Flood Risk Assessment

Port Ellen Terminal Development

September 2024

This page left intentionally blank for pagination.

Mott MacDonald
Moray House
16-18 Bank Street
Inverness IV1 1QY
United Kingdom

T +44 (0)1463 239323
mottmac.com

Flood Risk Assessment

Port Ellen Terminal Development

September 2024

Mott MacDonald Limited. Registered in
England and Wales no. 1243967.
Registered office: 10 Fleet Place, London
EC4M 7RB, United Kingdom

Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
0A	25/09/2024	Holly Clark	James Franklin	Laurence Cload	First Issue

Document reference: 10115031 | 0001 | A | 115031-MMD-PE-XX-RP-O-0020

The report is a factual assessment of flood risk based on the existing available statistical information. The flood risk assessment has been based on the information available at the time of study, to be used to inform relevant parties, and to assist in the planning process. Mott MacDonald is not liable for, and will not give a warranty against, actual flooding of the site, or damage as a result of said flooding.

Contents

Tables

Figures

1 Introduction

1.1 Scope of Assessment

Mott MacDonald were commissioned to undertake a Flood Risk Assessment (FRA) to support a Harbour Revision Order and Marine Licence application for the Port Ellen Terminal Development, hereafter referred to as the 'Proposed Development'.

This FRA has summarised the potential flood risk to, and from, the Proposed Development. The likelihood of increased flood risk as a result of the Proposed Development has been discussed. The FRA has summarised the results from coastal flood and surface water (drainage) modelling.

This document should be read in conjunction with the Wave Overtopping Assessment[1] and Port Ellen Development Drainage Impact Assessment[2].

1.2 Development Description

1.2.1 Site Location

The existing Port Ellen Ferry Terminal is located on the south coast of the Isle of Islay, at the existing Port Ellen Ferry Terminal (NGR NR 36300 45000) within the town of Port Ellen. It currently services RORO (roll-on roll-off) ferry services to and from Kennacraig on the mainland of Scotland. It also provides infrastructure to support the import of grain to Islay, alongside wider harbour operations such as commercial, fishing and leisure activities.

1.2.2 Proposed Development

The Proposed Development is a full redevelopment of the ferry terminal including:

- Two new, longer ferry berths designed to accommodate the New Islay Vessels;
- a significant area of reclaimed land to facilitate the required marshalling area;
- unaccompanied trailer facilities;
- a new terminal building;
- improvements to passenger access;
- an improved traffic management layout and segregated commercial spaces; and
- dredging to facilitate access and mooring of the New Islay Vessels.

The layout of the Proposed Development can be seen in Figure A.1- 2. The majority of the development site will be reclaimed land. The development is classified as a Water Compatible Uses site under the SEPA Flood Risk and Land Use Vulnerability Classification[3].

[1] Mott MacDonald (2024) Wave Overtopping Assessment Estimating Wave Overtopping for Annual Exceedance Probabilities of 1% and 0.5% in the Flood Risk Assessment June 2024

[2] Mott MacDonald (2024) Port Ellen Development Drainage Impact Assessment (DIA) June 2024. Reference: 100115031|0002|A|115031-MMD-00-ZZ-RP-C-0002

[3] SEPA (2018) Flood Risk and Land Use Vulnerability Guidance LUPS-GU24 v.4

1.2.2.1 Embedded Mitigation

The following mitigation is embedded into the design to minimise the risk of flooding at the Proposed Development site:

- An extended 1.9m revetment along the western edge of the port to minimise wave overtopping;
- A 1.0m sea wall along the western revetment section of the development to allow for protection from wave overtopping at key receptors of the terminal building and the car park;
- Hardstanding camber to allow drainage of pluvial surface water directly to sea; and
- Sustainable drainage systems (SuDS) to convey surface water to minimise pooling of water on hardstanding, and to allow for attenuation of water which is at higher risk of pollution prior to discharge to sea, for example, run-off from car park.

1.3 Methodology

The FRA was compiled following the methodology set out by SEPA[4],and relevant guidance documents as referenced throughout this document.

To determine the impacts on flood risk from the Proposed Development, the FRA assessed the existing ('baseline') scenario against the 'proposed development' scenario. The Proposed Development has been outlined in Section 1.2.2. To quantify the impacts of the Proposed Development scenario on flood extents, relevant flood modelling and assessment was undertaken for each identified source of flooding. For the purposes of this study, coastal flood modelling, surface water modelling and assessment of foul drainage were undertaken, the methodology for which have been discussed in Section 1.3.2.

For this study, the land use classification of the Proposed Development is a Water Compatible Uses site[5]. The functional floodplain assessed is land where there is a 0.5% or greater annual probability of flooding in any year (1:200-year event).

The outline methodology for the FRA is as follows:

- Desk-based review of baseline conditions including collation of publicly available information on historical floods and flood incident reports (refer to Section 1.3.1).
- Data requests to relevant consultees to obtain any non-publicly available information (refer to Section 1.3.1).
- Determine potential flood sources (refer to Section 2).
- Wave Overtopping Assessment to determine risks to key receptors from coastal flooding (refer to Section 2.1, 3.1 and Appendix A.2).
- Assess surface water and infrastructure (sewage) flooding through drainage modelling (refer to Sections 2.2 and 3.2).
- Determine flood impact as a result of coastal, surface and infrastructure flooding.
- Outline additional mitigation and operational measures (refer to Section 4).

1.3.1 Data Collation

The following publicly available data sources have been reviewed to inform the FRA:

---

[4] SEPA (2022) Technical Flood Risk Guidance for Stakeholders: SEPA requirements for undertaking a Flood Risk Assessment

[5] SEPA (2018) Flood Risk and Land Use Vulnerability Guidance LUPS-GU24 v.4

- SEPA Flood Maps[6];
- Data extracts from the FEH Webservice[7];
- Highland and Argyll Local Flood Risk Management Plan (2022-2028)[8];
- Ordnance Survey Maps.
- Scotland's environment webmap[9]
- British Geological Survey Onshore GeoIndex[10]

The following organisations have been consulted to request information in regard to historical flooding and flood incidents reported at, and within a 1km vicinity of the site, to identify any information relevant to the FRA which is not publicly available:

- Scottish Environment Protection Agency (SEPA).
- Marine Directorate.
- Scottish Water; and
- Argyll & Bute Council.

The above organisations were contacted on 6th June 2024 to request information. Responses were received from the following:

- Scottish Water provided information on flooding incidents from sewer blockage and surcharge and plans of the sewerage system.
- Argyll & Bute Council's response identified flood incidents and locations within the area including a relevant flood risk assessment report (2011)[11] and photographs.
- Marine Directorate provided a response determining that they hold no relevant information for the site.
- SEPA provided a response outlining the number of records and type (source) of flooding within 1km of the site between October 2004 and December 2014. Information on the specific location of the incidents cannot be provided by SEPA (Regulation 11(2) of EIRs[12] – personal data).

1.3.2 Assessment Approach

1.3.2.1 Coastal Flood Modelling

A wave overtopping assessment has been conducted by Mott MacDonald to estimate the wave overtopping for a range of extreme conditions at Annual Exceedance Probabilities (AEP) of 1% and 0.5% (a 1:100-year, and 1:200-year storm event, respectively), and has been provided in Appendix A.2.

The assessment used representative cross-sections of the revetment infrastructure, established mean tide levels for Port Ellen and assessed climate change allowances (based on UKCP18) to conduct a dependence assessment for wave and water level extremeness using the EA/DEFRA

---

[6] SEPA (2024) SEPA Flood Maps [Available Online] Flood Maps | SEPA - Flood Maps | SEPA

[7] UK Centre for Ecology and Hydrology (CEH) (2024) Flood Estimation Handbook Web Service [Available Online] Home Page - FEH Web Service (ceh.ac.uk)

[8] The Highland Council (n.d) Highland and Argyll Local Flood Risk Management Plan (2022-2028) [Available Online] Highland and Argyll Local Flood Risk Management Plan 2022 - 2028 (argyll-bute.gov.uk)

[9] Scotland's environment (2024) Scotland's environment webmap [Available Online] Scotland's environment map | Scotland's environment web

[10] BGS (2024) GeoIndex Onshore [Available Online] GeoIndex (onshore) - British Geological Survey (bgs.ac.uk)

[11] URS Scott Wilson (2011) Frederick Crescent, Port Ellen, Islay Flood Risk Assessment – Phase 1

[12] Environmental Information (Scotland) Regulations 2004

Joint Probability Approach[13]. Tidal water levels and current speeds were assessed using an in-house calibrated and validated hydrodynamic model of Port Ellen[14]. Wave characteristics around the proposed development site were based on this existing wave modelling study.

This allowed the 1:100-year and 1:200-year events to be defined (for wave height and water level) to calculate the mean wave overtopping discharges using the EurOtop Artificial Neural Network. The wave overtopping discharge estimated were assessed against safe tolerance limits for key receptors including people, structures, property, and vehicles.

1.3.2.2 Drainage Impact Assessment

A drainage impact assessment (DIA) has been conducted by Mott MacDonald to assess the surface water (pluvial) drainage requirements and outline the drainage system design.

An InfoDrainage flood model was developed to assess flood potential, level of flood protection for key infrastructure and receptors, and to determine drainage volume requirements to design an appropriately scaled sustainable drainage system (SuDS). The SuDS design was at an outline design stage at the time of writing, it may be modified as the project progresses through detailed design.

In line with NPF4, any new development will reduce flood risk by controlling the water at source through a SuDS and considering the exceedance flow route when the capacity of the drainage system is exceeded.

Foul drainage was also assessed as part of the DIA and informs the assessment on infrastructure failure flood risk (Section 3.3).

[13] JPA FD2308

[14] Mott MacDonald (2024) Port Ellen Development Detail Design – Wave modelling Report

2 Baseline Flood Risk

This section has identified the source of potential flooding i.e., fluvial, coastal, surface water (pluvial and/or drainage), or a combination of sources e.g., fluvial and coastal.

2.1 Coastal

Coastal flooding originates from the sea (open coast or estuary) where the normal tidal range is exceeded and flood the onshore or man-made structures that define the coast. The mechanisms which cause flooding are high astronomical tide, storm surge, wave action, and local bathymetric effects, often in combination.

The SEPA Flood Maps show that there is a high likelihood of coastal flooding (10% Annual Exceedance Probability (AEP)) at the location of the Proposed Development as shown in Figure 2-1. Consultation with SEPA determined they hold 4 records of coastal flooding incidents within 1km radius of the site, noting that the SEPA record may not be complete.

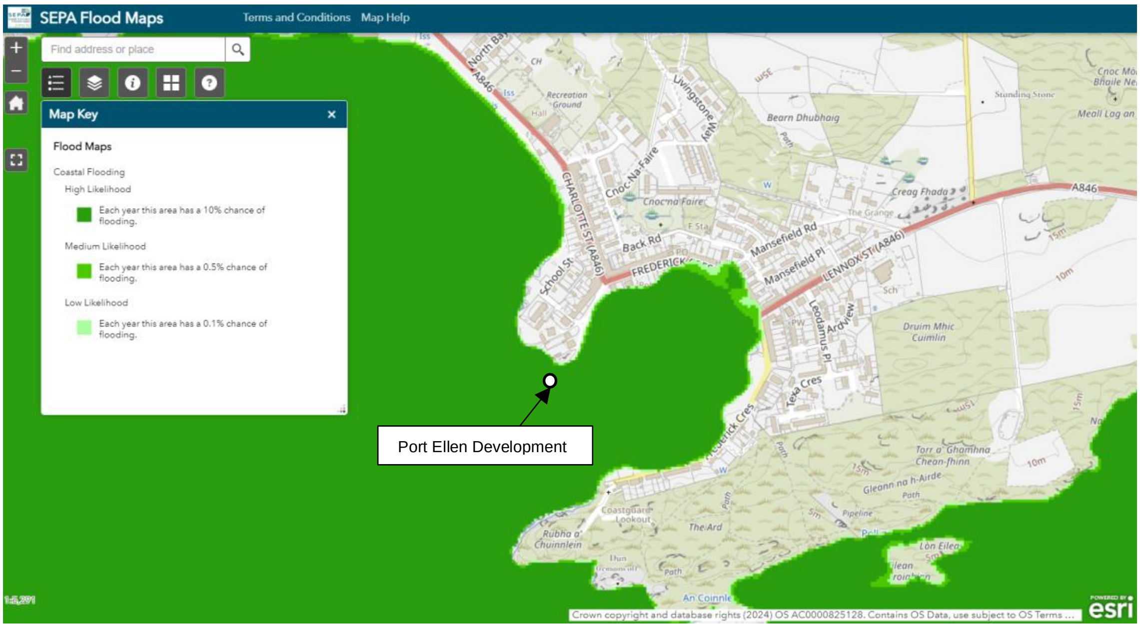


Figure 2-1: SEPA Flood Maps coastal flooding probability

Source: SEPA Flood Maps

Argyll and Bute Council provided information of historical coastal flooding incidents at the eastern end of Frederick Crescent, flooding from Loch Leòdamais. A Flood Risk Assessment from 2011[15], provided by the Council, identified that the Frederick Crescent area is susceptible to flooding on an almost annual basis. This flooding is exacerbated by a low-lying area of road, sea levels being above the height of the drainage holes in the sea wall, and above the sea wall top height during high spring tides, and no surface water drainage provisions on the road beside the sea wall.

