East Coast Railway Infrastructure Protection Projects

Phase 3 Design Report

Bray Head to Greystones North Beach

COASTAL CELL AREA 5

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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 Bray Head and Greystones North Beach (the 'Project').

The Project aims to deliver improved coastal protection measures to the railway infrastructure, addressing vulnerabilities related to coastal erosion, wave overtopping and cliff instability that are projected to worsen due to climate change. To improve resilience, the Project will be designed to withstand against a1 in 200 year return period event, for a minimum of 50 years (i.e. to year 2075).

This report presents the Phase 3 designs for Bray Head to Greystones North Beach, Coastal Cell Area 5 (CCA5) that will subsequently inform the detailed design phase.

The Project is situated in the rural area between Bray Head and Greystones North Beach. For the purpose of design, this Project is considered in two sections:

- Bray Head, covering the hard rock coast of Bray Head between Naylor's Cove, near Raheen Park in Bray, to the exit of Tunnel 3 on the southern side of Bray Head near Greystones. This section comprises c. 4km of railway with intermittent coastal defences up to 100m in length between three tunnelled sections.
- Greystones North Beach covering the predominantly soft cliff frontage. It extends from the exit of Tunnel 3 on the southern side of Bray Head to Greystones marina. This section comprises c. 3km of railway that includes a single tunnel.

The objectives at Bray Head are to provide improved resilience against coastal erosion and sea-level rise to existing masonry structures that carry the railway around the Bray Head. At Greystones North Beach the objective is to manage erosion of the soft cliffs to ensure the railway is not at risk over the design life.

In both locations, the engineering measures comprise rock revetments that are placed at the base of the existing structures (Bray Head) or at the toe of the cliff (Greystones North Beach). Structures have typically been designed to address wave energy under projected sea-level rise to 2075 and 1:200 year return period storm, however, because of the difficulty in site access, those at Bray Head have been designed for projected sea-level to 2100. Scour potential has been mitigated through design of the revetment toe, which varies according to the substrate material. Revetments in Bray Head are founded on bedrock, but those in Greystones North Beach are founded in glacial sediments.

Future adaptability in Bray Head has been considered during the design process. Due to the construction constraints in this area of the coastline a design for the year 2100 (i.e. 75 years) has been developed. Over longer timescales that are beyond the project scope, additional rock and structure reprofiling can be undertaken to respond to changes in coastal hazard.

Greystones North Beach design considered future adaptability in terms of the revetment structures. The concept of the design is to create two hardpoints along the cliff where toe erosion will be halted, and an intervening undefended section where erosion will be permitted to continue until a naturally stable bay of predictable form will develop. The design can be adapted by adjustments to the structures, which may be lengthened or increased in number.

The principal construction risks relate to interactions between construction equipment and people during construction, cliff failures during construction at Greystones North Beach, construction in the intertidal zone and/or from barges and unforeseen ground conditions. These risks will be mitigated through careful planning and additional site-specific ground investigation.

The next phase of the Project covers Statutory Process focussed on preparation of an environmental assessments which will comprise the preparation of an Environmental Impact Assessment Report (EIAR), Appropriate Assessment (AA) Screening Report and Natura Impact Statement (NIS).

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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 Irish Rail (IÉ) to provide improved coastal protection against predicted climate change effects of sea level rise and coastal erosion on the east coast railway corridor between Merrion Gates (Co. Dublin) and Wicklow Harbour (Co. Wicklow) (Figure 1-1)

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

This report presents the Phase 3 designs for Bray Head to Greystones North Beach, Coastal Cell Area 5 (CCA5) (hereafter referred to as the 'Project').

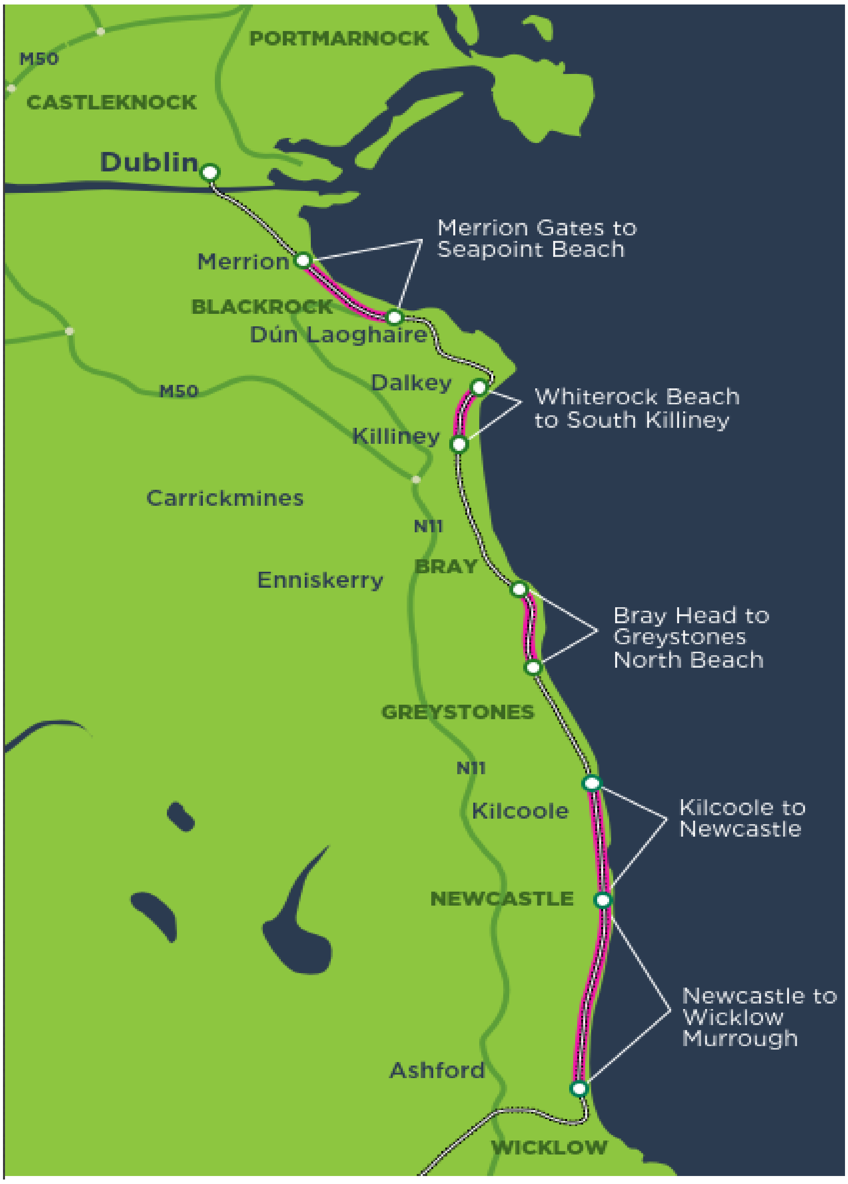

Figure 1-1 ECRIPP locations

1.2 Project location and description

The Project is situated in the rural area between Bray Head and Greystones North Beach. For the purpose of design this Project is considered in two sections:

- Bray Head (CCA5A), (Figure 1-2) covering the hard rock coast of Bray Head between Naylors Cove, near Raheen Park in Bray, to the exit of Tunnel 3 on the southern side of Bray Head. This section comprises c. 4km of railway with intermittent coastal defences up to 100m in length between three tunnelled sections.
- Greystones North Beach (CCA5B) (Figure 1-3) covering the predominantly soft cliff frontage. It extends from the exit of Tunnel 3 on the southern side of Bray Head to Greystones Harbour Marina. This section comprises c. 3km of railway that includes a single tunnel.

This section of railway has experienced problems from cliff instability and erosion, leading to several sections being rerouted inland, often though tunnels. The railway around Bray Head has been particularly susceptible to rockfalls, and extensive areas of rock netting and rockfall catch fences have been constructed. The section of railway between Tunnels 3 and 4 at the southern part of Bray Head, have also been susceptible to debris runouts onto the line. Issues relating to drainage and rock slope stability are addressed by ongoing IÉ maintenance work and do not form part of this Design Report.

The main hazards here are related to cliff instability above and below the railway and coastal defence undermining below the railway. Beach erosion at the northern and southern parts of the Project area are increasing exposure of the coastline to other hazards.

