project Scope and Approval (completed);
- Phase 2 – Concept, Feasibility and Options (completed);
- **Phase 3 – Phase3 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.

7.1 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.

7.2 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-CCA6_2-P2-PLA-EV-JAC-0010). There was generally support for the scheme and the rock revetment proposals. Continued access along the shoreline and to the beach was a key requirement from the public. Access provision has been assessed during the Phase 3 design and access behind the proposed defences has been retained and access points to the beach have been included at regular intervals. Impact on views were also raised. During Phase 3 Design the height of structures have been minimised as far as possible whilst maintaining the standard of protection.

A second round of consultation (PC2) will be undertaken in September 2025.

The Project will now undertake an environmental assessment and report it 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.

7.3 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.

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.

7.4 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.

8. 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 (CCA)	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
Computational Fluid Dynamics (CFD)	Numerical modelling to analyse and solve complex fluid dynamics problems based on the application of the Navier-Stokes equations. In coastal engineering design, CFD can help refine the design of coastal structures.
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 corridor.

Term	Description
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
Joint Probability Analysis (JPA)	Analysis combining the probability of two variables occurring at the same time to determine representative design conditions.
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
Option Selection Report (OSR)	Phase 2 deliverable documenting option selection for the Project
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.
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.

Term	Description
Return Period (RP)	Interval of reoccurrence of an event (e.g. storm, water level); indicates the expected frequency in years that an event can be typically expected to occur over the very long term.
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
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.
Road Rail Vehicle (RRV)	Dual mode vehicle that can operate on tracks and road.
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.
Unmanned Aerial Vehicle (UAV)	Drone or other unmanned aircraft used to capture high resolution topographic data through photogrammetry.
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

9. References

Causeway Geotech Ltd., 2025, East Coast Railway Infrastructure Protection Projects (ECRIPP); CCA6.1 Greystones – Ground Investigation, Factual Ground Investigation Report

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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

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.

Appendix A. Modelling outputs

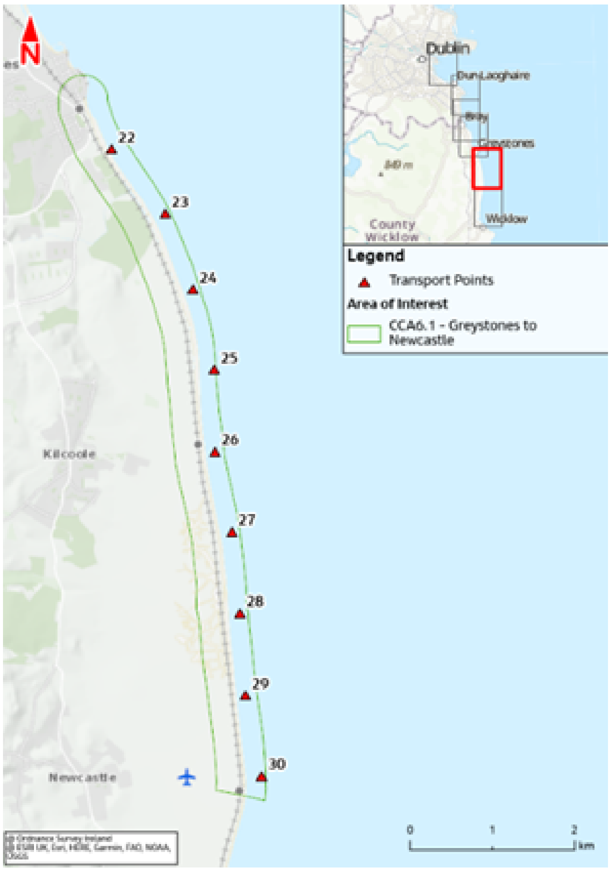

Figure A-1. Wave model extraction locations along CCA6.1.