[15] URS Scott Wilson (2011) Frederick Crescent, Port Ellen, Islay Flood Risk Assessment – Phase 1

The high astronomical tidal levels and coastal design sea levels for extreme high tide sea level at Port Ellen are outlined in Table 2.1. A climate change allowance of 0.61m is applied based on RCP8.5 emissions scenario by 2085 for Port Ellen[16].

Table 2.1: High Tidal Water Levels for Port Ellen

Water Level	Chart Datum (mCD)	Ordnance Datum (mAOD)	Ordnance Datum + Climate Change
Coastal Flood Boundary (CFB) Extreme Sea Level (1 in 200-year return period)	-	+2.24m	+2.85m
Highest Astronomical Tide	+1.08m	+0.89m	+1.5m
Mean High Water Springs	+0.90m	+0.71m	+1.32m
Mean High Water Neaps	+0.80m	+0.61m	+1.22m
Highest Observed Water	+1.92m	+1.73m	+2.34m

The lowest elevation at the existing Port Ellen site is 2.66m above ordnance datum (AOD) at the connection to Pier Road. This results in areas which are exposed to tidal inundation under the 1 in 200-year extreme sea level high tide plus climate change scenario which has an expected tide level of +2.85mAOD (see Table 2.1). The hardstanding area from the entrance to the port (at Port Ellen Grain Store) towards the existing terminal building is at elevations below 2.85mAOD and therefore exposed to tidal inundation (see Figure A.1- 1).

The baseline inshore wave characteristics model predictions (Port Ellen MIKE 21 FMSW model) show that the dominant wave direction at the entrance to Port Ellen originate from the south-east to south-west (82% of the total wave record), with wave heights of less than 1m occurring 62% of the time. Maximum wave heights are reached when waves propagate from a southerly direction with the maximum wave overtopping observed in the model at the south wall of the port (location of linkspan berth and the finger pier).

A wave overtopping assessment of the existing infrastructure at Port Ellen was undertaken to determine the baseline wave overtopping. The results of the baseline wave overtopping assessment, provided in Section 3.1.2, allow for direct comparison with the Proposed Development scenario.

2.2 Surface Water

The existing site has a topographic high point of 7.11 mAOD at the end of the finger pier, sloping north-eastwards towards the land at 2.66 mAOD.

Currently, there is no drainage system on site with water draining directly to sea via the quay walls into Kilnaughton Bay and Loch Leòdamais with no pollution prevention treatment or attenuation (e.g. SuDS systems). There are no known surface water drainage connections to Scottish Water's combined sewerage system.

SEPA Flood Maps do not identify a potential surface water flood risk at the location of the Proposed Development as shown in Figure 2-2. Consultation with SEPA identified four flooding incidents within 1km of the site as a result of surface water (pluvial).

[16] Met Office (2024) UKCP18 Marine Climate Change. Available Online: UK Climate Projections (UKCP) - Met Office

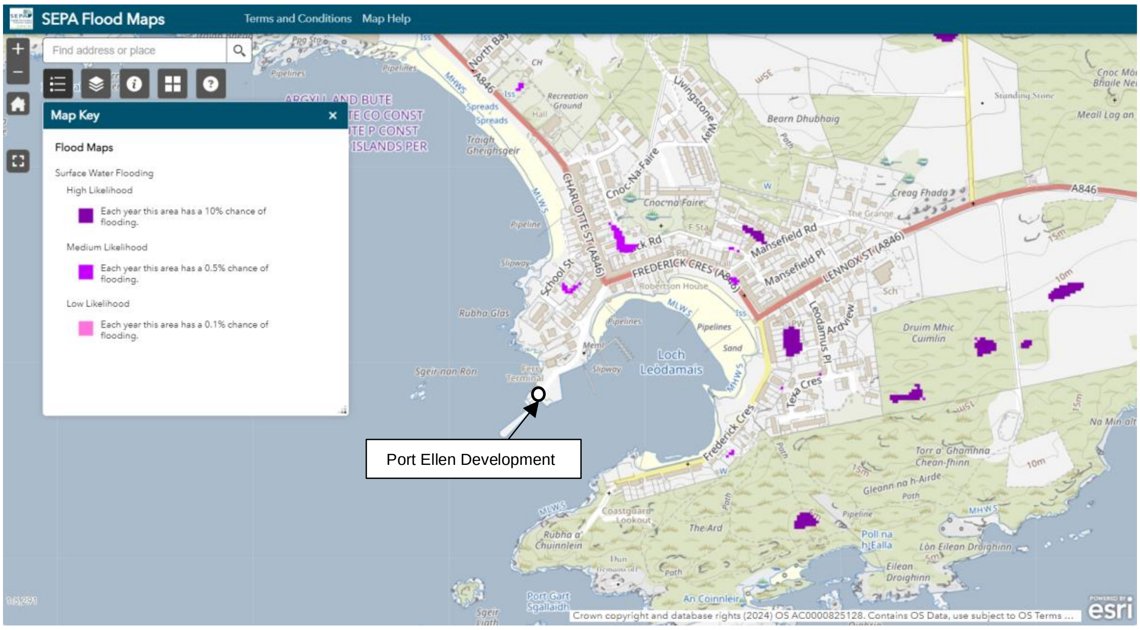


Figure 2-2: SEPA Flood Maps surface water flooding probability

Source: SEPA Flood Maps

A Flood Risk Assessment for Frederick Crescent[17], located on the south-eastern side of Loch Leòdamais, (provided by Argyll and Bute Council) identifies the eastern extent of Frederick Crescent as susceptible to pluvial flooding in combination with high coastal flooding.

Assessment of surface water flooding risk from pluvial and drainage sources at the Proposed Development has been provided in Section 3.2.

2.3 Fluvial

SEPA Flood Maps do not identify a potential fluvial flood risk at the location of the Proposed Development. Consultation with SEPA identified two incidents of river flooding within 1km radius of the Proposed Development. The location of the incidents was not available. The incidents are likely to be hydrologically separated from the Proposed Development as no fluvial waters are identified within the catchment of the site.

2.4 Groundwater

Groundwater flooding in Scotland is considered to be linked to other sources of flooding, such as fluvial or surface water, and can be difficult to differentiate from other types of flooding.

British Geological Survey (BGS) maps determine the onshore component of the Proposed Development is located on a very low to low productivity aquifer partially overlain by low permeability superficial deposits. The Port Ellen Phyllite Formation, which underlies the site of the Proposed Development, comprises of pelite and psammite which have limited water-bearing capacity, with groundwater flow primarily through fractures. Groundwater in close proximity to the shoreline is in hydraulic continuity with the sea and tidally influenced.

[17] URS Scott Wilson (2011) Frederick Crescent, Port Ellen, Islay Flood Risk Assessment – Phase 1

Due to limited groundwater flow and low water-bearing aquifer units the potential for, and risk of, groundwater flooding is assessed as negligible.

2.5 Infrastructure Failure

Flooding caused by infrastructure failure is characterised as flooding as a result of structural, hydraulic or geotechnical failure of a man-made structure including hydro-powered dams, water supply reservoirs, canals, flood defence structures, sewerage and water treatment tanks.

Sewerage and drainage system were identified in plans provided by Scottish Water, located approximately 100m north-east of the of the Proposed Development site. Scottish Water also provided information on sewer related flooding incidents and surcharging. The cause of flood is not always reported, however where the cause is reported it is as a result of blockage within the system or pump station failure. Scottish Water were not able to provide specific locations where sewer flooding incidents have occurred.

Argyll and Bute Council identified incident(s) of flooding at the Co-op convenience shop and southern end of Lennox St due to capacity issues with a surface water drain that outfall on to the beach. No further information in regard to this flooding incident was available.

Foul drainage at the site as identified on utility plans indicate a pipe network which conveys foul water to a 2,800-litre septic tank. Further detail on the foul drainage system is provided in Section 2.6 of the DIA.

No other relevant infrastructure is identified within the vicinity of the Proposed Development.

2.6 Baseline Flood Risk Summary

Table 2.2 has summarised the baseline flood risk. The need for detailed assessments was identified for coastal and surface water flood risk (see Section 3).

Table 2.2: Baseline Flood Risk Summary

Flooding Source	Baseline Risk	Justification	Detailed assessment required?
Coastal	High	SEPA flood map determines a High level of coastal flood risk. No current mitigation for coastal flooding at existing port. Areas of port hardstanding including the site entrance at elevations below the extreme high tide level.	Yes – wave overtopping assessment and tidal levels assessment
Surface Water	Moderate	No current mitigation for surface water flooding (e.g. SuDS, drainage) at existing port.	Yes – drainage impact assessment
Fluvial	Negligible	No source of fluvial flood risk.	No – screening sufficient
Groundwater	Negligible	No source of groundwater flood risk.	No – screening sufficient
Infrastructure	Low	Scottish water drainage and sewerage assets are not in hydraulic connection to the existing port site. No other relevant infrastructure identified.	Yes – assess foul drainage for proposed development in DIA

3 Post Development Flood Risk

3.1 Coastal

3.1.1 Tidal

The Proposed Development site is elevated comparative to the baseline (present day) development with elevations rising gradually from 2.66mAOD at the site entrance to 3.81mAOD at the terminal building and northern revetment quay wall. The Proposed Development quay walls and terminal building floor level is above the CFB extreme sea level, for 1 in 200-year scenario plus climate change, of +2.85mAOD (see Table 2.1).

The elevations at the site entrance are required to remain the same as existing (baseline) elevations. This is required to allow for a tie-in point to Pier Road which has multiple uses outside of the ferry terminal operations including access for an existing grain store building.

As per the baseline assessment, the elevation of the site entrance varies from 2.66mAOD on the south-eastern side to 2.81mAOD on the north-western side below the 1 in 200-year +cc CFB extreme sea level of +2.85mAOD. Due to the requirement to tie the site into the existing road, the site entrance remains at risk from tidal inundation under an extreme high tide scenario.

The Proposed Development's impact on tidal flood risk is assessed to be an improvement comparative to the baseline with the development quay walls and with the land at the terminal building being raised above the extreme high tide level including an allowance of 0.96m freeboard.

A risk of tidal flooding remains at the site entrance as per the baseline flood risk, which impacts the ability to ensure safe dry access and egress from the Proposed Development site for both pedestrians and vehicles. The Proposed Development does provide an area of safe refuge in the terminal building which is elevated above the extreme high tide level with a freeboard allowance of 0.96m. It is also noted that the extreme flood level for tidal flooding is of short duration because of natural tidal cycles.

Operational measures have been developed to manage residual risks and to ensure safe access and egress of people and vehicles during 1:200-year + CC storm events (Section 4).

3.1.2 Wave Overtopping

A wave overtopping assessment was undertaken for the Baseline and Proposed Development infrastructure. The mean wave overtopping discharges were calculated at representative cross-sections as illustrated in Figure 3-1 and Figure 3-2 for the baseline and Proposed Development scenarios respectively.

Wave parameters at the toe of the relevant structures were extracted from transformed wave heights using the wave model[18], for the 0.5% and 1% AEP events (2085 epoch climate change allowance), at the cross-section location B1-B3. Overtopping calculations were performed for each event, and the maximum discharge over the structures was determined for the predominant wave direction.

Appendix A.2 should be referred to for further details and limitations on the modelling undertaken to inform the wave overtopping assessment.

[18] Mott MacDonald (2024) Port Ellen Development Detail Design – Wave modelling Report

Figure 3-1 shows the baseline layout for the existing Port Ellen Pier along with the cross-sections (B1-B3) representative of the structures used to conduct the baseline overtopping assessment. The representative cross-sections for the Proposed Development scenario have been provided in Figure 3-2.

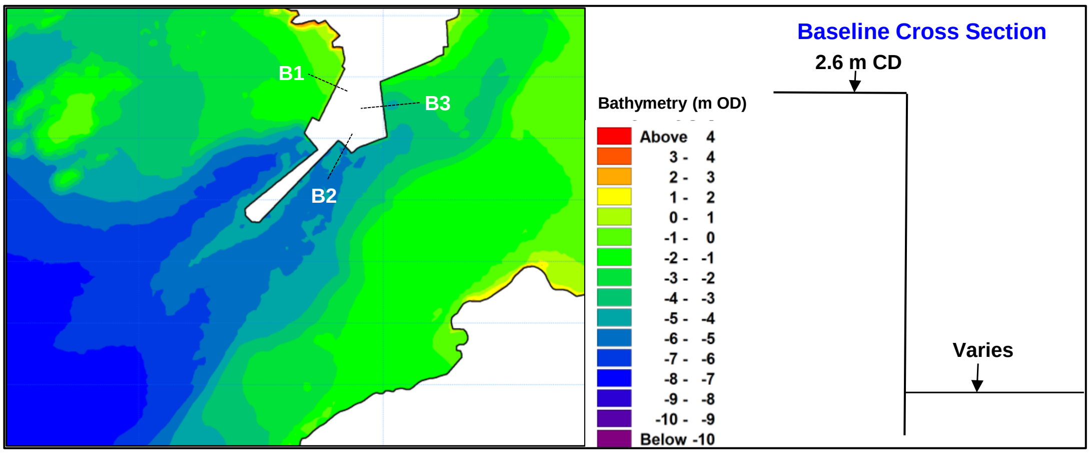


Figure 3-1: Port Ellen existing layout and cross sections for baseline Wave Overtopping Assessment