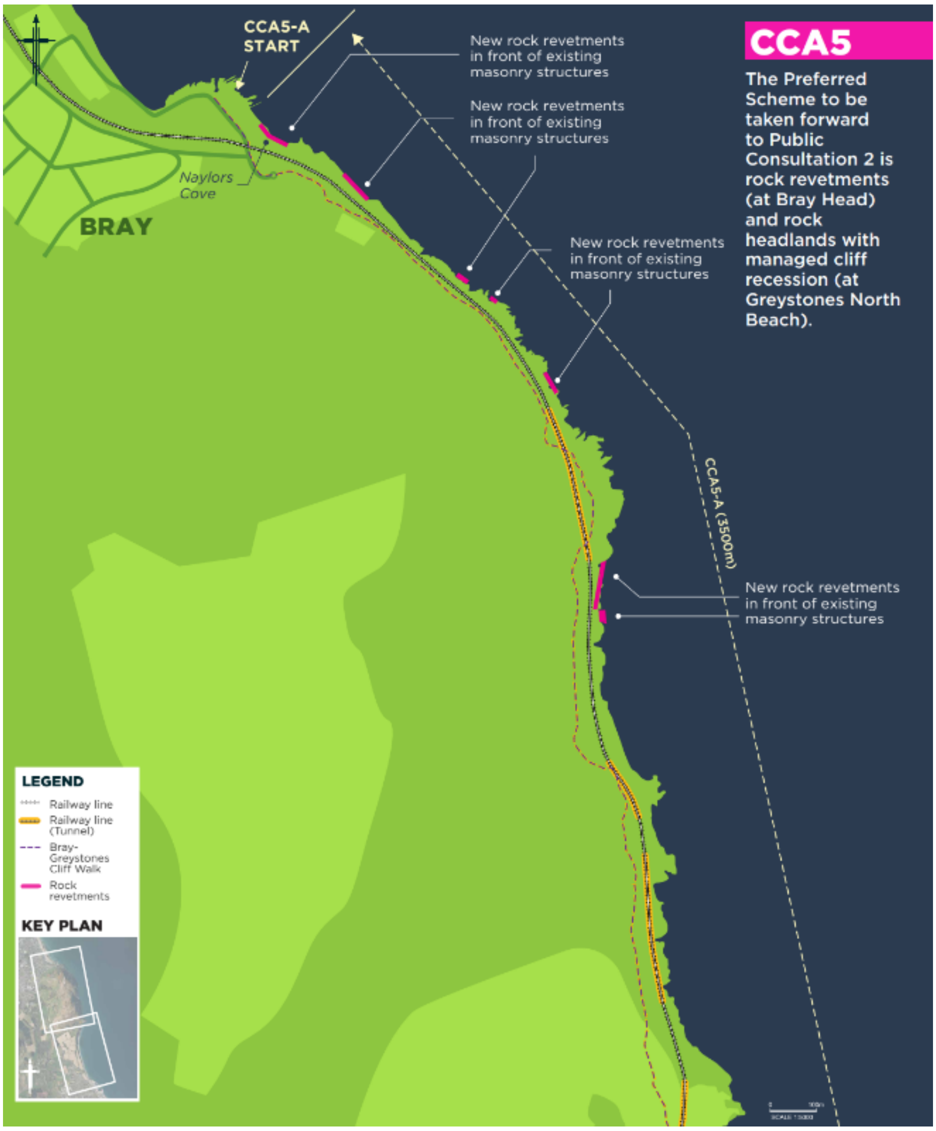

Figure 1-2 Bray Head location plan

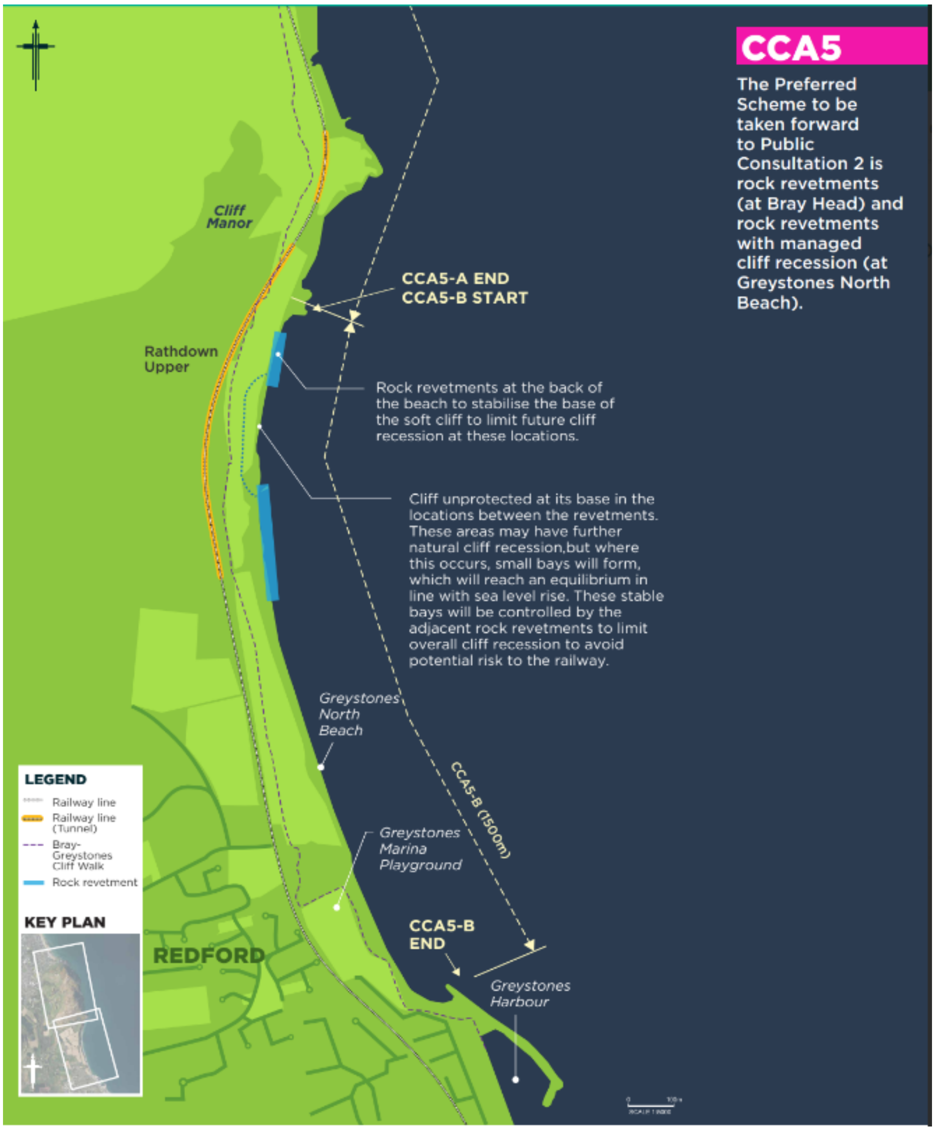

Figure 1-3 Greystones North Beach location plan

1.3 Project objectives

The objectives of engineering interventions for the Project are two-fold:

- At Bray Head (CCA5A), rock revetments have been designed to afford protection to existing masonry structures that span bedrock gullies or protect embankments. The existing structures are sound but will not afford the necessary protection to the railway against the combined impact of rising sea-levels and extreme weather events.
- At Greystones North Beach (CCA5B), two rock revetments are designed to control erosion of soft cliffs. At present, the cliff recession rate is rapid and some sections of the railway could be threatened within a 50-year period. The design comprises two revetments to inhibit toe erosion at sections of soft cliffs where the railway is most at risk. The design of the revetments allows the remainder of the cliffed frontage to continue to erode and ultimately form a static equilibrium bay ('stable bay') that will afford natural protection to the remainder of the railway.

1.3.1 Transport benefits

The proposed works will ensure that the railway remains safe to operate over the next 50 years. At Bray Head, the proposed works will provide resilience to undermining, structural failure of the existing lower coastal defences from wave loads and wave overtopping. At Greystones North Beach works will provide resilience against coastal erosion and cliff instability.

IE infrastructure at the site comprises a single-track railway with overhead electrification equipment (OHLE) that forms the southern limit of the electrified DART service that links Dublin and Greystones. A diesel service that links Dublin with Wicklow and the Wexford Europort also uses the railway.

The CCA5 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 Stage (preliminary stage 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, feasibility studies and stakeholder consultation.

1.5 Purpose of this report

This document provides the Phase 3 Design Report for Bray Head to Greystones North Beach CCA5. The report defines the design that will subsequently inform detailed design.

This report should be read in conjunction with 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

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

Table 2-1 Design criteria

Criteria	Description	Reference
Design Life	▪ 50 years for new permanent works ▪ Variable for existing structures, beaches and soft solutions	Scope of Services
Proposed Standard of Protection – Damage to structures	0.5% AEP (1 in 200RP)	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Proposed Standard of Protection – Reduction of disruption to services	10% AEP (1 in 10RP) for damage to rolling stock / lineside assets 100% AEP (1 in 1RP) for temporary line speed restrictions	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Proposed Standard of Protection – Pedestrian Safety	100% AEP (1 in 1RP)	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002
Wave overtopping thresholds	Design protection measures to limit wave overtopping to: ▪ 20 l/s/m or 2000 l/m under a 0.5% AEP storm	Refer to technical note 7694-ZZ-P1-MMO-CV-JAC-0002. Note, the limit was increased from 20 to 30l/s/m given this is a cliffed section. Refer to Section 5.2.3.
Maintenance requirements	For new permanent works: zero heavy maintenance for up to 25 years.	Scope of Services

2.2 Design standards

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

Table 2-2 Relevant design standards and codes of practice

Discipline	Code/Standard	Application
Chief Civil Engineer (CCE), IE Requirements	PWY-1101 Requirements for Track and Structures Clearances	Geometrical constraints on proposed solutions, including installation and maintenance
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-389 Drawing Certification Process	All drawings produced on the project
Chief Civil Engineer (CCE), IE Requirements	CCE-TMS-399 Glossary of Civil and Permanent Way Engineering Term	All technical reporting relating to railway terminology

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

Discipline	Code/Standard	Application
Structural	EN 1992 Eurocode 2	Applies to the design of civil engineering works in concrete. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design in EN 1990.
Structural	EN 1996 Eurocode 6	Applies to the design of civil engineering works, or parts thereof, in masonry. The execution is covered to the extent that is necessary to indicate the quality of the construction materials and products that should be used and the standard of workmanship on site needed to comply with the assumptions made in the design rules
Structural	BS EN 206-1:2000 Concrete – Part 1: Specification, performance, production and conformity	Additional reference where Eurocode does not cover a specific topic adequately for the design of concrete structures
Geotechnical	Eurocode 7: Geotechnical Design	Default standard for geotechnical design, but may require other supporting documentation e.g. British Standards
Geotechnical	Engineers Ireland Specification and Related Documents for Ground Investigation in Ireland, 2016	For defining approach and content of the Ground Investigation Interpretive Report
Coastal	The Rock Manual: The use of rock in hydraulic engineering (Ciria/CUR/CETMEF, 2007)	Design of rock structures, including: armour stability, scour, toe design
Coastal	BS6349 Maritime Works	Design of breakwaters, dredging, geotechnical design and materials used in maritime works
Coastal	Manual on wave overtopping of sea defences and related structures (EurOtop, 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).

At Greystones North Beach, an alternative design to the detached headlands presented at concept design has been developed as part of the Phase 3 design. The revetments described in this report have been adopted to maximise safe access to the beach by:

- Avoiding the creation of a narrow walkway between the structure and the base of the periodically unstable cliff, and
- Ensuring that tidal cut off occurs for the minimum length of the time for beach users.

The Phase 3 design also results in both the plan area of the rock structure and the volume of materials being used being greatly reduced.

2.4 Design element

2.4.1 Rock revetments

At Bray Head, rock revetments will be placed directly in front (seaward) of the existing exposed masonry structures (those remaining structures that did not have their resilience improved in the 2000's with the same rock revetment solution). These existing structures are at risk of undermining due to erosion at the toe. The rock revetments will prevent the loss of the foreshore in front of the masonry structures which leads to undermining, whilst also dissipating wave energy and therefore significantly reducing the wave energy impacting the structures. The rock revetments also significantly reduce wave overtopping onto the upper apron and retaining structures during storm events. The rock revetments will comprise of one layer of large-sized graded armour rock. The rock grading has been selected to provide stability over the scheme life using modelled wave conditions that allow for sea level rise.

At Greystones North Beach, two rock revetments will be constructed on the beach at the toe of the soft cliffs. The structures will comprise of two layers of graded armour rock. The rock grading has been selected to provide stability over the scheme life using modelled wave conditions that allow for projected sea level rise. The rock revetments will dissipate wave energy, reduce wave run up and overtopping. This will reduce the toe erosion of the cliffs behind the structures as the wave energy reaching the toe of the cliffs will be significantly reduced. Erosion of the beach and backshore will continue to occur either side of the revetments, but at locations where the railway line is not threatened.

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. Surveys of the beach and hinterland have therefore been derived from a 2m resolution aerial LiDAR survey captured in 2011 and 0.1m resolution digital elevation model derived by photogrammetry from high resolution drone imagery flown in 2023 under 'normal' and 'post-storm' conditions. These surveys extend to around mean low water. Where-ever possible, designs are based on the most recent data. In the absence of long-term beach monitoring data, the three available datasets have been used to assess the magnitude of variation in beach levels since 2011, with 2D profiles extracted at locations where structures are proposed.

1m or 2m resolution bathymetric data are available for the majority of the offshore area of the site, which were surveyed in the last 10 to 15 years.

The beach at the southern end of Greystones North Beach is very thin and has been completely stripped out in the past. The absence of any significant beach volume means that there is no significant sediment volume to transport, and therefore sediment transport and beach evolution modelling is not appropriate. Therefore, the main driver of erosion risk along the Project area is cliff recession and estimation of shoreline changes has been based on historical rates of change of the cliff top position. Beach evolution modelling may be required in subsequent stages if the design changes.

Rock has been assumed to be delivered by barge for all sections of this Project.

2.6 Safety certification and approvals

2.6.1 Workplace safety: roles and responsibilities

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

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

The 2013 Regulations ensure that health and safety is:

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

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

Clients must:

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

Project Supervisor for the Design Process (PSDP) must:

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

Project Supervisor for the Construction Stage (PSCS) must:

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

Designers must:

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

Contractors, including subcontractors, must:

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

2.6.2 Notification and training

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

In relation to training and competence:

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

2.6.3 Iarnród Éireann safety standards

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

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

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

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

The objectives of the safety certification and approval are to ensure:

- The compliance with applicable legal and technical requirements;
- The credible hazards identified, and their impact assessed; and
- Safety associated with the work sufficiently controlled and managed.

The following will be considered to support the safety certification and approval:

- The detailed definition of the change (i.e. scope of work and activities);
- Project team with the roles and responsibilities defined for project delivery and safety assurance;
- Identification of compliance requirements;
- Identification of potential affected stakeholders;
- Hazards identification and risk assessment to support the identification, assessment, control and management of safety hazards and risks; and
- Gathering evidence of demonstrating these requirements achieved.

3. Modelling results

3.1 Cross-shore and shoreline modelling

No cross-shore sediment movement modelling, or shoreline change modelling has been conducted up to this phase of design. This is because the beach is thin, and the erosion hazard is related to cliff recession (Section 2.5). If the design changes, shoreline modelling may be necessary to quantify beach response to alongshore sediment transport gradients.

3.2 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.

Nearshore wave data are extracted at regular intervals at approximately every 1km. The seabed level at the nearshore extraction points along the Project area is generally between -6m and -8m Mean Sea Level. The wave roses show that nearshore waves are mainly from the SE and ESE along this frontage (Figure 3-1). However, the largest waves are from the NE and ENE. The wave height exceeded 1% of the year is about 2m (1.95m to 2.10m) and the median annual wave height is about 0.3m (0.27m to 0.31m) for wave climate simulated for 1988 to 2021.

