AECOM

South West St Helier Visioning Framework

Coastal Defence - Concept Design Report

States of Jersey Development Company

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Quality information

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Abbreviations and Acronyms

AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
BGS	British Geological Survey
DTM	Digital Terrain Model
FoS	Factor of Safety
GoJ	Government of Jersey
HAT	Highest Astronomical Tide
JPA	Joint Probability Analysis
MBGL	Metre Below Ground Level
MHWS	Mean High Water Springs
OD	Ordnance Datum
OT	Overtopping
QT	Discharge-Time
RP	Return Period
SMP	Shoreline Management Plan
SoP	Standard of Protection

Non-Technical Summary

Background

The States of Jersey Development Company (JDC) has commissioned AECOM to undertake concept design of the coastal defences which are required to support the South West St Helier development (Figure 1-1) Improved defences are required to provide a minimum 1:200 year Standard of Protection against flooding to the development and adjacent areas behind for the next 50 years. Climate change impacts have also therefore been considered and included in the study.

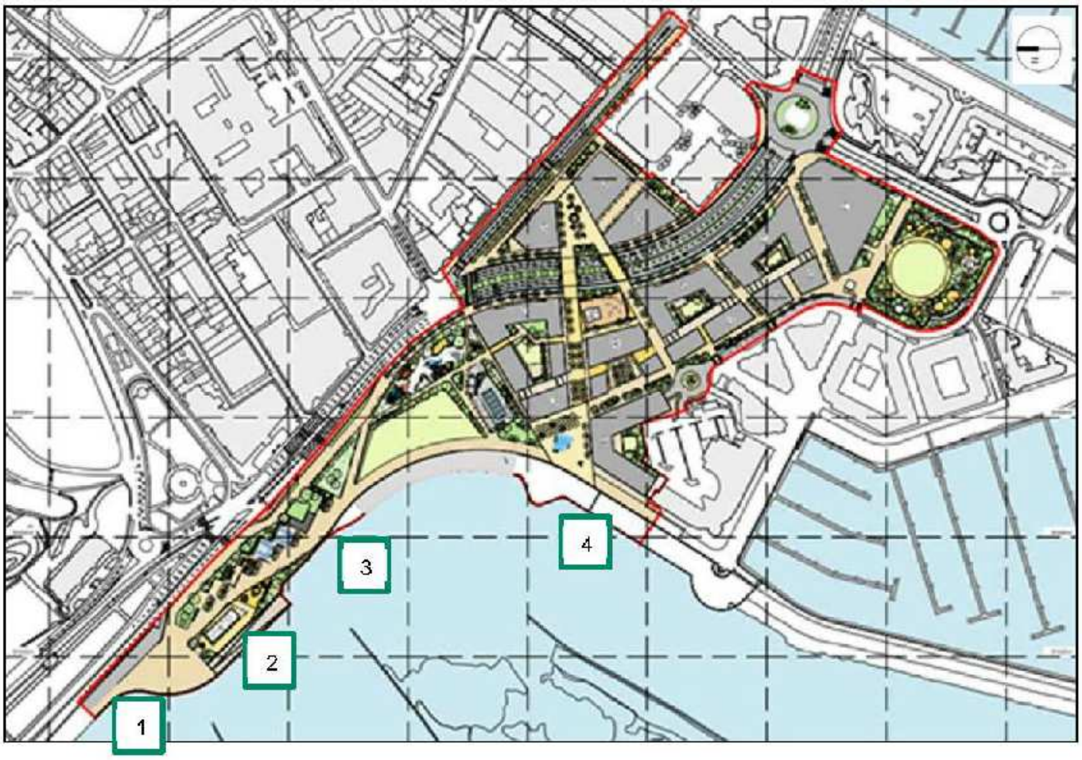

Figure 1-1: Proposed Development

Concept Design Development

The baseline design parameters and supporting data such as waves, water levels, risk and soil properties have been reviewed and calculated to support the design of the required coastal structures to support the design.

The flood risk due to wave overtopping discharges in storm conditions has been estimated using overtopping modelling and a 2D TUFLOW modelling software. The existing conditions and the proposed development conditions have been analysed and the flood maps for different time periods (present, 2070 and 2120) produced both wit hand without the scheme (Appendix A.1). The study shows that with the proposed developed, the flood risk in the study area is reduced to required levels.

In order to provide the required improvement, existing defences would need to be raised considerably (2m+) which would lead to significant constraints and detrimental impacts for the development and existing urban areas behind. To provide the protection, mitigate the adverse impacts, and meet the development requirements, the realignment of defences seawards of the existing line allows more efficient coastal structures to be designed and incorporated to mitigate flood risk without such significant impacts. It will also facilitate space to deliver other key objectives such as sustainable transport and allows more beneficial landscaping.

The shortlisted options for the edge protection structures included:

- mass gravity retaining wall
- conventional cantilever reinforced concrete stem wall,
- and cantilever reinforced concrete stem wall with piled heels.
- rock armour revetment
- secondary raised floodwalls / bunds

Selection of preferred option and concept design of coastal structures

Based on the key assessment and selection criteria which included buildability, overtopping performance, environment and sustainability, wider benefits opportunities and cost, the recommended options for the edge protection structures were chosen and subsequently ratified and agreed via three specific concept design development milestone review meetings held with JDC, and with the asset owners (Department of Infrastructure housing and Environment at the Government of Jersey – "IHE").

The concept design comprises four key coastal structure types to achieve the required protection (see numbered locations on Figure 1-1):

Location 1) Gravity sea wall with a slight angle of reclination.

Location 2) Block work quay wall with a slight angle of reclination.

Location 3) Gravity sea wall with a slight angle of reclination.

Location 4) Raised crest wall

As part of the development, a new slipway, replacing the current one adjacent to the bunker and next to the La Fregate Café is proposed (location 2 on Figure 1-1). Whilst providing flood risk benefits compared to the existing slipway, this will also provide an improved facility for the Duck boats and other users.

The proposed slipway width of 10m is wider than the existing slipway and more than adequate to accommodate a 2-lane traffic. The current ferry operator states the required turning circle is 15m and both at the top / bottom of the ramp and the turning circle provided with the new slipway is more than 20m.

Potential scour and beach lowering due to storms may be experienced at the base of the slipway due to wave concentration and therefore sheet pile structures are proposed in the design to reduce the risk of scour.

1 Introduction

The States of Jersey Development Company (JDC) has commissioned AECOM to undertake concept design of the coastal defences which are required to support the development (Figure 1-1). The study also includes Marine Environmental Impact Assessment support to feed into the overall Environmental Impact Assessment required of the outline Planning Process.

The available site information and an option appraisal study to determine the suitable edge protection structures and slipway structures has been discussed in the Data Review and Optioneering Report (AECOM,2021) and included in Appendix A.1.

This document details the concept design of the required coastal structures and relocated slipway for the western part of the development. The concept design of the raised walls for the terrace blocks and the rock armour sections for the eastern part of the development is provided in the "Coastal Assessment Report" (AECOM 2021) and included in Appendix A.2.

The report presents the findings of the study as follows:

Chapter 2 – Scope and Outputs. A summary of the key tasks of the study.

Chapter 3 – Ground and Groundwater Conditions. Summary of the geological conditions at the study area and assumed geotechnical parameters.

Chapter 4 –Geometry and Concept Design of Blockwork Quay Wall. Analysis approach and proposed geometry of the proposed blockwork quay wall.

Chapter 5 – Structural Stability of the Parapet Wall. Outlines the structural stability of the proposed parapet wall.

Chapter 6 -Concept Design of Slipway. Provides an overview of the slipway dimensions including gradient and base slab details.

Chapter 7 -Inundation Assessment. Provides inundation mapping for 1:200 RP for the present scenario, 2070 and 2120 with the proposed masterplan.

Chapter 8 -Summary. Provides an overview of the study undertaken and the key conclusions derived from the study.

2 Study Scope and Outputs

The overall scope of the study was divided into five key tasks:

1) **Task 1: Data review, optioneering, option appraisal and confirm preferred alignment.** Review of the available Metocean and geological information, review the proposed alignment and carry out an options appraisal for the edge protection structures and slipway structures. The outcomes include a general overview of the data available and constraints on the proposed agreed alignment and the recommended preferred edge protection option for further analysis/design.

2) **Task 2: Overtopping Assessment and Scheme Flood Modelling.** Overtopping assessment carried out (for the present day (2020), 2070, and 2120 1:200yr event) to determine the crest level of the proposed structures for the selected preferred option. The existing flood model was updated to include the proposed agreed alignment and levels for different design events (the same overtopping scenarios). This task informed the requited crest levels of the different coastal edge protection structures along the proposed agreed alignment as well as informing the understanding of the baseline and scheme flood risk due to wave overtopping in the study area.

3) **Task 3: Structural analysis and concept design of coastal defences.** Assessment of structural stability and provide geotechnical concept design for 2no. sections. The stability analysis undertaken for the raised seawalls (2no. sections) in the "Coastal Assessment Report" has also been updated to produce adequate general arrangement and section drawings to support the outline planning application.

4) **Task 4: Concept Design of Slipway.** The slipway slope, width, length and details have been determined/designed to concept level in line with the British Standards (BS)/International Standards. The scour protection options at the toe of the slipway have been recommended and the general arrangement drawings provided along with the concept design sections.

5) **Task 5: Summary Statement.** Non-technical summary outlining the key findings and outcomes of the study developed. This gives a concise summary of the technical work and presents the proposed alignment, rationale for preferred options selected and key design concepts.

This document summarises the concept design of the blockwork quay wall, slipway and details the structural stability of the parapet wall. It also provides the flood risk inundation assessment for the proposed development.

3 Ground and Groundwater Conditions

The geotechnical conditions were derived from the following sources:

- Amplus Ltd (2000) Ground Investigation Report Ref No. 0019
- Gillespies (2021) Visioning framework for the waterfront, southwest St. Helier, Jersey
- AECOM (2019). Coastal Erosion and Beach Analysis Desk Study
- British Geological Survey (1982) Map Sheet 31 Jersey Channel Islands Sheet 2 1:25:000 scale.

The following references have been used:

- Barnes (2015) Concrete Advice No.54 Friction between Materials, Concrete Society.

3.1 Proposed Geotechnical Conditions

Available Ground Investigation (GI) in the area is limited and several assumptions have been made in respect of ground and groundwater conditions.

The British Geological Survey (BGS) Map Sheet 31 ("Jersey (Channel Islands Sheet 2)", 1982, 1:25,000 scale) indicates Superficial and Bedrock strata within the site. The Superficial stratum consists of Alluvium (beach deposits) whilst the Bedrock stratum consists of Granophyre, which is described as an igneous granite rock.

To the northwest of the site there is a boundary between the Granophyre and the Jersey Shale Formation bedrock strata. The Jersey Shale Formation is described as containing sediments of mudstone, siltstone and sandstone.

A previous Ground Investigation (Ref: 0019, October 2000) was carried out by Amplus Ltd in what is now The Waterfront Centre, located approximately 420m to the southeast of the site and included the following works:

- 6No. rotary cored boreholes to a maximum depth of 23mbgl, using dry core drilling methods in the soils and rotary rock coring in the bedrock;
- 10No. trial pits to a maximum depth of 4.1mbgl using a 360-degree track mounted excavator;
- Standard Penetration Tests (SPTs) in the rotary cored boreholes;
- Small disturbed and bulk sampling;
- Installation of 50mm standpipe piezometers in 5 no. rotary cored boreholes; and,
- Geotechnical laboratory testing.

The exploratory holes generally revealed Made Ground overlying Alluvium, overlying Andesite or Granite Bedrock. The GI was undertaken in an area of land reclamation; as such, the Made Ground varied in thickness from 9.1m to 11.0m. The Alluvium generally ranged in thickness between 1m and 3m and was immediately underlain by Andesite or Granite bedrock. The groundwater table was monitored over 5 No. visits and generally ranged in depth between 1.7mbgl and 2.5mbgl.

The Amplus Ltd GI referenced trial pits that were undertaken on the beach during a previous phase of GI works. The beach deposits comprised a variable mixture of soft to firm clay, silt and sand to depths of between approximately 1mbgl and 3mbgl, underlain by bedrock.

Based on the aforementioned information, the proposed site is envisaged to comprise Alluvium in the form of beach deposits to depths between 1mbgl and 3mbgl, underlain by the Granophyre (granite) igneous bedrock.

As a result of climate change and variable tidal conditions, groundwater may no longer be present at the levels previously recorded. Instead the groundwater table is expected to be in hydraulic conductivity with tidal sea levels. A site-specific ground investigation will need to be conducted to confirm the local ground and groundwater conditions.

The assumed geotechnical parameters that were used for concept design purposes are provided below.

Table 3-1: Assumed Characteristic Geotechnical Parameters

Material	Bulk Unit Weight (kN/m³)	Angle of Friction (°)	Undrained Shear Strength (kPa)	Effective Cohesion (kPa)
Beach Material	20	31	-	0
Backfill Material	20	31	-	0
Precast Concrete Block	24	-	-	0

4 Geometry and Concept Design of Blockwork Quay Wall

This chapter details the analysis approach including assumptions made and proposed geometry of the blockwork quay wall.

4.1 Geometry

The following levels and dimensions are proposed for concept design purposes:

- Concrete blockwork wall founding level +0.35m to +0.50mAOD (2mbgl)
- Concrete blockwork wall coping level +9.90mAOD
- Top of fall protection barrier +11.00mAOD
- Thickness of concrete blockwork units 2m
- Base width of concrete blockwork units up to 6.5m
- Crest width of concrete blockwork units 3.6m
- Existing beach level +2.35m to +2.50mAOD
- Finish level of the slipway +2.35m to +9.90mAOD

The blockwork quay wall geometry is shown below, and detailed drawings are provided in Appendix A.3. (Drainage not shown)

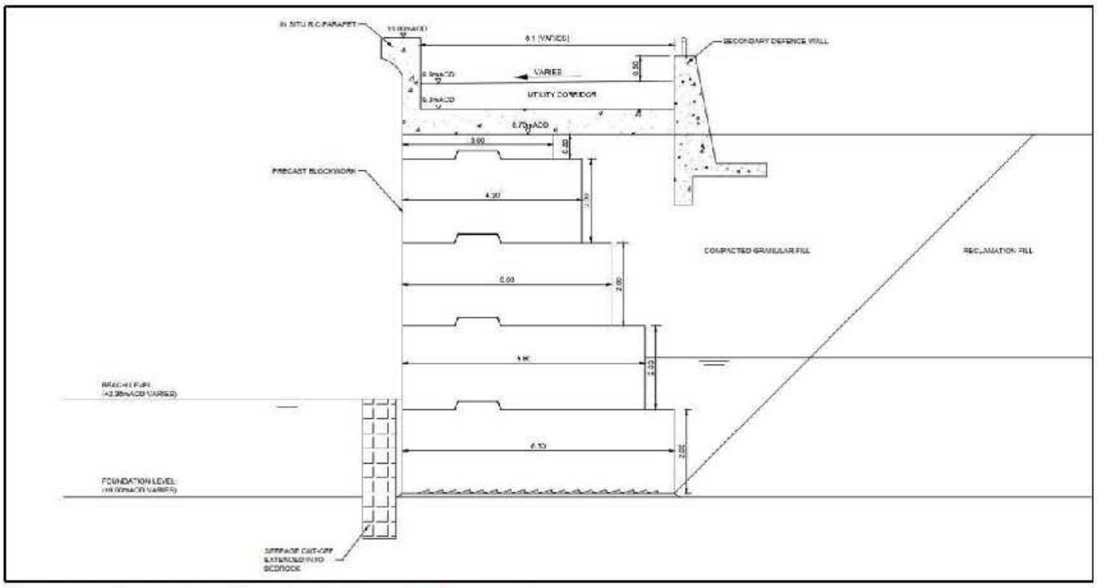

Figure 4-1: Indicative Blockwork Quay Wall Geometry

4.2 Assumptions

The following assumptions were adopted for the concept design:

- Blocks indicated are assumed monolithic.
- Wall is founded on bedrock.

- Groundwater is at ground level on the passive side (+2.35mAOD).
- 1m tidal lag has been assumed to determine groundwater on the active side (+3.35mAOD).
- A minimum 45° wedge of compacted granular backfill for the wall has been assumed.
- A maximum characteristic surcharge of 20kPa on top of wall has been considered.
- A friction coefficient of 0.4 has been assumed for internal checks between blocks.
- Drainage is sufficient to ensure no "rapid drawdown" occurs behind wall (i.e. exceedance of 1m groundwater differential).
- Active pressures have been assumed to be mobilised behind the wall.

4.3 Analysis Approach

Analysis using the limit equilibrium method was undertaken and checks were carried out for general wall sliding and overturning failures, allowing for uplift forces due to the assumed groundwater lag. Further checks were undertaken for internal sliding between the precast concrete blocks. Ultimate Limit State (USL) analyses were carried out using Eurocode 7 Design Approach 1 Combinations 1 & 2 and applying the relevant partial factors to Characteristic Actions and Resistances. Results are provided in the form of Overdesign Factors (ODF) where an ODF>1.0 indicates a pass of the assessment.

A bearing capacity check was deemed to be satisfied by inspection as it is assumed that the wall will be founded on competent bedrock. Load eccentricity was assessed and was found to be acceptable (i.e. resultant force acting within one third of the half-width from the centre of the wall base).

An assessment for global stability (limit equilibrium method for slip surfaces through backfill and beneath the wall) has not been carried out for the concept design stage but it is deemed to be satisfied by inspection in view of the anticipated ground conditions.

4.4 Results

A summary of the check results is provided below:

Table 4-1: Summary of Analysis Results

Design Component	Overdesign Factor (ODF)	
	Sliding Failure	Overturning Failure
External Wall Checks	1.2	2.5
Internal Check (6m below finished level)	1.5	>5
Internal Check (4m below finished level)	2.1	>5
Internal Check (2m below finished level)	3.4	>5

4.5 Conclusions and Recommendations

The proposed wall has passed the checks undertaken for concept design stage, proving the feasibility of the solution from a technical perspective, provided that actual ground and groundwater conditions are no worse than assumed. The block dimensions and toe protection details are shown in Appendix A.3. The following geotechnical recommendations are made in order to facilitate further stages of design and construction:

- A Ground Investigation is proposed to be designed and undertaken in order to inform following stages of design and construction.
- Beach deposit will need to be excavated and the wall foundation constructed on top of competent bedrock. Thus, allowance needs to be made for bulk excavation and reuse. The excess beach material not returned to the beach is expected to be used as backfill material within the proposed reclamation area, with testing undertaken as part of future investigation works to confirm suitability.

- The placement of compacted backfill behind the proposed retaining structure needs to be suitably designed and specified in accordance with industry-recognised standards such as BS 6349- Part 5 and 7. Adequate supervision and compliance testing are required during the construction works.

5 Structural Stability of the Wave Wall

This chapter details the structural stability of the wave wall proposed for the study area. The crest level of the wave wall has been estimated based on the overtopping calculations.

The wave water levels for the study were extracted from AECOM's existing Mike 21 local wave model. Based on an inspection and desktop analysis, it was established that the worst-case wave direction for the site was from a 240° sector and therefore the wave conditions in this sector shall be considered for the overtopping analysis and for the concept design of the edge protection structures.

The significant wave height plots for different scenarios are provided in Figure 5-1, Figure 5-2, and Figure 5-3.