Source: Mott MacDonald (2024) Wave Overtopping Assessment

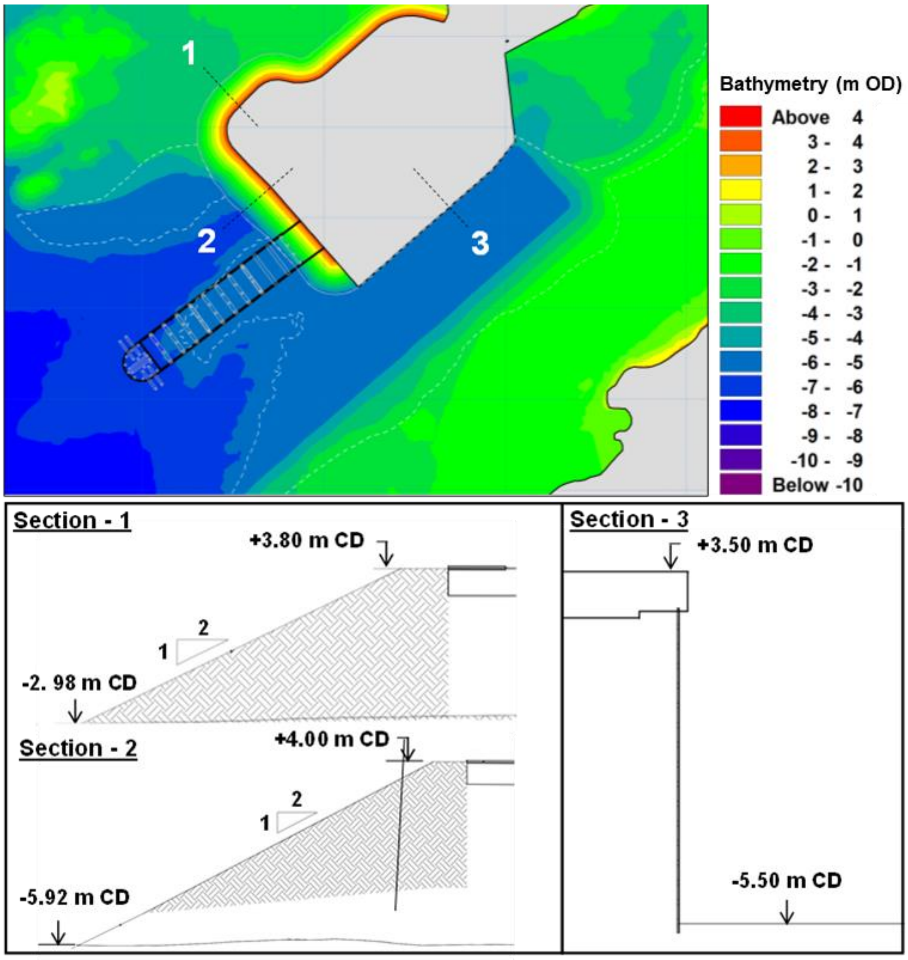


Figure 3-2: Cross-section locations and profiles for wave overtopping assessment

Source: Mott MacDonald (2024) Wave Overtopping Assessment

Maximum average overtopping discharges (provided in litres per second per metre) was calculated at each cross-section for the Baseline and Proposed Development scenario. These were compared to advised mean overtopping discharge limits for specific receptors, as specified in the EurOtop manual[19], shown in Table 3-1.

Table 3-1: Tolerance limits for wave overtopping

Receptor type	Mean discharge (q) (l/s/m)
Rubble mound breakwaters; Hs > 5 m; no damage	1
Rubble mound breakwaters; Hs > 5m; rear side designed for wave overtopping	5 to 10
Building structure elements; Hs = 1 to 3 m	⩽1
Damage to equipment set back 5 to 10m	⩽1
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping

Source: EurOtop (2018) Manual on wave overtopping of sea defences and related structures.

The calculated maximum average overtopping discharge for wave conditions for the Baseline and Proposed Development scenarios are presented in Table 3.2 and Table 3-3 respectively.

Table 3.2: Maximum average overtopping discharge for wave conditions (2085 Epoch) from the dominant wave directions for Baseline scenario cross-sections

Section	AEP Event	Structure	Maximum average overtopping discharge (l/s/m)
Section B1	1.0%	Quay	2.14
Section B1	0.5%	Quay	3.70
Section B2	1.0%	Quay	7.07
Section B2	0.5%	Quay	10.90
Section B3	1.0%	Quay	5.28
Section B3	0.5%	Quay	8.50

Table 3-3: Maximum average overtopping discharge for wave conditions (2085 Epoch) from dominant wave directions for Proposed Development design

Section	AEP Event	Structure	Maximum average overtopping discharge (l/s/m)
Section 1	1.0%	Revetment	0.62*
Section 1	0.5%	Revetment	0.62*
Section 2	1.0%	1.9 m revetment berm and 1.0m sea wall	0.25
Section 2	0.5%	1.9 m revetment berm and 1.0m sea wall	0.73
Section 3	1.0%	Quay	1.91*
Section 3	0.5%	Quay	1.91*

* The values are identical as they are rounded to two decimal places

The calculated mean overtopping discharge rates for the baseline scenario exceed the tolerance limits at all cross-sections for receptors including building structures, equipment and people.

[19] EurOtop (2018) Manual on wave overtopping of sea defences and related structures.

The calculated mean overtopping discharge rates for the Proposed Development are shown to be within the tolerance limits at Section 1 and Section 2 for 1:100-year (1% AEP) and 1:200-year (0.5%) storm events.

For Section 3, along the commercial berth quay wall of the Proposed Development, there is calculated to be an exceedance in the safety tolerance limits for building structure elements, damage to equipment to a set-back of 5-10m for 1:100-year and 1:200-year storm events.

3.1.3 Impact of development on coastal flood risk

The results from the existing wave modelling study[20] showed that the proposed reclamation and associated dredging had no adverse impacts on the prevailing wave climate. Thus, the wave conditions around Port Ellen are not expected to change significantly from the present-day 'baseline' condition as a result of the Proposed Development.

The wave overtopping rate for the Proposed Development infrastructure was calculated to be within EurOtop tolerance limits on north and western revetment quay walls (cross sections 1 and 2 respectively on Figure 3-2). The design was adjusted to include a 1.0m wave-wall and an increased berm width (to 1.9 m) along the western revetment. This was necessary to reduce the calculated wave over topping rate to a tolerable value.

The commercial berth of the Proposed Development (cross section 3) was calculated to experience wave overtopping at rates which exceed the tolerance limits for receptors including building structures and people. In line with the drainage assessment the site will be graded to route overtopping water away from operational areas and discharge directly to the sea. At this location it was not possible to include a wave wall or revetment in the design as access is required for fishing vessels. There are no buildings located along the commercial berth, and no public access within a 2m security fence set back from the quay wall. Further details on operational measures and safe access and egress in storm events has been outlined in Section 4.

The calculated wave overtopping rate for the existing infrastructure (Baseline scenario) predicts an exceedance of tolerance limits at all three locations. This would endanger key receptors including people, buildings, and equipment - as outlined in Table 3-1. The Proposed Development reduces wave overtopping to tolerable values at two of the three quay walls assessed.

3.2 Surface Water

The existing development relies on overland flow spilling over the quay walls, discharging directly to coastal waters. Presently there is no treatment or attenuation of surface water. The Proposed Development will have a greater hardstanding area but will employ SuDS to allow flood alleviation and pollution prevention measures, protecting key receptors in a 1:200-year + CC flood event. All drainage discharges to the sea (either directly or via SuDS) and will have no impact on a downstream receptors or catchments.

The DIA [Document Reference: 115031-MMD-00-ZZ-RP-C-0002] presents the outline drainage design. It has assessed potential surface water flooding, to and from, the Proposed Development using InfoDrainage flood modelling software; further details and results have been provided in the DIA. A summary of the level of protection at key infrastructure as assessed by the InfoDrainage model is presented in Table 3-4.

[20] Mott MacDonald (2024) Port Ellen Development Detail Design – Wave modelling Report

Table 3-4: Modelled Flood Risk Protection Levels for Key Infrastructure

Key Infrastructure	Modelled Flood Risk Protection Level
Terminal Building and Car Park	1 in 200-year event + CC
Marshalling Area	1 in 30-year event + CC
Unaccompanied Trailer Area	1 in 200-year + CC
Roads	1 in 30-year +CC
Finger Pier	Not applicable (pier will be cambered and surface water will discharge to sea without control or treatment)

The Proposed Development's impact on surface water flood risk was judged to be an improvement compared to the baseline.

Operational measures have been developed to manage residual risks and to ensure safe access and egress of people and vehicles during 1:200-year + CC storm events (refer to Section 4).

3.3 Infrastructure Failure

. A review of surface water pathways[21] and topography indicated that overland flow from the location of Scottish Water assets will drain to sea (south to Loch Leòdamais and north-west to Kilnaughton Bay) and will not flow towards the site of the Proposed Development, as shown in Figure 3-3.

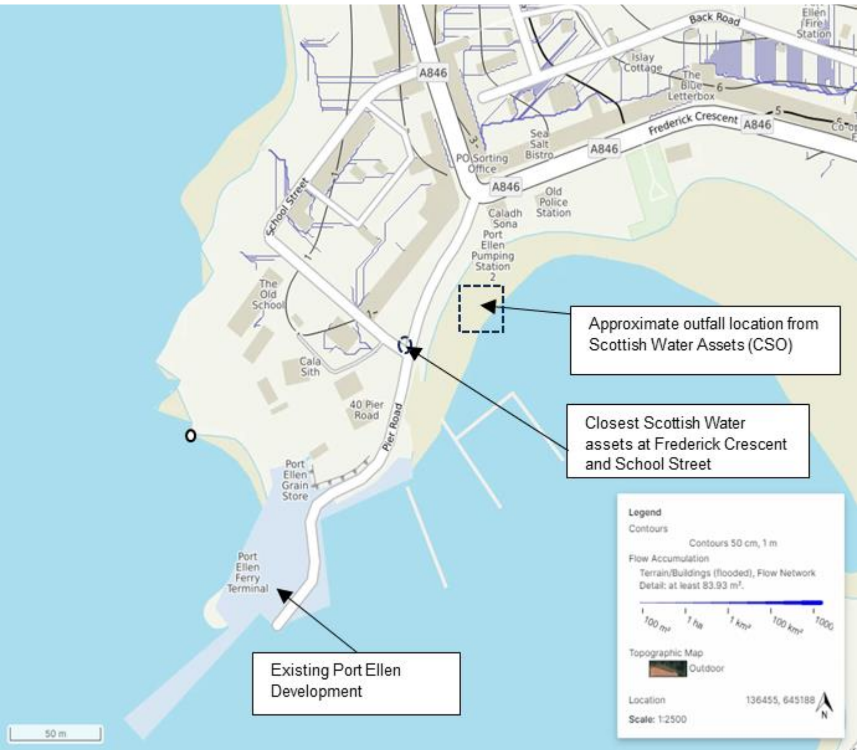


Figure 3-3: Overland flow pathways and topography from location of wastewater assets

Source: SCALGOLive. Map Data. © OpenStreetMap contributors. Contains OS data © Crown copyright and database right 2024.

[21] Scalgo Live Flow Network Detail (2024) based on data from SEPA LiDAR for Scotland Phase I-VI DTM (2023) and Ordnance Survey Open Rivers (2021), OS Open ZoomSTack land (2016) and OpenMap Local Building (2016)

The closest Scottish Water sewerage and drainage assets are located approximately 100m north-east of the site underneath Frederick Crescent and School Street. The review indicates that there will be a negligible impact on the Proposed Development should an infrastructure failure incident occur. Therefore flood risk from infrastructure failure was judged to be negligible.

3.4 Post-Development Flood Risk Summary

Table 3.5 summarises the flood risk for the Proposed Development with the inclusion of embedded mitigation (summarised in Section 1.2.2.1). A requirement for additional mitigation and operational measures can be seen, this has been outlined in Section 4. The assessment of flood risk following implementation of mitigation measures has been summarised in Table 5.1 in Section 5.

Table 3.5: Post-Development Flood Risk Summary

Flooding Source	Baseline Flood Risk	Embedded mitigation	Post-development flood risk	Additional mitigation
Coastal (tidal)	High	Floor level elevated above the extreme high tide level; risk remains at site entrance. Safe refuge provided at terminal building.	Moderate	Yes – safe access and egress procedures Safe refuge also possible
Coastal (wave overtopping)		Western revetment of 1.9m; Sea wall of 1.0m along the western revetment	Minor	
Surface Water	Moderate	Sustainable drainage systems and hardstanding camber	Minor	Yes – safe access and egress procedures
Fluvial	Negligible	Not applicable	Negligible	No
Groundwater	Negligible	Not applicable	Negligible	No
Infrastructure Failure	Low	No specific embedded mitigation – assessed as not being in hydraulic conductivity with the site due to Proposed Development site elevation	Low	No

4 Safe Access and Egress

The impact of flood risk has been reduced compared to the baseline scenario. The majority of the Proposed Development site is predicted to be safe from coastal flooding for a 1:200-year + CC (0.5% + CC AEP) storm event. Wave overtopping will be experienced at the commercial berth however it was not feasible to implement safe access and egress protection measures in the Proposed Development design (e.g sea wall, revetment) as this would hinder the functional operation of fishing vessels.

A risk of tidal inundation remains at the site entrance under 1 in 200-year (+cc) extreme high tide level which is comparative to the present-day risk at the existing Port Ellen site. This will limit the ability to ensure safe access and egress to and from the site and requires operational management to ensure safe dry access and egress.

Key receptors, including the terminal building are also predicted to be safe from surface water flooding for a 1:200-year + CC (0.5% + CC AEP) storm event. For other receptors, including roads and marshalling areas, the level of protection for surface water flooding is 1:30-year (3.3% AEP). This reduction in performance was due to space and elevation limitations.