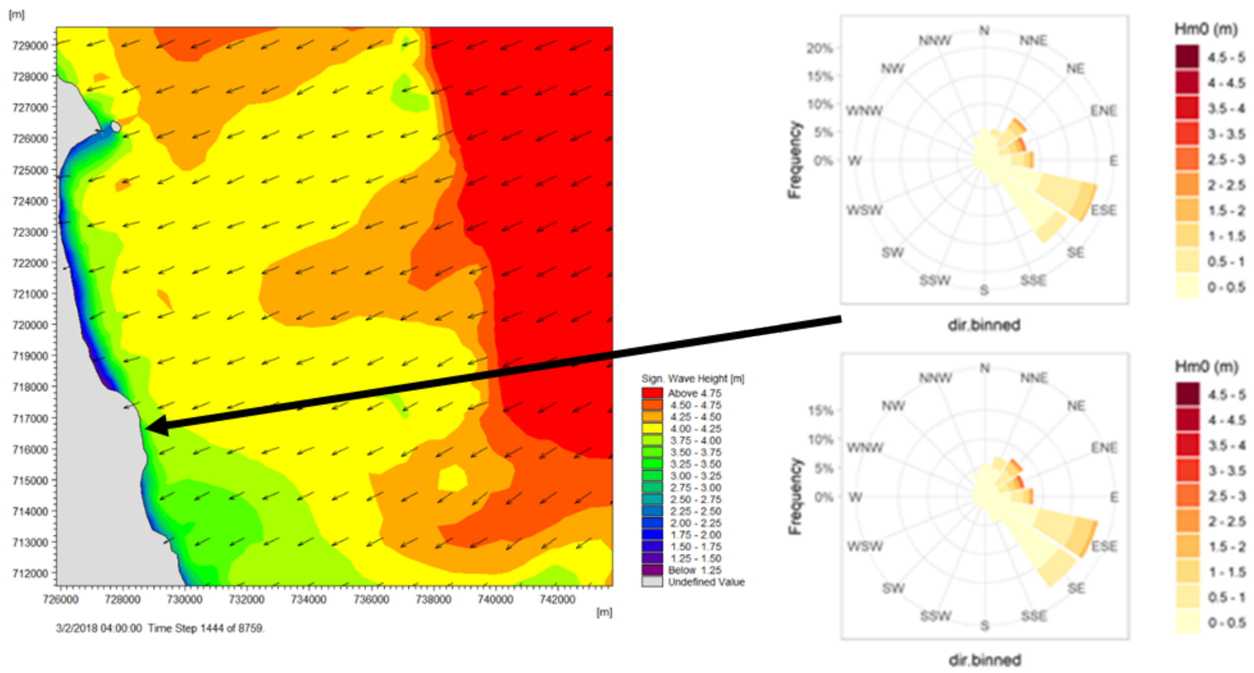

Figure 3-1 The Project area contour plot showing event of 3rd March 2018 (left), wave height roses - Jan/1988-Dec/2021 (top right) & Jan/2056-Dec/2100 (bottom right)

4. Coastal processes assessment

4.1 Introduction

Bray Head forms a major boundary along the east coast that inhibits sediment transport to adjacent cells. The headland is formed of resistant bedrock and erosion rates are negligible, however zones of weakness at joints and faults have created gullies that are spanned by masonry structures. These structures are vulnerable to sea-level rise and require additional measures to ensure climate resilience.

Greystones North Beach is backed by glacial sediment cliffs that are eroding rapidly, particularly at times of low beach levels. The railway is generally inland of the cliffs and not vulnerable to erosion. However, there are areas of particular concern where there is a risk of erosion, and these areas are proposed for engineering to limit erosion and cliff recession. The beach receives no sediment supply from south because Greystones Marina inhibits along shore transport. There is a limited supply of sediment from cliff recession, but most of the material is too small to be retained on the beach. Preserving the protective beach is therefore important in moderating the rate of cliff recession under projected sea-level rise.

4.2 Long-term shoreline rates of change

Long-term shoreline rates of change have been assessed for Greystones North Beach (CCA5B) using georeferenced aerial imagery and historical mapping in a GIS. These shoreline rates of change have been used to project future recession of the cliff at Greystones to understand the potential implications of future coastal erosion on the railway under different scenarios.

The input data for this assessment were georeferenced historical OSI mapping and aerial imagery. Historical OSI mapping and aerial imagery were provided for a range of epochs with different resolution and accuracy. The mapping error was calculated by comparing the position of known features in each dataset with the same features shown in the 2023 survey captured for this project. The positional error of each dataset is indicated by the root-squared mean error (RMSE), shown in Table 4-1. The combined error between the given epoch and the 2023 aerial imagery indicates the margin of error when comparing change between the two datasets. The combined error is a sum of the error in the two epochs, divided by the number of years between the epochs.

Table 4-1 Georeferenced datasets and uncertainty

Data Epoch	Data Type	Mean RMSE (±m)	Combined Error to 2023 (±m)
1830 B&W 6 inch	OS	10.34	0.07
1900 25 inch	OS	3.55	0.05
1940	OSI	6.68	0.11
1995	Aerial	4.46	0.25
2000	Aerial	3.23	0.25
2005	Aerial	2.75	0.29
2012	Aerial	1.69	0.37
2017	Aerial	2.43	0.41
2023	Aerial (UAV) and DEM	0.03	-

Long-term rates of shoreline change were analysed and assessed using the GIS-based Digital Shoreline Analysis System (DSAS). The cliff top / headscarp was mapped in each dataset and attributed with the year and RMSE associated to each epoch. Expert judgement was required to interpret the historical OS maps, and to interpret the headscarp in areas where dense vegetation obscured the view of the headscarp, and where aerial imagery resolution and quality was poor. Figure 4-1 shows cliff top position change over c. 200 years, illustrating the effect of differential erosion rates on the cliff line position, with the greatest change apparent in the southern and central parts of the frontage. Change in the north of the bay may in part reflect rerouting of the railway inland (though Tunnel 4) in the early 20th century. Change in the south of the bay is more marked and reflects both natural erosion processes acting on the soft cliffs and also accelerated erosion in recent years that result from outflanking of defences at Greystones Marina.

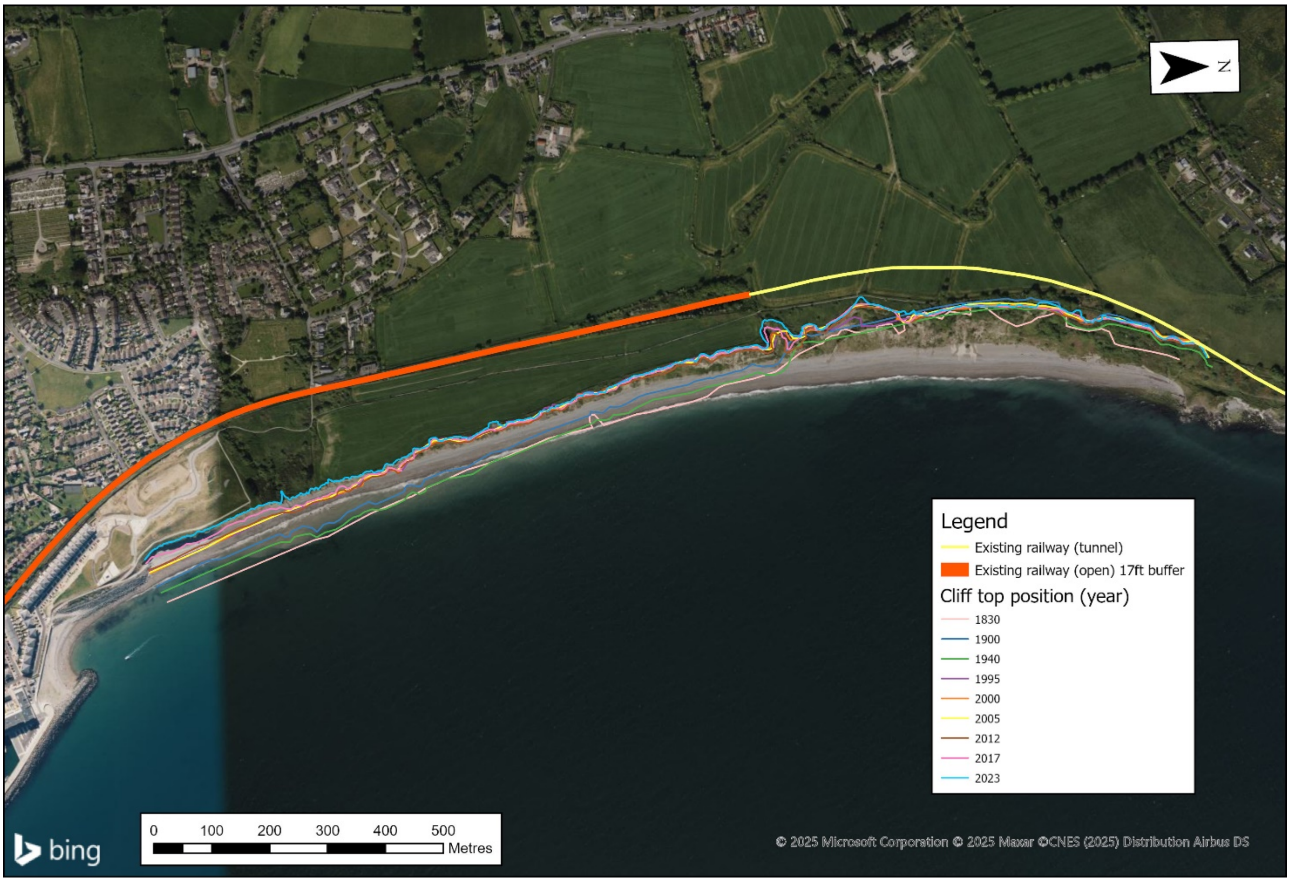

Figure 4-1 Cliff top position between 1830 and 2023

Transects were cast along a digitised baseline at 10m intervals. The distance is measured from the baseline along each transect to the intersection with the mapped shoreline. These distances are used to calculate rate of change statistics:

- End point rate (EPR)/Overall rate of change (OCR): annual change rate (m/year) calculated by dividing the distance of shoreline movement by the time elapsed between the oldest and the most recent shoreline.
- Weighted linear regression (WLR): annual change rate (m/year) determined by fitting a least-squares regression line to all shoreline points for a transect. More reliable data are given greater emphasis or weight towards determining a best-fit line.

The rate of change statistics were calculated for the epoch range 1830 to 2023. The rate of change statistics for the individual transects were mapped and summarised to provide the averages for distinct cliff behavioural units (CBUs) within the coastal unit that display similar geomorphological characteristics and patterns of change.

Historical long-term shoreline rates of change are shown in Table 4-2, including the end point rates (EPR) and weighted linear regression (WLR) rate of change statistics. The results show rates of shoreline recession at the cliff top are notably greater at the south of Greystones North Beach and lowest to the north.

Table 4-2 Shoreline change assessment results

Epoch	CBU	Average EPR (m/year)	Maximum EPR (m/year)	Uncertainty EPR +/- (m)	Average WLR (m/year)	Max. WLR (m/year)	Uncertainty WLR +/- (m)
1830 to 2023	1 – South	0.42	0.69	0.05	0.52	0.63	0.20
	2 – Central	0.30	0.37	0.05	0.32	0.45	0.11
	3 – North	0.13	0.31	0.05	0.07	0.44	0.10

A map showing the spatial variation in rates of change (WLR) between the epochs 1830 and 2023 are shown in Figure 4-2. Note that the inland extent of transects is arbitrarily placed and is not a prediction of future change or the location of the present-day shoreline. The transects are colour coded by a scale of change rates as shown in the map legend.