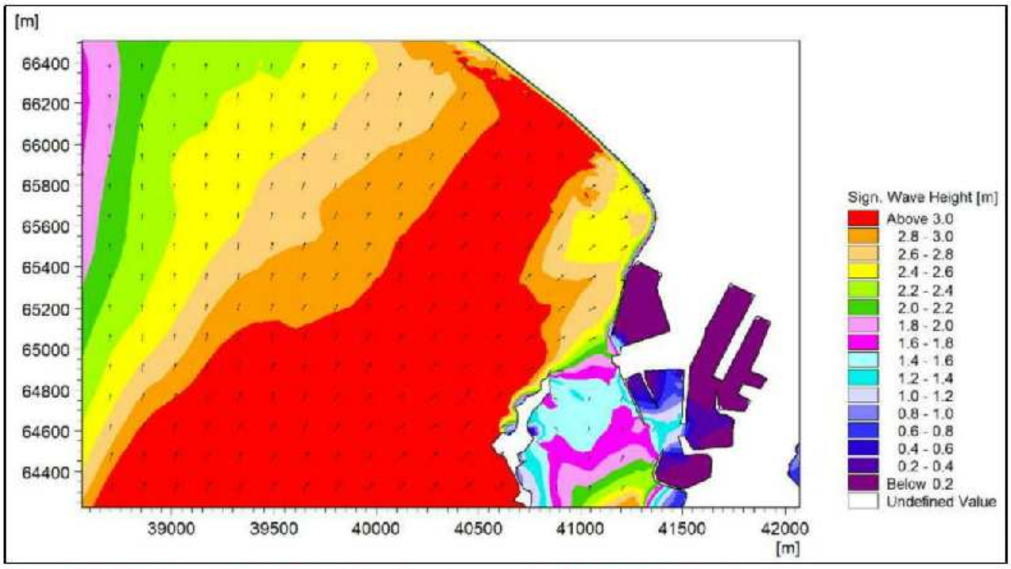

Figure 5-1: Modelled Wave Height for the present day,2020 (240° direction)

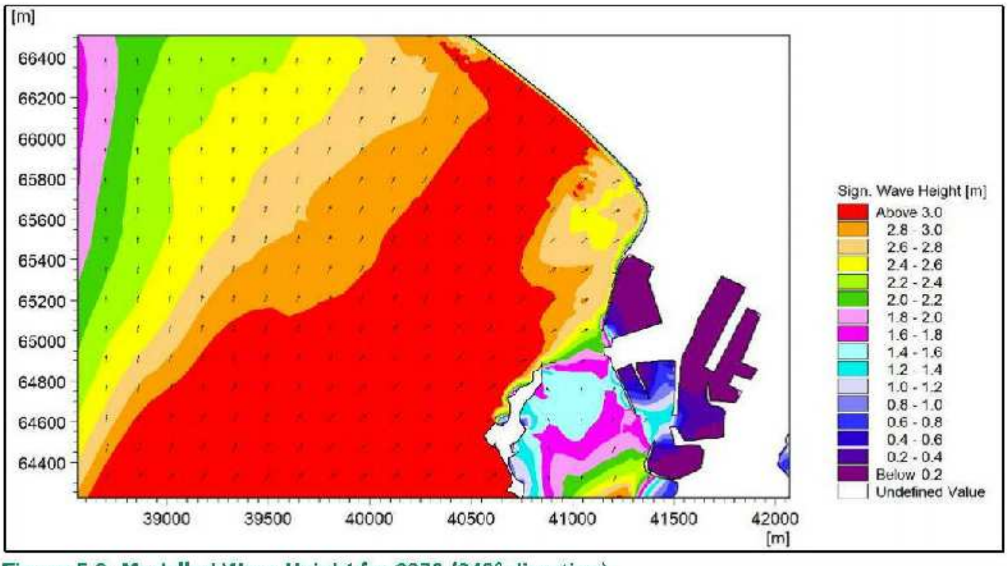

Figure 5-2: Modelled Wave Height for 2070 (240° direction)

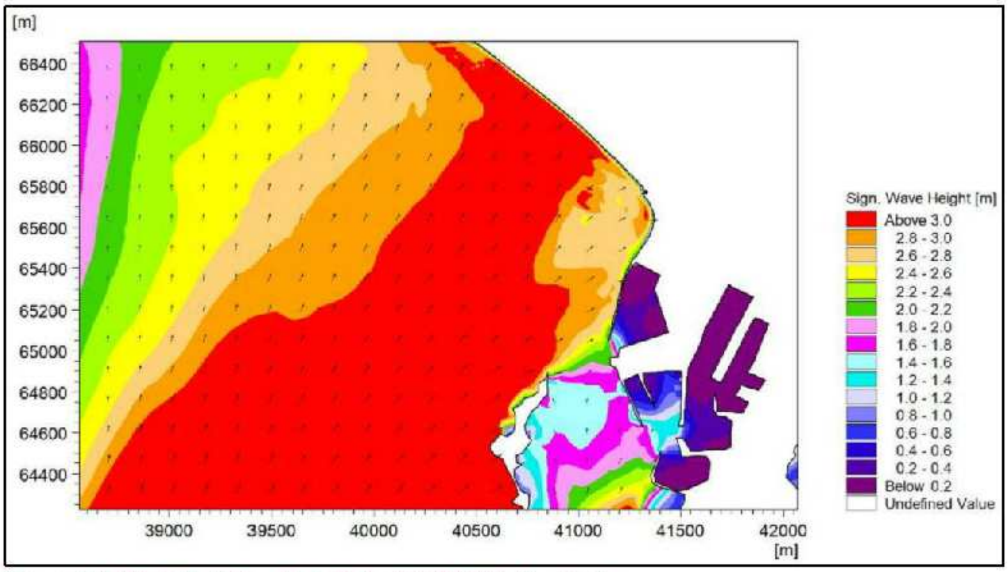

Figure 5-3: Modelled Wave Height for 2120 (240° direction)

The wave-water levels extracted for different agreed design events at a depth of +2.0mAOD at the proposed slipway location (**Error! Reference source not found.**) has been provided below in Table 5-1.

Table 5-1: Hs & SWL Combinations at +2.5mAOD at the Proposed Slipway Location-1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.71	7.9	231.2	7.21	0.72	7.7	231.2	7.67	0.73	7.6	231.2
6.85	1.04	10.6	231.6	7.21	1.04	10.6	230.8	7.67	1.04	10.6	230.7
6.85	1.14	10.8	232.0	7.21	1.14	10.8	230.9	7.67	1.14	10.8	230.8
6.85	1.60	12.3	234.2	7.21	1.59	12.2	232.4	7.67	1.59	12.2	232.1
6.85	1.79	12.9	235.8	7.21	1.78	12.8	233.8	7.67	1.80	12.9	234.3
6.62	2.30	14.3	237.9	6.97	2.34	14.3	238.6	7.42	2.34	14.2	238.4
6.55	2.37	14.6	238.0	6.90	2.42	14.6	239.0	7.35	2.47	14.6	238.1
6.52	2.39	14.6	238.4	6.88	2.44	14.7	239.4	7.33	2.50	14.6	238.1
6.46	2.47	15.2	237.8	6.81	2.53	15.2	239.3	7.26	2.57	15.1	238.5
6.42	2.50	15.5	238.0	6.77	2.58	15.5	239.8	7.21	2.63	15.4	238.8
6.39	2.50	15.7	238.9	6.74	2.59	15.7	240.8	7.18	2.69	15.7	239.0
6.32	2.54	16.1	237.3	6.67	2.59	16.0	240.4	7.11	2.73	16.1	239.2

The extreme waves will be experienced during the relatively short high tide periods when the water is deepest.

The overtopping calculations has been undertaken in accordance to EurOtop (2018) manual. A slope of 20V:1H has been considered and following conditions were provided to accommodate the battered wall condition.

Step 4b (battered walls) Mean overtopping discharges for battered walls under impulsive conditions are observed to be slightly in excess of those for a vertical wall over a wide range of dimensionless freeboards. Multiplying factors are given in Equation 7.11.

10:1 battered wall:	$q_{10:1\,batter} = q_{vertical} \times 1.3$	7.11
5:1 battered wall:	$q_{5:1\,batter} = q_{vertical} \times 1.9$	

where $q_{vertical}$ is arrived at from the procedure for fully-vertical walls under impulsive overtopping conditions. These influences are shown in Figure 7.12. It should be observed that these adjustments are arrived at from a relatively small number of data, and that they represent fairly small changes given the scatter inherent in the underlying prediction methods.

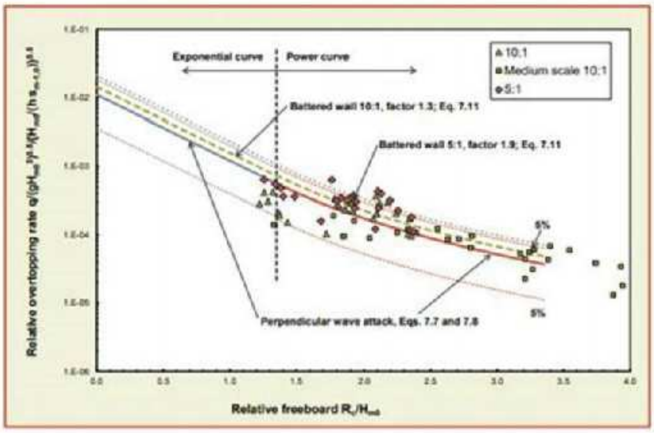

Figure 7.12: The influence of steeply battered (10:1 and 5:1) walls on wave overtopping, for impulsive conditions

Figure 5-4: Overtopping rates for the battered wall condition (Ref: EurOtop Manual,2018)

Since there is no specific guidelines provided for 20:1 battered wall, and also considering the uncertainties in the guidelines provided in Figure 5-4, the overtopping rates estimated has been increased by a factor more than 15% as a conservative approach.

The maximum overtopping rates (l/s/m) are given below:

Table 5-2: Maximum Overtopping Rates (l/s/m)

AEP	RP (years)	2020	2070	2120
0.50%	200	2.50	3.50	30

The proposed wave wall with bullnose crest level estimated is +11.0m AOD and the promenade levels has been estimated based on the landscape requirements. The drainage system needs to be developed at the subsequent design stage for the drainage of the overtopping discharge.

The structural stability of the section representing the wave wall crest level of +11.00m AOD and +8.70m AOD has been assessed as the wall height is 2.3m from the promenade level and is higher when compared to other sections.

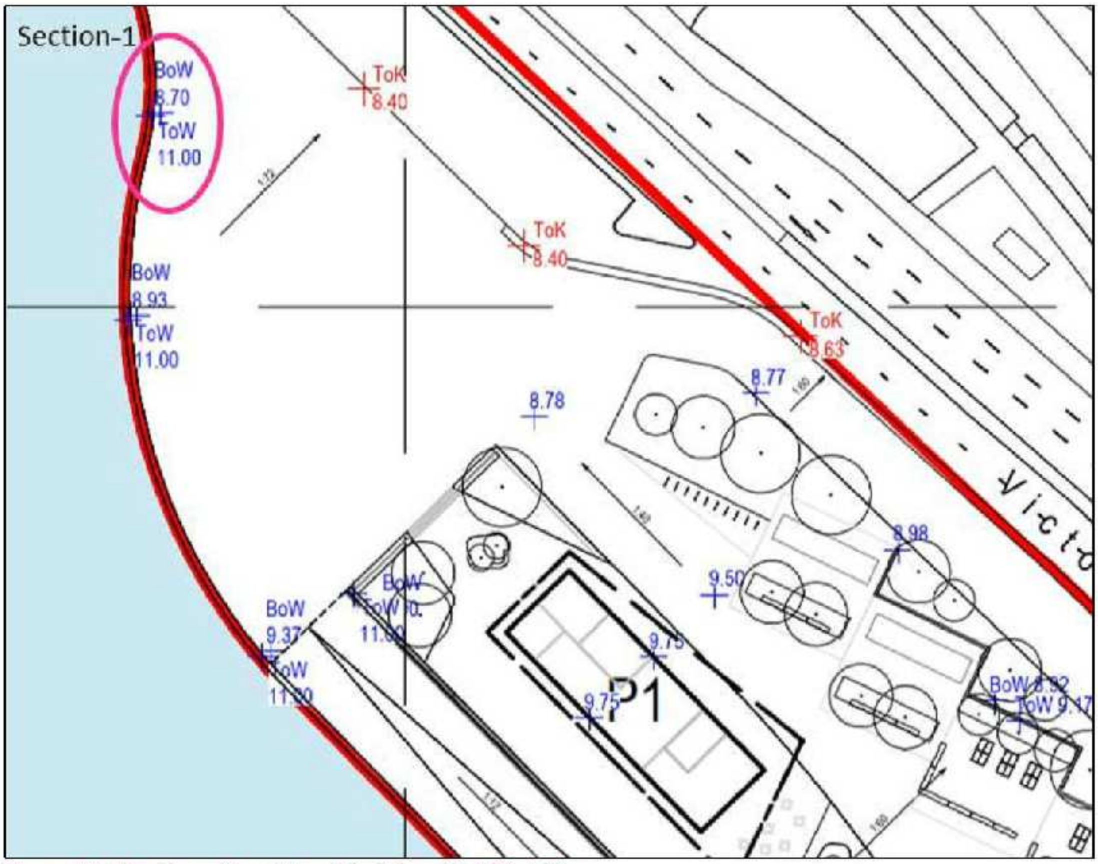

Figure 5-5: Sections Considered for Wave Wall Stability

The design assumptions and basis of design mentioned in the "First Tower to West Park – Coastal Defence Scheme Outline Design Report (2020)" have been used in the present study.

The different design scenarios that have been considered for the study are "Normal Operation" and "Storm" scenario. The normal operation scenario loads are vertical loads, hence not posing any risk of sliding or overturning.

- **Section 1**

The proposed wave wall for this section is provided in Figure 5-6 below:

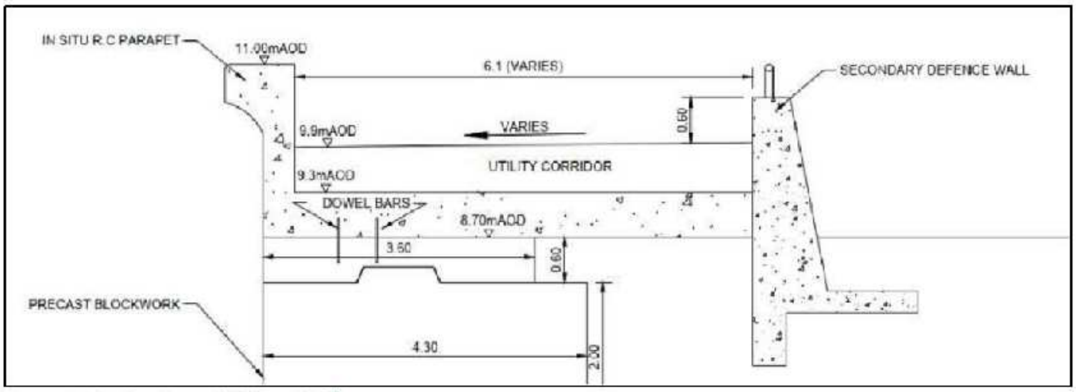

Figure 5-6: Section -1 Wave Wall

Overturning and Sliding

Wave loads, for the storm scenario, as defined in the design approach, may vary significantly along the frontage. Wave loads adopted for the design section are, according to Goda (1974) and a significant wave height (Hs) of 2.59m - AEP 0.5% in 2070 (Table 5-1):

- Overturning moment (unfactored): 178 kNm/m
- Horizontal Load (unfactored): 117 kN/m

The parapet structure remains stable for the given loads for overturning with a FoS greater than 1. To mitigate the risk of sliding, keys or alternatively dowel bars (2 No. of 25mm dia at 500mm c/c spacing) have been proposed. Other potential engineering solutions could also be considered during design, including increasing the promenade thickness against sliding and uplift, as required.

Structural Resilience of the Parapet

This parapet will be subject to shear and bending moment due to wave action, so wave loading according to Goda and a significant wave height (Hs) of 2.59m, results in:

- Shear at the base of the parapet (unfactored): 90 kN/m
- Bending Moment at the base of parapet (unfactored): 116 kNm/m

The parapet remains in the order of a utilisation of below 52% factor for both loads and is considered acceptable.

6 Concept Design of Slipway

It is intended that the new slipway will be located north-west of the existing slipway to form a new facility. The new facility will be replacing the existing slipway and is intended to accommodate similar vessels.

The below figure depicts the proposed agreed alignment, developed by Gillespies to IHE, JDC and key stakeholders brief, which has been adopted for this study and is provided in Figure 6-1.

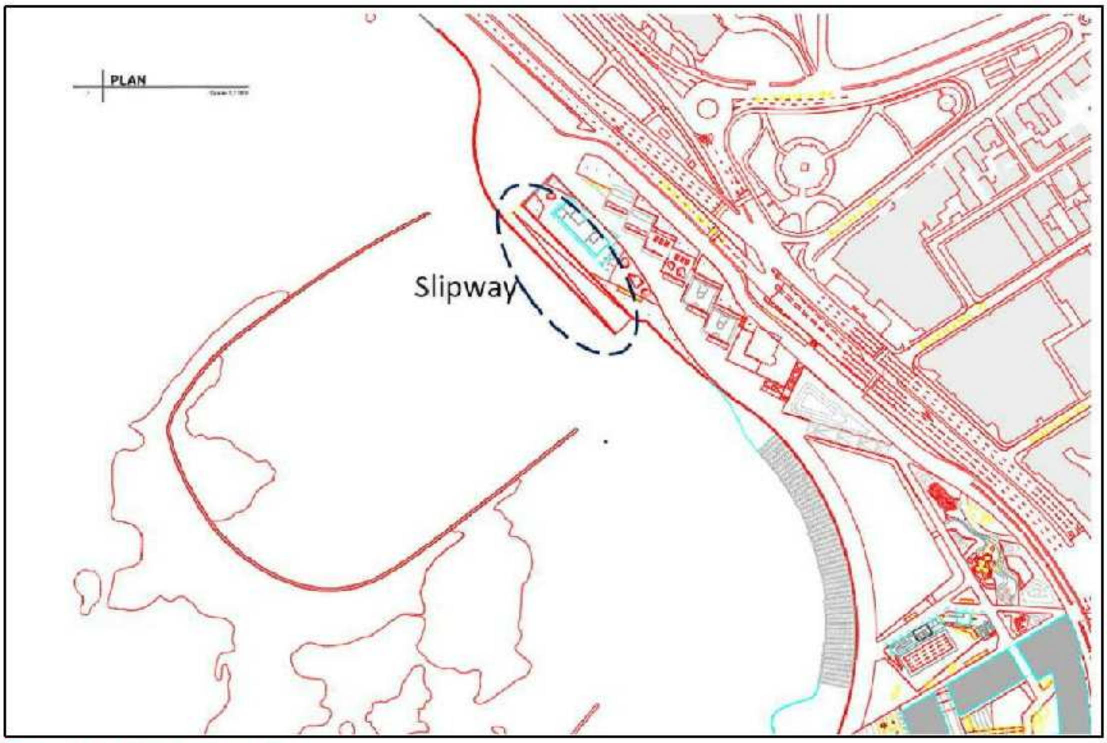

Figure 6-1: Slipway Location

Figure 6-1 indicates the new proposed slipway to swap over from the existing slipway position and to be parallel to the shoreline.

Generally it's advisable to have the new alignment perpendicular to the direction of the main wave attack, but to avoid extensive reclamation, the development proposes the slipway to be parallel to the shoreline. It is important to discuss the new proposed alignment with the current ferry operators and Jersey navigation authority to ensure that this assumption is correct and that all parties agree with the new alignment.

For this report it's assumed the ferry operator to only takes people out during good weather conditions and the ramp is mainly used to drive down to the beach for launching from the foreshore.

It further assumed that parking at the top of the slipway will only be for the Elizabeth Castle amphibious vehicle ferry operator.

6.1 Concept Design

Given the early stage of development, little information is available about the proposed future use of the slipway, thus the concept design is based on the operational needs of the current user, Elizabeth Castle amphibious vehicle ferry operator. The following vessel data was received from the amphibious vehicle touring operation, Table 6-1:

Table 6-1: Vessel Data

Amphibious Vehicle Data	Value
Unladen Weight	8300 kg
Maximum Cross Weight	14000 kg
Overall Width	2.485 m
Overall Length	9.300 m

The new slipway will be created by placing it between two retaining walls and will be from quay side to a sandy rocky beach.

6.1.1 Geometry of New Slipway

As mentioned, it's generally advisable to have the new alignment perpendicular to the direction of the main wave attack, but for concept design purposes it is assumed the proposed alignment is acceptable by current ferry operator and local authorities.

As per Figure 6-1, a slipway width of 10 to 12m with a gradient of 1:12 is proposed for this development. The 1:12 gradient ensures the extend of the foreshore construction at the seaward side will be minimised whilst still maintaining the required slope for launching and recovery of small crafts as well as independent wheelchair users.

The proposed new slipway width of 10 to 12m is greater than the existing slipway (8m) and is more than adequate for a 2-lane towing vehicle slipway if so required.

The preferred turning circle for towing vehicles at crest level is 20m and according to the current ferry operator the amphibious vehicles only require 15m, therefore the proposed master plan layout from Gillespies should be adequate.

At the toe of the slipway a turning circle of 20m needs to be confirmed following the finalisation of the edge protection and scour protection structure. The required space or adequacy should also be discussed with the current operator and navigation authorities for approval.

A tidal hump with adequate transition or alternatively stop-logs should be considered at the crest of the slipway.

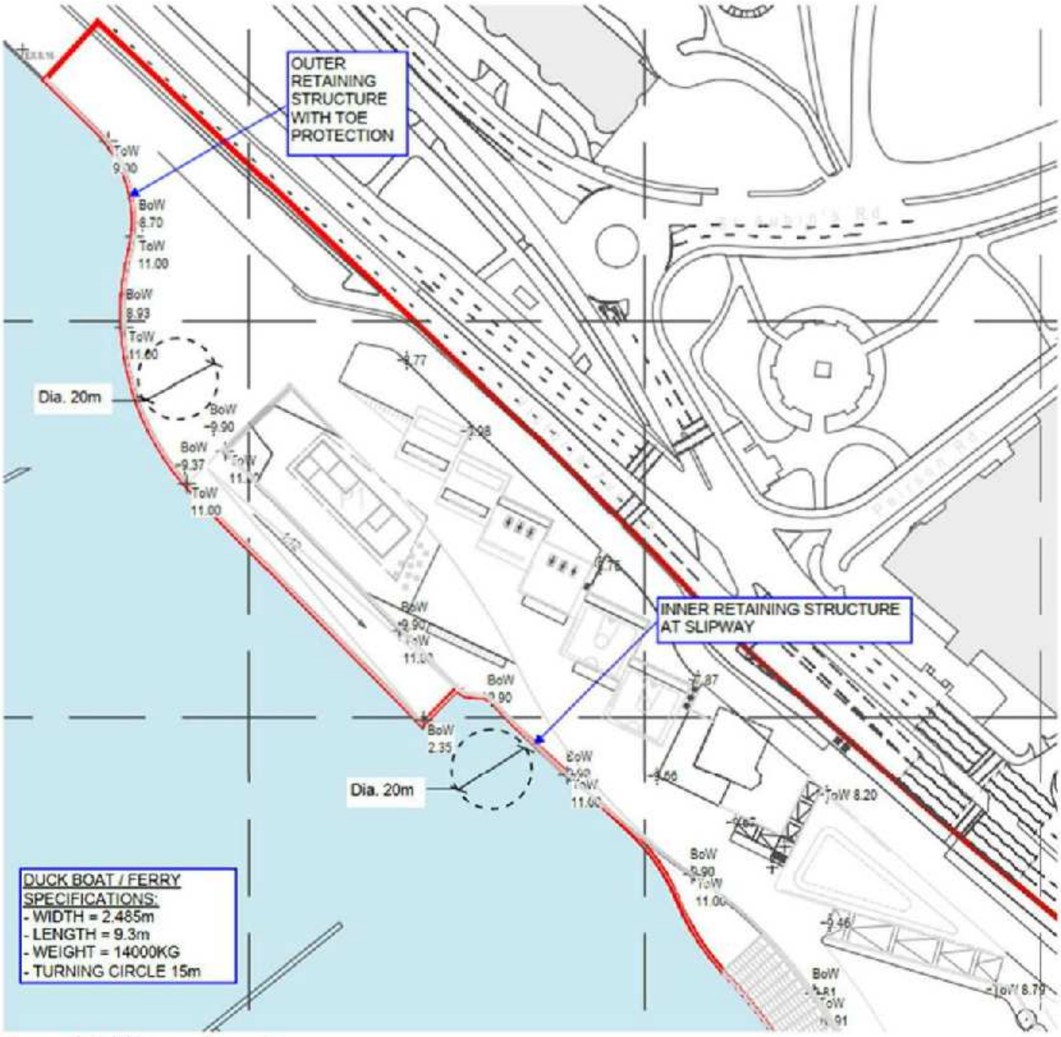

Figure 6-2: Slipway Impacts

Furthermore, the concept design assumes calm sea condition during launching, but if this where to change in the future, impact damping measures fixed to the sides of the vertical retaining structures both sides of the slipway may be considered to prevent damage to small crafts during launching in rougher sea conditions.