During operation of the Proposed Development safe access and egress is required, residual risks from surface water flooding and wave overtopping must be managed to achieve this. Operational control measures have been outlined in Section 2.6 of the Environmental Impact Assessment Report[22].

[22] Mott MacDonald (2024) Port Ellen Terminal Development Environmental Impact Assessment Report

5 Summary of Findings

The Port Ellen Terminal Development, which includes expansion of the existing Port Ellen terminal, dredging and provision of a new terminal building, has been assessed for flood risk. Overall flood risk from the Proposed Development has been reduced in comparison to the baseline (present day) scenario.

All sources of flood risk were investigated, a detailed assessment was undertaken for coastal and surface water flood risk. Wave modelling was undertaken to assess coastal flood risk, and a hydraulic model was developed to assess drainage flood risk.

The site has been designed to minimise flood risk potential through incorporating sustainable drainage system (SuDS) and hardstand grading (camber) to allow surface and coastal waters to discharge to sea, either directly over quay walls or via SuDS.

The Proposed Development is predicted to be protected for a 1:200-year (0.5% AEP + CC) storm event for surface water flooding at key receptors, including the terminal building and public car park. For other receptors including roads and marshalling areas, the predicted level of protection for surface water flooding is 1:30-year (3.3% AEP).

Proposed coastal defences (sea walls and revetments), along the northern and western quays, were predicted to limit the over-topping rates during a 1 in 200-year event (plus an allowance for climate change) to an acceptably safe level around key receptors.

The Proposed Development site, including key receptors of the terminal building, car park, and marshalling areas, is elevated above the 1 in 200-year (plus an allowance for climate change) extreme high tide level. A risk of tidal inundation remains at the site entrance and will limit the ability to ensure safe access and egress to and from the site and requires operational management to mitigate against this risk. The terminal building is a safe refuge under an extreme high tide scenario. It is not possible to raise the floor level at the site entrance as it is required to tie-in to the existing Pier Road.

Flood risk from groundwater and infrastructure failure was judged to be negligible.

A safe access, egress and site operations summary has been provided in Section 2.6 of the Environmental Impact Assessment.

A summary table outlining the identified flood risks for the Proposed Development is provided in Table 5.1.

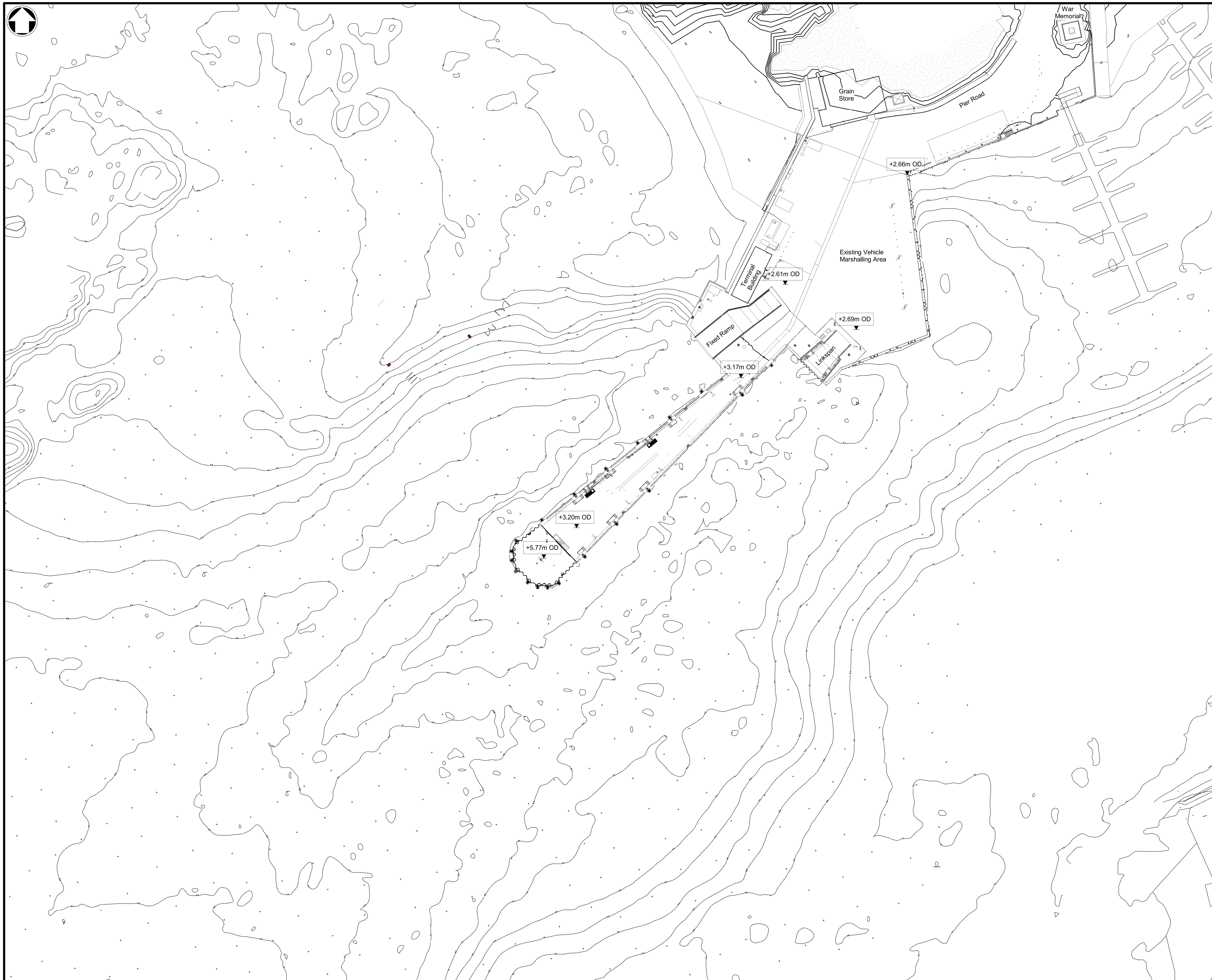
Table 5.1: Flood Risk Summary

Flood Source	Baseline Flood Risk	Post-Development Flood Risk (with embedded mitigation)	Additional Mitigation	Residual Risk
Coastal (tidal)	High	Moderate	Employ safe access and egress operations as set out in Section 2.6 of the Environmental Impact Assessment Report including control measures during storm events. Safe refuge also possible	Minor
Coastal (wave overtopping)		Minor		Negligible – reduced from baseline up to 1:30-year event. After 1:30-year, areas of roads and marshalling area will be same as baseline and will be manged through port operated access and egress control.
Surface Water	Moderate	Minor		Negligible
Fluvial	Negligible	Negligible	Not applicable	No identified risk (negligible)
Groundwater	Negligible	Negligible	Not applicable	No identified risk (negligible)
Infrastructure Failure	Low	Low	Not applicable	Low

Appendices

A.1 Site Layout

Figure A.1- 1: Existing Port Ellen Layout



Plan
1 : 500

1:500 0 25m 50m

Notes
1. All dimensions in millimetres unless noted otherwise.
2. All levels in metres relative to Chart Datum (mCD) unless noted otherwise.
3. DO NOT SCALE. Follow written dimensions only.
4. This drawing has been prepared to coordinate the outline design and is suitable for Client review and comment. Not for construction.
5. All structural details are preliminary and subject to detailed design.
6. Levels in boxes are to Ordnance Datum (m OD)

Key to symbols

Reference drawings

External Drawing:
Topographical and Bathymetric
Reference: A8024
Date of Survey: 13.05.2022

Rev	Date	Drawn	Description	Ch'k'd	App'd
P1	24.09.2024	Tech'n	WORK IN PROGRESS	Eng	PM

Status Stamp
FOR INFORMATION

MOTT MACDONALD
St Vincent Plaza
319 St Vincent Street
Glasgow, G2 5LD
United Kingdom

T +44(0)141 222 4500
F +44(0)141 221 2048
W - www.mottmac.com

Client
Caledonian Maritime Assets Limited
Municipal Buildings
Fore Street
Port Glasgow
PA14 5EQ

CMAL Caledonian Maritime Assets Ltd Stòras Mara Caileannach Eta

Title
Port Ellen Terminal Development
General Arrangement
Existing Site Layout

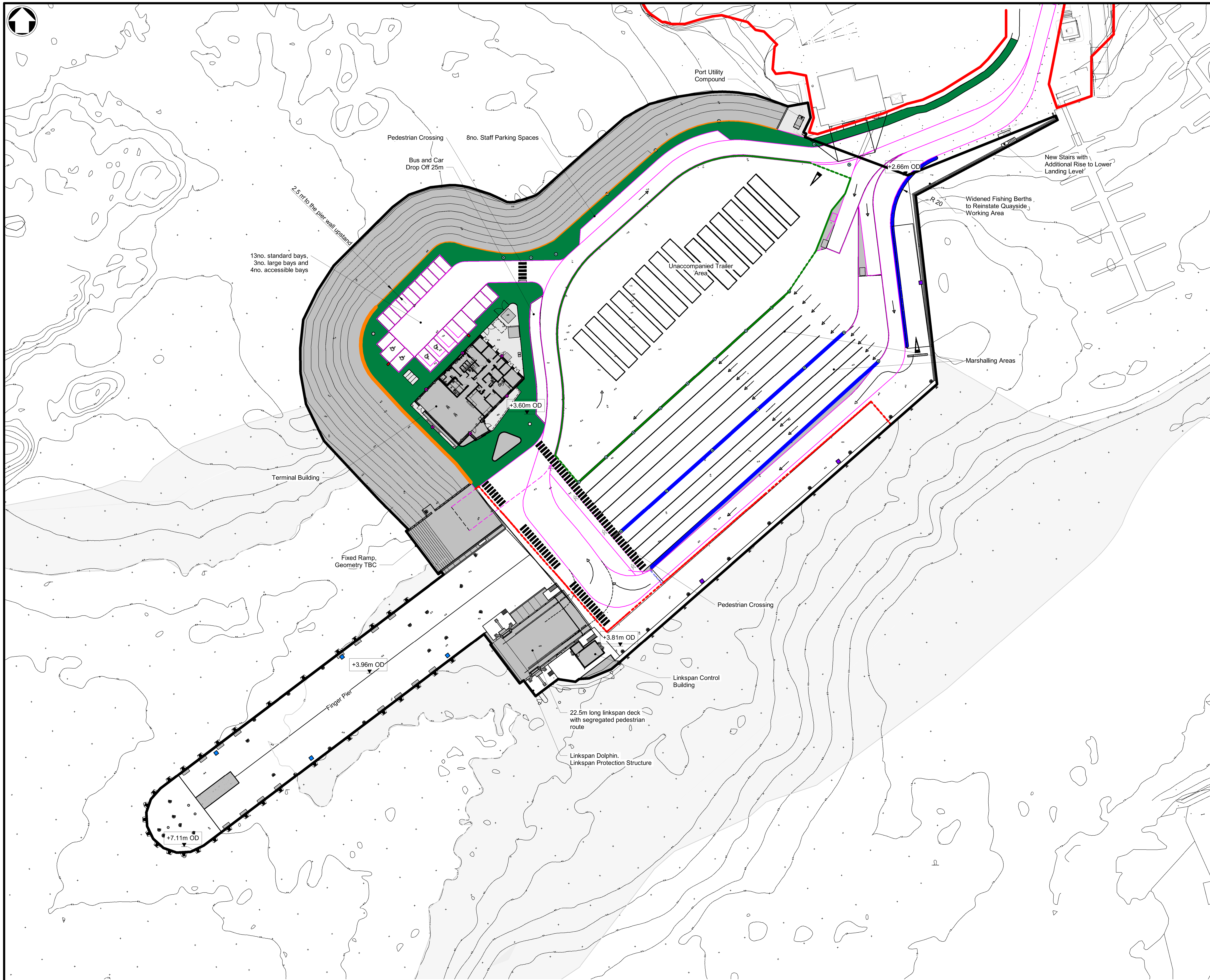
Designed	Engineer	Eng. Check	Project Technical Lead
Drawn	Technician	Coordination	Project Manager
Dwg. Check	Engineer	Approved	Project Manager

MMD Project Number	Scale at A1	Security
115031	1:500	STD

Suitability Description	Suit. Code
For Information	S1

Drawing Number	Rev
115031-MMD-PE-XX-DR-Z-0110	P1

Figure A.1- 2: Proposed Port Ellen Terminal Development Layout



Refined Layout

1 : 500

This document is issued for the party which commissioned it and for specific purposes connected with the captioned project only. It should not be relied upon by any other party or used for any other purpose.
We accept no responsibility for the consequences of this document being relied upon by any other party, or being used for any other purpose, or containing any error or omission which is due to an error or omission in data supplied to us by other parties.