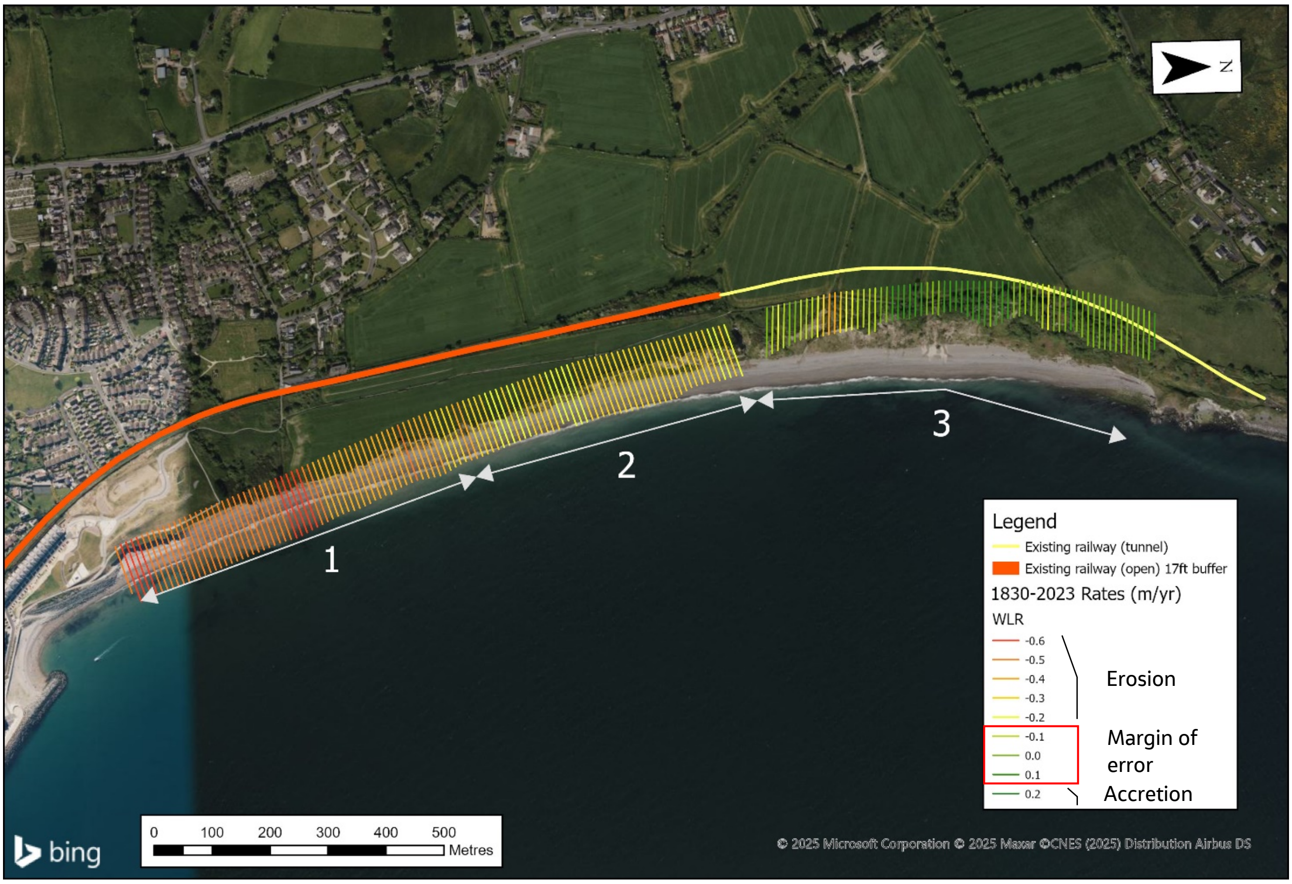

Figure 4-2 Shoreline change assessment long-term pattern of change between 1830 and 2023

The results of this assessment are summarised as follows:

- CBU 1 – South: High long-term recorded rates of cliff top retreat, where the mean rate is -0.42m/y, and the maximum is >0.69m/y. Analysis of cliff recession data between individual data epochs shows that cliff recession rates have increased notably in the south of Greystones North Beach in the past decade. This trend follows the construction of Greystones Marina and coastal defence structures immediately to the south of the cliffs, indicating erosion is potentially being accelerated by outflanking of the structures.
- CBU 2 – Central: Medium to high rates of cliff top retreat, where the mean rate is 0.30m/y, and maximum rates is 0.37m/y.

- CBU 3 – North: Mostly negligible change at the headscarp. Localised moderate rates of erosion (<0.4m/year) were shown to occur towards the incised stream valley. The accuracy and reliability of the historical rates of change are poor in this location due to difficulties in identifying the headscarp in aerial imagery because of dense vegetation. In addition, the coastal slope and headscarp have been significantly modified between the different epochs of mapping by the construction and dismantling of the railway on the coastal slope during the 20th century. This gives rise to potentially misleading indicators of natural cliff change which are instead attributable to anthropogenic modification of the slope and headscarp. However, it is clear that this CBU has consistently eroded at a very low rate.

4.3 Future cliff recession projections

Future recession projections were calculated by the linear projection of the rates of historical long-term change. The results show the potential implications of future coastal erosion on the railway under the Do-Nothing scenario.

The overall change rate or the median recession rate underpin the linear projection, depending on which statistic best fits with the observed data. The overall change rate is the modelled long-term rate of change between 1830 and 2017. The median recession rate is modelled for each transect in several steps:

1. Generate annual recession rates for each interval between epochs
2. Remove recession rates that are smaller than the mapping error (combined RMSE)
3. Where 10 or more data points remain, calculate the median rate (Probability 50%) and the 15th percentile (~1 standard deviation below median) to indicate fast rates of recession (Probability 15%)
4. Where less than 10 data points remain, the overall change rate is used

In order to determine whether the overall change rate or the median are the better fit to the observed data, a time series was created to show observed shoreline movement and modelled distances by transect. The modelled data with the lowest RMSE were selected as the best fit with the observed data. The recession rates were summarised by calculating the mean annual recession rate for both the Optimistic (Probability 50%) and Worst Case (Probability 15%) scenarios for each cliff behavioural unit. Cliff recession distances were projected for years: 2075, 2100 and 2125, and plotted on a series of maps to show the proximity of the railway to the cliff top edge and infer potential implications of future coastal erosion and cliff top recession. Projected recession rates do not take into account changes to coastal defences and assumes uniform change over the cliff behavioural unit over time.

The projections under the Do-Nothing scenario are provided in Table 4-3 and illustrated in Figure 4-3.

Table 4-3 Optimistic and Worst Case cliff recession projections for Do Nothing scenario

CBU	Recession Rates by CBU – 50% Probability (Optimistic)				Recession Rates by CBU – 15% Probability (Worst Case)			
	Rate (m/year)	Cliff top recession - Distance (m)			Rate (m/year)	Cliff top recession - Distance (m)		
		2075	2100	2125		2075	2100	2125
1 - South	0.41	20.72	31.07	41.43	1.06	52.98	79.47	105.97
2 - Central	0.30	14.93	22.40	29.86	0.69	34.49	51.73	68.98
3 - North	0.14	6.85	10.27	13.70	0.42	21.00	31.50	42.00

In summary, the results of the future cliff top recession under the Do-Nothing scenario show that:

- At Greystones North Beach, significant cliff top recession is projected for the Optimistic and Worst-case scenarios. The railway emerges from the tunnel at the north of the frontage and may be impacted within 50 to 100 years for the Worst-case scenario. However, for most of this unit, grassland provides a wide buffer between the railway, including a potential loop on the seaward side of the line, and the cliff top edge and no impacts are projected. It is therefore recommended that the coastline in the vicinity of the southern tunnel portal and incises stream gully are defended.
- The historical recession rates used to calculate the projected recession distances in CBU 3 – North have a high degree of uncertainty, but since the tunnel is close to the cliff edge, it may be impacted in 50 to 100 years, particularly in the north of the bay. Behaviour of these cliffs is response to climate change and sea-level rise is uncertain, and a longer record magnitude and frequency of cliff failure events resulting in headscarp retreat is required to make more reliable project reliable projections. Given this uncertainty, and the vulnerability of the railway tunnel to erosion, it is recommended that this section of shoreline be defended

Figure 4-3 Do Nothing scenario projections over 100 years at Greystones North Beach

4.3.1 Historical record of cliff instability

The cliffs at Greystones North Beach are formed of soft sediment, with bedrock cropping out at the toe in the northern part of the bay. In the northern part of the frontage, the cliffs have a 'slope over wall' morphology, whereby the basal part of the cliff is formed in greywacke and shale bedrock, overlain by glacial sediments including tills and granular outwash. The bedrock dips below beach level in the south in the bay, where cliffs are exclusively formed of glacial materials. Historically, the railway was cut into the coastal slope and benched on the glacial sediment. Erosion resulted in the railway being rerouted inland in the early 20th Century, with remnants of structures still found on the beach.

Greystone North Beach is fronted by an unprotected shingle beach which provides a line of defence at the toe of the cliffs against the sea. Beach material at the southern and northern margins of the bay is particularly susceptible to being drawn alongshore/offshore in storm conditions to expose a steeply sloping underlying clay platform that reduces the protection of the toe of the soft cliffs. This can lead to increased rates of toe erosion and headscarp retreat. Furthermore, elevated pore water pressures caused by sustained wet weather weaken the soft cliffs and promote cliff failures that may cause cliff retreat or debris runout, even where beaches are healthy and there is negligible toe erosion. This is particularly apparent along gullies and drainage channels which cut through the coastal slope.

The mechanisms of cliff instability comprise:

- Shallow mudslides in glacial sediment and debris mantling the cliff which can lead to localised headscarp retreat and debris runout onto the beach. These are triggered by toe erosion and surface water runoff following sustained wet weather.
- Shallow slides in debris mantling the cliff which lead to debris runout onto the beach and occasionally to headscarp retreat. This is likely to be triggered by toe erosion, sustained wet weather and potentially wave splash during storms coincident with high tides.
- Deep-seated rotational failures which can lead to significant headscarp retreat, debris runout and potentially toe heave. This is triggered by toe erosion aided by sustained wet weather.
- Rock/debris fall from steep cliff/slope which can lead to limited headscarp retreat. Failed rock blocks and/or debris falls, bounces or rolls onto the beach below the cliff toe. This is likely to be triggered by sub-aerial weathering, toe erosion and undercutting, and sustained wet weather.

A review of historical evidence has been undertaken to establish a record of past cliff instability events, including the magnitude (volume and extent), frequency and mechanisms of failure (Figure 4 and Figure 5). Historical evidence includes archive OSI georeferenced digital aerial imagery over seven epochs from 1995 to 2018, and thirteen epochs of Google Earth imagery from 1995 to present. Recent aerial imagery collected by the project in 2023 was also analysed. Debris volumes were estimated based on available digital elevation models and aerial imagery. The data have previously been used to support an assessment of historical cliff recession and a protection of future change. Refer to Section 4.2 of this report for detailed results of this assessment.

The database indicates that erosion of the soft cliffs at Greystones North Beach over the last c. 200 years has been widespread, however failures resulting in cliff top retreat has predominantly occurred in the southern and central parts of the bay. The following describes the main characteristics of cliff instability observed along the frontage in the historical record, as shown in Figure 4-4 and Figure 4-5.

At the southern part of the frontage, where no engineering is proposed, the following events were observed:

- This section of frontage is characterised by widespread frequent slumping and deep-seated slides on the cliffs formed in limestone gravels and sands, resulting in cliff top retreat.
- Smaller magnitude events of less than 20m^3 occur multiple times annually, and moderate to high magnitude events of between 20m^3 to 100m^3 occur up to twice per year on average. The high magnitude and frequency of slumping of the cliff was particularly notable in imagery dated 2013 and 2023, which may be associated to the effects of the construction of Greystones Harbour and beach lowering trends at the southern end of the bay.
- Very high magnitude debris volumes of between 100m^3 to180m^3 associated with deep-seated slides were recorded, with three of these events occurred in 2023, and one in 2016, 2017 and 2020.
- Runout distances of slumped debris onto the beach is typically 5m or less, however in higher magnitude events, debris runout extends 8m to 11m.

In the central part of the frontage, where the southern rock revetment is proposed, the following were observed:

- This section of frontage is characterised primarily by widespread shallow slides and mudslides in glacial till. Deep-seated slides and slumping also occur within the limestone gravels.
- Mudsliding occurs in the pre-failed debris which mantles the slope. Mudslides occur multiple times annually and typically result in debris runout onto the beach of between 5m to 10m, and up to 14m for higher magnitude events.
- Shallow slides were evident throughout the central frontage which led to limited headscarp retreat. Small magnitude events of less than 20m^3 occur annually, whilst moderate to high magnitude events of 20m^3 to 90m^3 occur every several years on average, mainly within in the cliffs formed of glacial tills. Runout distanced are generally between 5m to 10m, and up to 15m for the higher magnitude events.
- Slumping and deep-seated slides are evident in the limestone gravels occur less frequently than other modes of failure, between 1 to 2 years on average, and are of moderate and high (50m^3 to 100m^3) to very high (>100m^3) magnitude, and result in headscarp retreat. The highest magnitude event recorded at Greystones North Beach occurred in the till cliffs in 2021, resulting in 240m^3 of debris on the beach, whereby the runout distance was 17m.