6.1.2 Retaining Wall Structures

The construction of the landfill retaining structures (blockwork quay wall) is seen as the primary construction activity with the slipway`s compacted granular sub-base and surfacing constructed between an inner and outer retaining wall.

The inner landfill retaining wall will remain constant as per the landfill design requirement, while the outer retaining wall will follow the slope of the new spillway.

The slipway section is shown below in Figure 6-3 and Appendix A.3.

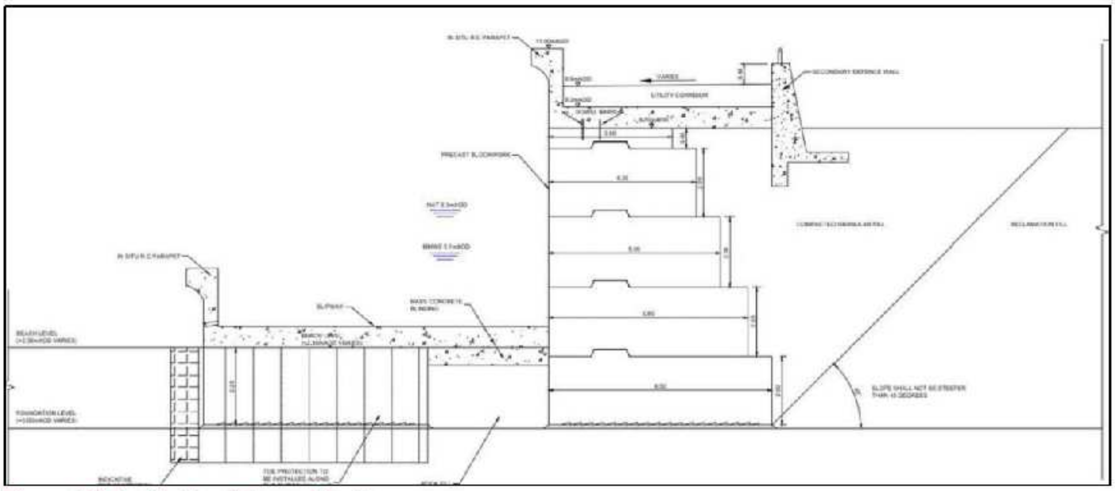

Figure 6-3: Indicative Slipway Section

6.1.3 Surfacing and Layer Works

It is intended that the new slipway will comprise a simple concrete ramp sloping down on to the foreshore. The new slipway will be constructed between the inner and outer retaining wall as mentioned in the previous section.

The new slipway will comprise a constant gradient reinforced concrete slab anchored to prevent the slipway migration towards the sea.

The new parapet wall, on the seaside of the slipway, and the slipway ramps slab will form a monolithic concrete structure. Due to stability requirements for the parapet wall, a 600mm thick reinforced concrete slab is required and will be constructed on top of a bed comprising of rock fill, although this will have to be confirmed when there is more information available on the ground conditions, imposed loadings, design works has been undertaken and construction methods confirmed.

6.1.4 Toe Scour

At this stage, it is envisaged that cut-off wall formed of sheet piles shall be installed along the entire wall and the slipway perimeter. Indicative drawings are provided in Appendix A.3.

6.1.5 Slipway Furniture

Consideration of anchorage points at the top for provision of winches for recovery of small craft should be given by the client/developer. Anchorage points can be added at a later stage if required by the client.

7 Inundation Assessment

7.1 Modelling Approach and Software

TUFLOW modelling software was used to simulate wave overtopping discharge within a 2D model representation of Jersey. This was undertaken TUFLOW version 2020-01-AB. TUFLOW is a two-dimensional (2D) hydraulic modelling software that simulates the hydrodynamic behaviour of water using a grid-based approach. TUFLOW allows hydraulic modelling of surface water flows by applying discharge-time (QT) boundary conditions to the model grid catchment at specific locations. In this instance QT boundaries would be applied along defence locations where overtopping rates have been calculated for a specific joint probability wave event.

The existing site conditions have been modelled and provided in in the "Coastal Assessment Report" (AECOM 2021) and included in Appendix A.2.

7.2 2D Model – Topography

The underlying topographical data is comprised of a composite Digital Terrain Model (DTM) with a 1m grid resolution sourced from the 1m LiDAR supplied by GoJ. The LiDAR survey was undertaken in 2017. The 2D TUFLOW model was set up with a grid resolution of 2m.

7.3 Manning's Roughness Coefficient ('n')

Spatial variations of land cover within the model domain were defined using JsyData_Polygons provided by Gillespies. These data categories were used throughout the model to define appropriate Manning's Roughness Coefficients shown below:

Table 7-1: Manning's 'n' Roughness Coefficients

Surface	'n'
Building	0.3
Roads and Paved Areas	0.025
Grass	0.06
General Surface	0.03

No representation of infiltration of permeable surfaces, groundwater interaction or surface water sewers have been taken into consideration with the overtopping model.

7.4 Overtopping Rates

The overtopping rates are applied to the model as a localised discharge-time boundary. The overtopping rates are calculated in l/s/m for each defence whereas the rates are converted to model compatible units, ready for application within the model in m3/s per grid cell.

For the proposed development, the overtopping rates estimated in Section 5 has been applied. The overtopping rates estimated at the terrace blocks and rock revetment sections at the Radisson Blu Waterfront Hotel as mentioned in "Coastal Assessment Report" (AECOM,2021) have been applied along the respective sections.

7.5 Model Timestep and Simulation Duration

The model timestep was set to 1s, to be half of the model grid size. The peak overtopping for all defences occurs at 3hr into the simulation. The duration of the simulation is 8hr to allow the overtopping water to propagate throughout the model.

7.6 Model Adjustments

The model has been updated from the Overtopping model created for Jersey Shoreline Management Plan. This includes adjustments to the defences; Section 1 has been raised to 11.00mAOD, Section 4 and 5 have been raised to 10.91mAOD. The secondary wall located behind Section 1 defence has been raised by 0.6m. The modified promenade levels and layout have been taken from P12157-00-003-GIL-0101-01-Site Levels and Grading. The materials layer has been updated to reflect the extended promenade.

7.7 Limitations

The model has been updated from the Overtopping model created for Jersey Shoreline Management Plan to look at the defence overtopping inundation for the whole island at a wider scale than the investigation area for this project.

The model is a stand-alone assessment of overtopping inundation at defences for St Helier and does not consider water interactions from any other sources including overtopping inundation from neighbouring defences. No representation of drainage or water egress at defences or water ingress at slipways have been modelled for this project. An overtopping rate was not provided for the sloping slipway defence, so it has been assumed to have a constant defence height of 11.00mAOD and the Overtopping Rate for the adjacent Section 1 has been used. No overtopping rate has been applied to the defence line located behind the slipway.

The levels provided in P12157-00-003-GIL-0101-01-Site Levels and Grading contain spot levels and some gradients. Where gradient or more detailed spot level information has not been provided for an area, the topography has been interpolated between two known points which will affect the accuracy of the flood depth figures. It is recommended to re-simulate the model with more detailed surface elevation data for the modified promenade landscape.

7.8 Results

The crest level of the primary defences has been raised to 11.00mAOD for Section 1 and 10.91mAOD for Section 4 and 5. The secondary defence walls located approximately 18m behind Section 1 are raised by 0.6m.

For the 1 in 200 return period for 2070 and 2020 the low depth flow paths are contained to the promenade, modified park areas behind the defences and adjacent modified building area to the East, all south of the A1.

During a 1 in 200 return period for 2120, a significant portion of the promenade experiences low depth flood water flow paths. The topography of the modified promenade slopes to the east, which is the direction of the primary flow path across the promenade to the adjacent residential area north-east of the A1 for the 1 in 200 return periods events for 2120. The north-west end of the promenade also slopes down to existing ground levels and experiences overtopping inundation flooding.

While the secondary defence does provide a slight amendment to the flow path for all events, the gaps in the wall to allow pedestrians to access the amenities allows the water to pass and flow down walkways.

The existing defence and levels north-west of Section 1 defence and the modified promenade are unchanged in this model, the flooding continues to occur on the defence walkway and some areas of Victoria Avenue for 1 in 200 return period.

The flood maps showing extent and depths are provided in Appendix A.4.

8 Summary

Based on the available met ocean and geotechnical data, option appraisal study has been undertaken (as mentioned in Data Review and Optioneering Report AECOM,2021). Following the discussion with JDC/Government of Jersey IHE, the "Blockwork Quay Wall" is considered as the preferred edge protection option.

Based on the existing geotechnical information available, concept design of the blockwork wall has been developed. Further refinement to the geometry of the structure including toe protection and drainage will be undertaken during the detailed design stages. following the completion of the site-specific GI works.

For a 1 in 200-year return period storm event the overtopping rates have been estimated for different design conditions, i.e. present scenario, 2070 and 2090, at the proposed development area. Based on the estimated overtopping rates, the wave wall crest level of the proposed development is set as +11.00m AOD.

The structural stability of the parapet wall has been assessed for the critical condition where the parapet wall height is 2.3m from the promenade level. In order to reduce the risk of sliding, stainless steel dowel bars (2 No. of 25mm dia at 500mm c/c spacing) have been proposed. Further refinement of the geometry of the parapet wall could be undertaken in the subsequent detailed design stage, if required.

The new slipway will be constructed between the inner and outer retaining wall as mentioned in the previous section and comprised of a constant gradient reinforced concrete slab anchored to prevent the slipway migration towards the sea.

The new parapet wall, on the seaside of the slipway, and the slipway ramps slab will form a monolithic concrete structure. At this stage, it is anticipated that a 600mm thick reinforced concrete slab is required, however, needs to be confirmed following the completion of the site-specific GI works and the confirmation of the construction method.

Inundation Modelling has been undertaken using the existing flood model, however by updating the flood model to include the proposed development for a 1 in 200 return period events for 2020,2070 and 2120. The flood depths and extent are provided in Appendix A4. The coastal structures proposed provide the development and land behind with the required level of flood risk protection for the next 50 years, with only shallow and localised residual wave overtopping flood risk to the promenade areas under the most extreme events which can be managed through drainage and signage.

9 References

AECOM (2021). Data Review and Optioneering Report

AECOM (2021). Coastal Assessment Report

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HR Wallingford (1991), Jersey Coastal Management Study. HR Wallingford Report EX2490, December 1991.

A.1 Data Review and Optioneering Report

A.2 Coastal Assessment Report

A.3 Drawings

A.4 Inundation Maps (Future Development Scenario)

South West St Helier Waterfront Development

Data Review and Optioneering Report

States of Jersey Development Company

Project number: 60650295

14 June 2021

DRAFT

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Quality information

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Abbreviations and Acronyms

AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
BGS	British Geological Survey
DTM	Digital Terrain Model
FoS	Factor of Safety
GoJ	Government of Jersey
HAT	Highest Astronomical Tide
JPA	Joint Probability Analysis
MBGL	Metre Below Ground Level
MHWS	Mean High Water Springs
OD	Ordnance Datum
OT	Overtopping
QT	Discharge-Time
RP	Return Period
SMP	Shoreline Management Plan
SoP	Standard of Protection

1 Introduction

1.1 Overview

The Jersey Development Company (JDC) has commissioned AECOM to undertake concept design of the coastal defences which are required to support the development. The study also includes Marine Environmental Impact Assessment support to feed into the overall Environmental Impact Assessment required of the outline Planning Process.

As part of the main scope, this document reviews the available site information and proposed alignment and provides an option appraisal study to determine suitable edge protection structures and slipway structures.

The below figure depicts the proposed agreed alignment, developed by Gillespies, which has been adopted for this study and is provided in Appendix A.

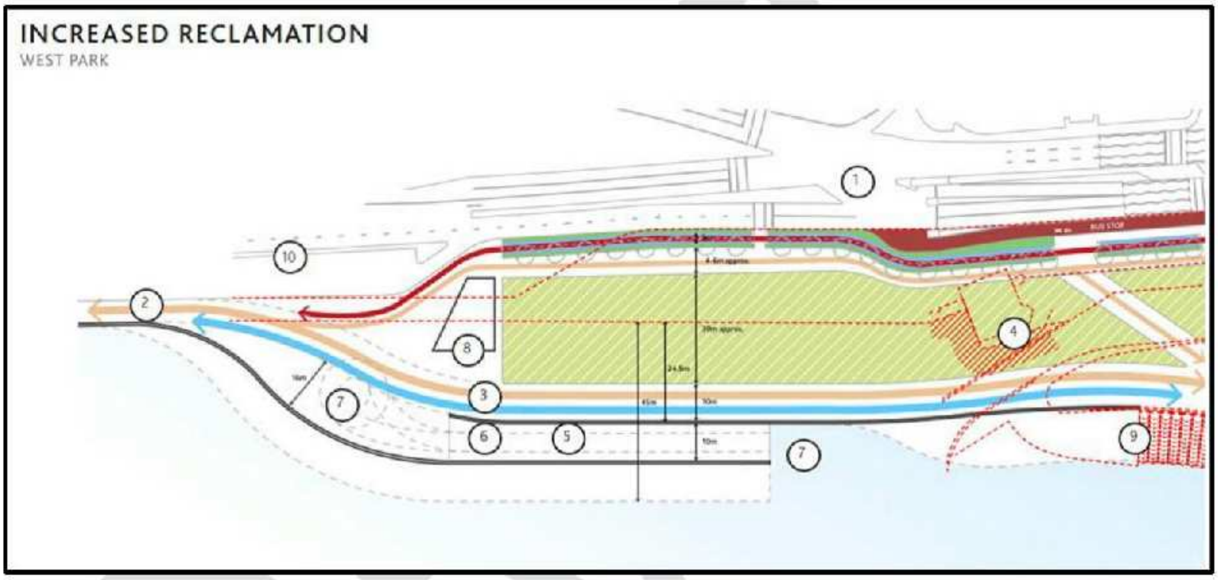

Figure 1-1: Proposed Agreed Alignment

The report presents the findings of the study as follows:

Chapter 2 – Scope and Outputs. A summary of the key tasks of the study.

Chapter 3 – Baseline Review. Summary of data reviewed and general Metocean and Geological conditions at the study area.

Chapter 4 – Review of the Proposed alignment. Evaluation of proposed alignment with respect to coastal process and hydrodynamics, slipway impacts, duck boat requirements/turning circles and other anticipated environmental constraints.

Chapter 5 – Option Appraisal Study. Outlines the pros and cons of the different coastal protection structures (edge protection and slipway structures) in terms of buildability, likely overtopping performance, sustainability, wider benefits opportunities, and cost.

Chapter 6 -Summary. Provides an overview of the study undertaken and the key conclusions derived from the study.

2 Study Scope and Outputs

The overall scope of the study has been divided into five key tasks:

1) **Task 1: Data review, optioneering, option appraisal and confirm preferred alignment.** Undertake a review the available Metocean and geological information, review the proposed alignment and carry out an option appraisal for the edge protection structures and slipway structures. The outcomes include a general overview of the data available and constraints on the proposed agreed alignment and the recommended preferred edge protection option for further analysis/design.
2) **Task 2: Overtopping Assessment and Scheme Flood Modelling.** To undertake an overtopping assessment (for the present day (2020), 2070, and 2120 1:200yr event) to determine the crest level of the proposed structures for the selected preferred option. The existing flood model shall be updated to include the proposed agreed alignment and levels for different design events (same as OT scenarios). The study outcome shall be the crest level of the coastal edge protection structures along the proposed agreed alignment and an understanding of the flood extent due to wave overtopping in the study area.
3) **Task 3: Structural analysis and concept design of coastal defences.** Assess structural stability and undertake geotechnical concept design for 2 no. sections. The stability analysis undertaken for the raised seawalls (2 no. sections) in the "Coastal Assessment Report" shall be updated to produce adequate general arrangement and section drawings to support the outline planning application.
4) **Task 4: Concept Design of Slipway.** The slipway slope, width, length and details shall be determined/designed to concept level in line with the BS/International standards. The scour protection options at the toe of the slipway shall be considered for this study and the general arrangement drawings shall be provided along with concept design deriving the geometry.
5) **Task 5: Summary Statement.** A non-technical summary document outlining the key findings and outcomes of the study shall be developed to feed the wider Stakeholder Engagement activities. This will provide a plain English concise summary of the technical work and will present the proposed alignment, rationale for preferred options selected and details of defence heights, design concepts etc.

The document summarises the optioneering study and confirmed preferred option / alignment and tie-in details (Task 1) for further discussion with JDC, GIL and IHE to select and confirm the preferred edge protection / slipway structures for further design development up to a concept stage.

3 Baseline Review

The historic data received and the information from the previous studies in the study area have been reviewed and the key findings are provided in below sections. In addition, other online sources available has also been checked to extract the baseline site information.

The documents that were reviewed are as follows:

- AECOM. Coastal Assessment report (2021)
- AECOM. First Tower to West Park -Coastal Defence Scheme Outline Design Report (2020)
- AECOM. Jersey Shoreline Climate Resilience Management Plan (2019)
- AECOM. Coastal Erosion and Beach Analysis Desk Study (2019)
- Cundall Johnson & Partners Amplus Ltd. Waterfront Development, St Helier, Jersey- Ground Investigation Report-Factual (2000)
- British Geological Survey (BGS) Map. http://www.largeimages.bgs.ac.uk/iip/mapsportal.html?id=1003934

3.1 Metocean Conditions

The bathymetry contours at the study area is shown in Figure 3-1 and the seabed levels where the anticipated footprint of the proposed agreed alignment shall be in the order of +2.5 to +3.0mAOD and the levels shall be confirmed once the Masterplan is finalised.

Figure 3-1: Bathymetry contour lines at site (All levels are in mAOD)

The wave water levels for the study were extracted from AECOM's existing Mike 21 local wave model. Based on an inspection and desktop analysis, it was established that the worst-case wave direction for the site was from a 240° sector and therefore the wave conditions in this sector shall be considered for the overtopping analysis and for the concept design of the edge protection structures.

The significant wave height plots for different scenarios are provided in Figure 3-2, Figure 3-3, and Figure 3-4.