26/09/2024 09:34:48 Autodesk Docs://100115031 - Port Ellen Development Detail Design - Fixed Price/115031-MMD-PE-XX-M3-C-0001.rvt

Notes

1. All dimensions in millimetres unless noted otherwise.
2. All levels in metres relative to Chart Datum (mCD) unless noted otherwise.
3. DO NOT SCALE. Follow written dimensions only.
4. This drawing has been prepared to coordinate the outline design and is suitable for Client review and comment. Not for construction.
5. All structural details are preliminary and subject to detailed design.
6. Levels in boxes are to Ordnance Datum (m OD)

Key to symbols

- Pedestrian Area
- Pedestrian Guardrail
- Wave Wall
- 2m Security Fence
- Painted delineator with 1.2m Mesh Fence
- Painted Delineator
- Road Contours
- EV Charging Point
- 6m Hinged Lighting Column
- 8m Hinged Lighting Column
- Building Mounted Lighting Column
- Combined water and power (110V) service bollard - locations TBC

Reference drawings

External Drawing:
Topographical and Bathymetric
Reference: A8024
Date of Survey: 13.05.2022

Rev	Date	Drawn	Description	Ch'k'd	App'd
P1	24.09.2024	Tech'n	WORK IN PROGRESS	Eng	PM

Status Stamp

WORK IN PROGRESS

MOTT MACDONALD

St Vincent Plaza
319 St Vincent Street
Glasgow, G2 5LD
United Kingdom

T +44(0)141 222 4500
F +44(0)141 221 2048
W - www.mottmac.com

Client
Caledonian Maritime Assets Limited
Municipal Buildings
Fore Street
Port Glasgow
PA14 5EQ

CMAL Caledonian Maritime Assets Ltd Stòras Mara Caileannach Eta

Title
Port Ellen Terminal Development
General Arrangement
Refined Site Layout

Designed	Engineer	Eng. Check	Project Technical Lead
Drawn	Technician	Coordination	Project Manager
Dwg. Check	Engineer	Approved	Project Manager

MMD Project Number	Scale at A1	Security
115031	1:500	STD

Suitability Description	Suit. Code
Work In Progress	S0

Drawing Number	Rev
115031-MMD-PE-XX-DR-C-0312	P1

A.2 Port Ellen Development Wave Overtopping Assessment

Wave Overtopping Assessment

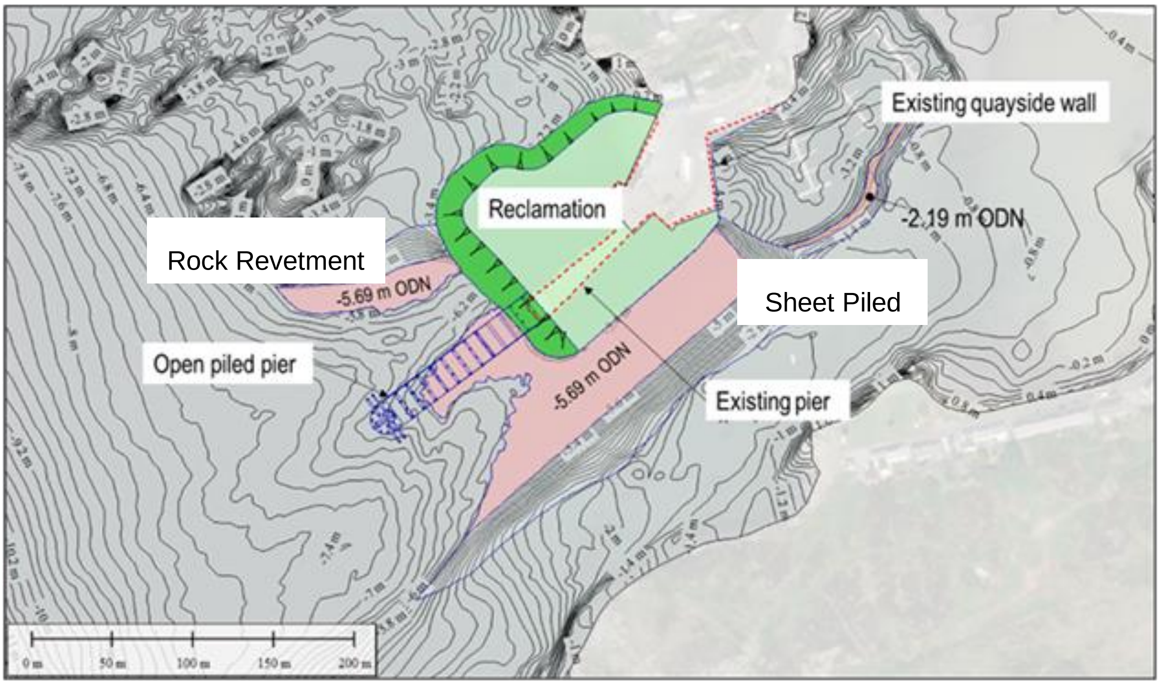
Estimating Wave Overtopping for Annual Exceedance Probabilities of 1% and 0.5% in the Flood Risk Assessment

Project:	Port Ellen Terminal Development – Flood Risk Assessment		
Our reference:	<Insert Text>	**Your reference:**	<Insert Text>
Prepared by:	Sathish Kumar	**Date:**	27 June 2024
Approved by:	Adrian Wright	**Checked by:**	Jon Williams
Subject:	Wave Overtopping Assessment for 0.5% and 1% AEP Events		

1 Introduction

Mott MacDonald has been commissioned to undertake a Flood Risk Assessment (FRA) to support the planning process for the Port Ellen Terminal Development (Figure 1.1). The development is protected by a rock revetment with a slope of 1 in 2 along the southwest and northwest of the reclaimed area, and a quay wall facing the northeast. As part of the FRA a wave overtopping assessment has been carried out for a range of extreme conditions (1 and 0.5% Annual Exceedance Probability - AEP). This technical note describes the overtopping assessment methodology and conclusions.

Figure 1.1: Port Development Layout Option 5B



Source: Mott MacDonald, 2024

2 Background met ocean data

2.1 Water levels

The astronomical tide at Port Ellen is classified as micro-tidal, with a mean tidal range of around 0.6m. (Table 2.1).

Table 2.1: Tide levels at Port Ellen

Tidal level	Chart datum (m CD)	Ordnance datum (m OD)
Highest astronomical tide (HAT)	1.10	0.91
Mean high-water springs (MHWS)	0.90	0.71
Mean high-water neaps (MHWN)	0.80	0.61
Mean sea level (MSL)	0.46	0.27
Mean low-water neaps (MLWN)	0.50	0.31
Mean low-water springs (MLWS)	0.30	0.11
Lowest astronomical tide (LAT)	-0.30	-0.49

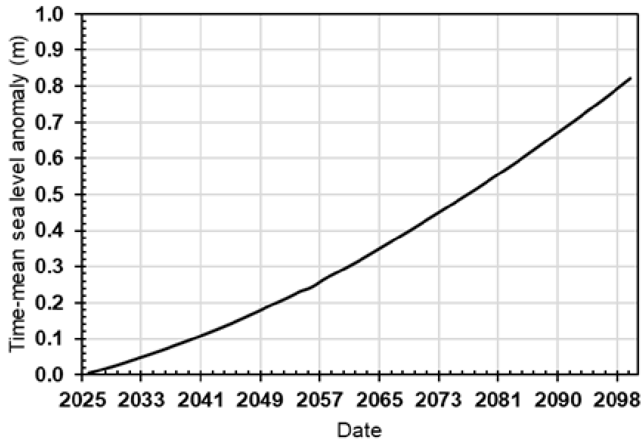
Source: Admiralty Total Tide, 2021

2.2 Climate Change

A fixed water level set at MHWS (0.71 m ODN) was used for the overtopping assessment. According to the Defra Joint Probability guidance (FD2308), the area is classified as modestly correlated to waves and water levels, meaning that this assumption represents a reasonable worst-case (see Section 1-4). A sea level rise of +0.6 m, based on the UKCP18 95th percentile RCP 8.5 scenario for 2085, was applied to the 2025 ‘baseline’ tidal condition.

UKCP18 suggests that changes in wave heights could occur due to increased water depth or alterations in the frequency, duration, and severity of storms. This study assumes the period from 2000 to 2055, and then from 2056 to 2125, wind speed and wave heights may experience an increase of 5% and 10%, respectively[1].

Figure 2.1: Projected time-mean sea level anomaly (m) for RCP 8.5 at 55.61N, -6.08E 2025 to 2100 (i.e. corrected for the base year of 2025)



[1] https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

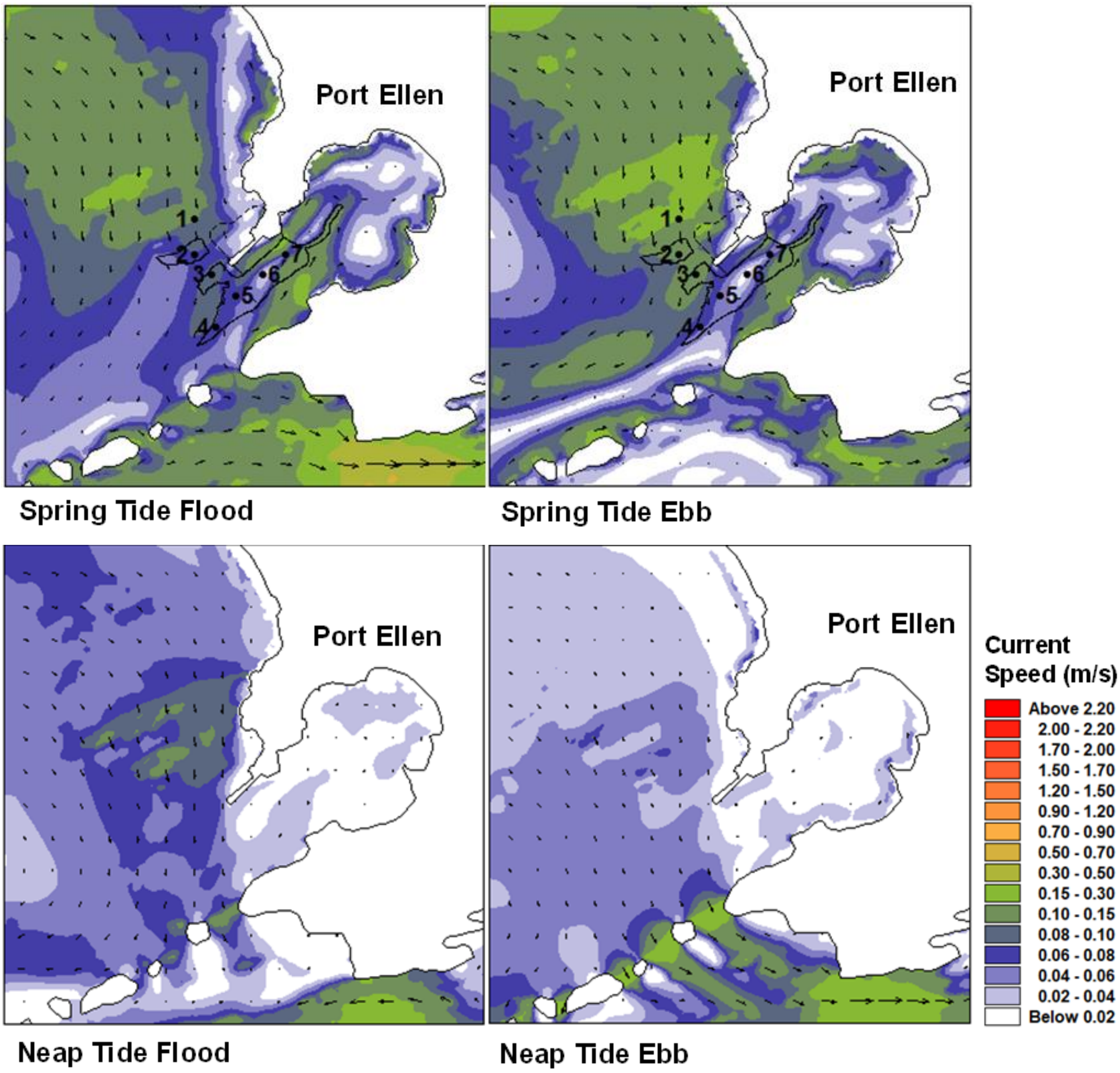
Source: Climate Change, 2024

2.3 Hydrodynamic flow characteristics

The dynamic interaction between the North Atlantic and the Irish Sea influences the flows around Islay. This tidal interaction significantly influences the flow characteristics south of Islay. In particular, it leads to a resonance effect at the tidal wave frequency, forming an amphidromic point from which the tide diverges.

The tidal flow and circulation into Port Ellen shows a weak circulation influenced by the strong flow in the deeper channel. Here, the Port Ellen site is characterised by a small tidal range (0.6 m on spring tides) and slow flow speeds, and tidal currents within the bay are from the south to the east, generating a weak circulation during spring and neap tides. The flow around the Port Ellen development from the northwest is generally aligned parallel to the coast. Wind stress over shallow bathymetry within Kilnaughton Bay results in higher current speeds (0.3 m/s) than tide-only conditions (0.1 m/s). Figure 2.2 shows the variation in tidal currents around Port Ellen during the spring and neap flood and ebb tides.