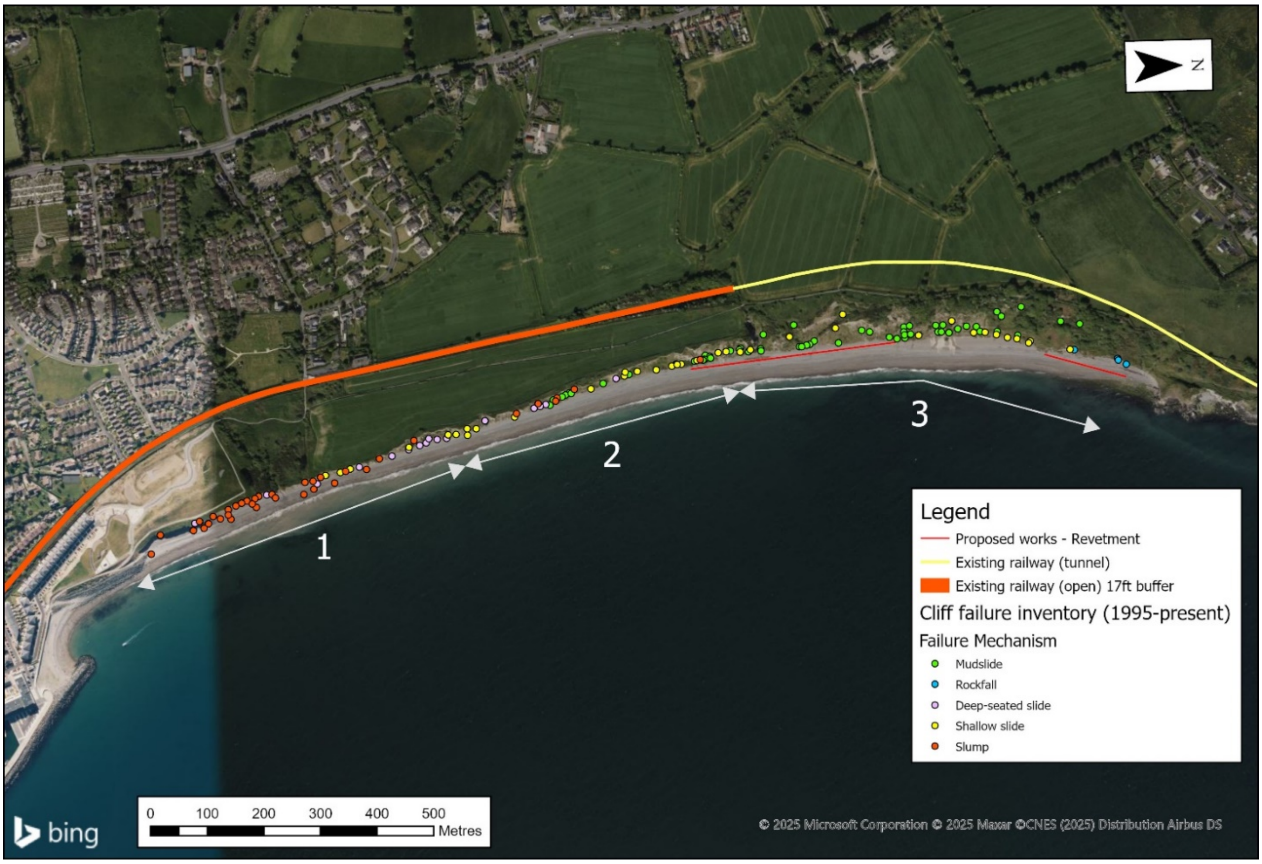

Figure 4-4 Cliff failure inventory between 1995 and present, shown by failure mechanism type (Cliff units: 1-South, 2-Central, 3-North)

In the northern part of frontage, where the northern rock revetment is proposed, the following were observed:

- This section of frontage is predominantly characterised by localised mudslide features in pre-failed debris which mantles the slope. Most mudslide activity was recorded on the cliff face, particularly in the vicinity of the drainage channels cut in the glacial tills (one at the southern portal of Tunnel 4 is natural, a second further north was cut by IE to drain the tunnel). Both of these channels are a focus of slope instability through shallow mudslides, due to saturation of debris mantling the slope, and entrainment of sediment and loss of support at the base of the channels. Mudsliding occurs multiple times annually. There is evidence of episodic headscarp retreat, however headscarp retreat occurs infrequently at a very low annual rate. Headscarp retreat due to mudsliding was notable in 2021 whereby the cliff top path was severed. Mudslide debris runout distances are typically 5m to 10m, however in exceptional instances, runout distances can extend to 20m.

- Shallow slides on the pre-failed debris mantling the slope were also recorded throughout the historical record, though they occur less than once a year on average. These events were primarily moderate to high magnitude (20m^3 to 100m^3), with only two very high magnitude (100m^3 to 180m^3) events occurring earlier in the record. Debris runout distances were generally between 5m to 11m, and 18m in the highest magnitude event. Aerial imagery shows that beach level fluctuates considerably, burying and exposing the railway remnants between epochs. Beach lowering and cliff toe erosion of the softer sediment is a primary trigger for shallow failures in the slope.
- There are several recorded rockfall events in the far northern extent of the frontage, where the glacial till overlies bedrock. Rockfall and collapse of overlying glacial till over the cliff edge was recorded in three instances in 2023, and once in 2016. Rockfall is episodic and has not lead to headscarp retreat. Debris volumes range from 6m^3 to 45m^3, though runout distances are relatively short between 3m to 7m.

While relatively frequent, these processes all involve movement of relatively small volumes of material that will not destabilise the proposed rock revetments in the central a d northern part of the bay. It is likely that some fine-grained material will enter voids in the revetment and reduce permeability, however this is expected to be a temporary effect, with material washed out during a subsequent storm.

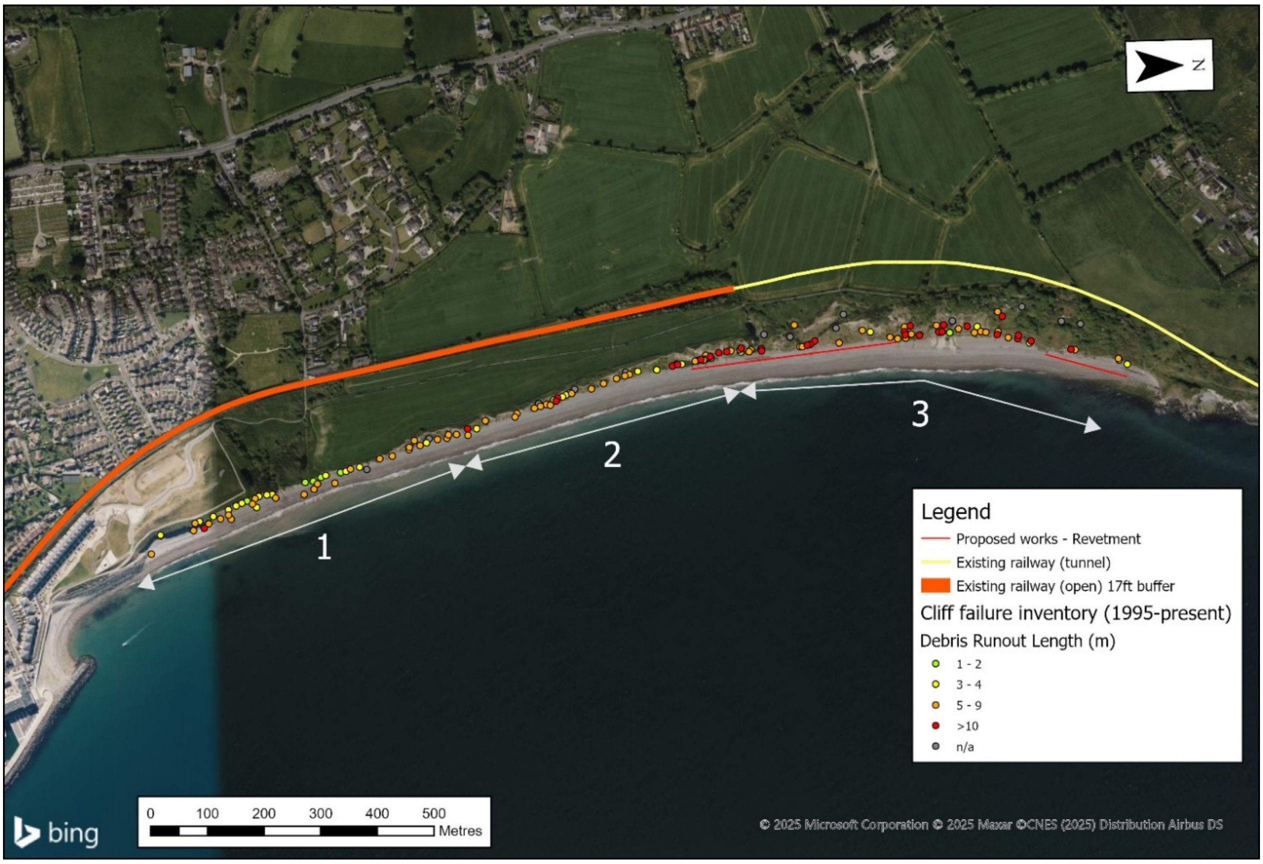

Figure 4-5 Cliff failure inventory between 1995 and present, shown by debris runout distance (Cliff units: 1-South, 2-Central, 3-North)

4.3.2 Projected change in cliff instability (50-100 years)

At Greystones North Beach, the shingle beach provides the main coastal defence, however there is no new source of beach shingle feeding into this coastal cell to increase beach levels in line with sea level rise. It is expected that with sea level rise and increased storminess, there will be a relative reduction in beach volume above the high tide level as shingle is drawn offshore, allowing waves to break more frequently at the cliff toe.

Toe erosion at the soft cliffs is likely to accelerate, initiating shallow and deep-seated landsliding and cliff top recession. However, while this will lead to an increase in sediment supply, the predominantly fine-grained nature of the glacial sediments means little coarse sand or gravel will be contributed to the beach.

The proposed engineering for this Project is not projected to affect beach sediment budgets, however it will provide additional protection to the cliff toe behind the rock revetments from direct wave action and allow cliff debris to build up at the cliff toe behind the structure. Cliff recession behind the rock revetments would be expected to cease over time as the coastal slope stabilises to a reduced angle with continued weathering at the headscarp. However, mudslides on the debris mantling the slope may still be initiated by sustained or intense rainfall, and localised shallow slides may be triggered during extreme storm conditions whereby waves overtopping the rock revetment wash material away from the cliff toe behind the structure, destabilising the debris apron. Erosion of the cliff toe, landsliding and cliff top retreat would be expected to continue between the rock revetment as the stable bays develop.

Projected increases in sustained and/or high intensity rainfall events are likely to elevate groundwater levels, which may trigger more frequent cliff failures, particularly mudslides and shallow slides, resulting in debris runout to the beach. This is expected to be most prominent at the gullies and drainage channels on the coastal slope in the northern part of the bay, where surface water runoff is concentrated over the cliff face.

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). Further analysis has been undertaken considering the design alterations required to increase the design to the year 2100. In order to achieve design criteria in the year 2100 the damage numbers for the rock sizing (a key parameter in the rock stability analysis) can be altered to reflect minor damage (roughly a damage number of 3). This shows that the rock revetment will continue to perform until 2100, but additional damage and increasing maintenance should be expected towards the end of its design life.

At Bray Head (CCA5A) the designs are to provide cliff protection in an open sea environment, while Greystones North Beach (CCA5B) considers cliff protection at the back of the beach. Since the structures are providing the same type of protection the design process remains universal.

CCA5A and CCA5B provide different design constraints. CCA5A is limited based on construction safety and access to the construction site. In order to minimise the effect of this, the design in areas of particular risk or difficulty, have been designed using one type of rock only with a simple slope and rock crest only. In the case of CCA5B, minimal negative impact on access to the beach has been designed for in the present-day scenario. It must be noted that in the absence of a natural feed of beach sediment, as beach material is lost offshore, access around the rock structure will become increasingly more difficult over the course of its design life Access could always be restored through future beach nourishment (if needed).

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 wave 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 construction is complex, toe details may not be possible to construct and as such extra consideration has been taken to determine a conservative crest level.