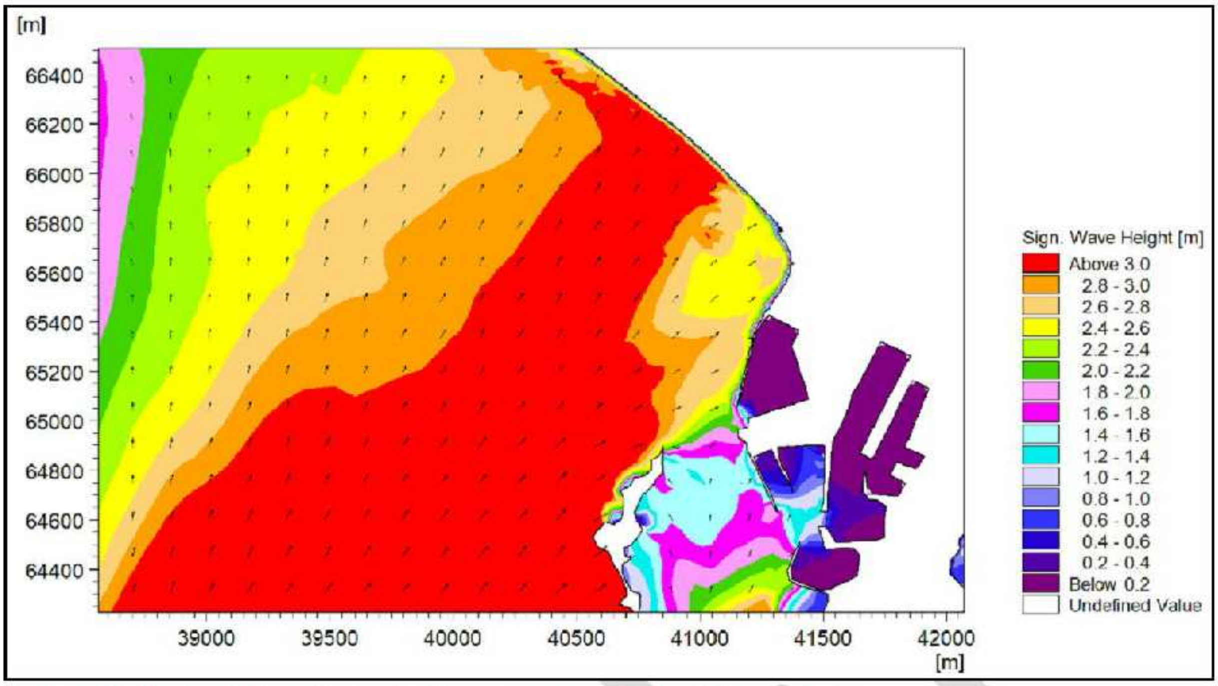

Figure 3-2: Modelled Wave Height for the present day,2020 (240° direction)

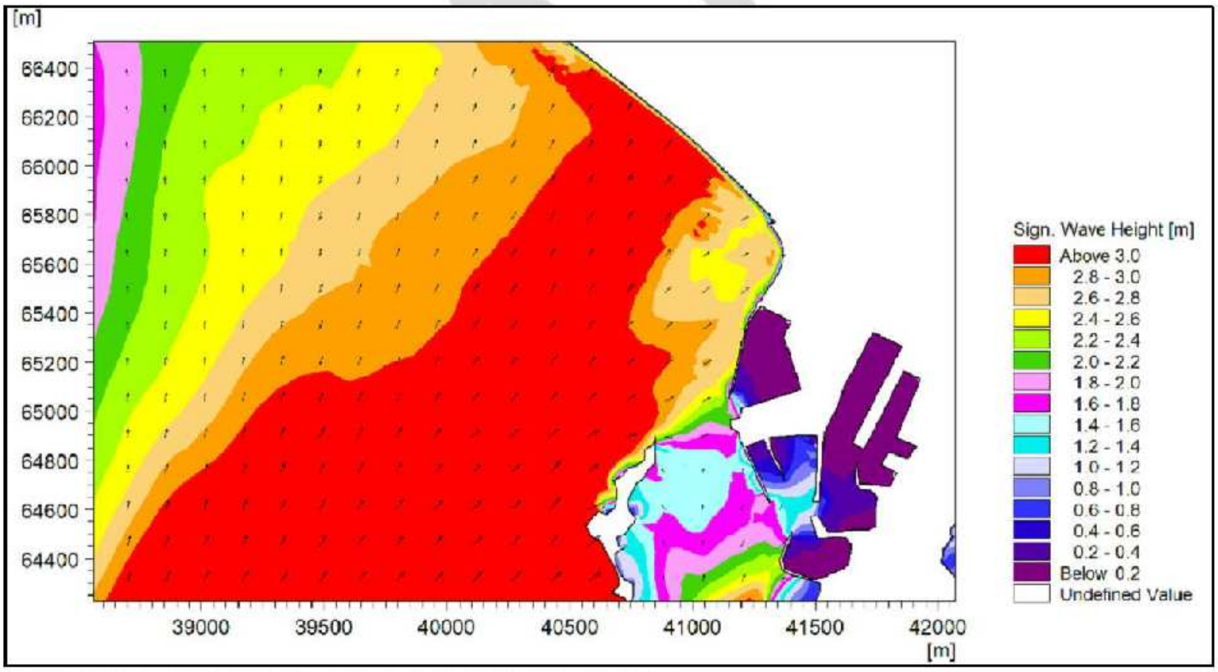

Figure 3-3: Modelled Wave Height for 2070 (240° direction)

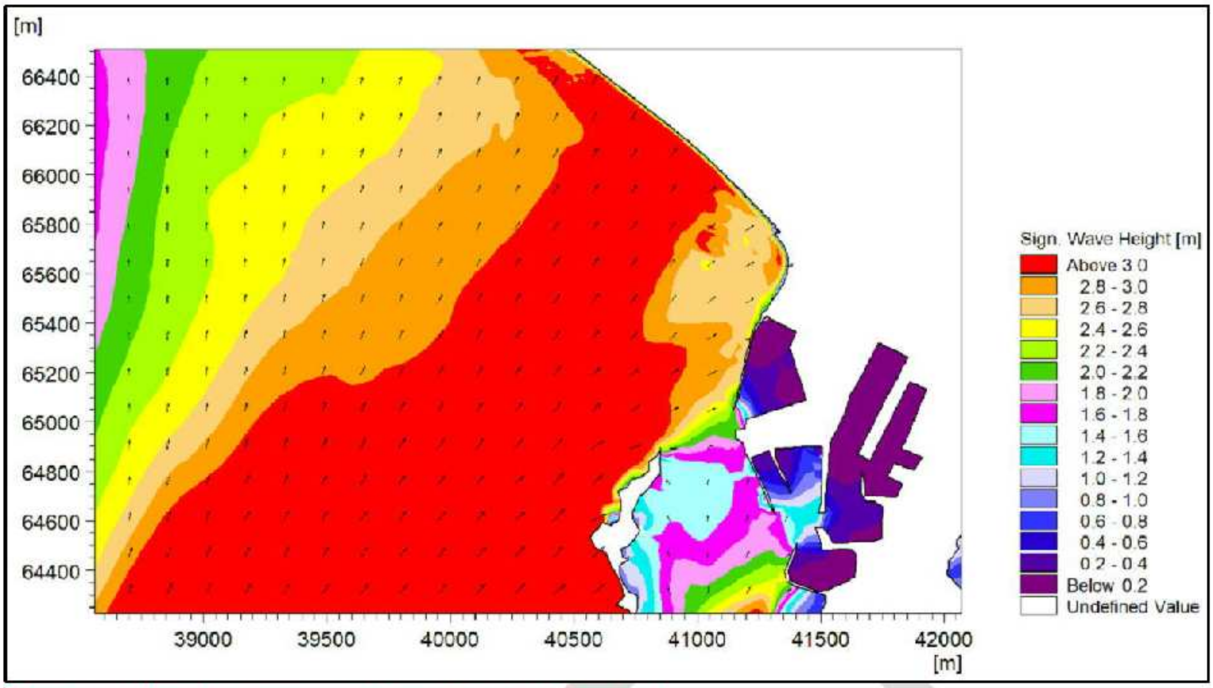

Figure 3-4: Modelled Wave Height for 2120 (240° direction)

The wave-water levels extracted for different agreed design events at a depth of +2.5mAOD at the proposed slipway location (Figure 1-1) has been provided below in Table 3-1. Once the Masterplan is finalised, the cross-sections for the overtopping and design of the edge protection structures shall be identified and the wave-water data for the different design scenarios shall be updated.

Table 3-1: Hs & SWL Combinations at +2.5mAOD at the Proposed Slipway Location-1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.72	8.0	232.4	7.21	0.73	7.8	232.2	7.67	0.74	7.7	232.0
6.85	1.06	10.7	233.3	7.21	1.06	10.6	231.8	7.67	1.05	10.6	232.0
6.85	1.17	10.9	233.6	7.21	1.17	10.9	231.7	7.67	1.16	10.8	232.7
6.85	1.64	12.3	236.0	7.21	1.63	12.2	233.4	7.67	1.61	12.1	234.0
6.85	1.84	12.9	237.7	7.21	1.83	12.9	235.1	7.67	1.81	12.8	235.4
6.62	2.30	14.3	239.6	6.97	2.36	14.3	240.6	7.42	2.42	14.2	239.5
6.55	2.37	14.6	238.9	6.90	2.43	14.6	240.9	7.35	2.50	14.5	240.1
6.52	2.35	14.6	239.3	6.88	2.48	14.7	240.4	7.33	2.52	14.6	240.4
6.46	2.40	15.1	239.1	6.81	2.51	15.1	241.4	7.26	2.58	15.1	240.6
6.42	2.42	15.4	239.8	6.77	2.52	15.4	241.1	7.21	2.62	15.4	241.1
6.39	2.41	15.6	240.0	6.74	2.57	15.7	242.4	7.18	2.64	15.7	240.9
6.32	2.41	16.0	239.7	6.67	2.52	16.0	240.8	7.11	2.71	16.1	240.5

3.2 General Geotechnical Conditions

3.2.1 Beach Survey Data

AECOM previously produced a Coastal Erosion and Beach Analysis Desk Study (April 2019) summarising beach profiles across Jersey between the years 2003 and 2017. The Desk Study included the proposed site and indicated that the beach level is approximately +2.0m AOD, with "marginal accretion" of beach deposits.

3.2.2 Geological Maps & Beach Monitoring Profiles

The British Geological Survey (BGS) Map Sheet 31 ("Jersey (Channel Islands Sheet 2)", 1982, 1:25,000 scale) indicates Superficial and Bedrock strata within the site. The Superficial stratum consists of Alluvium (beach deposits), and the Bedrock stratum consists of Granophyre which is described as an igneous granite rock.

To the northwest of the site is a boundary between the Granophyre and the Jersey Shale Formation bedrock strata. The Jersey Shale Formation is described as containing sediments of mudstone, siltstone and sandstone.

3.2.3 Existing Ground Investigations

A previous Ground Investigation (Ref: 0019, October 2000) was carried out by Amplus Ltd in what is now The Waterfront Centre located approximately 400m to the southeast of the site and included the following works:

- 6 no. rotary cored boreholes to a maximum depth of 23mbgl using dry core drilling methods in the soils and rotary rock coring in the bedrock;
- 10 no. trial pits to a maximum depth of 4.1mbgl using a 360-degree track mounted excavator;
- Standard Penetration Tests (SPTs) in the rotary cored boreholes;
- Small disturbed and bulk sampling;
- Installation of 50mm standpipe piezometers in 5 no. rotary cored boreholes; and,
- Geotechnical laboratory testing.

The exploratory holes generally revealed Made Ground overlying Alluvium, overlying Andesite or Granite Bedrock. The Ground Investigation was undertaken in an area of land reclamation; as such, the Made Ground varied in thickness from 9.1m to 11m. The Alluvium generally ranged in thickness between 1m and 3m and was immediately underlain by Andesite or Granite bedrock. The groundwater table was monitored over 5 visits and generally ranged in depth between 1.7mbgl[1] and 2.5mbgl.

The Amplus Ltd Ground Investigations referenced trial pits that were undertaken on the beach during a previous phase of ground investigation works. The beach deposits comprised a variable mixture of soft to firm clay, silt and sand to depths of between approximately 1mbgl and 3mbgl, underlain by bedrock.

3.2.4 Preliminary Ground Profile

Based on information obtained from BGS Map Sheet 31 and the Amplus Ltd Ground Investigation, the proposed site is envisaged to comprise Alluvium in the form of beach deposits to depths between 1mbgl and 3mbgl, underlain by Granophyre (granite) igneous bedrock. A shallow groundwater table between 1.7mbgl and 2.5mbgl is anticipated. The groundwater table is expected to be in hydraulic conductivity with tidal sea levels.

[1] Metre below ground level

4 Review of the Proposed agreed alignment

The proposed alignment shown in Figure 4-1 has been reviewed against the different engineering/environmental aspects and are detailed in the below sections for further discussion with JDC, GIL and IHE.

4.1 Metocean Constraints

Based on the inspection and desktop analysis on the wave conditions at the study area using AECOM's existing Mike 21 local wave model, it has been observed that the critical wave direction is from 240°N and therefore with reference to the proposed agreed alignment orientation, the critical waves acts perpendicular to the proposed edge protection and slipway structures. Therefore, the proposed edge protection types should be designed in such a way that the structural performance (buildability and stability) and the overtopping performance are enough to maintain the structure for its intended use and to minimise flooding behind the study area.

The key Metocean constraints that are anticipated at this stage are as follows:

1) Potential Scour at the base of the slipway toe due to wave concentration: Adequate scour protection is required to limit the scouring of the slipway toe, thereby affects the structure stability and localised beach erosion. In addition, periodic monitoring and inspections are suggested to record any localised erosion and subsequent possible accretion in the nearby areas due to the sediment movement and to ensure that the slipway operations (functional and structural) are not compromised.

2) Adequate toe protection should also be provided to the edge protection structures to minimise the beach erosion and this is quite common for vertical, or near vertical, wall structures.

3) The type of structure and geometry (crest level) should be provided in such a way that the wave overtopping is minimal and at the same the crest level should not hinder the sea view.

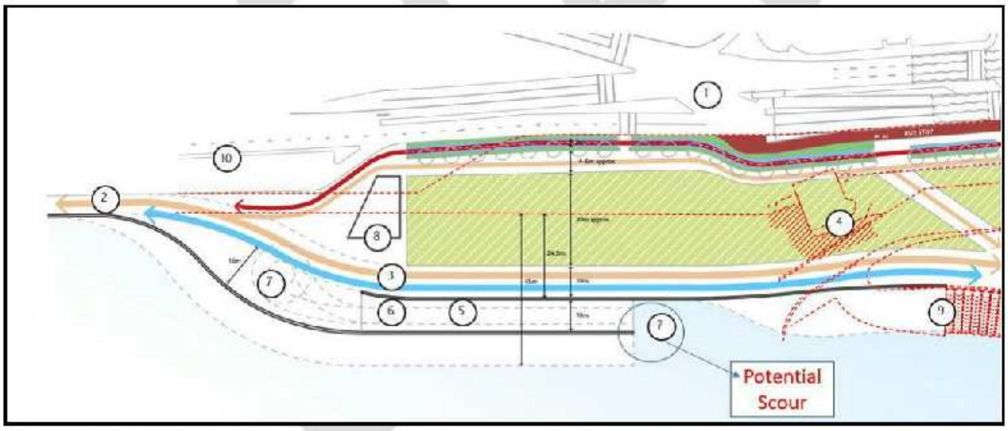

Figure 4-1 : Potential Scouring at the Slipway Toe.

4.2 General Engineering Constraints

This section outlines the geotechnical, construction and other tie-in constraints to be considered within the study.

Any solutions for retaining and supporting the proposed alignment shall be based one of on the following options:

1. Mass gravity wall solution in which the wall gains resistance to sliding and overturning through the bulk weight of the structure and friction on its underside;

2. Cantilever reinforced-concrete stem wall solution in which the wall gains resistance to sliding and overturning through the weight of backfilled material placed behind the stem on the wall heel;

3. Earthworks solution in which select fill is utilised to form a compacted embankment, with rock armour or similar placed along the side slopes to withstand wave action. A suitable filter system would need to be incorporated between the compacted fill and rock armour to prevent the washing out of fines due to wave action; and,
4. A piled upstand solution, such as a cantilever reinforced-concrete stem wall with piled heel. A piled upstand solution would operate in a manner similar to Option (2) above but require a smaller foundation area due to the resistance generated by the piles.

Each of the above options may be implemented in various shapes and forms which are discussed in more detail in Section 5. It is likely that a combination of the above options will be required as part of the overall scheme due to local design constraints and to tie the proposed structure to the existing sea defences. It is likely that some form of mass gravity wall solution or a piled solution (Options 1 and 4) will be required at the tie-ins of the proposed structure to the existing sea defences, where there is not enough space to form earthworks or a cantilever stem wall (Options 2 and 3).

In addition to the above, the following geotechnical constraints are highlighted as part of the proposed alignment:

- Land reclamation and foundation and wall construction will require management of the tidal water. This may require the construction of a cofferdam or controlling construction to work around the tide level. Cofferdam construction may present difficulties due to the hard granite bedrock at relatively shallow depths. Dewatering and waterproofing measures shall be required to ensure dry construction across the proposed alignment footprint.
- Alluvial beach deposits will need to be removed and replaced to construct the wall foundation on the competent bedrock. Thus, allowance will need to be made for bulk removal and disposal (or temporary storage) of material during construction. Reuse of Alluvial beach deposits as a select backfill material within the proposed reclamation area is possible, provided a suitable testing regime is undertaken as part of future investigation works.
- The construction of compacted backfill behind the proposed retaining structure will need to be suitably designed and specified in accordance with industry-recognised standards. Adequate supervision and compliance testing will be required during the construction works.
- A drainage strategy will need to be developed to adequately manage high tidal levels (including during storms), groundwater and overtopping actions. Any drainage strategy will need to be suitably integrated with the retaining wall geotechnical design.

4.3 Slipway considerations

Boat launching ramps or slipways are generally provided for the launching and recovery of hire boats, transient craft, dinghies and for public boat access to the waterway it services. Boat launching ramps should be designed to suit the type and size of boats that will be using them. Signs should be provided to indicate any loading limit for vehicles using the ramp.

4.3.1 Existing slipway

The existing slipway comprises a 250mm thick layer of cobblestone placed on top of a 300mm thick concrete ground slab which abuts a higher-level granite talus groyne on its seaside with a parapet wall. Granite talus with a coping and balustrading has been provided on the leeside.

The slipway is mainly used by small amphibious vehicles / ferries which run to and from Elizabeth Castle, a fortress set on a tidal island just offshore in St Aubin's Bay on the edge of St Helier. The ferries presently operate by reversing down the cobbled slipway that leads from quay side to a sandy rocky beach. It's also understood that the ferry service is only run in calm to mildly rough sea conditions.

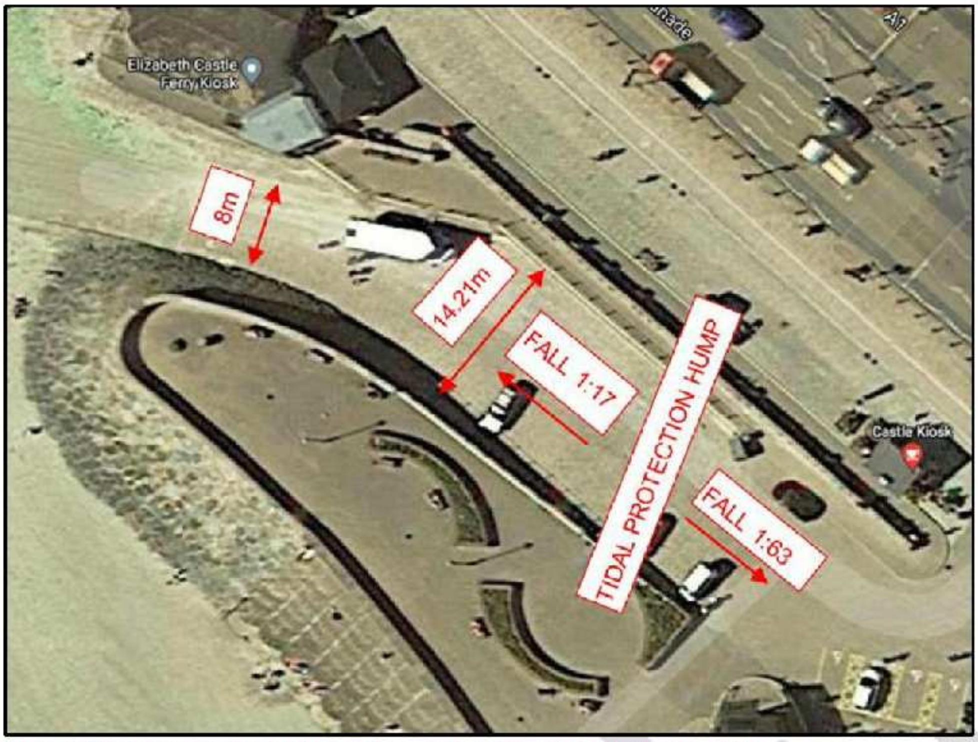

Figure 4-2 : Layout of existing slipway. Source: Google Earth, 2021

The existing slipway is approximately 8 metres wide at beach level and just under 14,5 metres wide at the top. The slipway is orientated at an angle of approximately 30 degrees to the shoreline and meets the uneven concrete causeway/footpath that runs across the beach to the castle. The slipway slopes at a gradient of 1:17 with the fall changing direction at the tidal protection hump as shown on Figure 4-2.

4.3.2 General design guidelines for small craft launching ramps

A launching ramp normally consist of one or more lanes of uniform gradient extending from above the high-water mark to below the lowest predicted water level. The land approaches should be level, perpendicular to the ramp centre-line and uniformly graded parallel to the centre-line to assist with the backing of slipway users.

a) Dimensions

The ramp length will depend on local tidal conditions and the period of tide during which launching is intended. Following a review of local and international slipway design guidelines, the following should be considered for small craft vessels launching ramps/slipway:

- Head of the ramp should be a minimum of 500mm above the highest astronomical tide with a suitable vertical curve grading provided to allow a smooth transition and satisfactory vehicle clearances to the land approach.
- The land approach should extend at least 20m landward of the head of the ramp.
- A single lane ramp should be a minimum of 4m wide between kerbs or at least 4.5m for a single lane without kerbs.
- A multi-lane ramp should have minimum width per lane of 3.7m.

b) Gradient

For boat trailers the ramp gradient should be within the range of 1:9 to 1:7 to ensure quick and easy launching of the boat from the trailer without the towing vehicle having to enter the water. Where local needs and conditions require a grade outside this range, the variation and its associated use limitations should be clearly shown on a sign adjacent to the head of the ramp.

c) Vehicle manoeuvring areas

Towing vehicle manoeuvring areas should allow for a minimum vehicle turning path as show in Figure 4-3.

d) Dry storage and retrieval facilities

The type of boat storage system and required retrieval facilities is generally governed by the type and size of the boat and the required capacity of the retrieval system. These facilities are client dependant.

e) Traffic and parking studies

Traffic and parking studies are recommended and should, as a minimum, consider existing availability of car parking and traffic generation rates during low and high season to determine operational constrains.