Figure 2.2: Spatial current flow around Port Ellen during spring and neap for flood and ebb tide, respectively

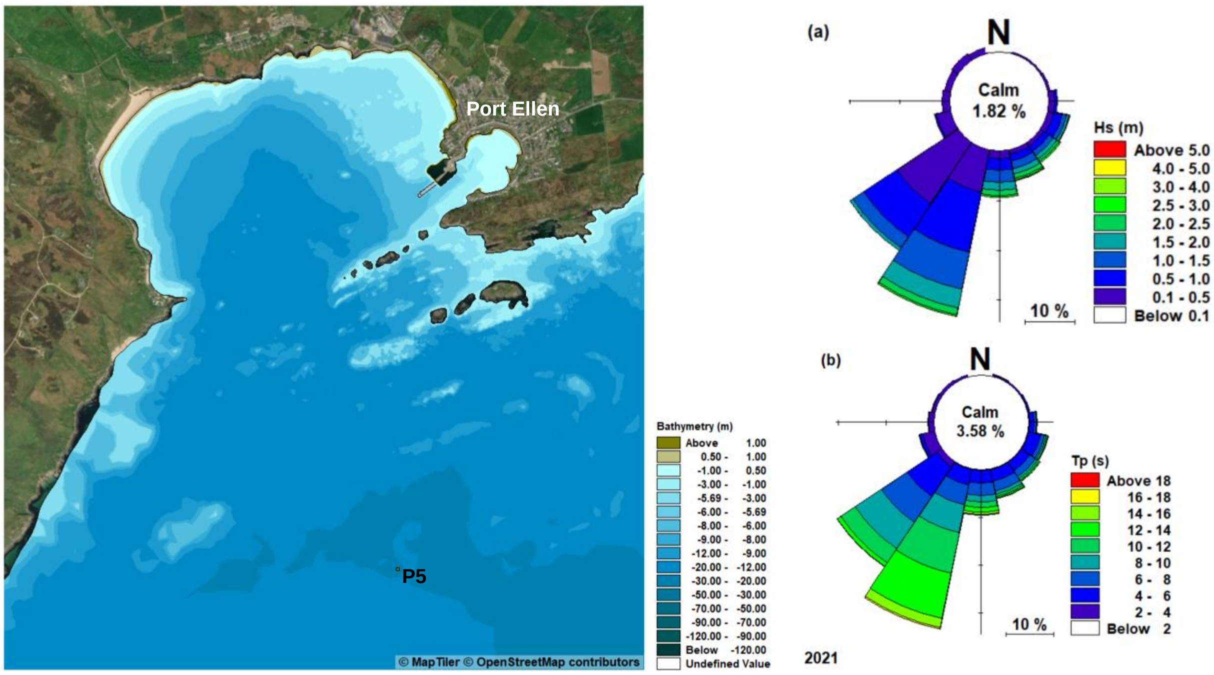


Source: Mott MacDonald, 2024

2.4 Inshore wave characteristics

The existing calibrated MML Port Ellen MIKE 21 FMSW model[2] generated wave data covering 43 years from 1979 to 2022. The data show that the dominant waves at the entrance to Port Ellen (Figure 2.3) originate from the southeast to the southwest and account for approximately 82% of the total wave record (1979 to 2022). Wave heights of less than 1m occur approximately 69% of the time.

Figure 2.3: Inshore wave data extraction Point P5 (136016.8 E, 642808.2 N, BNG-OSGB36) with (a) significant wave height, and (b) peak wave period roses from data from 1979 to 2022



Source: Mott MacDonald, 2024

The island chain at the entrance provides a natural breakwater aligned northeast to the southwest that shelters the port from the dominant wave directions (Figure 2.3). An analysis of wave propagation within the harbour at three locations (Figure 2.4) showed a reduction in wave height and a narrowing of the dominant wave direction as waves propagate inshore and refract. Importantly, the reduction in wave height near Port Ellen exceeds 70% compared to the maximum significant wave height at the entrance to the bay (Figure 2.3).

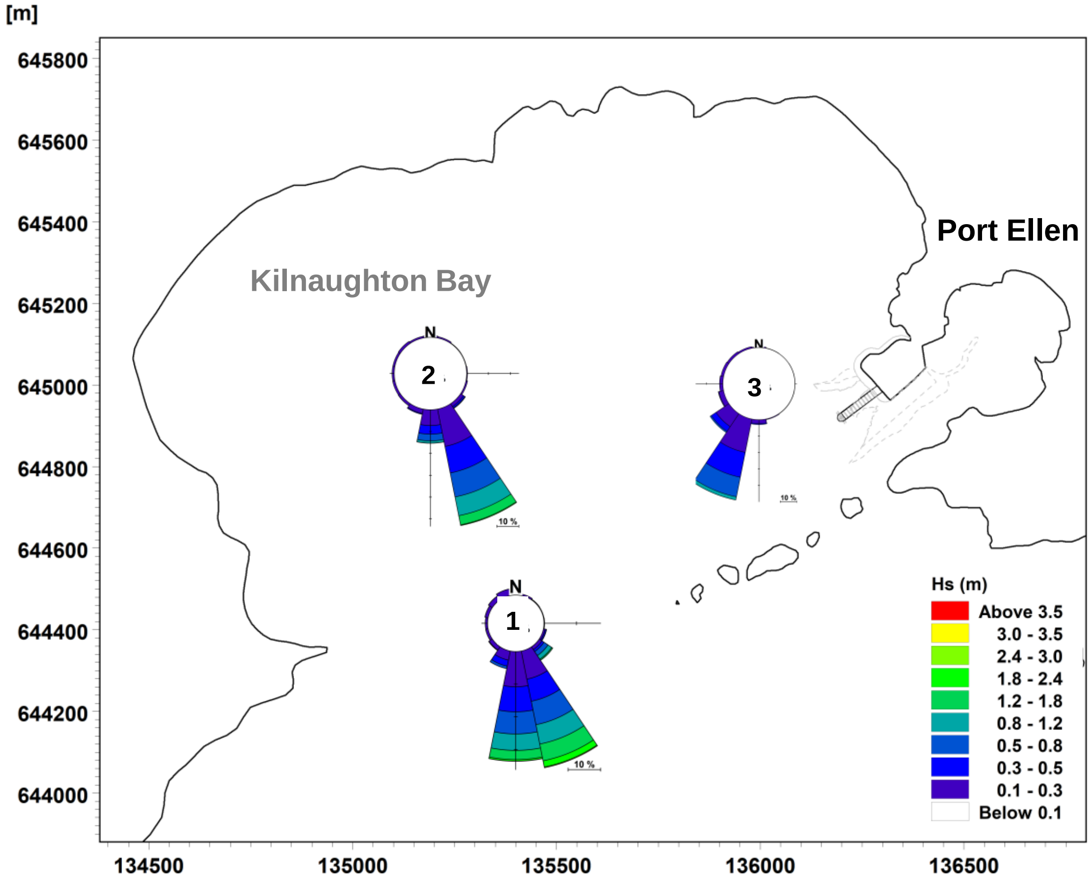
Typically, wave heights near Port Ellen are less than 0.3m and 0.5m for over 75% and 90% of the time. This indicates that the wave climate at and around the port is typically small and significantly less than in the adjacent coastal waters.

2.4.1 Scheme Impacts

The results from the wave modelling (MML, Port Ellen Wave Modelling, June 2024) showed that the proposed reclamation and associated dredging had no adverse impacts on the prevailing wave climate. Thus, the wave conditions around Port Ellen are not expected to change significantly from the present-day ‘baseline’ condition.

[2] Port Ellen Development Detail Design – Wave modelling Report

Figure 2.4: Annual average wave height rose, showing offshore to nearshore wave transformation characteristics.



Source: Mott MacDonald, 2024

3 Overtopping assessment methodology

The approach for estimating wave overtopping around the development area is as follows:

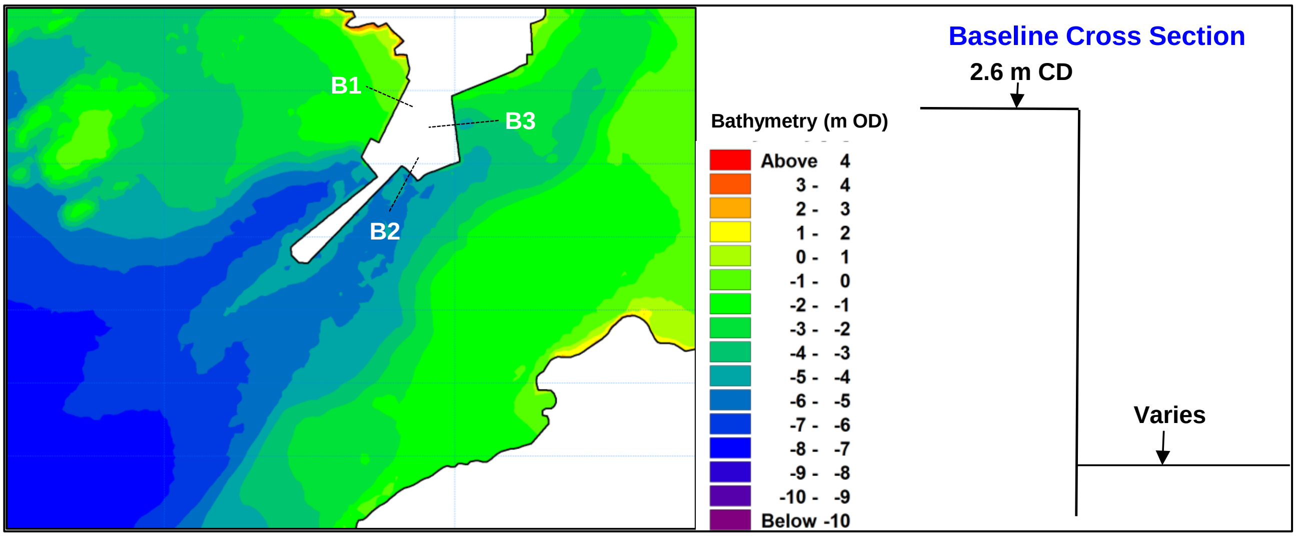
- Derive representative cross-sections based on the crest elevation and bed level at the toe of the structure for sections facing northwest, southwest and southeast;
- Establish mean tide levels for Port Ellen using the Admiralty tide tables;
- Assess climate change allowances for sea-level rise (SLR) and changes in wind speed and wave heights based on UKCP18 and EA/SEPA recommendations;
- Conduct a dependence assessment for wave and water level extremes using the EA/DEFRA Joint Probability approach (JPA, FD2308);
- Assess the tidal water level and current speeds using the in-house calibrated and validated MIKE by DHI hydrodynamic model of Port Ellen;
- Assess the wave characteristics around the Port Ellen development area based on the previous wave modelling studies;

- Estimate the Annual Exceedance Probability (AEP) for 1% and 0.5% AEP events and define the wave height and water level combinations for overtopping assessment;
- Calculate the mean wave overtopping discharges at the Port Ellen terminal structures using the EurOtop Artificial Neural Network (ANN) tool; and
- Summarise for design purposes the wave overtopping discharge estimates and analysis based on the structural configuration and evaluate the wave overtopping discharge estimates against safe limits for people, structure type, property, and vehicles;

3.1 Port Ellen Baseline Layout

The baseline infrastructure at Port Ellen includes the pier, the CalMac Ferry Terminal, fish quays, and floating pontoons. The existing structures at the pier and the CalMac Ferry Terminal consist of vertical walls. These walls are composed of various materials, including sheet piles, concrete, and stone blocks. Figure 3.1 shows the baseline layout along with the cross-section of representative of the structures used in the overtopping discharge assessment.

Figure 3.1: Port Ellen Baseline Layout and cross section

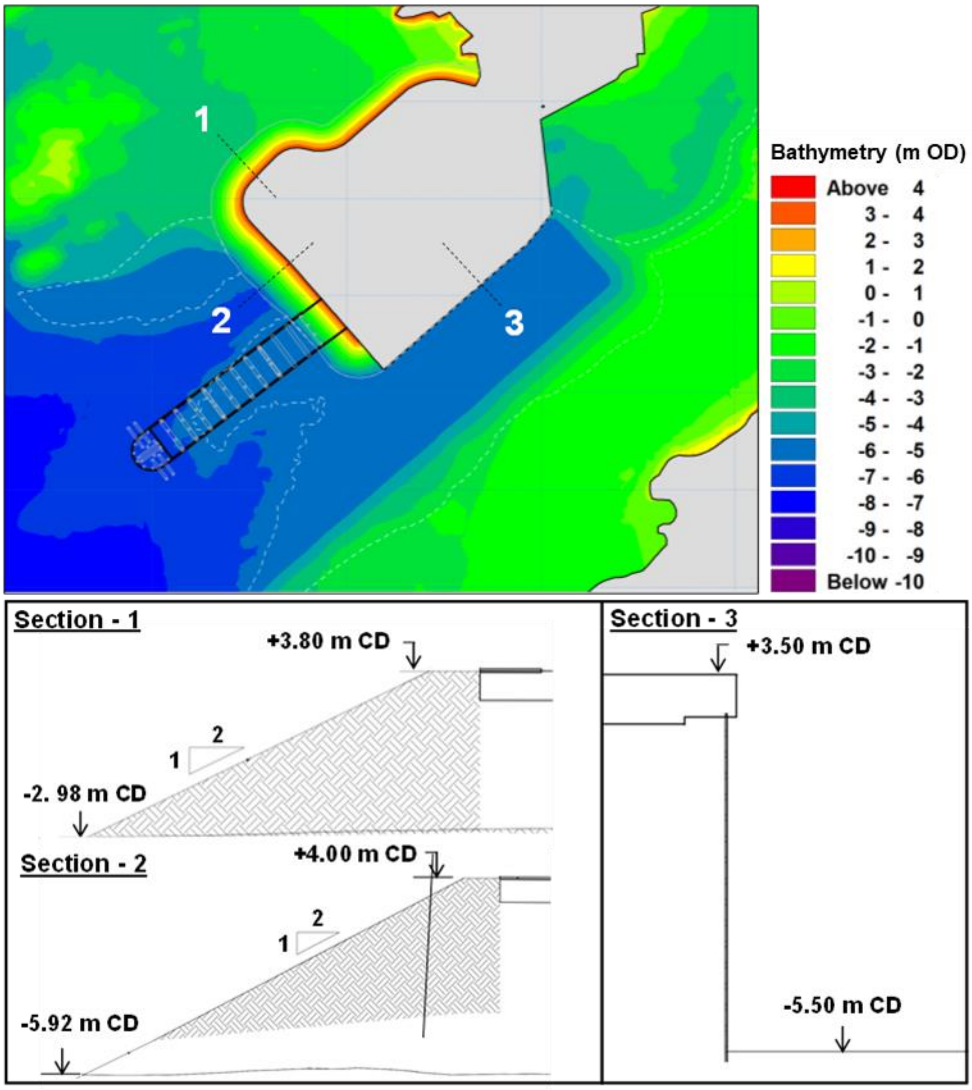


Source: Mott MacDonald, 2022

3.2 Port Ellen Layout Option 5B - cross sections

Figure 3.2 shows the layout of Option 5B and the cross-sections of these representative coastal structures used in the overtopping discharge assessment. The marshalling area does not include any crest wall but would have a sloping gradient for the drainage. A rock revetment, with a slope of 1 in 2, on both the northwest and southwest sides of the reclamation area represented by Sections 1 and 2. The Quay wall, with a dredging depth of -5.50 m CD, is located in the southeast, and its cross-section is represented by Section 3.