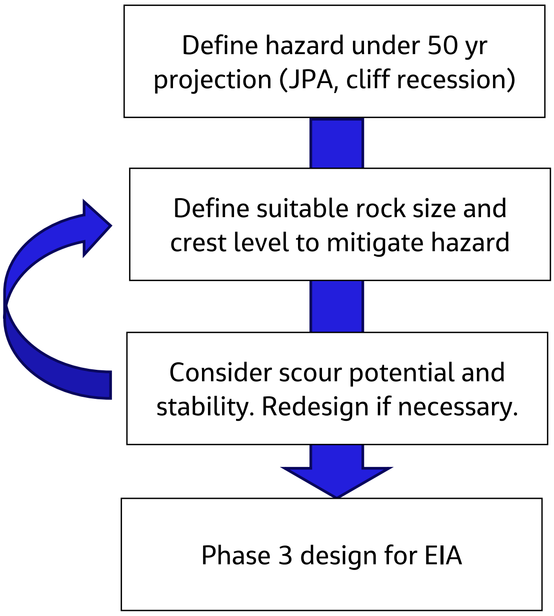

Figure 5-1 Design methodology

5.2 Design parameters

All structures proposed have been designed to recognised and proven current codes, standards, or regulations. Principal design assumptions used in this Project are shown in Table 5-1.

Table 5-1 Principal design parameters

Design Assumption	Value	Reference
Rock Density	2650kg/m³	CIRIA C683 (2007)
Water Density	1025kg/m³	CIRIA C683 (2007)
Storm Duration	12 Hours	This study
Coefficient of Gravity	9.81m/s2	
Plunging Coefficient	6.2	CIRIA C683 (2007)
Surging Coefficient	1.0	CIRIA C683 (2007)
Nominal Permeability	0.1	CIRIA C683 (2007)
Wave Obliquity	0 degrees	Assumed for conservativism
Damage Number (start of damage)	2	CIRIA C683 (2007) van der Meer and van Gent assessments for start of damage. Where the Year 2100 is also considered

5.3 Coastal engineering design

The Project area has undergone wave condition modelling to determine a joint probability analysis (JPA) of combined wave and water levels under deepwater conditions at extraction points along the coastline. An optimal conservative design has been adopted. This assumes that parameters for waves approaching perpendicular to the coast are used since these are the most damaging events. However, if wave conditions are significantly worse for those approaching the coast ±22.5° from perpendicular, these have been selected instead (CIRIA 2007). The toe of each structure has been assumed based on current profile data. Since the nearshore at the bottom of the Bray Head (CCA5A) cliffs shows presence of harder rock, it is unlikely that the toe of these structure will vary significantly over its the design life. As such, the lowest toe level determined based on several lidar surveys has been used to determine nearshore wave conditions.

At Greystones North Beach (CCA5B), a scour assessment using a mixture of Carpenter and Powell, and Sumer and Fredsoe equations (CIRIA 2007) have been used to determine potential erosion toe levels. These lower scour levels have then been considered when transforming the waves nearshore using Goda, creating an erosion extreme wave condition JPA.

Using the toe levels determined, rock stability assessments (using van der Meer and van Gent equations in CIRIA 2007) and overtopping can be considered allowing for a suitable slope and crest to be determined. In the case of CCA5B these assessments have occurred for the year 2075 and year 2100.

5.3.1 Assessment of scour

At CCA5A, an assessment of scour has taken place for structure CDR061D (Structure T) only. The structure is situated on an area of beach with limited geotechnical data and therefore information from adjacent areas has been used in combination with engineering judgement to support the scour assessment. Given the exposed nature of the other structures, it is unlikely that safe construction of a toe structure required to mitigate scour will be feasible and therefore no scour assessment has been undertaken.

For CCA5B the scour depth used an estimate of beach level reduction to give a possible lowered erosion toe level. The design of the revetments in this location considered the lowered toe level. The calculated scour depths calculated for CCA5 are detailed in Table 5-2.

Table 5-2 Scour depth calculations

Location	Estimated Scour Depth (m)	Scour Level (mODM)
CCA5A - CDR061D (Structure T)	0.85	-1.35
CCA5B - Revetment 1	1.57	-0.57
CCA5B - Revetment 2	1.47	-0.47

The scour calculations are considered conservative. The equations assume the beach is exclusively formed from sand, when the sites comprise a mixed sand and gravel beach that has a lower scour potential. Results from these calculations shall be used in conjunction with site-specific results from geotechnical investigations to determine the suitable depth of the structure toe.

5.3.2 Rock armour sizing

The sizing of the armour has been based on the wave action of 200-year Return Period (RP) waves in accordance with the Van der Meer (1988) for non or marginally overtopped structures formulation and van Gent et al. (2004; in CIRIA 2007). Rock structures have been geometrically optimised to limit the size of the required armour units to 6-10t for ease of construction. Results are summarised in Table 5-3.

Table 5-3 Rock armour sizing results

Location	Primary Armour Sizing (t)	Slope (1:X)
CCA5A - CDR061D (Structure T)	6-10	2
CCA5A – CDR063D (Structure S)	6-10	3
CCA5A – CDR067D (Structure Q)	6-10	3
CCA5A – CDR070D (Structure O)	6-10	3
CCA5A – CDR074D (Structure M)	6-10	3
CCA5A – CDR079D (Structure K)	6-10	3
CCA5A – CDR080D (Structure J)	6-10	3
CCA5B - Revetment 1	6-10	2
CCA5B - Revetment 2	6-10	2

Given the very difficult access and construction difficulties for the structures in CCA5A, the rock slope and armour sizing has been considered under 1 in 200 yr RP for the Year 2100 for a start of damage scenario (as opposed to 2075 adopted elsewhere). By doing this, the chance of failure of the structure is reduced in the design life, potentially increasing the design life.

5.3.3 Wave overtopping assessment

To evaluate the overtopping at the revetment structures for the defined crest levels, guidance was followed from EurOtop II (2018). The Overtopping limit considered in analysis is 30l/s/m for reducing cliff erosion. Results for the year 2075 and 2100 are summarised in Table 5-4. At this stage Vmax has not been considered in the overtopping limits.

Table 5-4 Wave overtopping assessment results

Location	Year 2075 Rock Crest Level (mODM)	Year 2100 Rock Crest Level (mODM)
CCA5A - CDR061D (Structure T)	6.10	6.56
CCA5A – CDR063D (Structure S)	6.24	6.71
CCA5A – CDR067D (Structure Q)	6.22	6.47
CCA5A – CDR070D (Structure O)	5.95	6.51
CCA5A – CDR074D (Structure M)	6.70	7.20
CCA5A – CDR079D (Structure K)	6.35	6.85
CCA5A – CDR080D (Structure J)	6.60	7.12
CCA5B- Revetment 1	6.14	N/A
CCA5B- Revetment 2	6.21	N/A

5.4 Cliff instability considerations

Cliff instability is the principal cause of erosion at Greystones North Beach, where rapid retreat of soft cliffs has been observed in recent decades (see Section 4.2). The mechanisms of cliff instability comprise 'bottom-up' failures driven by to erosion by waves, and 'top down' failures caused by elevated ground water levels following sustained wet weather. Since the Project is focussed solely on the climate change effects on coastal processes, the proposed stabilisation measures will only address toe erosion and limited cliff top retreat in response to sustained wet weather is possible.

5.4.1 Implications & design risk mitigation recommendations

There are minor implications of cliff instability to the proposed scheme at Greystones North Beach:

- The proposed northern rock revetment is located along a frontage affected by rockfall and mudsliding. These failure mechanisms are likely to continue to some degree over the design life of the revetment, due to sustained or intense rainfall and surface water runoff over the cliff face, and sub-aerial weathering of rock and soil. Toe protection by the revetment is expected to reduce their size and frequency. Mudslide runout is likely to interact with the structure but loading on the structure is not expected to be damaging. Fine sediment from mudslide runout will likely accumulate behind the structure and subsequently be washed away and through the structure during high tide and storm conditions.
- The proposed rock revetment in the central frontage is located in area mainly affected by mudsliding on the pre-failed debris mantling the slope associated to gullies, and shallow slides. Mudsliding is likely to continue over the design life of the revetment, due to sustained or intense rainfall and surface water runoff over the cliff face and gullies. Toe protection by the revetment is expected to reduce their size and frequency. Debris runout is unlikely to put excessive loading on the structure. Localised shallow slides

may occur as the coastal slope angle reduces and stabilises over time with continued sub-aerial weathering, however the frequency and magnitude of these events are likely to be significantly reduced as the toe is protected from coastal erosion. Debris runout has the potential to interact with the structure, however the volume of debris is unlikely to put excessive loading on the structure.

- Overall, the type of events expected over the design life of the revetment are not likely to cause serious damage to the proposed structures and debris runouts will be trapped behind the structure. However, periodic maintenance of the space behind structure may be required to ensure sufficient space to retain debris remains, and loading upon the structure is minimised.

5.5 Geotechnical analysis

An engineering ground model for the site has been developed in the GIR for this Project. This is supported by boreholes, dynamic probes and geophysical surveys at track level and on the beach.

The solid geology at Bray Head comprises the Bray Head Formation of greywackes and quartzites from the Cambrian period. A small section in the south features greywacke & shale of the Caledonian Devil's Glen Formation. The bedrock within Bray Head is occasionally cut by faults, which typically trend in an east-west direction, these faults represent weaker more fractured rock, which are more susceptible to erosion. The unfaulted bedrock is resistant to coastal erosion, but progressive weathering of the craggy cliffs causes occasional block detachments and rockfalls.

Bray Head has minor superficial deposits, with only thin patches of glacial sediment, while Greystones North Beach is predominantly formed of glacial sediments. Beach sand and gravel at Greystones North Beach and in small pocket beaches at the base of Bray Head. Cliffs of Bray Head are formed in bedrock, while those at Greystones North breach at formed in glacial deposits, comprising poorly sorted sands, gravels and diamictons, with localised sections of cliff formed in rockfall debris and bedrock.

Ground investigations from November 2023 and May 2024 gathered information on soil, rock, and groundwater for designing coastal defences, supervised by Jacobs to ensure accurate data collection and reporting. Methods and results are summarised in the factual report (Causeway Geotech, 2025).

The geotechnical design comprised rock revetments. At Bray Head the bedrock is expected to be at or near to the surface. No geotechnical analyses were undertaken at the Phase 3 Design stage. At Greystones North Beach, structures will be founded on superficial deposits. The conducted analysis confirmed that the bearing capacity, stability and settlement of the structures are acceptable.

The geotechnical risks identified at this stage of the project comprise scour in front of the proposed structures, variable or unforeseen ground conditions which may include unexpectedly soft or thick glacial sediments.

5.5.1 Structure stability assessment

The geotechnical analysis at the Phase 3 design stage was not undertaken for Bray Head because the sub-seabed stratigraphy is not known since there is no GI in the foreshore area, and there is no bathymetry available, meaning the seabed topography in the foreshore area is uncertain.

For more details about Bray Head geotechnical assessment refer to Sections 6.1, 8 and 9 of the Geotechnical Interpretive Report (Appendix B).

Structure assessment at Greystones North Beach consisted of: Ultimate Limit State (ULS) bearing check, stability assessment of rock armour slope, 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 rock armour (unfavourable permanent loading).

A bearing check was undertaken for marine beach sands and glacial till. The bedrock contribution to bearing was ignored. Maximum utilization of 99% was achieved for marine beach sands, whereas slightly lower utilization of 94% was achieved for glacial till. Hence, the design is considered safe because the degree of utilisation is less than 100%.

The settlement check considered the effects of immediate settlement and primary consolidation and was undertaken for two the ground models present at the two rock revetments where rock level is at -4m OD and -12m OD. Settlement in fully saturated granular materials was defined by immediate settlement, whereas settlement in cohesive material was defined by primary consolidation settlement. Bedrock was considered an incompressible stratum. The results showed that majority of consolidation settlement occurs within five years after construction (30mm). The total settlement at the landward edge of the structure is insignificant (equals 5mm) and should not affect global stability of the cliff slope. Therefore, the obtained settlements of the structure are deemed acceptable.

Factor of safety (FoS) was calculated for rock armour with two internal friction angles (40 and 55 degrees) and was compared against the steepest slope with slope ratio of 1:1.5 (approx. 33.7˚). The slope stability was satisfied because the minimum FoS obtained was ≥ 1.3.

For more details about Greystones North Beach geotechnical assessment refer to 6.2, 8 and 9 of the Geotechnical Interpretive Report (Appendix B).