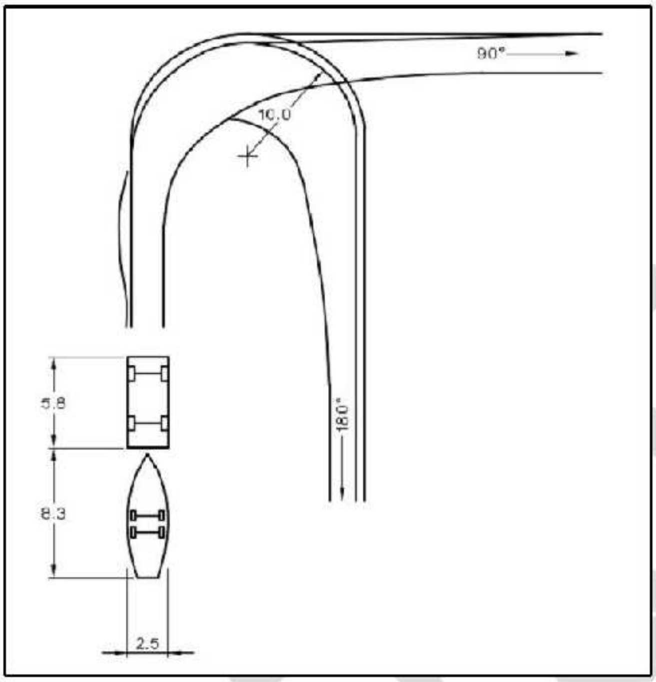

Figure 4-3: Car, boat and trailer turning path (Extract from AS3962-2001 Guidelines for design of marinas)

4.3.3 Alignment and launching of new proposed slipway

The working concept Masterplan diagram received from Gillespies, shown in Figure 4-4 , indicates a slipway width of 10m with an approximate gradient of 1:14. The new proposed alignment swaps over from the existing and is parallel to the shoreline.

Generally, it's advisable to have the new alignment perpendicular to the direction of the main wave attack, but development constraints apply. Therefore, it is important to discuss the new proposed alignment the current ferry operators and Jersey navigation authority to ensure the it is acceptable.

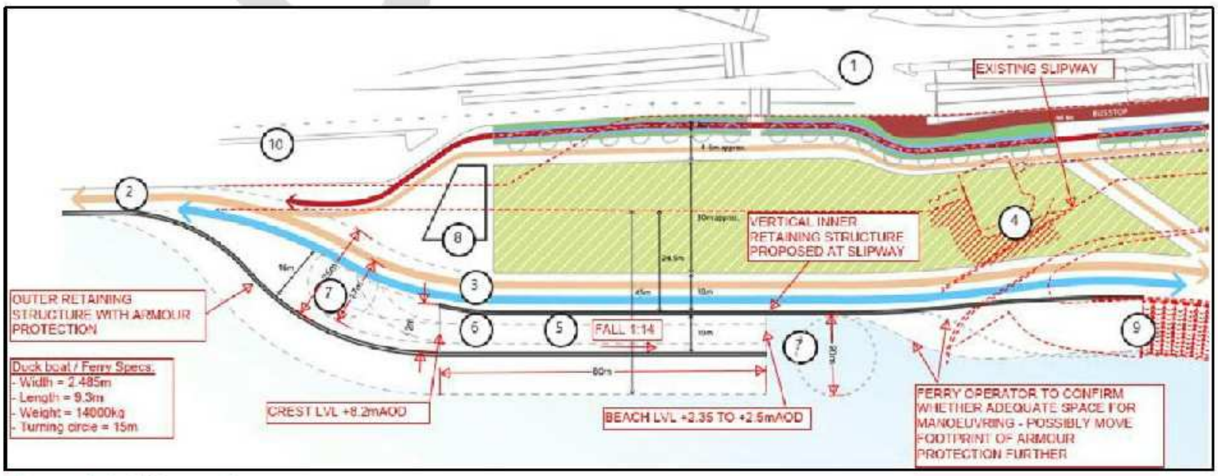

Figure 4-4: Slipway Impacts

The proposed new slipway width (10m) is greater than the existing slipway (8m) and is more than adequate for a 2-lane towing vehicle slipway. The addition of a pedestrian walkway may be considered going forward.

The proposed gradient is less than the existing, but still sufficient for independent wheelchair users. Presently the amphibious ferries do not require a minimum slope as the operation is different to a boat and trailer launching, however its recommended not to exceed the 1:12 slope recommended for independent wheelchair users.

The preferred turning circle for towing vehicles is 20m and according to the current ferry operator the amphibious vehicles only require 15m. At the top of the ramp the available space appears to be approximately 25m, but this might possible be reduced to allow for the Le Petit Train or vehicle parking. At the bottom of the ramp a turning circle of 20m will most likely fit, but its recommended to understand the impact of the proposed armour toe structure may reduce the turning circle and needs to be confirmed following the finalisation of the edge protection and scour protection structure. The required space or adequacy thereof should also be discussed with the current operator and navigation authorities for approval.

Furthermore, impact damping measures fixed to the sides of the vertical retaining structures both sides of the slipway may be considered to prevent damage to small crafts during launching in rougher sea conditions.

As mentioned in Section 4.3.2e), undertaking traffic and parking studies are recommended and should, as a minimum, consider existing availability of parking or loss thereof, as well as traffic generation rates during low and high seasons.

For ramp toe protection, its recommended to extend the outer retaining structure to the toe level of the ramp and build up from rock level to ensure the toe is not structurally weakened through scouring episodes during large storms events.

4.4 Environmental Constraints

Residential properties line much of the St Aubin's Bay coastline, with many within 200m of the site and an array of commercial properties (retail/restaurants) are also located in the vicinity of the site.

There are no designated heritage assets within the project site boundary. There are a large number of designated assets within 250m of the site, which include listed buildings and listed places. There are no non-designated assets within 250m of the project site.

The site is not located within any protected sites designated for marine features. The closest protected site is the Southeast Coast Ramsar which is approximately 1.3 km to the south of the project site. This Ramsar site provides important winter habitat for nationally important populations of waders and wildfowl.

St Aubin's Fort Ecological Site of Special Interest (SSI) is located approximately 3 km to the south-west of the site. This SSI is protected on the basis of special archaeological, architectural, historical, botanical, zoological and ecological features. The marine habitat in the direct footprint of the project consists of upper intertidal sandy soft sediments supporting infaunal invertebrates such as polychaetes, bivalves and amphipods. It has been advised by Paul Chambers – Government of Jersey (August 2020) that previous surveys undertaken in the area indicate the intertidal sands generally have low biological diversity and abundance and are considered to be of low value.

5 Option Development and Appraisal

A desktop analysis has been carried out to shortlist the different coastal protection solutions that fall under the possible coastal structure options identified in Section 4.2. The key considerations and criteria used for shortlisting the coastal protection structures for the proposed alignment and for the slipway structures are as follows:

- Buildability – *To understand the Metocean and geotechnical conditions at the study area and to recommend the best possible coastal structures from a structural buildability perspective.*
- Likely Overtopping Performance - *How the structure generally performs against overtopping in the severe weather conditions, predominantly in terms of likely crest level variations between the different coastline structures.*
- Env Sustainability – *To provide a preliminary indication of the impact on the environment during construction and operation of the proposed coastal structures and to understand the heritage, landscape, and ecological implications.*
- Wider Benefits Opportunities - *To understand the, likely footprint of the structure to identify the extent of the available beach frontage for public use, possibility of providing beach access to the beach users, and visual impact assessment.*
- Cost – *High level assessment of capital cost and maintenance cost variation between the different options.*

5.1 Edge protection Structures

5.1.1 Introduction

The edge protection or retaining structures shall be in different forms as mentioned in the Section 4.2. However, different structural forms including Rock Revetment, Gabions, Geotextile sand filled containers, Concrete Revetment with blocked surface (Seabees), and Stepped Concrete Revetment have not been considered due to different constraints including structural stability, wider footprint, and aesthetics.

The shortlisted edge protection structures are as follows:

- Mass Gravity Retaining Wall;
- Cantilever Reinforced-Concrete Stem Wall;
- Cantilever Reinforced-Concrete Stem Wall with Piled Heel.

5.1.1.1. Mass Gravity Retaining Wall

These structures depend on self-weight to resist the lateral earth pressure from the retained backfill and therefore, in general will be huge structures. Although different materials could be used for the construction of the gravity wall including concrete, stone and masonry, considering the height of the retaining fill (7m+), a concrete gravity retaining wall is considered suitable. An indicative sketch of the proposed gravity wall is shown below:

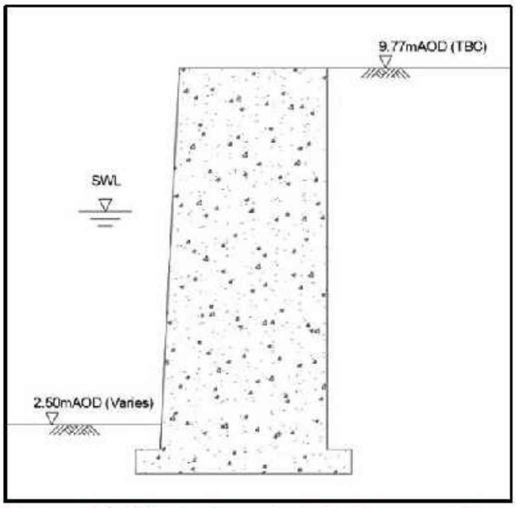

Figure 5-1: Illustrative sketch of a mass Concrete Gravity Retaining Wall (not to scale).

- Buildability

  Alluvial beach deposits will need to be removed and replaced to construct the wall foundation on the competent bedrock. Construction will require the management of the tidal water as stated in Section 4.2.

- Likely Overtopping Performance

  Due to the impermeable nature of the structure and minimal wave dissipation, a higher crest level is expected when compared to conventional rock revetment systems. However, with the provision of a wave wall/recurve wall/bullnose the wave overtopping shall be minimised.

- Env Sustainability

  It is considered that each of the three options would result in similar impacts on landscape and heritage receptors due to their scale and appearance. This option is likely to require the largest amount of material resource when compared to the others and have the greatest embodied carbon. Of the three edge protection structures this is considered to have the greatest permanent footprint within the foreshore but this can be mitigated by locating the front face on the required alignment.

- Wider Benefits Opportunities

  Sea view impacts could be reduced ort mitigated by providing an adequate wave wall as stated above. The beach encroachment is limited compared to sloped structures such as a revetment and therefore minimizes the impact on the beach available for public use. Amenities could be provided at the leeside of the wall considering the safe setback distance from the face of the wall that will be predominantly depends on the wave overtopping.

- Cost

  The capital costs are expected to be higher when compared to conventional revetment systems, including rock and concrete armour block revetments, however, minimal maintenance costs are anticipated during the typical design life of 50 years.

5.1.1.2. Cantilever reinforced-concrete stem wall

Cantilever retaining wall is one of the most common types of retaining walls and composed of stem and base slab. The base slab portion below the retained earth fill is termed as "heel slab" and other part is termed as toe.

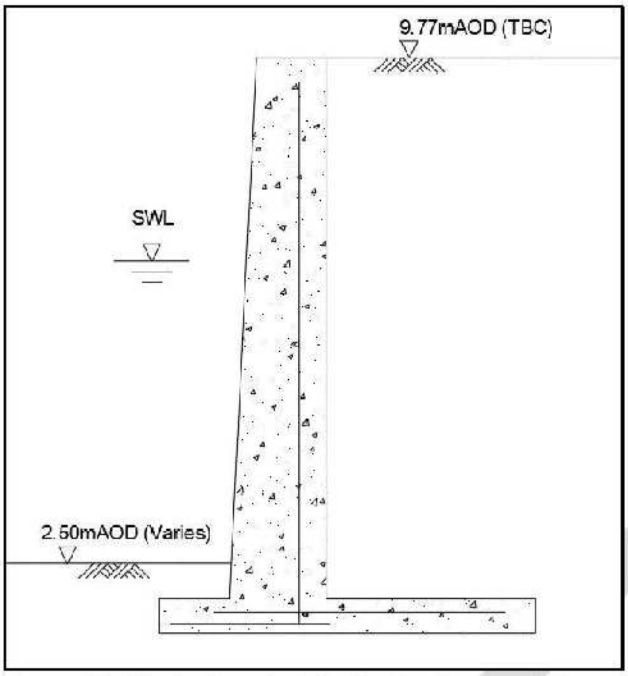

Figure 5-2: Illustrative sketch of a Cantilever reinforced-concrete stem wall (not to scale).

- Buildability

  Similar to a gravity wall, alluvial beach deposits will need to be removed and replaced to construct the base slab foundation on the bedrock. Construction will require the management of the tidal water as stated in Section 4.2.

- Likely Overtopping Performance

  Overtopping performance is similar to the gravity wall structures and the wave overtopping could be minimised by providing a wave wall/recurve wall/bullnose.

- Env Sustainability

  It is considered that each of the three options would result in similar impacts on landscape and heritage receptors due to their scale and appearance. This option is considered to have a higher embodied carbon than the Cantilever Reinforced-Concrete Stem Wall with Piled Heel, but lower than the Mass Gravity Retaining Wall on the basis that it would require less concrete.

- Wider Benefits Opportunities

  The beach encroachment is limited and thereby has minimal impact on the extent of the beach available for public use. Considering the extent of the overtopping flooding and adequate drainage facilities, amenities could be provided at the leeside of the wall.

- Cost

  The capital and maintenance costs are expected to be in the same order as that of the mass concrete gravity wall.

5.1.1.3. Cantilever Reinforced-Concrete Stem Wall with Piled Heel

This type of structure is similar to Cantilever reinforced-concrete stem wall, however the foundation area required will be minimal as the resistance generated by the piles.

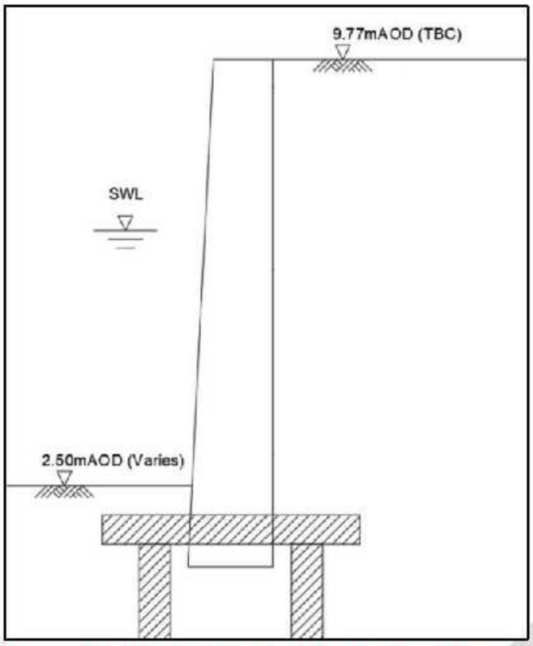

Figure 5-3: Illustrative sketch of a Cantilever Reinforced-Concrete Stem Wall with Piled Heel (not to scale).

- Buildability

  Although the required foundation area is minimal, the key buildability constraint will be hard drilling of the piles on the bed rock. This means special drill bits and heavy rigs are required to get the required torque to drill the piles.

- Likely Overlopping Performance

  Overtopping performance is similar to the gravity wall structures and the wave overtopping could be minimised by providing a wave wall/recurve wall/bullnose.

- Env Sustainability

  It is considered that each of the three options would result in similar impacts on landscape and heritage receptors due to their scale and appearance. This option is considered to have the lowest embodied carbon through the use of steel sheet piles. Of the three edge protection structures this is considered to have the greatest construction footprint within the foreshore however it would have the smallest permanent footprint. As this option would require special drill bits and heavy rigs this option is considered to have the greatest potential to result in nuisance effects related to noise, vibration and air quality. It is also considered that these works could disturb marine wildlife through noise, vibration and air quality.

- Wider Benefits Opportunities

  Minimal beach encroachment and beach extent available for public use will not be much affected. Amenities could be provided at the lee side of the wall considering the overtopping zone.

- Cost

  The capital cost is expected to be much higher than the gravity/cantilever wall with heel slab as the pile drilling operations could be expensive. However, the maintenance cost is expected to be in the same order as that of the gravity/cantilever wall with heel slab.

5.1.2 Option- Benefits Matrix

An option-benefits matrix has been provided below summarising the key assessment points. The options are ranked based on the overall anticipated performance to support the decision-making process.

Table 5-1: Option- Benefits Matrix for Edge Protection Structures

<table>
<tr><th>Structure</th><th>Buildability</th><th>OT performance</th><th>Environment</th><th>Wider Benefits</th><th>Cost (Capital/ Maintenance</th><th>Ranking</th></tr>
<tr><td>Mass Gravity Retaining Wall</td><td rowspan="2">Requires removal and replacement of alluvial deposits for foundation construction. Management of tidal water for land reclamation and wall construction</td><td rowspan="3">Moderate crest level with provision of wave wall/recurve wall/bullnose.</td><td>Larger permanent footprint and greater embodied carbon</td><td rowspan="3">Reduced structural footprint. Amenity potential.</td><td rowspan="2">Medium, Low</td><td>1</td></tr>
<tr><td>Cantilever reinforced-concrete stem wall</td><td>When compared to the points above, this option would have a smaller permanent footprint and lower embodied carbon.</td><td>1</td></tr>
<tr><td>Cantilever Reinforced-Concrete Stem Wall with Piled Heel</td><td>In addition to above points, the drilling of piles in the hard rock is challenging.</td><td>Could result in additional nuisances effects (noise, vibration and air quality) as a result of piling requirements</td><td>Expensive, Low</td><td>2</td></tr>
</table>

5.2 Slipway Structures

5.2.1 Introduction

The options for edge protection structures for the slipway could include rock armour, sheet piles, vertical retaining walls. However, the rock armour and sheet pile options are not considered further in this study due to aspects such as structural feasibility, wider benefits (including wider footprint), coastal processes and aesthetics.

The edge protection structures for the proposed alignment (Section 5.1) could be considered for the slipway to ensure a smooth transition. However, an additional option, i.e. Concrete block armour revetment is provided for further discussion with the client to accommodate more wave dissipation and thereby minimal toe erosion.

The shortlisted slipway edge protection structures are as follows:

- Mass Gravity Retaining Wall;
- Cantilever Reinforced-Concrete Stem Wall;
- Cantilever Reinforced-Concrete Stem Wall with Piled Heel.
- Concrete Block Armour Revetment.

Since the first three options are the same as that provided in Section 5.1.1, only concrete armour revetment is discussed further in this section.

5.2.1.1. Concrete Block Armour Revetment

Concrete block armour revetments are typically provided in severe wave conditions, where the procurement/transportation of large rocks (in the order of 10t or more) to the site is challenging.

The different types of concrete block armour include Cubes, Dolos, Tetrapod's, Accropode's, Core-loc units, and Xbloc units, and the choice of the concrete block is generally based on the structural and hydraulic stability requirement, the number of layers required, cost, and aesthetic requirements.

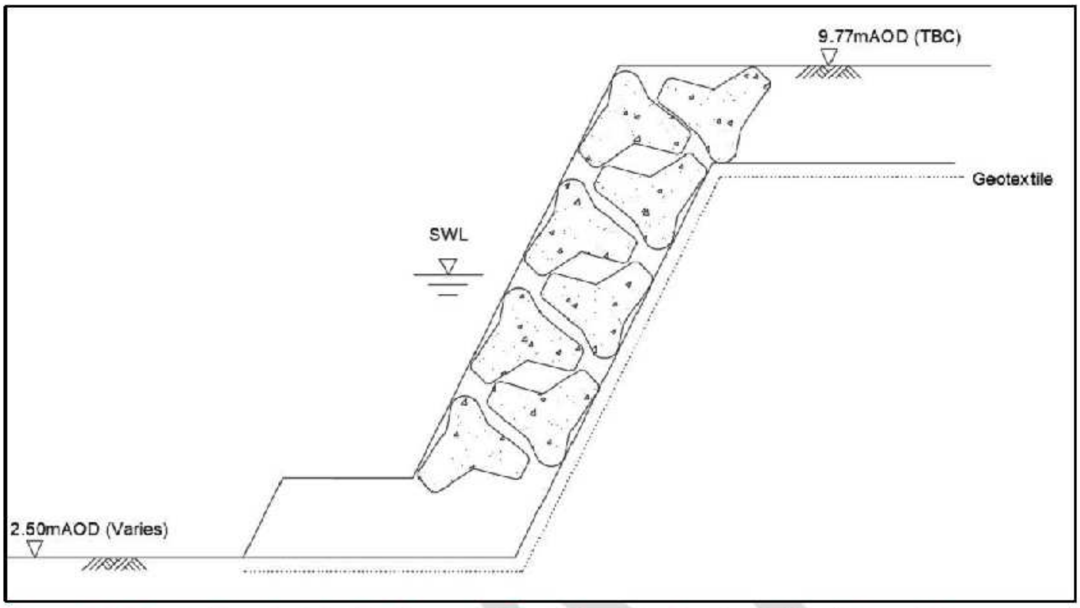

Figure 5-4: Illustrative sketch of a Concrete Block Armour Revetment (not to scale).

- Buildability

  Alluvial beach deposits will need to be removed and replaced depending on the structure toe configuration.

- Likely Overtopping Performance

  Absorbs wave energy through the voids between the blocks and therefore limits overtopping and flooding.

- Env Sustainability

  This option would have a larger construction and permanent footprint than the edge protection structures. It is considered that this option is likely to have slightly adverse landscape effects, heritage effects depending on final design due to change in appearance from the rest of the existing sea defence structures.

- Wider Benefits Opportunities

  Although the structural footprint is wider, the toe erosion will be minimal when compared to the vertical structures.

- Cost

  The capital costs are typically lesser when compared to impermeable concrete coastal protection structures, and the maintenance costs are minimal.

5.2.2 Option- Benefits Matrix

An option-benefits matrix has been provided below and the options are ranked based on the overall anticipated performance to support client decision making process.