Figure 3.2: Cross-section of the quay wall and revetment defence around the development area



Source: Mott MacDonald, 2024

3.3 Annual exceedance probability (AEP)

Using the DHI extreme value analysis (EVA) tool, a marginal EVA of wave data at the entrance to Port Ellen (Figure 2.3) was undertaken using a 'Peaks Over Threshold' approach to determine the characteristics of 1% and 0.5% AEP wave events for 22.5-degree directional wave sectors between 135 to 225 deg. N. The approach defined the 95% confidence limits for Epoch 2085 and accounted for the 10% increase in wave heights due to climate change (Section 2.2).

Table 3.1 and Table 3.2 shows the estimated EVA wave extremes for significant wave height (Hs) and peak wave period (Tp) across each directional sector for Epoch 2085 at the representative sections. These estimate account for a projected 10% increase in wave heights due to climate change for both the baseline and layout 5B.

Table 3.1: EVA: Hs for the 95th percentile and Tp by direction sectors for Epoch 2085 accounting for the 10% increase in wave height for projected climate change for baseline layout.

		Section B1		Section B2		Section B3	
AEP	Direction	Hs (m)	Tp (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
1%	South-Southwest - Swell	0.9	17.6	1.1	17.6	1.0	17.6
	South - Swell	0.9	19.5	1.2	19.5	1.1	19.5
	Southwest - Swell	0.5	13.0	0.7	13.0	0.6	13.0
	Southeast - Swell	0.7	19.1	1.0	19.1	0.9	19.1
	South-Southeast - Swell	0.8	16.7	1.1	16.7	1.0	16.7
0.5%	South-Southwest - Swell	0.9	17.7	1.2	17.7	1.1	17.7
	South - Swell	1.0	19.6	1.2	19.6	1.1	19.6
	Southwest - Swell	0.5	13.1	0.7	13.1	0.6	13.1
	Southeast - Swell	0.8	19.3	1.0	19.3	0.9	19.3
	South-Southeast - Swell	0.9	16.9	1.2	16.9	1.1	16.9

Source: Mott MacDonald, 2024

Table 3.2: EVA: Hs for the 95th percentile and Tp by direction sectors for Epoch 2085 accounting for the 10% increase in wave height for projected climate change for Layout 5B.

		Revetment NW (section 1)		Revetment SW (Section 2)		Quay (Section 3)	
AEP	Direction	Hs (m)	Tp (s)	Hs (m)	Tp (s)	Hs (m)	Tp (s)
1%	South-Southwest - Swell	1.4	17.6	1.7	17.6	1.1	17.6
	South - Swell	1.5	19.5	1.8	19.5	1.2	19.5
	Southwest - Swell	0.8	13.0	1.0	13.0	0.7	13.0
	Southeast - Swell	1.2	19.1	1.4	19.1	1.0	19.1
	South-Southeast - Swell	1.4	16.7	1.6	16.7	1.1	16.7
0.5%	South-Southwest - Swell	1.5	17.7	1.8	17.7	1.2	17.7
	South - Swell	1.5	19.6	1.9	19.6	1.2	19.6
	Southwest - Swell	0.9	13.1	1.1	13.1	0.7	13.1
	Southeast - Swell	1.3	19.3	1.6	19.3	1.0	19.3
	South-Southeast - Swell	1.5	16.9	1.8	16.9	1.2	16.9

Source: Mott MacDonald, 2024

3.4 Overtopping results

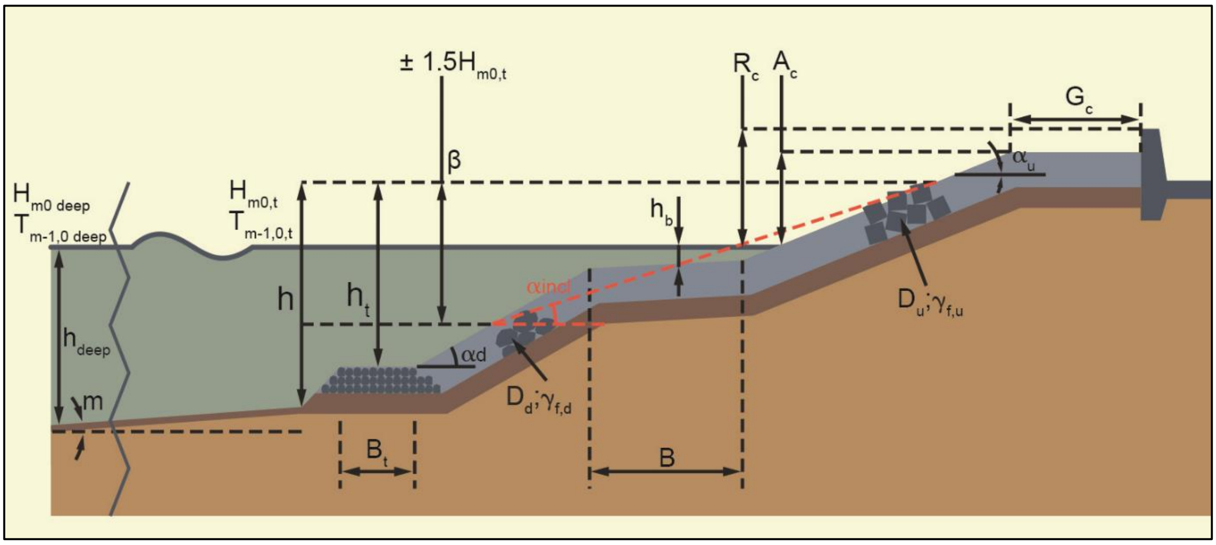
The assessment of wave overtopping discharge over the crest of the structures along the Port Ellen terminal was conducted using EurOtop (2018) equations. Overtopping discharge rates were calculated for Epoch 2085, considering the AEP (Annual Exceedance Probability) of 1% and 0.5% wave events.

For each AEP wave event, wave parameters at the toe of the structures were extracted from the transformed wave heights using the MIKE3 Wave FM wave model at three locations along the Baseline and Layout 5B terminal (Figure 3.1 and Figure 3.2). Overtopping calculations were performed for every event, and the maximum discharge over the structures was determined for waves from the dominant directions (Table 3.2 and Table 3.2). This analysis aimed to identify the worst-case scenario associated with the AEP of 1% and 0.5%.

3.5 EurOtop artificial neural network overtopping tool

The assessment of wave overtopping discharges at the Port Ellen terminal structures was conducted using the artificial neural network (ANN) tool developed by the University of Bologna[3] and adopted from EurOTop (2018)[4]. The ANN tool is trained using the comprehensive CLASH[5] database, which contains measured overtopping data from model and field tests. The database covers many defence geometries and wave conditions, providing a high prognostic skill. This tool estimates three parameters: **q** (overtopping discharge), **Kr** (reduction factor for roughness), and **Kt** (reduction factor for the presence of a berm). Figure 3.3 shows the schematisation of the structural geometry and associated hydraulic parameters defied in the CLASH database. Table 3.3 defines these parameters and their usage based on the input cross-section geometry.

Figure 3.3: Schematisation of the structure geometry and hydraulic parameters in the CLASH database.



Source: Mott MacDonald, 2024

Table 3.3: Geometric and hydraulic inputs used in the ANN overtopping tool

#	Parameter	Unit	Definition of the parameter	Remarks
1	m		The cotangent of the foreshore slope	If no foreshore is present, it is equal to 1000
2	h	(m)	Water depth at the structure toe	
3	$H_{s,t}$	(m)	Significant wave height at the structure toe	
4	$T_{m-1,0,t}$	(s)	Spectral wave period at the structure toe	
5	B	[°]	Wave obliquity	
6	h_t	(m)	Toe submergence	If no toe is present, it is equal to h
7	B_t	(m)	Toe width	If no toe is present, it is equal to 0
8	H_b	(m)	Berm submergence	If no toe is present, it is equal to 0
9	B	(m)	Berm width	If no toe is present, it is equal to 0

[3] www.unibo.it/overtopping-neuralnetwork/

[4] http://www.overtopping-manual.com/eurotop/neural-networks-and-databases/

[5] The CLASH database stands for Crest Level Assessment of Coastal Structures by full scale monitoring, neural network prediction, and Hazard analysis on permissible wave overtopping.

#	Parameter	Unit	Definition of the parameter	Remarks
10	$cot_{\alpha d}$	(-)	The cotangent of the angle that the structure part below the berm makes with a horizontal	The cotangent of the angle that the structure part below the berm makes with a horizontal
11	$cot_{\alpha u}$	(-)	The cotangent of the angle that the structure part above the berm makes with a horizontal	If no berm is present, it is equal to cotαd
12	γ_{fd}	(-)	Roughness factor for $cot_{\alpha d}$	
13	γ_{fu}	(-)	Roughness factor for $cot_{\alpha u}$	
14	D_d	(m)	Size of the structure elements along $cot_{\alpha d}$	For smooth structures, it is equal to 0
15	D_u	(m)	Size of the structure elements along $cot_{\alpha u}$	For smooth structures, it is equal to 0
16	A_c	(m)	Crest height references to still water level	In the absence of wave walls and/or promenades, it is equal to Rc
17	R_c	(m)	Wall height references to still water level	In the absence of wave walls and/or promenades, it is equal to Ac
18	G_c	(m)	Crest width	

Source: Università di Bologna, contains data from EurOtop (2018)

3.6 Schematisation of structures used in the overtopping assessment

All water and structural elevations used in the overtopping assessment were referenced to the Ordinance Datum. The water depth (h) at the toe of the structure is the sum of the water depth and the MHWS water level. Table 3.4 and Table 3.5 present the schematisation of the ANN input parameters for a selected cross-section's schematisation (as shown in Figure 3.1 and Figure 3.2) for both the baseline and Layout 5B.

Note, in some cases (e.g. armour size), expert judgment and sensitivity testing have been undertaken to confirm the suitability/ sensitivity of these parameters to the predicted mean overtopping volumes. Where applicable, the default or mean values have been applied.

Table 3.4: Schematisation of input parameters for selected baseline cross sections assessment

Cross Section	h (m)	h_t (m)	B_t (m)	H_b (m)	B	$cot_{\alpha d}$	$cot_{\alpha u}$	γ_{fd}	γ_{fu}	D_d (m)	D_u (m)	A_c (m)	R_c (m)	G_c
Section B1	3.9	3.9	0	0	0	0	0	0	0	0	0	1.7	1.7	0
Section B2	5.9	5.9	0	0	0	0	0	0	0	0	0	1.7	1.7	0
Section B3	4.9	4.9	0	0	0	0	0	0	0	0	0	1.7	1.7	0

Source: Mott MacDonald, 2024

Table 3.5: Schematisation of input parameters for selected cross sections assessment

Cross Section	h (m)	h_t (m)	B_t (m)	H_b (m)	B	$cot_{\alpha d}$	$cot_{\alpha u}$	γ_{fd}	γ_{fu}	D_d (m)	D_u (m)	A_c (m)	R_c (m)	G_c
Section 1	3.88	3.88	0	0	0	1.83	1.83	0.5	0.5	1.0	1.0	2.9	2.9	1.8
Section 2	6.82	6.82	0	0	0	1.83	1.83	0.5	0.5	1.0	1.0	3.1	3.1	1.8
Section 3	6.40	6.40	0	0	0	0	0	0	0	0	0	2.6	2.6	0

Source: Mott MacDonald, 2024

3.7 Summary of overtopping assessment for baseline and layout 5B

Overtopping discharge for 1% and 0.5% AEP events for the 2085 epoch (including climate change) were estimated at 3 cross sections along the Port Ellen baseline and development area representative of the revetment and quay wall sections. Overtopping discharge estimates were carried out for the AEP events from the dominant wave directions with the worst-case volumes presented below in Table 3.6 and Table 3.7 corresponding to the baseline and development scheme respectively.