5.6 Landscape design

Landscape design is not needed at this stage of the project given the isolated location of many of the proposed works. The designed rock revetments will use natural materials, similar to that already employed elsewhere along the coast. At Bray Head, the proposed structures will not affect the form of the coastline, which is already defended. At Greystones North Beach, the proposed structures are designed to allow natural evolution of the shape of the bay at locations where there is no risk the railway.

5.7 Access

Vehicular access to Bray Head is not possible from land or the railway and all works will need to be supported from the coast. At Greystones North Beach, limited vehicular access is possible to the shoreline once enabling works have been undertaken. However, the surrounding narrow residential roads are unsuitable for large plant and delivery of rock by barge will be necessary.

5.8 Environmental enhancement/biodiversity design

The Phase 3 design will be further modified having regard to the potential environmental effects to be identified in the EIAR.

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-CCA5-P2-ENG-CV-JAC-0001).

6.1.1 Design at Bray Head

Rock revetments are proposed to protect existing masonry/concrete structures that are subject to direct wave loading (those remaining that were not protected by rock in the 2000's). The structures comprise large 6t - 10t rock only where the construction is difficult. Where construction is likely to be less problematic, an underlayer is specified. A large underlayer grading shall be used for the profile purposes and to allow the continued discharge of existing weep holes and outfall pipes. The revetments will be founded on existing rock shore platforms that have localised veneers of gravel.

6.1.2 Design at Greystones North Beach

The proposed engineering solution for Greystones North Beach has evolved through the design process. Rock headlands that were detached from the cliff base were originally proposed. However, these structures constrained access to the beach at high tide and also created a potentially unsafe narrow pathway between the base of the cliff and the structure. These problems have been removed by placing the structures further up the beach to directly abut the cliff. Further analysis of projected coastal erosion and structure performance indicated that the southernmost structure was unnecessary over an initial 50-year time period and therefore this has been removed from the Phase 3 design.

Two rock revetments are proposed where the projected erosion poses a risk to the railway. The revetments shall be constructed close to the cliff to maximise public access to the beach in all tidal states. The revetments shall be made from 6-10t grading of rock with a underlayer of 0.3-1t. The crest height has been determined based on overtopping limits for erosion and the revetment will protect the cliff against toe erosion. The toe of the revetment has been designed to be founded on, or slightly below, the surface of the clay shore platform. Following installation, the toe will be buried by beach material to restore the current shore profile and ensure beach access is maximized through the tidal cycle.

6.2 Future adaptability of the design

Future adaptability has been considered during the design process. Due to the construction constraints at Bray Head a design for the year 2100 has been developed. Beyond this date, since the design for a revetment structure is straightforward, it is envisaged that additional rock and structure reprofiling can be undertaken when the effectiveness of the revetment in reducing undermining is reduced. As stressed in earlier sections, the rock will only protect the cliff against undermining and will not influence erosion of the wider coastal slopes and cliffs caused by the impacts of drainage.

Adaptability was considered at Greystones North Beach in terms of the revetment structures. The concept of the design is to create two hardpoints along the cliff where toe erosion will be halted, and an intervening undefended section where erosion will be permitted to continue until a naturally stable bay of predictable form will develop. This bay will retain beach material and create more sheltered wave conditions, reducing the rate of toe erosion until an equilibrium form is achieved. The design can be adapted by adjustments to the structures, which may be lengthened or increased in number. The design can also be adapted using nature-based solutions such as beach recharge to accelerate the formation of a stable bay.

6.3 Interfaces between sub-cells and existing structures

Given the construction limitations at Bray Head and since the rock crest level required for the year 2100 criteria is a marginal increase compared to year 2075 (and the rock armour size required is comparable), it is recommended that structures are designed for the year 2100.

During the design process, 3D drawing models have been used to inform the interfaces of the structures into the coastline in which they are to be situated. Where two structures are present, these have been combined and the worst-case minimum design criteria adopted.

In terms of interfaces into the existing cliff faces, it is proposed that the cross section will either rotate around the crest until the profile meets the cliff face, or the sides of the main body of the revetment will slope using a 1 in 2 slope down to bathymetry. In the case of CDR061D (Structure T), where interfacing with the existing concrete steps unit is required, the crest of the structure shall be sloped at 1 in 2 down to half a rock above the steps unit. From there, the remaining unsupported cross section shall be rotated around it highest point until it has reached the concrete. This can also help protect the toe of the steps against undermining.

6.4 Drawing list

Drawings prepared for this site are summarised in Table 6-1.

Table 6-1 Drawings for CCA5

Drawing No.	Title	Description
7694-CCA5-P3-DWG-CV-JAC-0010	Site location plan	Overview of Bray Head and Greystones North Beach site
7694-CCA5-P3-DWG-CV-JAC-0100	Location plan	Location of structures at Bray Head
7694-CCA5-P3-DWG-CV-JAC-0200	GA plan and section CCA5A CDR061D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0201	GA plan and section CCA5A CDR063D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0202	GA plan and section CCA5A CDR067D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0203	GA plan and section CCA5A CDR070D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0204	GA plan and section CCA5A CDR074D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0205	GA plan and section CCA5A CDR079D and CDR080D	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0300	General arrangement plan CCA5B	Location of structures at Greystones North Beach
7694-CCA5-P3-DWG-CV-JAC-0400	General arrangement cross sections CCA5-B1	Structure location, 3D model and cross section
7694-CCA5-P3-DWG-CV-JAC-0401	General arrangement cross sections CCA5-B2	Structure location, 3D model and cross section

6.5 Buildability

Buildability in this area is largely constrained by poor access, tidal working and wave conditions.

In the case of Bray Head it is has been determined that materials are likely to be delivered from and constructed entirely by sea. Structures benefit from placement between existing headland outcrops with smaller stumps from eroded cliffs present at the toe of the structure. Although this is beneficial for interfacing the structure, it provides an additional constraint as the vessels carrying the rock are not able to get sufficiently close to the shoreline, which limits long range rock placement. Vessels will be subjected to exposed wave conditions during placement and, as such, may be limited as to when placement can occur.

Greystones North Beach is largely limited by the tidal working in the area. Access to the construction area to vehicles is via the beach from Greystones. This route is not suitable for delivery of rock, which will transported to site via the sea. The beach is low lying meaning at high tide much of the construction area will be submerged.

In order to maximise beach access, the revetments will be constructed as close to the cliff as possible. This will be achieved through excavation of debris from the cliff toe. The potential for cliff instability being triggered by toe excavation will be mitigated by installation of temporary sheet piling, which will be removed once the revetment has been 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.

6.7 Health and safety

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 is presented in Appendix C. 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.

Table 6-2 Top five risks identified in the DEHERR

Risk ID.	Activity	Potential Hazard	Design to Reduce Risk	Residual Risk	Action By	Comments
34	Working on the coast	Working on beach or barges on an exposed coast has a heightened risk of drowning and loss of equipment due to tides or storms.	Plan work around tides and calm sea-state conditions. Ensure construction me	Construction in tidal zone and on barges is unavoidable, but risk will be minimised through careful planning.	Designer / Contractor	Plan works to ensure work from barges and on beach is undertaken as quickly and safety as possible.
2	Use of vehicles/plant on site – Public or Construction Staff	Transportation over foreshore and access ramps, etc. Potential plant overturning leading to potential for injury/death to staff or public with access to the foreshore.	Access points to be identified and to be incorporated during design development. Management of public access	Transportation over foreshore and access ramps Potential plant overturning leading to potential for injury/death to members of public with access to the foreshore.	Contractor	Contractor to include appropriate traffic management and works segregation in method statements with mitigation and reduction measures to separate vehicles and public
12	Cliff Material Slip	Risk of injury to personal and damage to the new revetment under significant cliff slippage during construction that involves excavation of material from the cliff toe	The design involves use of temporary sheet piles to support loose cliff material at CCA5b during constriction of revetment	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).
10	Unforeseen services present	Striking of live services causing electrocution, explosion, flooding and / or disruption of services.	Full services survey to be undertaken during design development.	Striking of live services causing electrocution, explosion, flooding and / or disruption of services.	Contractor	Full services search to be undertaken at detailed design stage. Contractor to survey location prior to excavation works, where reasonable.
13	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/loss or damage to plant.	Inform contractor of risk of soft ground from GI and geotechnical analysis in detailed design.	Potential for site operatives or plant to become stuck in pockets of soft ground. Instability of plant working in area of low soil strength. Risk of suffocation, crash injuries from sinking into ground/loss or damage to plant.	Designer / Contractor	Contractor to prepare method statement and safe systems of work. Risk to be updated following completion GI and geotechnical analysis.

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 to CCA5A, 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).

No maintenance of the designed engineering is proposed because the revetments are designed to respond to beach movement and toe scour.

6.8 Recommendations for refinement at detailed design

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

The use of plant will be carefully planned and managed during construction to ensure the safety of workers. Members of the public will only be at risk at Naylor's Cove in CCA5A and CCA5B, where access will be carefully managed to ensure their safety.

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.

In addition, value engineering will be undertaken to optimise volumes of rock being used for construction. It may necessary to undertake modelling of beach behaviour, including CFD, to determine likely responses of the beaches to the proposed structures.

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 – Phase 3 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 EIAR, AA Screening Report, NIS 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 beach was a key requirement from the public. Beach access provision has been assessed during the Phase 3 design which was one of the reasons for moving from rock headland to rock revetments. The Phase 3 design proposals now maximise access seaward of the structures.

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 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 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	A spatial model which subdivides the coast based on the variation in physical characteristics, including the geology, geomorphology, shoreline topography and orientation, and existing defence type
Coastal erosion	Loss or displacement of land, or long-term removal of rocks and sediment along the coastline due natural impact of waves, wind, rain and tides
Coastal flooding	Submergence of normally dry and low-lying land by seawater
Coastal protection	Measures aimed at protecting the coast, assets and inhabitants from coastal flooding and erosion. Coastal protection may involve structural, non-structural or nature-based solutions
Coastal spit	A coastal landform, whereby a stretch of beach material projects out to the sea and is connected to the mainland at one end
Concept level design	Foundational phase of the design process which lays the groundwork for the entire project. The design work undertaken for the concept design is sufficient to confirm that the options will work from a technical perspective and will meet the Project objectives.
Concrete armour	Precast concrete units placed to form breakwaters or revetments to dissipate wave energy
Constructability	Also known as buildability. The extent to which a design facilitates the each and efficiency of construction

Term	Description
Design horizon	The period of time over which the scheme provides the required Standard of Protection (SoP) to the railway line.
Design life	The service life intended by the designer, which is the period of time after installation during which the structure meets or exceeds the performance requirements.
Dilapidation survey	A detailed survey that examines the existing state of the coastal structure
Dune regeneration	Stabilisation and enhancement of existing dune systems to deliver additional resilience
Embankment	Linear grassed earth structure providing flood protection; typically used along riverbanks
Emergency works	Works in response to an event that is unexpected and serious such that it presents a significant risk to human life, health and property or the natural environment and involves the need for immediate action to manage the risk
Feasibility study	An assessment of the practicality of a proposed project plan or method.
Flood proofing	Structural, and non-structural, solutions that can prevent or reduce flood damages to a property or its content.
Flood warning and preparedness	Measures undertaken to better prepare, respond and cope with the immediate aftermath of a flood event
Foreshore	The part of a shore between high- and low-water marks
Freeze-thaw weathering	Form of mechanical weathering whereby water enters cracks in rocks, freezes and expands, widening the cracks. Repetition of this cycle causes gradual break down of the rock.
Gabions	A basket or container filled with earth, stones, or other material
Geomorphology	The interaction between Earth's natural landforms, processes and materials
Geotextile	Permeable fabrics which, when used in association with soil, have the ability to separate, filter, reinforce, protect, or drain
Geotubes/ Geotextile Tubes	Tube shaped bags made of porous, weather-resistant geotextile and filled with sand slurry, to form artificial coastal structures such as breakwaters or levees
Groyne	Linear structure constructed perpendicular to the shoreline which helps retain beach material in place.
Hazard	A process or material that has the potential to cause harm.
High tide mark	A point that represents the maximum rise of a body of water over land
Hydrodynamic modelling	Used in the analysis of coastal hydrodynamic processes, it is employed to simulate major physical phenomena in the coastal region
Maintenance burden	The level of maintenance (repair, monitoring, rebuilding) required over the design life of the structure to retain the Standard of Protection of the coastal defence structure
Managed realignment	A coastal management strategy that involves setting back the line of actively maintained defences to a new line inland and creating inter-tidal habitat between the old and new defences
Mudslides	Mass of saturated sediment that moves downslope. Typically comprises distinct source, transport and debris accumulation zones
Multi criteria analysis (MCA)	A structured approach to determine overall preferences among alternative options, where the options should accomplish multiple objectives.
Nature-based solutions	The use of natural materials and processes to reduce erosion and flood risk to coastal infrastructure
Pore water pressure	The pressure of groundwater within voids between sediment particles. High pore water pressures push particles part, reducing the shear strength which may trigger slope failure.