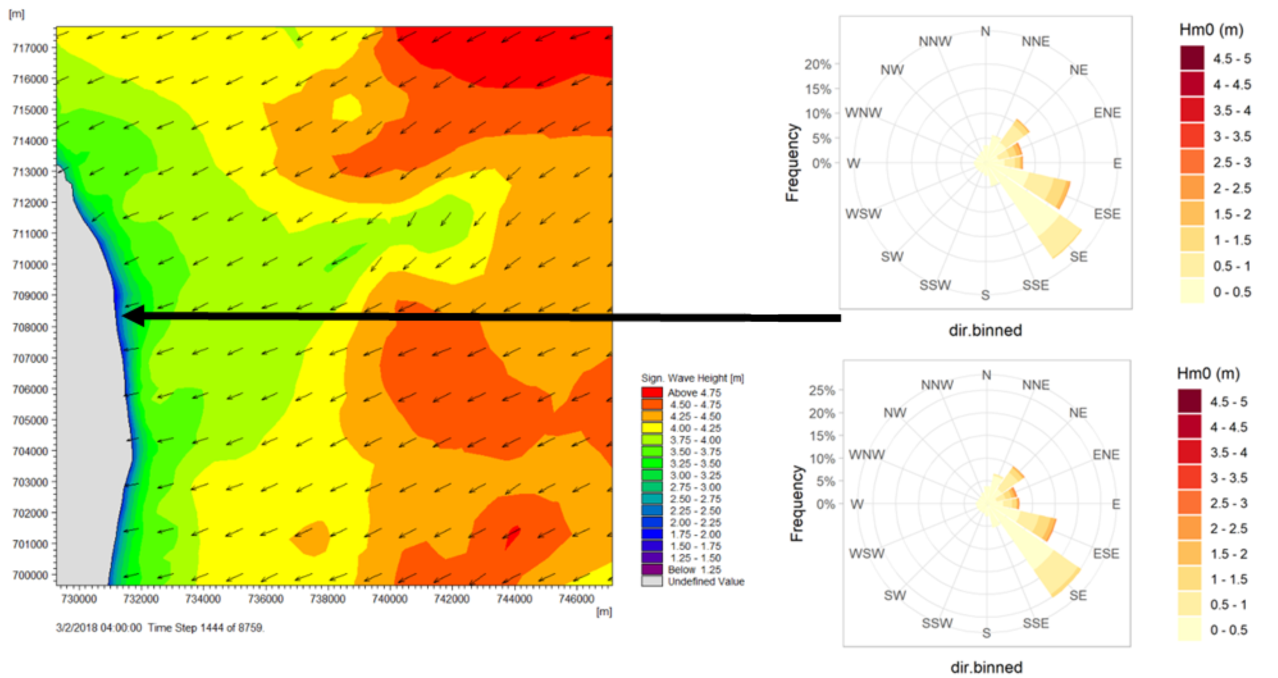

Figure A-2. CCA6.1: Contour plot showing event of 3rd March 2018 (left), wave height roses - Jan/1988-Dec/2021 (top right) & Jan/2056-Dec/2100 (bottom right)