Table 5-2: Option- Benefits Matrix for Edge Protection Structures

<table>
<tr><th>Structure</th><th>Buildability</th><th>OT performance</th><th>Sustainability</th><th>Wider Benefits</th><th>Cost (Capital/ Maintenance</th><th>Ranking</th></tr>
<tr><td>Mass Gravity Retaining Wall</td><td rowspan="2">Requires removal and replacement of alluvial deposits for foundation construction.
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Amenity potential.</td><td rowspan="2">Medium, Low</td><td>1</td></tr>
<tr><td>Cantilever reinforced-concrete stem wall</td><td>When compared to the points above, this option would have a smaller permanent footprint and lower embodied carbon.</td><td>1</td></tr>
<tr><td>Cantilever Reinforced-Concrete Stem Wall with Piled Heel</td><td>In addition to above points, the drilling of piles in the hard rock is challenging.</td><td>Could result in additional nuisances effects (noise, vibration and air quality) as a result of piling requirements</td><td>Expensive, Low</td><td>3</td></tr>
<tr><td>Concrete Block Armour Revetment</td><td>Requires removal and replacement of alluvial deposits depending on the toe configuration.</td><td>Better wave dissipation and hence minimal overtopping</td><td>Likely to have adverse landscape and heritage effects as a result of the introduction of a new feature within the landscape that deviates from the existing sea defence structures</td><td>Minimal scouring at the structure toe.</td><td>Low, Low</td><td>2</td></tr>
</table>

6 Summary

The baseline metocean and geotechnical information has been reviewed. The seabed levels at the proposed agreed alignment is in the order of +2.5 to +3.0mAOD, and levels shall be validated once the alignment is confirmed and concept Masterplan is finalised.

The critical wave direction is from 240°N and therefore the critical wave climate will be acting nearly perpendicular to the proposed agreed alignment.

Based on the available geotechnical information from BGS Map Sheet 31 and the Amplus Ltd Ground Investigation, the proposed study area is envisaged to comprise Alluvium in the form of beach deposits to depths between 1mbgl and 3mbgl, underlain by Granophyre (granite) igneous bedrock.

Potential Scour at the base of the slipway toe is expected due to wave concentration. The geotechnical constraints include the management of the tidal water during the land reclamation and wall construction, removal and replacement of the alluvial deposits for wall foundation construction, and drainage strategy to manage high water levels, groundwater and overtopping.

The proposed slipway width of 10m is wider than the existing slipway and more than adequate to accommodate a 2-lane traffic. The proposed gradient of 1:14 is less than the existing slipway, but still sufficient for independent wheelchair users. The amphibious ferries do not require a minimum slope however it is recommended not to exceed the 1:12 slope considering the independent wheelchair users.

The current ferry operator states the required turning circle is 15m, although the preferred turning circle for towing vehicles is 20m. At the top of the ramp, the available space is approx. 25m, however could be reduced due to Le Petit train or vehicle parking.

At the bottom of the ramp, a turning circle of 20m is available, but the impact of the proposed armour toe structure may reduce the turning circle and needs to be confirmed following the finalisation of the edge protection and scour protection structure. The required space available for manoeuvring should be discussed with current operator and navigation authorities for approval.

The proposed alignment and advancement of the shoreline is confirmed to provide a suitable arrangement to support the design of new edge protection structures and slipway.

The shortlisted options for the edge protection structures include mass gravity retaining wall, conventional cantilever reinforced concrete stem wall, and cantilever reinforced concrete stem wall with piled heels. Based on the key assessment criteria including buildability, overtopping performance, sustainability, wider benefits opportunities and cost, the recommended options for the edge protection structures shall be gravity wall and conventional cantilever wall. At the transition with the existing structures, either gravity wall or cantilever with pile foundation could be considered as "tie-in structures" where there is the space constraint.

The above-mentioned coastal structures shall be considered for the edge protection for the slipway structure, however a concrete block armour revetment has also been included for client discussion as this offers better wave dissipation and would help minimise scouring at the structure toe.

Based on the above-mentioned key criteria, the recommended options comprise a gravity wall and conventional cantilever wall accommodating a smooth transition between the slipway and the proposed alignment edge protection structures.

From the previous wave overtopping and flood risk assessment studies undertaken in the study area, it is understood that the primary defence alone could not provide the required SoP and therefore the secondary defence structures should be considered. Detailed flood risk assessment shall be undertaken in the next stage of the study for concept design of the primary and secondary defence structures.

6.1 Recommendations and Way forward

- The shortlisted edge protection options shall be discussed with the client to confirm the preferred edge protection solutions for the proposed alignment and slipway.

- Following the selection of the preferred edge protection structures, the structural and operational (in terms of overtopping) performance of the structures shall be analysed.

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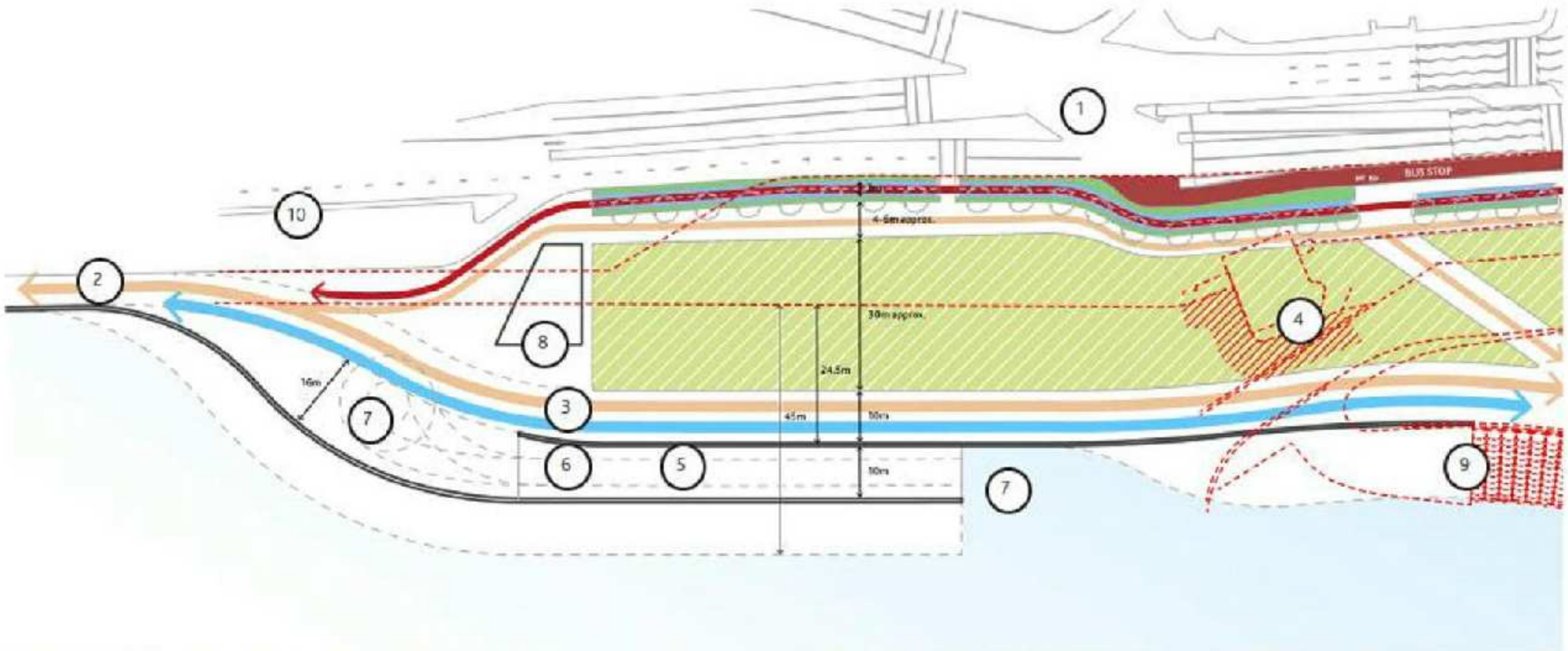
A.1 Proposed Agreed Alignment

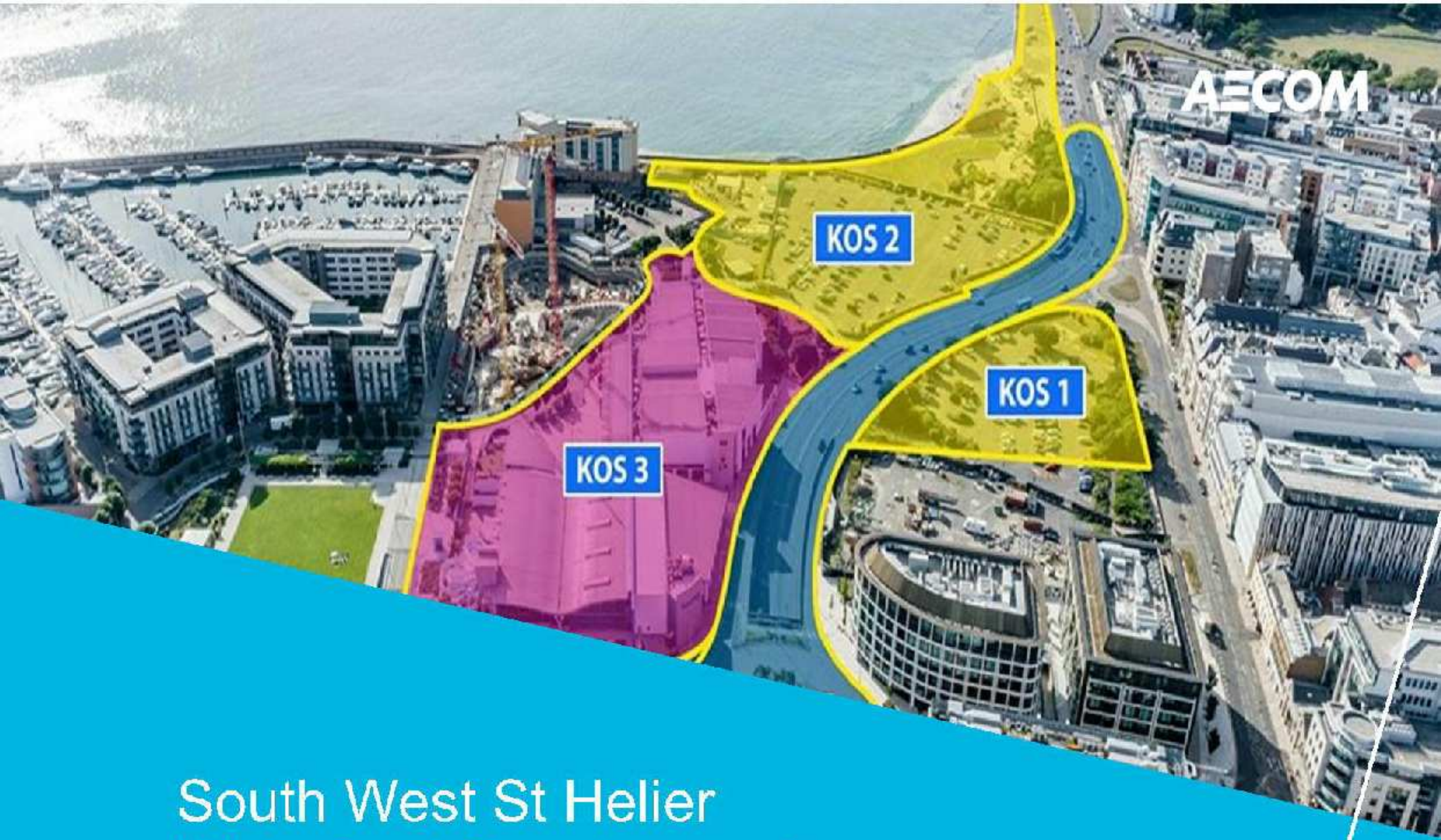
INCREASED RECLAMATION

WEST PARK



1. What is the likely extent of changes to the existing junction and subsequent space take?
2. How far West should these works be?
3. Potential for Le Petit Train to route along waterfront
4. Heritage value of the bunker -potential to create a void to reveal the existing wall opening
5. 10m wide slip way. Is this sufficient?
6. What are the key parking requirements, is it viable to park on the slipway or integrate along Victoria Avenue or as part of kiosk building?
7. What is the preferred turning circle; top or bottom of the ramp or on ramp?
8. Potential ticket kiosk and castle vehicle storage area
9. Interface to existing revetment (sheer seawall to stepped revetment?)
10. Loss of carparking

- Note: Diagram depicts design intent only for discussion. All works are subject to Hospital junction design, further flood defence requirements, transport engineer and IHE's input



South West St Helier Waterfront Development

Coastal Assessment Report

States of Jersey Development Company

Project Reference: St Helier Waterfront
Project Number: 60650295
001

19th April 2021

Quality information

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Revision History

Revision	Revision date	Details	Authorized	Name	Position
000	Initial Draft	Draft for Review	JCAS	Jon Short	Associate

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Abbreviations and Acronyms

AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
DTM	Digital Terrain Model
FoS	Factor of Safety
GoJ	Government of Jersey
HAT	Highest Astronomical Tide
JPA	Joint Probability Analysis
MHWS	Mean High Water Springs
OD	Ordnance Datum
OT	Overtopping
QT	Discharge-Time
RP	Return Period
SMP	Shoreline Management Plan
SoP	Standard of Protection

1. Introduction

The Jersey Development Company (JDC) commissioned AECOM to undertake a preliminary study to assess the coastal wave overtopping and flood risk for the South West St Helier Development site.

The study has assessed the current standard of protection provided by the structures fronting the site and has undertaken high level optioneering to provide the required standard of protection against storm events. High level stability assessment of the potential required design modifications has also been undertaken.

The study was developed in close liaison with JDC and Gillespies (undertaking the Masterplan for the site) and the outputs of the study have informed the outline design plans and landscaping for the site.

The report presents the findings of the study as follows:

Chapter 2 – Study Site and Coastal Structures. Overview of the site and types of coastal structures present.

Chapter 3 – Scope and Outputs. A summary of the key tasks of the study.

Chapter 4 – Option Appraisal. General Summary of the different flood risk management options and approach selected for the present study.

Chapter 5 – Wave Overtopping Assessment. Provides estimated overtopping discharges for different sections along the study area representing different types of coastal structures and anticipated crest levels for the same to accommodate design storm events.

Chapter 6 – Inundation Assessment. Provides inundation mapping for 1:200 RP for the present day, 2070 and 2120 for the study area.

Chapter 7 – Structural Assessment. Provides the high-level stability assessment findings undertaken for raising the existing coastal defence structure crest levels.

Chapter 8 -Summary. Provides an overview of the study undertaken and the key conclusions derived from the study.

2. The Site

The study frontage is located at the far eastern edge of St. Aubin's Bay. The site runs from just west of the bunker near the slipway, adjacent to Jersey's only dual carriageway the A2-Victoria Avenue, along the terrace blocks and down to the edge of the rock armour structure just north of the Radisson Hotel. The land behind the defences has been previously reclaimed.

The location of the project site is shown in Figure 2-1.

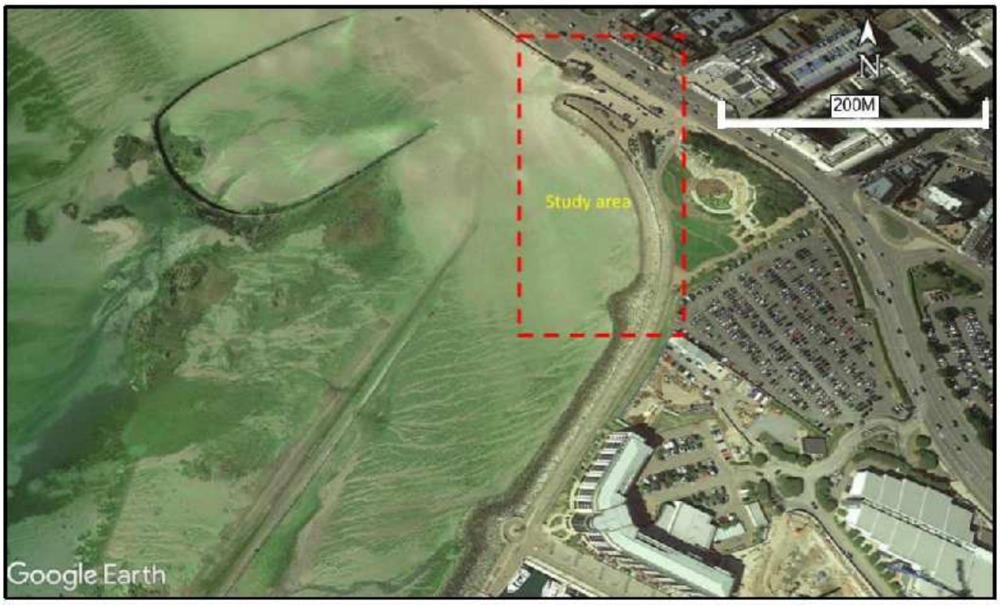


Figure 2-1. Frontage with extent markings. Source: Google Earth, 2021.

2.1 Coastal structures and characteristics

The site consists of different coastal structures and defences including, battered sea walls, slipway, granite talus, terrace blocks, and rock armour. The following sections (Figure 2-2) have been selected to define the same and to undertake the overtopping calculations.

The structural detailing used for the overtopping calculations has been extracted from the as-built drawings (dated 1993) provided by GoJ. These as-built drawings used Chart datum as the vertical datum system, and therefore the levels have been reduced to Ordinance Datum to align with the existing wave model results and to incorporate this information to update the existing inundation model.

It has been observed that the sea bed levels in as-built drawings has changed when compared with the latest available bathymetry data (LIDAR 2018). This change in bed level is most likely due to the coastal processes including, sedimentation. Therefore, to reflect the actual site conditions for the current baseline assessment, the sea bed levels have been obtained from the LIDAR data (2018).

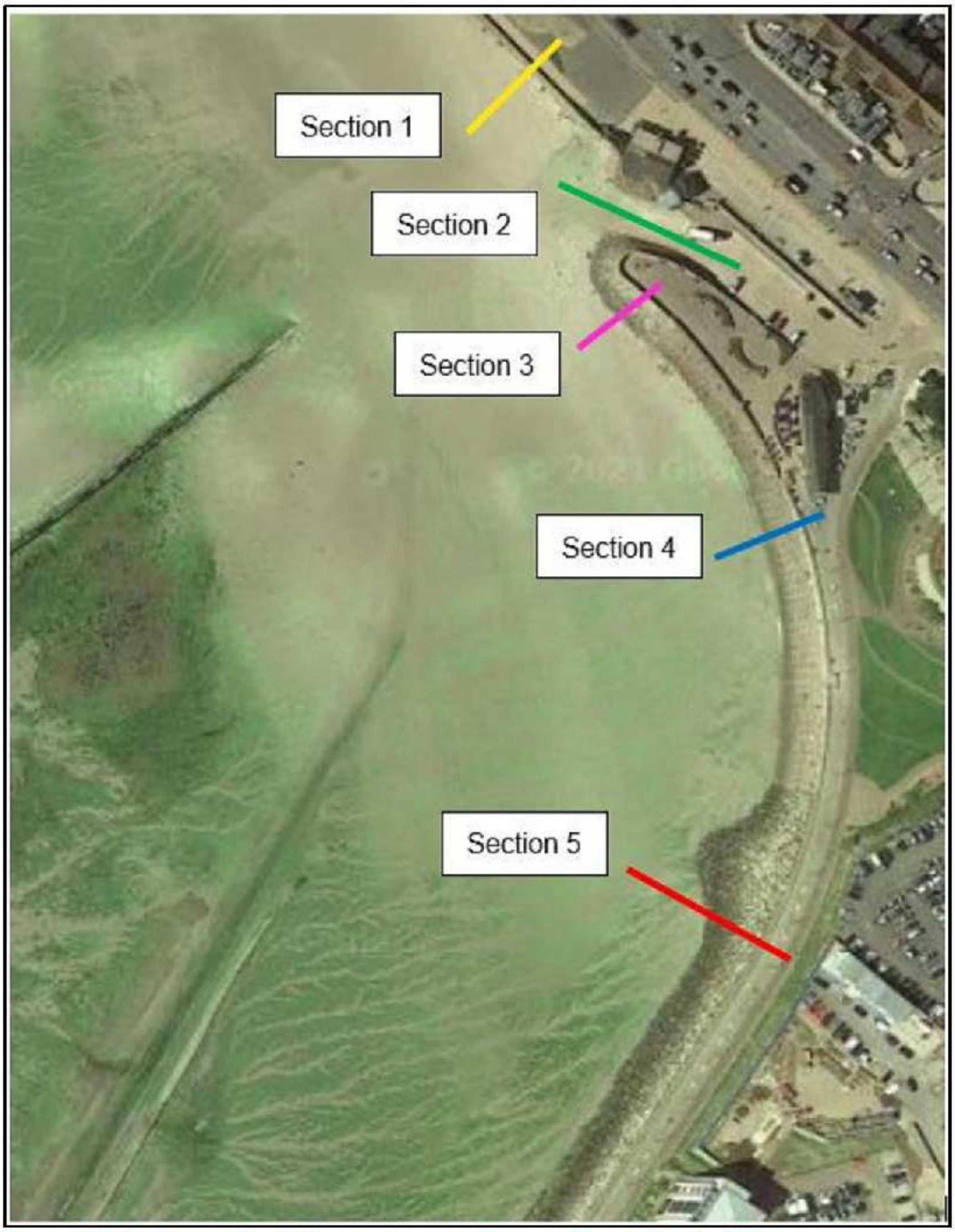


Figure 2-2. Sections selected for overtopping. Source: Google Earth, 2021.

The different sections selected for the overtopping calculations are provided below:

- Section 1 (Battered Sea Walls)

This section consists of near vertical wall with a crest level and sea bed level in the order of 9.1m Above Ordnance Datum (AOD) and 2.5m AOD, respectively.