Term	Description
Risk	The adverse consequence of a hazard event. Risk is typically described in financial terms, but may consider human harm, environment impact, programme delays or reputational damage.
Residual risk	The risk that cannot be completely eliminated by engineered mitigation measures. It is generally agreed to be at an acceptable level by the client.
Revetment	Sloping or stepped structure built parallel along the shoreline between the low lying beach and higher mainland to protect the coast from erosion and wave overtopping. The revetment may have a smooth or rough surface
Rock netting	A drapery system designed to control rockfall movement by guiding falling debris to a collection point at the toe of the slope
Saltmarsh	Coastal grassland that is regularly flooded by seawater
Sea level rise	An increase in the level of the oceans due to the effects of climate change and/or land-level change
Seagrass bed	Intertidal or sub-tidal beds of sea grass. Provides ecosystem benefits including carbon sequestration.
Seawall	Vertical or near-vertical impermeable structure designed to withstand high wave forces and protect the coast from erosion and/or flooding
Shellfish reefs	Sub-tidal or intertidal reefs formed of suitable material for settlement by oysters or mussels.
Sill	A low rock structure in front of existing eroding banks to retain sediment behind.
Standard of Protection	The expected frequency or chance of an event of a certain size occurring. Defined for this project as being a 0.5% Annual Exceedance Probability, also known as a 1 in 200 year storm protection level.
Storm surge	A temporary change in sea level that is caused by a storm event, which can lead to coastal flooding
Toe scour	Occurs when the toe (bottom) of the defence is worn away by the waves and can cause defences to fail.
Wave exposure	The degree to which a coast is exposed to wave energy
Wave overtopping	The average quantity of water that is discharged per linear meter by waves over a protection structure (e.g., breakwater) whose crest is higher than the still water level

9. References

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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

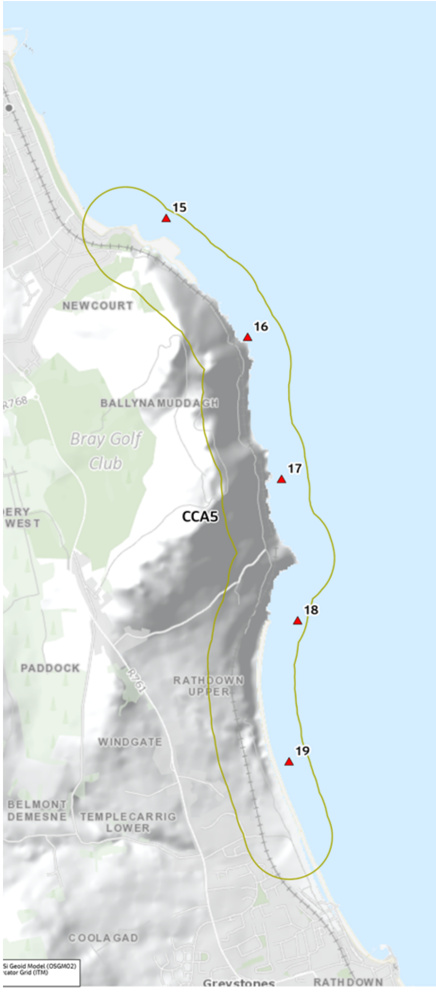

Figure A-1. Wave model extraction locations along CCA5.

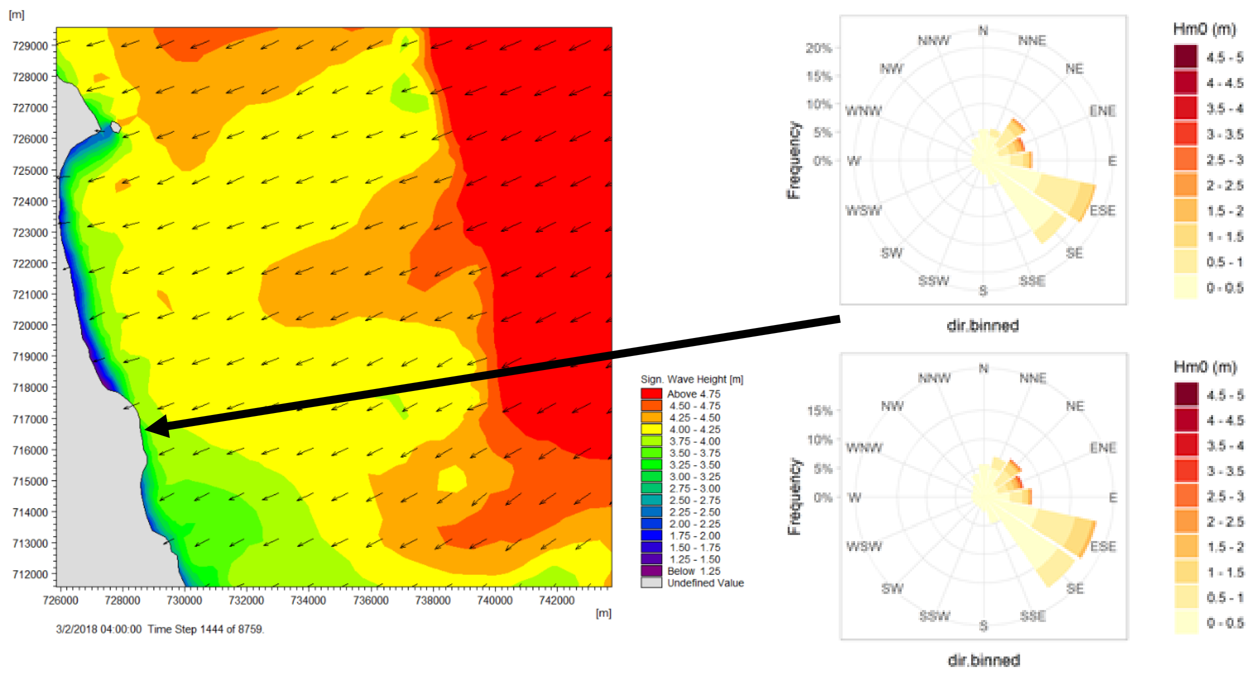

Figure A-2. CCA5: Contour plot showing event of 3rd March 2018 (left), wave height roses - Jan/1988-Dec/2021 (top right) & Jan/2056-Dec/2100 (bottom right)

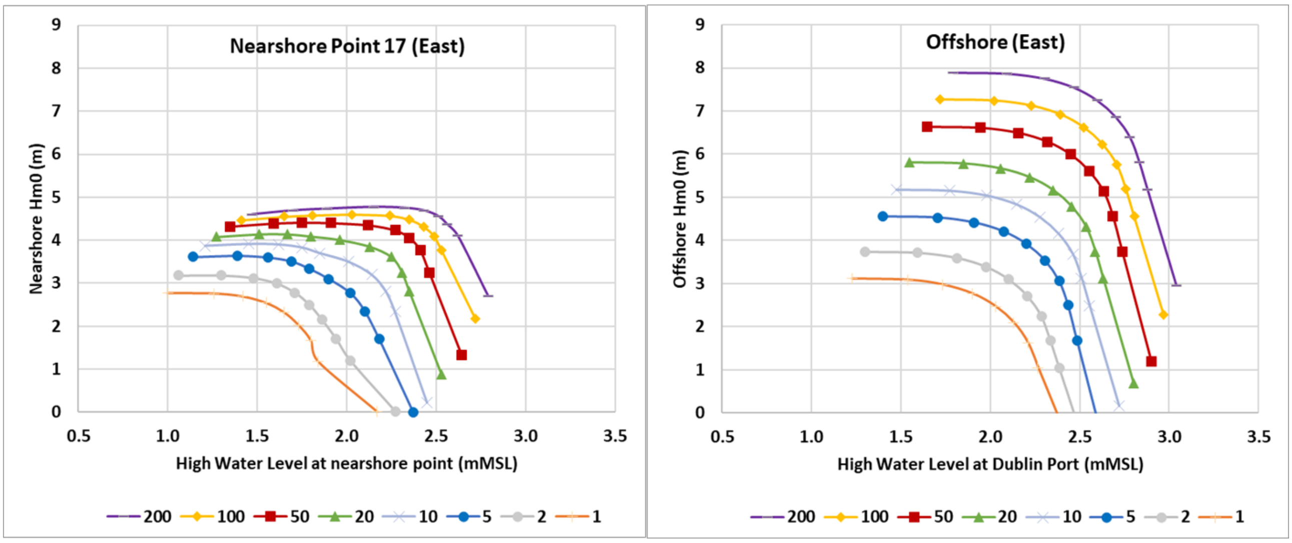

Figure A-3. Joint probability curve at nearshore point 17 in CCA5 (left) compared to offshore (right) for waves from the East. Nearshore wave extracted at depth of -8.10 mMSL. Note any changes in the high water levels from Dublin to the nearshore point is due to 2D variations in water level.

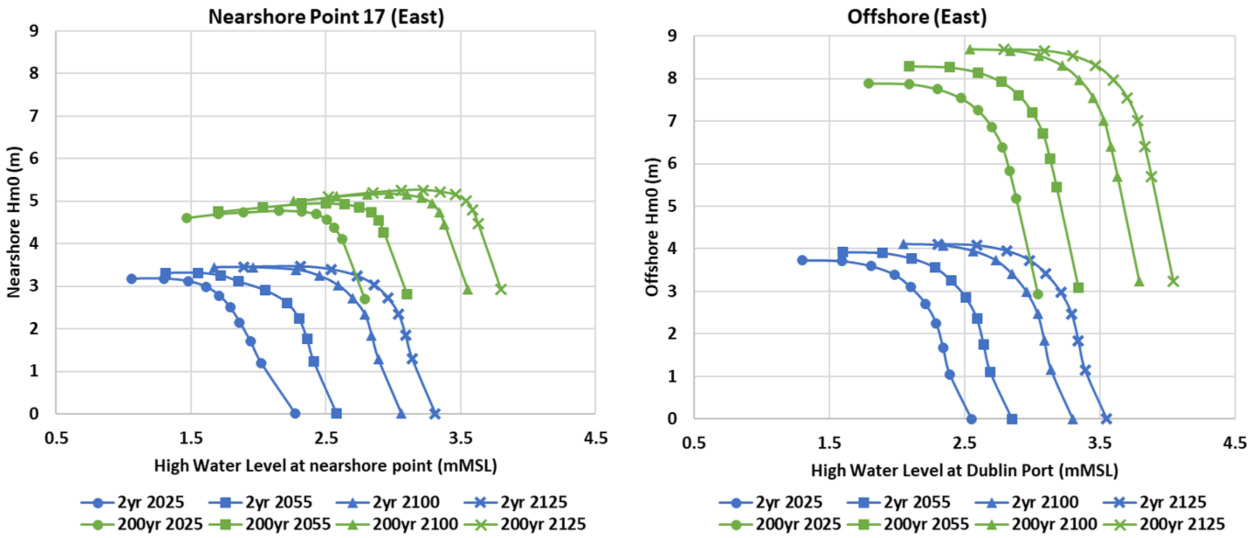

Figure A-4. Impact of climate change on joint probability curves for 1 in 2 year and 1in 200 year return periods at nearshore point 17 in CCA5 (left) and Offshore (right) for waves from East. Nearshore wave extracted at depth of -8.10 mMSL. Note any changes in the high water levels from Dublin to the nearshore point is due to 2D variations in water level.

Appendix B. Geotechnical outputs

Document Number	Document Title
7694-CCA5-P3-ENG-CV-JAC-0002	Geotechnical Interpretive Report

Appendix C. DEHERR – (designers' risk assessment)

Document Number	Document Title
7694-CCA5-P3-REG-CV-JAC-0004	CCA5 DEHERR