Table 3.6: Maximum average overtopping discharge for wave conditions (2085 Epoch) from the dominant wave directions for baseline sections

Sections	AEP Wave Event	Crest m CD	Crest m OD	Depth at Toe (m CD)	Depth at Toe (m OD)	Structure Type	Maximum average overtopping Discharge l/s/m
Section B1	1.0%	2.6	2.41	-3.0	-3.19	Quay	2.14
Section B1	0.5%	2.6	2.41	-3.0	-3.19	Quay	3.70
Section B2	1.0%	2.6	2.41	-5.0	-5.19	Quay	7.07
Section B2	0.5%	2.6	2.41	-5.0	-5.19	Quay	10.90
Section B3	1.0%	2.6	2.41	-4.0	-4.19	Quay	5.28
Section B3	0.5%	2.6	2.41	-4.0	-4.19	Quay	8.50

Source: Mott MacDonald, 2024

Table 3.7: Maximum average overtopping discharge for wave conditions (2085 Epoch) from the dominant wave directions for development sections

Sections	AEP Wave Event	Crest m CD	Crest m OD	Depth at Toe (m CD)	Depth at Toe (m OD)	Structure Type	Maximum average overtopping Discharge l/s/m
Section 1	1.0%	3.8	3.61	-2.98	-3.17	Revetment	0.62*
Section 1	0.5%	3.8	3.61	-2.98	-3.17	Revetment	0.62*
Section 2	1.0%	4.0	3.81	-5.92	-6.11	Revetment	1.96
Section 2	0.5%	4.0	3.81	-5.92	-6.11	Revetment	3.28
Section 3	1.0%	3.5	3.31	-5.50	-5.69	Quay	1.91*
Section 3	0.5%	3.5	3.31	-5.50	-5.69	Quay	1.91*

Source: Mott MacDonald, 2024

* The values are identical as they are rounded to two decimal places.

The advised mean overtopping discharge limits from the EurOtop manual are shown in Table 3.8 and compared against the estimated maximum average overtopping discharge. This table shows that the calculated mean overtopping discharge rates slightly exceed the safety tolerance limits for Section 2 and Section 3 (west and south Sections) with a toe depth greater than 5.5 m OD. Higher overtopping discharge can potentially impact the Terminal building and car parking area during operations

It’s important to note that these mean overtopping discharge values are associated with wave conditions during larger AEP events. It is expected, therefore, that during these extreme conditions, there will be limited port activity, with access restricted to people, operations, and machinery.

Table 3.8: Tolerance limits for wave overtopping

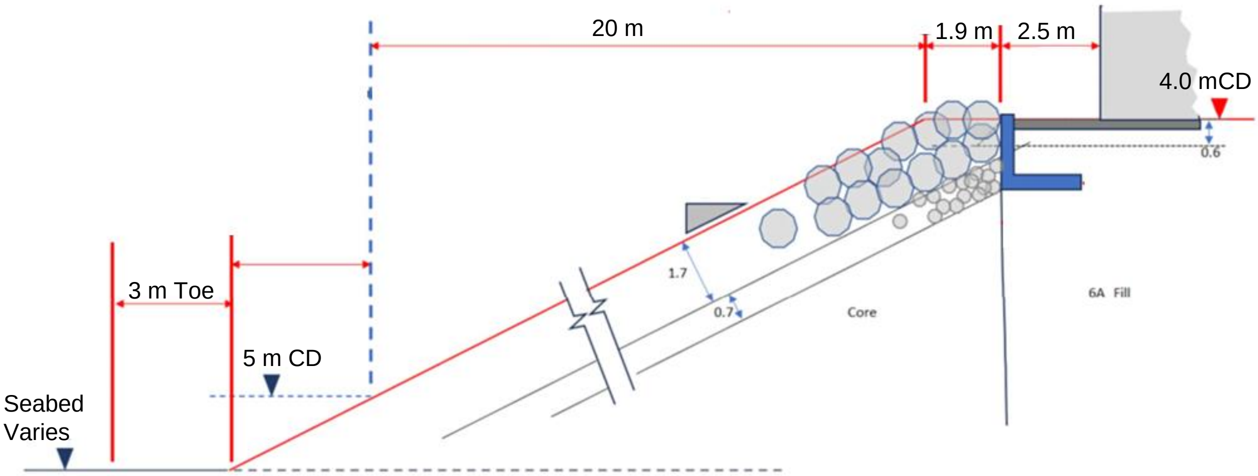
Hazard type and reason	Mean discharge q (l/s per m)
Rubble mound breakwaters; Hs > 5 m; no damage	1
Rubble mound breakwaters; Hs > 5m; rear side designed for wave overtopping	5 to10
Building structure elements; Hs = 1 to 3 m	⩽1
Damage to equipment set back 5 to 10m	⩽1
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping

Source: EurOtop, 2018

3.7.1 Overtopping assessment with wave wall for section 2 development layout 5B

The calculated mean overtopping discharge rates have marginally exceeded the safety tolerance limits (>1 l/s/m), specifically for Section 2 and Section 3 (the west and south sections). Section 2, where the Terminal building and car parking area are planned, is particularly affected. To limit the overtopping discharge rates, a wave wall was incorporated into the design of Section 2 (Figure 3.4). This concept was then used for further optimisation of the berm width and wave wall height to reduce overtopping discharge. Overtopping assessment utilised the AEP wave conditions, as shown in Table 3.2.

Figure 3.4: Updated section 2 cross section with provision for wave wall



Source: Mott MacDonald, 2024

Table 3.9 presents the worst overtopping discharge volumes for the AEP events, considering the dominant wave directions for the updated section 2. The assessment of overtopping began with a wave wall height ranging from 0.5 to 1.5 m. The data on overtopping discharge volumes (0.5% AEP event) indicates that a wave wall height of 1.0 m falls within the tolerance limits for the recommended mean overtopping discharge, as per the EurOtop manual, for the wave conditions projected for Epoch 2085.

The assessment of overtopping discharge was expanded to include a berm width of 2.5 m and a corresponding wave wall height of 0.5 to 1.5 m, to evaluate the reduction in discharge volume. The calculated overtopping discharge volume indicates that the 1 m wave wall reduces predicted mean overtopping volumes to an acceptable limit. It is, therefore, recommended to maintain a wave wall height of 1.0 m to ensure the discharge remains within these tolerances.

Table 3.9: Maximum average overtopping discharge for wave conditions (2085 Epoch) from the dominant wave directions for development section 2 (updated)

Sections	AEP Wave Event	Crest m CD	Crest m OD	Depth at Toe (m CD)	Depth at Toe (m OD)	Berm Width (m)	Wave wall height (m)	Maximum average overtopping Discharge l/s/m
Section 2	1.0%	4.0	3.81	-5.92	-6.11	1.9	0.5	0.45
Section 2	0.5%	4.0	3.81	-5.92	-6.11	1.9	0.5	1.41
Section 2	1.0%	4.0	3.81	-5.92	-6.11	1.9	0.75	0.32
Section 2	0.5%	4.0	3.81	-5.92	-6.11	1.9	0.75	0.99
Section 2	1.0%	4.0	3.81	-5.92	-6.11	1.9	1.0	0.25
Section 2	0.5%	4.0	3.81	-5.92	-6.11	1.9	1.0	0.73
Section 2	1.0%	4.0	3.81	-5.92	-6.11	2.5	1.0	0.23
Section 2	0.5%	4.0	3.81	-5.92	-6.11	2.5	1.0	0.68
Section 2	1.0%	4.0	3.81	-5.92	-6.11	1.9	1.5	0.17
Section 2	0.5%	4.0	3.81	-5.92	-6.11	1.9	1.5	0.44

Source: Mott MacDonald, 2024

References

Artificial Neural Network Tool, http://overtopping.ing.unibo.it/overtopping/

EurOtop, Second Edition 2018, Manual on overtopping of sea defences and related structures, 297 PP

Mott MacDonald, May 2024, Port Ellen terminal development – Dredging and Sediment Dispersion Modelling, PP 99

Mott MacDonald, May 2024, Port Ellen terminal development – Wave Modelling Report, PP 70

Metoffice, November 2018, UKCP18 Marine Report, 133 PP

A.3 SEPA Flood Risk Assessment Checklist

Scotland's 4th National Planning Framework has recently been published. This document is therefore being reviewed and updated to reflect the new policies. You can still find useful and relevant information here but be aware that some parts may be out of date and our responses to planning applications may not match the information set out here.

Flood Risk Assessment (FRA) Checklist

(SS-NFR-F-001 - Version 16 - Last updated 27/08/2019)

This document must be attached within the front cover of any Flood Risk Assessments issued to Local Planning Authorities (LPA) in support of a development proposal which may be at risk of flooding. The document will take only a few minutes to complete and will assist SEPA in reviewing FRAs, when consulted by LPAs. This document should not be a substitute for a FRA.

Development Proposal Summary

Site Name:		Port Ellen Terminal Development		
Grid Reference:	Easting:	136300	Northing:	645000
Local Authority:		Argyll and Bute Council		
Planning Reference number (if known):				
Nature of the development:		Infrastructure	If residential, state type:	
Size of the development site:		1.4	Ha	
Identified Flood Risk:	Source:	Coastal	Source name:	

Land Use Planning

Is any of the site within the functional floodplain? (refer to SPP para 255)	No	N.B. Coastal flooding (not needed for flow or storage) If yes, what is the net loss of storage?	m³
Is the site identified within the local development plan?	Yes	Local Development Plan Name: Local Development Plan 2; Year of Publication: 2024; Allocation Number / Reference: A3002	
If yes, what is the proposed use for the site as identified in the local plan?	Other	If Other please specify: Consideration of options to maintain and further develop ferry services between the mainland and Islay	
Does the local development plan and/or any pre-application advice, identify any flood risk issues with or requirements for the site.	No	If so, please specify:	
What is the proposed land use vulnerability?	Water Compatible	Do the proposals represent an increase in land use vulnerability?	No

Supporting Information

Have clear maps / plans been provided within the FRA (including topographic and flood inundation plans)?	Yes		
Has sufficient supporting information, in line with our Technical Guidance, been provided? For example: site plans, photos, topographic information, structure information and other site specific information.	Yes		
Has a historic flood search been undertaken?	Yes	If flood records in vicinity of the site please provide details:	See FRA
Is a formal flood prevention scheme present?	No	If known, state the standard of protection offered:	
Current / historical site use:	Ferry Terminal/ Coast		
Is the site considered vacant or derelict?	No		

Development Requirements

Freeboard on design water level:		0.75-0.96	m		
Is safe / dry access and egress available?		Neither	Min access/egress level:	2.81	m AOD
Design levels:	Ground level:	3.81	m AOD Min FFL:	0.3	mAOD

Mitigation

Can development be designed to avoid all areas at risk of flooding?	No	The terminal building is raised above flood levels. Other areas of the site are water compatable and resilient to flooding - See FRA
Is mitigation proposed?	Yes	
If yes, is compenstory storage necessary?	No	
Demonstration of compensatory storage on a "like for like" basis?	No	
Should water resistant materials and forms of construction be used?	No	

Flood Risk Assessment (FRA) Checklist

(SS-NFR-F-001 - Version 16 - Last updated 27/08/2019)

Hydrology

Is there a requirement to consider fluvial flooding?	No			
Area of catchment:		km^2	Is a map of catchment area included in FRA?	Select from List
Estimation method(s) used (please select all that apply):	Pooled Analysis ☐		If Pooled analysis have group details been included?	Select from List
	Single Site Analysis ☐			
	Enhanced Single Site ☐			
	ReFH2 ☐			
	FEH RRM ☐			
	Other ☐		If other (please specify methodology used):	
Estimate of 200 year design flood flow:		m^3/s		
Qmed estimate:		m^3/s	Method:	Select from List
Statistical Distribution Selected:	Select from List		Reasons for selection:	

Hydraulics

Hydraulic modelling method:		Select from List		Software used:	Select from List	
				If other please specify:		
Number of cross sections:						
Source of data (i.e. topographic survey, LiDAR etc):				Date obtained / surveyed:		
Modelled reach length:			m			
Any changes to default simulation parameters?				If yes please provide details:		
Model timestep:						
Model grid size:						
Any structures within the modelled length?		Select from List		Specify, if combination:		
Maximum observed velocity:			m/s			
Brief summary of sensitivity tests, and range:						
variation on flow (%)			%	Please specify climate change scenario considered:		
variation on channel roughness (%)			%			
blockage of structure (range of % blocked)			%			
boundary conditions:		**Upstream**			**Downstream**	
(1) type		Flow			Select from List	
	Specify if other			Specify if other:		
(2) does it influence water levels at the site?		Select from List			Select from List	
Has model been calibrated (gauge data / flood records)?		Select from List				
Is the hydraulic model available to SEPA?		Select from List				
Design flood levels:	200 year		m AOD	200 year plus climate change		m AOD
Cross section results provided?		Select from List				
Long section results provided?		Select from List				
Cross section ratings provided?		Select from List				
Tabular output provided (i.e. levels, velocities)?						
Mass balance error:			%			

Coastal

Is there a requirement to consider coastal / tidal flooding?	Yes			
Estimate of 200 year design flood level:	2.22 (+0.45)	m AOD		
Estimation method(s) used:	CFB		If other please specify methodology used:	
Allowance for climate change (m):	0.6	m		
Allowance for wave action etc (m):	See freeboard	m		
Overall design flood level:	2.85 (+0.45)	m AOD		

Comments

Any additional comments:	Lowest level of site at 2.58m at tie in to existing road, this is concurrent with the existing levels for access to and from the existing Port Ellen site. Pedestrian and vehicular access/egress level is 2.81mAOD. Limitations exist on being able to increase this elevation due to it tying in to existing road where other operations exist e.g. access to the grain store. No alternative access/egress routes exist.

Approved by: **L Cload**

Organisation:	Mott MacDonald	
Date:		25/09/2024

Note: Further details and guidance is provided in 'Technical Flood Risk Guidance for Stakeholders' which can be accesssed here:- CLICK HERE

mottmac.com