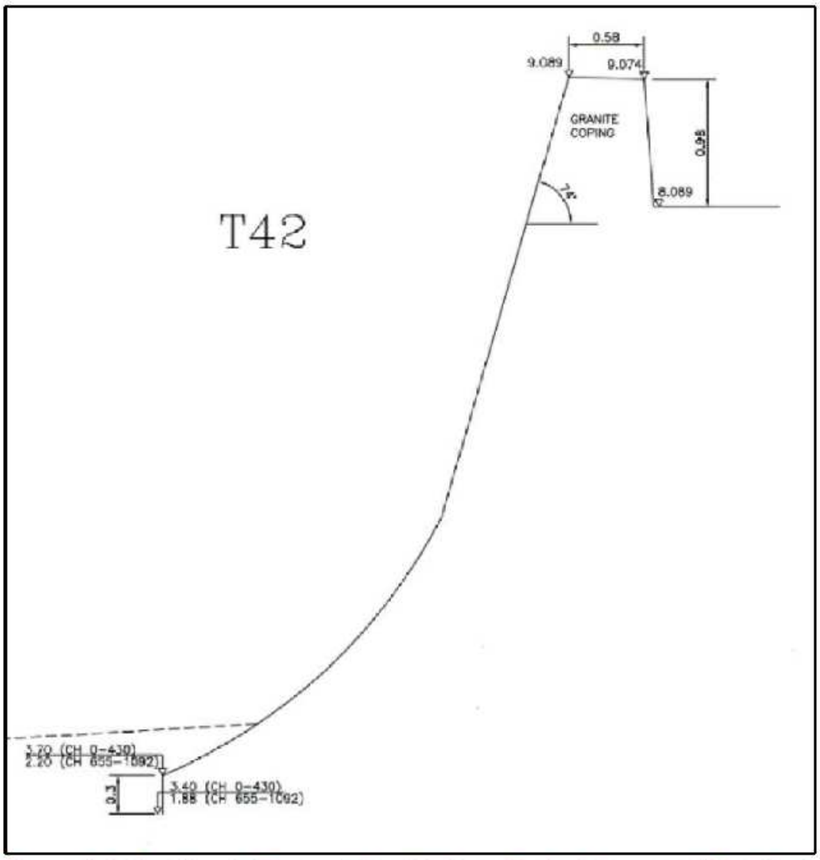


Figure 2-3: Section-1 (Battered Sea Walls) – Typical Structural Details (not to scale).

- Section 2 (Slipway)

This section represents the slipway with a crest level in the order of 8.2mAOD and the coastal sea bed level is assumed as 2.5mAOD.

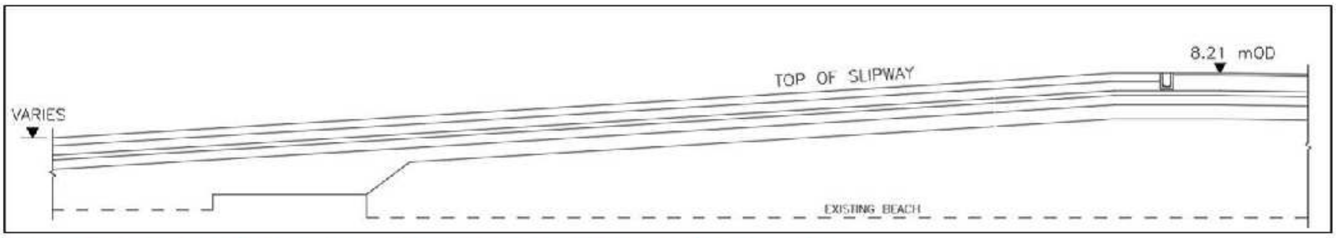


Figure 2-4: Section-2 (Slipway) – Typical Structural Details (not to scale).

- Section 3 (Granite Talus)

This section represents a sloped granite talus structure with a crest level in the order of 9.71mAOD and the coastal sea bed level is considered as 3.5mAOD.

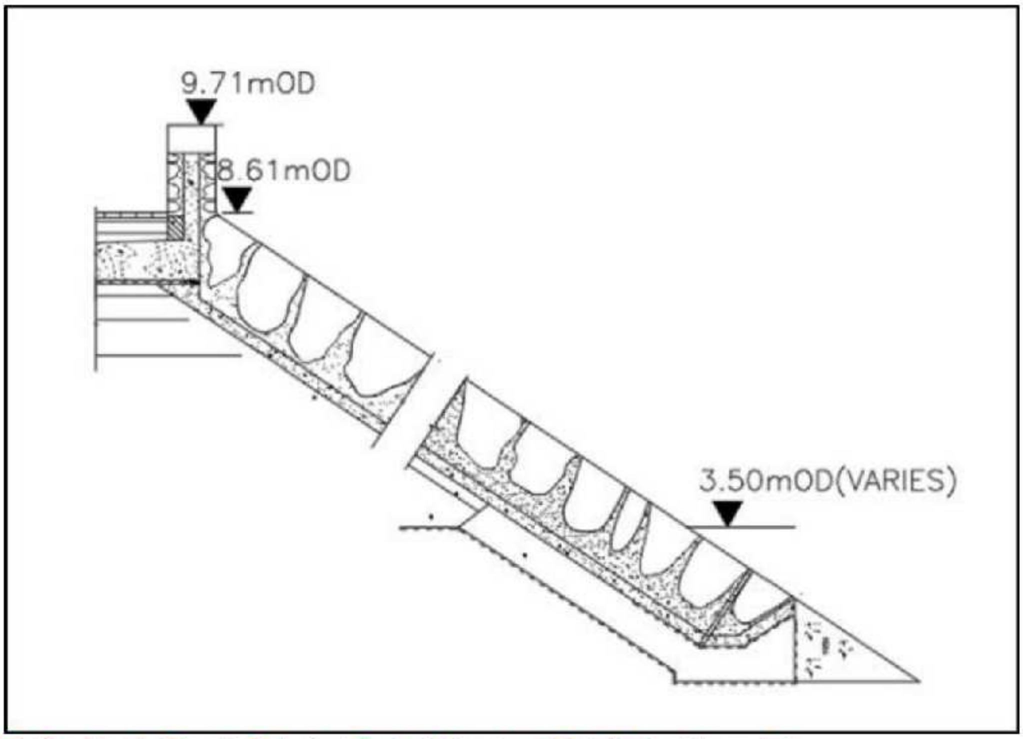


Figure 2-5: Section-3 (Granite Talus) – Typical Structural Details (not to scale).

- Section 4 (Terrace Blocks)

This section represents a stepped terrace block structure with a crest level in the order of 9.71mAOD and the coastal sea bed level is considered as 3.5mAOD.

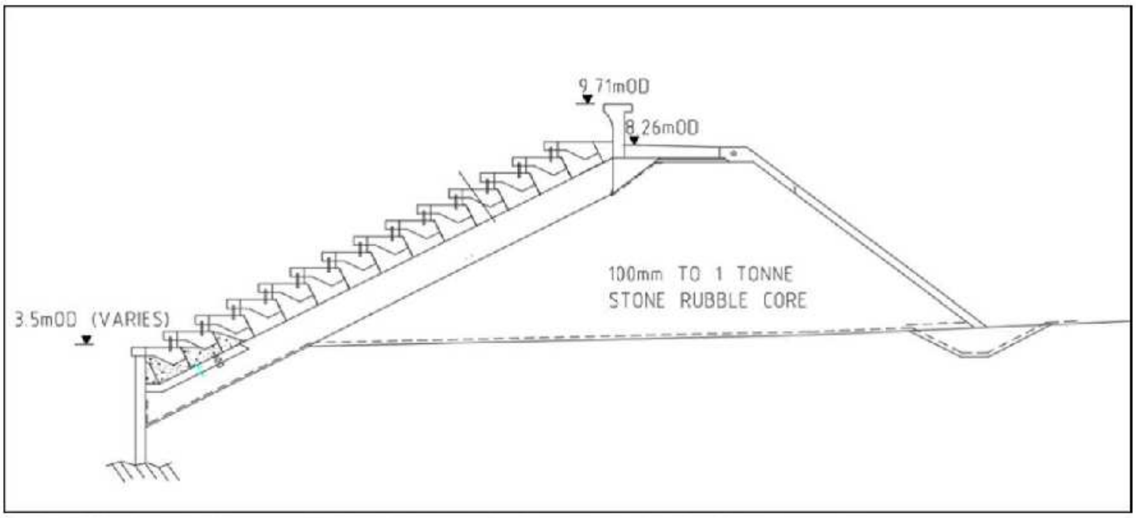


Figure 2-6: Section-4 (Terrace Blocks) – Typical Structural Details (not to scale).

- Section 5 (Rock Armour)

This section represents a sloped rock armour structure with a crest level in the order of 9.71mAOD and the coastal sea bed level is considered as 0.5mAOD.

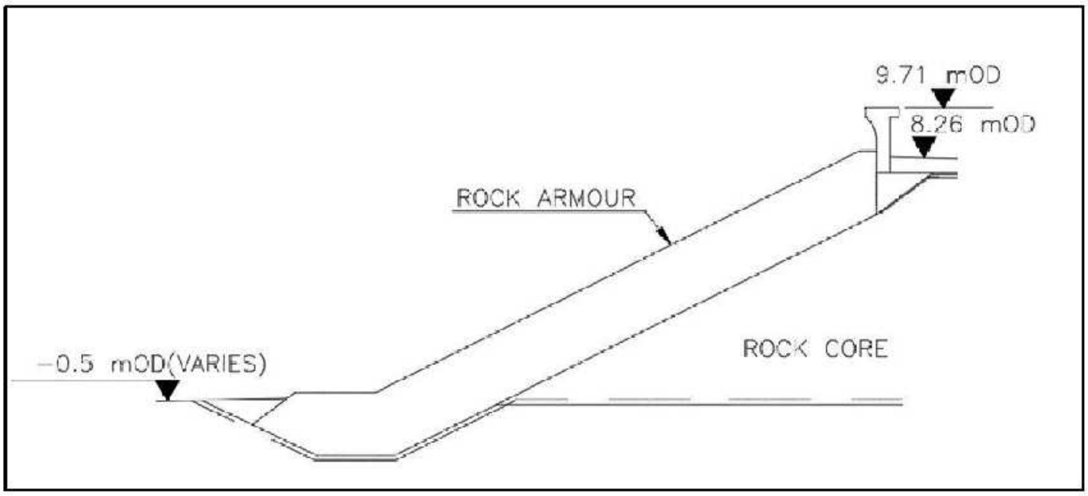


Figure 2-7: Section-5 (Rock Armour) – Typical Structural Details (not to scale).

3. Study Scope and Outputs

The key tasks and outputs of the study were to:

a) Undertake an overtopping assessment with the current defences (for the present day, 2070, and 2120 1:200yr event) to identify the current level of risk from wave overtopping.

b) Identify the level of required crest raising or structural modifications to mitigate storm events and provide the required standard of protection to the site.

c) Identify potential engineering options or structure modifications to provide the required standard of protection to the site.

d) Provide a high-level assessment of structural stability for the modifications required to the structures to assess viability of concept.

4. Option Appraisal

The flood risk in the coastal areas can be mitigated in different ways. The main options are:

- Do Nothing: Under this option there is no intervention allowing the flooding risk to remain unmitigated. Generally, this option is taken as the baseline condition to evaluate the performance of the other "do something options". Given the level of risk and the nature (see Chapter 5 and 7) of the development this option is not deemed viable for this project.
- Protect: This option includes the modification of the existing coastal structures including raising of the crest level or construction of new structures to limit the wave overtopping risks. Setback/Secondary wall options to reduce the primary defence height rise are also included in this option.
- Advance: This option results in reclaiming the coastal areas and still requires the significant raising of the primary defence wall, however, this provides space to accommodate defences and additional amenities for the community. The reclamation crest level depends on the type of primary defence structures.

For the present study, following the discussion with JDV and Gillespies, the preferred option concept is to raise the existing crest of the defences with wave return for the majority of the frontage; this concept has been used to test the required level of raising to mitigate the overtopping risk in the assessment (Chapter 5). Further specific detail behind the crest raising and preferred options around the slipway and the bunker are still being discussed and evaluated at the time of writing.

5. Wave Overtopping Assessment

5.1 Introduction

Wave overtopping calculations were undertaken for the coastal defence structures to identify the level of risk from coastal flooding for 1 in 200-year return periods (present day, 2070, and 2120). The calculations were carried out using EurOtop (2018) 'Manual on wave overtopping of Sea Defences and Related Structures' to determine the overtopping discharge (l/s/m) along the study area. EurOtop guidance (2018) is regarded as best practice within the industry. The required inputs to the calculation vary according to structure type. For the sea wall and the revetments, the inputs typically consist of:

- significant wave height (m);
- mean wave period (s);
- wave direction;
- structure freeboard (m);
- water depth at the structure toe (m);
- revetment slope.

The wave water levels for the study were extracted from AECOM's existing Mike 21 St. Helier local wave model. Based on an inspection and desktop analysis, it was established that the worst-case wave direction for the site was from a 240° sector, and therefore was applied in the overtopping analysis. The significant wave height plots for different scenarios are provided in Figure 5-1, Figure 5-2, and Figure 5-3below:

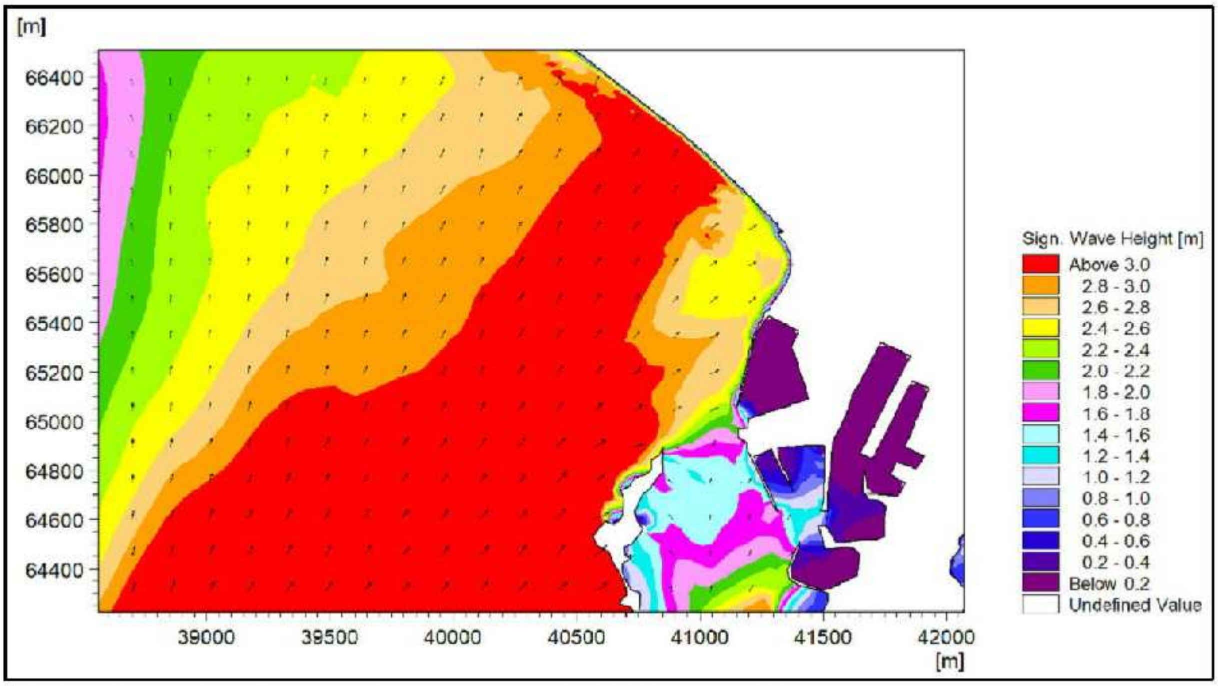


Figure 5-1: Modelled Wave Height for the present day,2020 (240° direction)

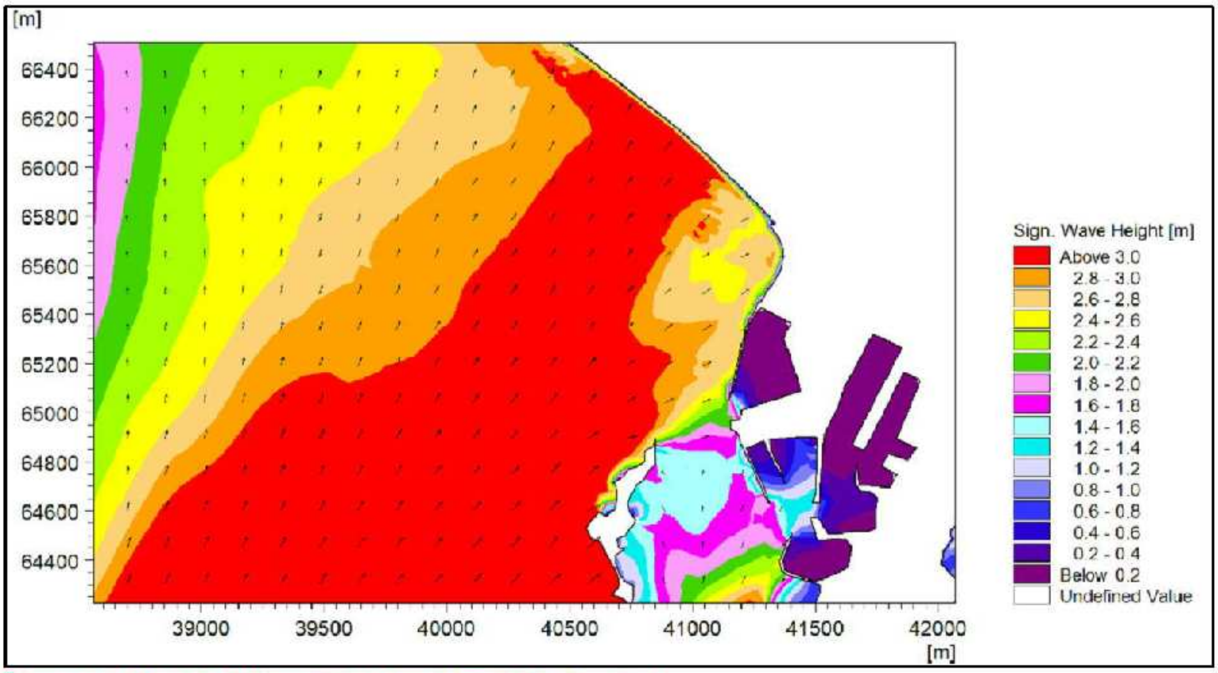


Figure 5-2: Modelled Wave Height for 2070 (240° direction)

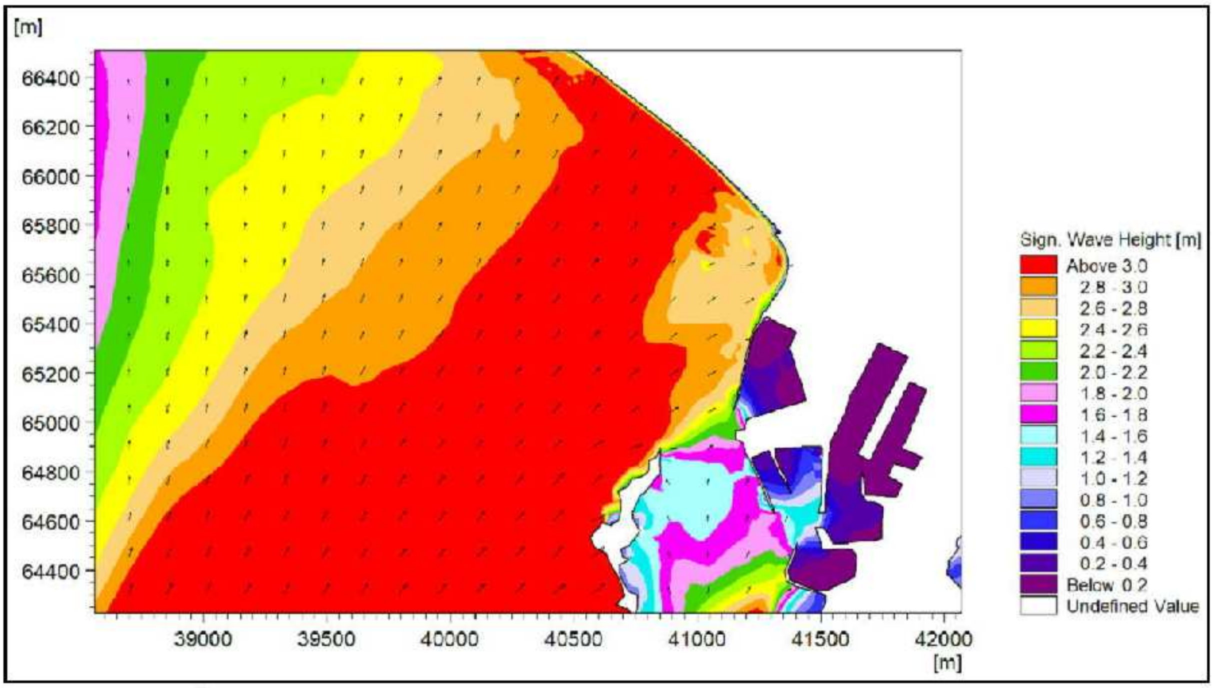


Figure 5-3: Modelled Wave Height for 2120 (240° direction)

The target residual OT limit for 1 in 200 SoP for 2070 is 0.1l/s/m, however considerations have been made to allow for the uncertainties in the prediction of the OT rates using the empirical equations. The overtopping tolerances provided in the EurOtop manual is shown below in

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Significant damage or sinking of larger yachts; H_{m0} > 5 m	>10	>5,000 – 30,000
Significant damage or sinking of larger yachts; H_{m0} = 3-5 m	>20	>5,000 – 30,000
Sinking small boats set 5-10 m from wall; H_{m0} = 3-5 m Damage to larger yachts	>5	>3,000-5,000
Safe for larger yachts; H_{m0} > 5 m	<5	<5,000
Safe for smaller boats set 5-10 m from wall; H_{m0} = 3-5 m	<1	<2,000
Building structure elements; H_{m0} = 1-3 m	≤1	<1,000
Damage to equipment set back 5-10m	≤1	<1,000

Figure 5-4: General limits of Overtopping for Property behind the Defence (EurOtop Manual, 2018)

Table 5-1: Guidance on Recommended Mean Overtopping Discharge Limits (EurOtop Manual, 2018)

Hazard Type	Mean Discharge Limits (l/m/s)		
	H_{m0}=1m	H_{m0}=2m	H_{m0}=3m
People at the seawall	10-20	1	0.3
Cars on seawall	75	10-20	5

5.2 Overtopping Rates

The estimated overtopping discharge rates for each of the options are provided in the below sections. The incoming wave direction has been considered as normal to the structure to provide a conservative (worst case) estimate of the overtopping rates.