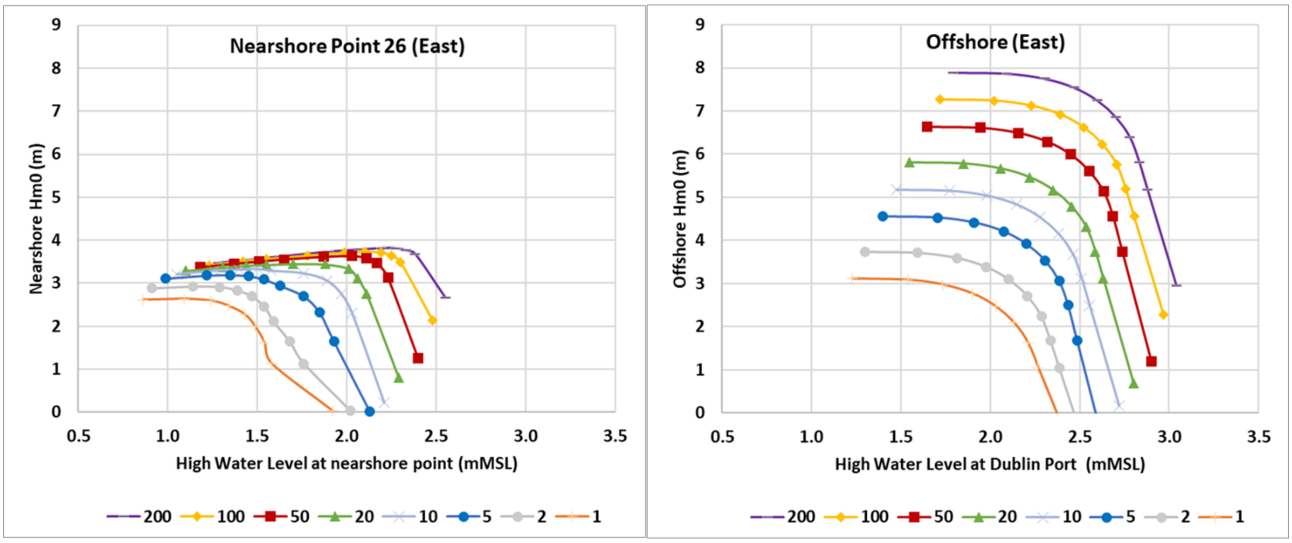

Figure A-3. Joint probability curve at nearshore point 26 in CCA6.1 (left) compared to offshore (right) for waves from the East. Nearshore wave extracted at depth of -5.6 mMSL Note any changes in the high water levels from Dublin to the nearshore point is due to 2D variations in water level.

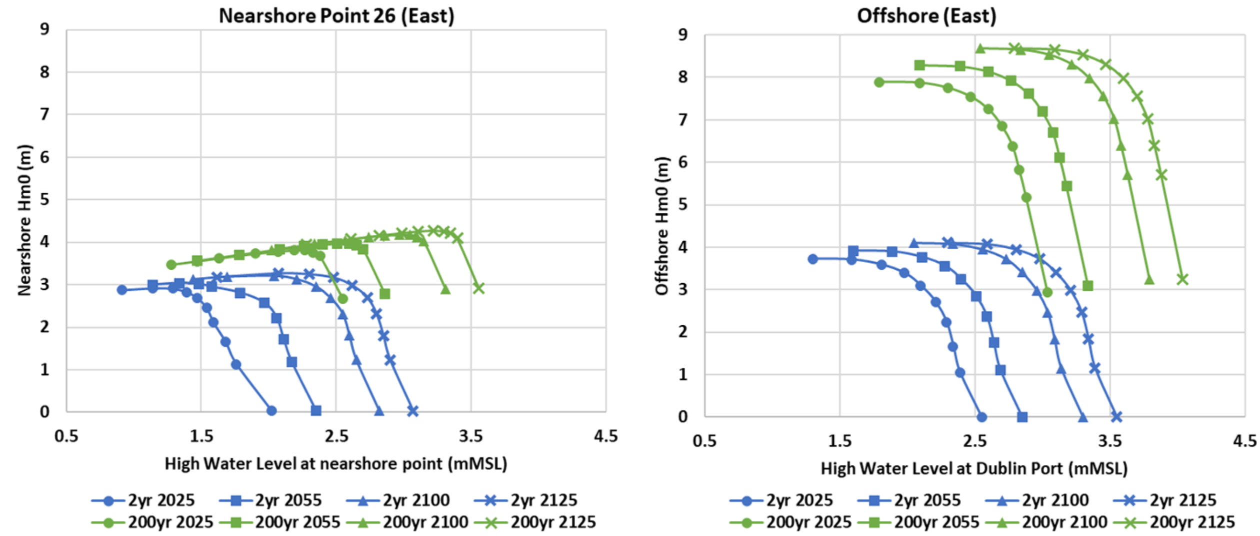

Figure A-4. Impact of climate change on joint probability curves for 1 in 2 year and 1in 200 year return periods at nearshore point 26 in CCA6.1 (left) and Offshore (right) for waves from East. Nearshore wave extracted at depth of -5.6 mMSL. Note any changes in the high water levels from Dublin to the nearshore point is due to 2D variations in water level.

Appendix B. Geotechnical outputs

Document Number	Document Title
7694-CCA6_1-P3-ENG-CV-JAC-0002	Geotechnical Interpretive Report

Appendix C. DEHERR – (designers risk assessment)

Document Number	Document Title
7694-CCA6_1-P3-REG-CV-JAC-0003	Design Hazard Elimination Risk Register