Section 1 (Battered Sea wall)

The typical structural details used for the overtopping calculations are shown in Figure 2-3. The wave-water level combinations extracted at the structure toe and the estimated overtopping rates are provided below in Table 5-2.

Table 5-2. Hs & SWL Combinations at Structure Toe (Section-1, Battered Sea Wall) 1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.72	8.0	232.4	7.21	0.73	7.8	232.2	7.67	0.74	7.7	232.0
6.85	1.06	10.7	233.3	7.21	1.06	10.6	231.8	7.67	1.05	10.6	232.0
6.85	1.17	10.9	233.6	7.21	1.17	10.9	231.7	7.67	1.16	10.8	232.7
6.85	1.64	12.3	236.0	7.21	1.63	12.2	233.4	7.67	1.61	12.1	234.0
6.85	1.84	12.9	237.7	7.21	1.83	12.9	235.1	7.67	1.81	12.8	235.4
6.62	2.30	14.3	239.6	6.97	2.36	14.3	240.6	7.42	2.42	14.2	239.5
6.55	2.37	14.6	238.9	6.90	2.43	14.6	240.9	7.35	2.50	14.5	240.1
6.52	2.35	14.6	239.3	6.88	2.48	14.7	240.4	7.33	2.52	14.6	240.4
6.46	2.40	15.1	239.1	6.81	2.51	15.1	241.4	7.26	2.58	15.1	240.6
6.42	2.42	15.4	239.8	6.77	2.52	15.4	241.1	7.21	2.62	15.4	241.1
6.39	2.41	15.6	240.0	6.74	2.57	15.7	242.4	7.18	2.64	15.7	240.9
6.32	2.41	16.0	239.7	6.67	2.52	16.0	240.8	7.11	2.71	16.1	240.5

The maximum overtopping rates (l/s/m) at Section-1 are given in Table 5-3

Table 5-3. Maximum Overtopping Rates (l/s/m) at Section 1 (Battered Sea Walls)

AEP	RP (yrs)	2020	2070	2120
0.50%	200	160	260	415

Section 2 (Slipway)

The typical structural details used for the overtopping calculations are shown in Figure 2-4. The wave-water level combinations extracted at the structure toe and the estimated overtopping rates are provided below in Table 5-4. Although the actual structure slope is in the order of 1 in 15, 1 in 10 slope has been considered for the study due to the limitations of the overtopping manual. Due to this uncertainty, the max. overtopping rates are represented in a set of range values and are on the conservative side as tabulated in Table 5-5.

Table 5-4. Hs & SWL Combinations at Structure Toe (Section-2; Slipway) 1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.76	8.5	234.5	7.21	0.77	8.3	232.3	7.67	0.76	8.1	233.3
6.85	1.16	11.0	235.6	7.21	1.17	10.9	231.7	7.67	1.14	10.9	232.3
6.85	1.28	11.2	235.8	7.21	1.30	11.2	232.0	7.67	1.27	11.1	232.6
6.85	1.76	12.5	237.3	7.21	1.80	12.4	233.3	7.67	1.82	12.4	233.6
6.85	1.93	13.1	238.4	7.21	1.99	13.0	234.7	7.67	2.05	13.0	235.3
6.62	2.17	14.3	239.1	6.97	2.33	14.3	240.1	7.42	2.49	14.3	238.5
6.55	2.19	14.6	239.4	6.90	2.35	14.6	240.9	7.35	2.52	14.6	238.9
6.52	2.18	14.6	239.9	6.88	2.36	14.7	240.6	7.33	2.54	14.7	238.8
6.46	2.19	15.1	239.9	6.81	2.36	15.1	241.0	7.26	2.56	15.1	239.2
6.42	2.20	15.4	240.2	6.77	2.34	15.4	240.8	7.21	2.57	15.4	239.4
6.39	2.19	15.7	240.7	6.74	2.37	15.7	240.8	7.18	2.58	15.7	239.3
6.32	2.18	16.0	240.8	6.67	2.32	16.0	241.2	7.11	2.58	16.1	240.6

The maximum overtopping rates (l/s/m) at Section-2 are given in Table 5-5.

Table 5-5. Maximum Overtopping Rates (l/s/m) at Section 2 (Slipway)

AEP	RP (yrs)	2020	2070	2120
0.50%	200	63-150	192-326	443-610

Section 3 (Granite Talus)

The typical structural details used for the overtopping calculations are shown in Figure 2-5. The wave-water level combinations extracted at the structure toe and the estimated overtopping rates are provided below in Table 5-6.

Table 5-6. Hs & SWL Combinations at Structure Toe (Section-3; Granite Talus) 1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.75	8.3	233.2	7.21	0.77	8.2	232.9	7.67	0.75	7.9	233.5
6.85	1.14	10.8	233.7	7.21	1.16	10.9	231.7	7.67	1.12	10.7	233.3
6.85	1.26	11.0	233.9	7.21	1.29	11.1	231.6	7.67	1.24	10.9	233.4
6.85	1.76	12.4	235.2	7.21	1.82	12.4	232.2	7.67	1.78	12.3	234.1
6.85	1.96	13.0	236.4	7.21	2.04	13.1	233.5	7.67	2.03	13.0	235.4
6.62	2.29	14.3	238.1	6.97	2.44	14.3	237.9	7.42	2.61	14.4	237.3
6.55	2.32	14.6	238.3	6.90	2.47	14.6	238.3	7.35	2.66	14.6	236.9
6.52	2.33	14.7	238.4	6.88	2.48	14.7	238.3	7.33	2.69	14.7	237.0
6.46	2.35	15.1	238.4	6.81	2.48	15.1	239.3	7.26	2.72	15.2	237.2
6.42	2.36	15.4	238.6	6.77	2.48	15.4	239.7	7.21	2.74	15.5	237.1
6.39	2.37	15.7	238.9	6.74	2.52	15.7	239.9	7.18	2.75	15.8	236.8
6.32	2.37	16.1	238.9	6.67	2.48	16.1	239.8	7.11	2.79	16.2	237.2

The maximum overtopping rates (l/s/m) at Section-3 are given in Table 5-7.

Table 5-7. Maximum Overtopping Rates (l/s/m) at Section 3 (Granite Talus)

AEP	RP (yrs)	2020	2070	2120
0.50%	200	17	44	124

Section 4 (Terrace Blocks)

The typical structural details used for the overtopping calculations are shown in Figure 2-6. The wave-water level combinations extracted at the structure toe and the estimated overtopping rates are provided below in Table 5-8.

Table 5-8. Hs & SWL Combinations at Structure Toe (Section-4; Terrace Blocks) 1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.82	8.8	240.6	7.21	0.82	8.7	241.2	7.67	0.83	8.7	240.8
6.85	1.24	10.6	240.2	7.21	1.24	10.7	240.9	7.67	1.24	10.8	241.4
6.85	1.35	11.0	240.5	7.21	1.35	10.9	241.1	7.67	1.35	10.9	241.7
6.85	1.78	12.1	241.6	7.21	1.78	12.0	242.1	7.67	1.78	12.0	243.3
6.85	1.94	12.7	242.4	7.21	1.94	12.6	242.9	7.67	1.94	12.6	244.4
6.62	2.02	14.0	244.1	6.97	2.04	14.0	243.9	7.42	2.19	13.8	246.5
6.55	1.98	14.3	245.7	6.90	2.01	14.3	244.2	7.35	2.17	14.0	246.7
6.52	1.96	14.4	246.0	6.88	2.00	14.3	244.1	7.33	2.19	14.1	246.1
6.46	1.91	14.8	246.6	6.81	2.22	14.8	244.7	7.26	2.42	14.5	246.6
6.42	1.86	14.9	247.0	6.77	2.19	15.1	245.3	7.21	2.39	14.7	246.9
6.39	1.84	15.2	247.5	6.74	2.17	15.4	245.5	7.18	2.37	15.0	247.3
6.32	1.77	15.4	248.5	6.67	2.12	15.7	246.7	7.11	2.32	15.2	247.6

The maximum overtopping rates (l/s/m) at Section-4 are given in Table 5-9.

Table 5-9. Maximum Overtopping Rates (l/s/m) at Section 4 (Terrace Blocks)

AEP	RP (yrs)	2020	2070	2120
0.50%	200	8	19	67

Section 5 (Rock Armour)

The typical structural details used for the overtopping calculations are shown in Figure 2-7 . The wave-water level combinations extracted at the structure toe and the estimated overtopping rates are provided below in Table 5-10.

Table 5-10. Hs & SWL Combinations at Structure Toe (Section-5; Rock Armour) 1:200yr for 240°

2020 scenario				2070 scenario				2120 scenario			
SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)	SWL (mAOD)	Hmo (m)	Tm (sec)	Wave Direction (degN)
6.85	0.76	8.0	234.4	7.21	0.77	7.8	234.5	7.67	0.77	7.8	234.3
6.85	1.10	10.3	234.7	7.21	1.10	10.3	234.4	7.67	1.10	10.3	234.2
6.85	1.20	10.6	234.9	7.21	1.20	10.5	234.5	7.67	1.21	10.5	234.3
6.85	1.64	11.9	235.8	7.21	1.64	11.9	235.4	7.67	1.64	11.9	235.1
6.85	1.82	12.6	236.4	7.21	1.82	12.6	236.3	7.67	1.83	12.6	236.2
6.62	2.30	14.1	237.4	6.97	2.33	14.1	237.6	7.42	2.37	14.1	238.4
6.55	2.39	14.4	237.8	6.90	2.43	14.4	238.4	7.35	2.48	14.4	238.8
6.52	2.42	14.5	238.1	6.88	2.46	14.5	238.4	7.33	2.51	14.5	238.8
6.46	2.48	15.0	238.2	6.81	2.54	15.0	238.9	7.26	2.61	15.0	239.8
6.42	2.53	15.3	238.5	6.77	2.59	15.3	239.4	7.21	2.67	15.3	240.3
6.39	2.56	15.6	239.0	6.74	2.64	15.6	239.9	7.18	2.70	15.6	240.5
6.32	2.61	16.0	238.9	6.67	2.72	16.0	241.0	7.11	2.77	16.0	240.6

The maximum overtopping rates (l/s/m) at Section-5 are given in Table 5-11.

Table 5-11. Maximum Overtopping Rates (l/s/m) at Section 5 (Rock Armour)

AEP	RP (yrs)	2020	2070	2120
0.50%	200	20	41	84

From the above overtopping calculations undertaken for the existing scenario, it is observed that the existing crest level of the structure shall not be sufficient to mitigate storm events with 1 in 200-year return period for the present day, 2070 and 2120. Following this, a sensitivity study has been undertaken to estimate the increase in the crest level of the existing structures to accommodate the potential flooding scenarios. The effect of the proposed secondary wall has not been considered in the overtopping calculations, however, a high-level engineering judgement has been provided to indicate the potential effectiveness of having secondary wall at the lee side of the primary defences.

Table 5-12. Maximum Overtopping Rates (l/s/m) at Section 1 (Battered Sea Walls) for different crest levels

Section 1	2020	2070	2120	With addition of secondary wall	Summary
Existing – 9.07 mAOD	160	260	415	Will not be able manage the overtopping as a standalone option	High OT rates, SoP not achievable
9.57mAOD (+0.5m increase)	101	170	277	Will not be able manage the overtopping as a standalone option	High OT rates, SoP not achievable
10.27mAOD (+1.2m increase)	57	95	157	Combined with primary wall raising will help minimise risk	Moderate OT rates, SoP may be achievable with raised land behind
10.57mAOD (+1.5m increase)	45	75	123	Combined with primary wall raising will help minimise risk	Moderate OT rates, SoP may be achievable with raised land behind

Table 5-13. Maximum Overtopping Rates (l/s/m) at Section 2 (Slipway) for different crest levels

Section 2	2020	2070	2120	Summary
Existing – 8.21 mAOD	63-150	192-326	443-610	High OT rates, SoP not achievable
8.71mAOD (+0.5m flood gate)	18-66	74-169	189-348	High OT rates, SoP not achievable
10.27mAOD (+1.2m increase)	3-20	17-61	52-145	Moderate OT rates, SoP may be achievable with raised land behind
10.57mAOD (+1.5m increase)	2-11	9-38	29-98	Moderate OT rates, SoP may be achievable with raised land behind

Table 5-14. Maximum Overtopping Rates (l/s/m) at Section 3 (Granite Talus) for different crest levels

Section 3	2020	2070	2120	With addition of secondary wall	Summary
Existing – 9.71 mAOD	17	44	124	1m high wall may manage the overtopping	Moderate OT rates, SoP achievable
10.21mAOD (+0.5m increase)	6	19	62	1m high wall may manage the overtopping	Moderate OT rates, SoP achievable
10.91mAOD (+1.2m increase)	2	6	22	NA	Low OT rates, SoP achievable
11.21mAOD (+1.5m increase)	1	4	14	NA	Low OT rates, SoP achievable

Table 5-15. Maximum Overtopping Rates (l/s/m) at Section 4 (Terrace Blocks) for different crest levels

Section 4	2020	2070	2120	With addition of secondary wall	Summary
Existing – 9.71 mAOD	8	19	67	1m high wall may manage the overtopping	Moderate OT rates, SoP achievable
10.21mAOD (+0.5m increase)	3	7	30	NA	Moderate OT rates, SoP achievable
10.91mAOD (+1.2m increase)	0.5	2	9	NA	Low OT rates, SoP achievable
11.21mAOD (+1.5m increase)	0.2	1	5	NA	Low OT rates, SoP achievable

Table 5-16. Maximum Overtopping Rates (l/s/m) at Section 5 (Rock Armour) for different crest levels

Section 5	2020	2070	2120	With addition of secondary wall	Summary
Existing – 9.71 mAOD	20	41	84	1m high wall may manage the overtopping	Moderate OT rates, SoP achievable
10.21mAOD (+0.5m increase)	9	19	41	1m high wall may manage the overtopping	Moderate OT rates, SoP achievable
10.91mAOD (+1.2m increase)	3	6	13	NA	Low OT rates, SoP achievable
11.21mAOD (+1.5m increase)	2	4	8	NA	Low OT rates, SoP achievable

6. Flood modelling

6.1 Modelling Approach and Software

TUFLOW modelling software was used to simulate wave overtopping discharge within a 2D model representation of Jersey. This was undertaken TUFLOW version 2020-01-AB. TUFLOW is a two-dimensional (2D) hydraulic modelling software that simulates the hydrodynamic behaviour of water using a grid-based approach. TUFLOW allows hydraulic modelling of surface water flows by applying discharge-time (QT) boundary conditions to the model grid catchment at specific locations. In this instance QT boundaries would be applied along defence locations where overtopping rates have been calculated for a specific joint probability wave event.

6.2 2D Model-Topography

The underlying topographical data is comprised of a composite Digital Terrain Model (DTM) with a 1m grid resolution sourced from the 1m LiDAR supplied by GoJ. The LiDAR survey was undertaken in 2017. The 2D TUFLOW model was set up with a grid resolution of 2m.

6.3 Manning's Roughness Coefficient ('n')

Spatial variations of land cover within the model domain have been defined using JsyData_Polygons provided by GoJ. The data categories used throughout the model to define appropriate Manning's Roughness Coefficients are shown below in Table 6-1.

Table 6-1. Manning's n coefficients applied in the inundation model.

Surface	'n'
Building	0.3
Roads and Paved Areas	0.025
Grass	0.06
General Surface	0.03

6.4 Overtopping Rates

The maximum overtopping discharge rates (Qmax) and the water profiles mentioned in Section 5 have been used in the inundation modelling. The overtopping rates entered into the model were obtained by linearly interpolating between zero overtopping to Qmax, returning to zero overtopping, between the start and end of the simulation.

The overtopping rates are applied to the model as a localised discharge-time boundary. The overtopping rates are calculated in l/s/m for each defence, whereas the rates are converted to model compatible units, ready for application within the model in m3/s per grid cell.

6.5 Model Timestep and Simulation Duration

The model timestep was set to 1s, to be half of the model grid size. The peak overtopping for all defences occurs at 3hr into the simulation. The duration of the simulation is 8hr to allow the overtopping water to propagate throughout the model.

6.6 Limitations

No representation of drainage or water egress at defences or water ingress at slipways has been included for this project. The model is a stand-alone assessment of overtopping inundation at defences for St Helier and does not consider water interactions from any other sources including overtopping inundation from neighbouring defences.

6.7 Options Modelled and Results

The existing coastal defence structures were included in the inundation model and the OT boundaries for different return period scenarios (2020,2070, and 2120) have been applied to 07-D17 and the 5 no. of the sections considered for the study.

For a 50-year design scenario, i.e. 2070, the flooding extent demonstrates the areas of low topography behind the defence and the likely flood flow paths / ponding. Most of this is adjacent to the defences but Victoria Avenue and parts St Helier are shown to be at risk of flooding without mitigation measures.

For the area of interest, the flood extent is generally confined to the immediate area behind defences, with a few exceptions where the park is located and along Victoria Avenue towards town. Once the revised masterplan is finalised, the updated flood extent can be mapped and included.

The flood extent for 2020, 2070 and 2120 are included in Appendix B.

Figure 6-1: Areas of Overtopping Flooding for a 1:200SoP (2070) event (Existing Coastal Defence Structures)

7. Structural Assessment

The structural stability of Section-4 (Terrace blocks) and Section-5 (Rock armour) has been checked to understand the potential feasibility of raising the existing crest wall level by 1.2m, i.e.10.91mAOD. Through dialogue with Gillespies and JDC, the backfill level has been proposed to be raised up to 9.81mAOD, to accommodate sea view and landscape requirements.

At this stage, it is envisaged that the existing parapet front wall to be chipped off and new parapet wall with raised crest levels to be constructed.

A typical detail of the proposed new parapet wall is shown below in Figure 7-1.

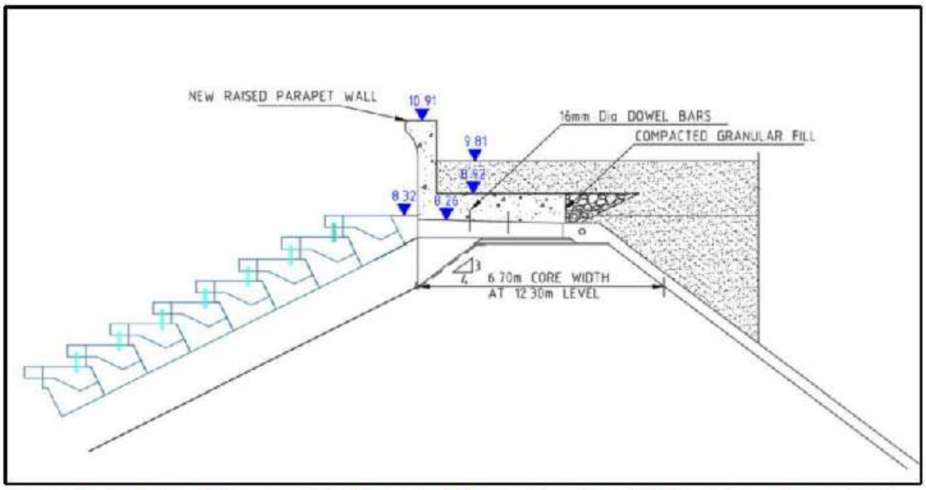


Figure 7-1: Raised Parapet Wall Typical Cross-Section (All levels are in mAOD)

7.1 Structural Stability

The design assumptions and basis of design mentioned in the "First Tower to West Park – Coastal Defence Scheme Outline Design Report (2020)" have been used in the present study.

The different design scenarios that have been considered for the study are "Normal Operation" and "Storm" scenario. The normal operation scenario loads are vertical loads, hence not posing any risk of sliding or overturning.

- **Section 4 (Terrace Blocks)**

Overturning and Sliding

Wave loads, for the storm scenario, as defined in the design approach, may vary significantly along the frontage. Wave loads adopted for the design section are, according to Goda (1974) and a significant wave height (Hs) of 2.22m - AEP 0.5% in 2070 (Table 5-8):

- Overturning moment: 55.32kNm/m
- Horizontal Load: 54.89kN/m

The structure remains stable for the given loads for overturning with a FoS greater than 1. In order to mitigate the risk of sliding, dowel bars (16mm dia) have been proposed. Other potential engineering solutions could also be considered during design, including increasing the promenade thickness, as required.

Structural Resilience of the Parapet

This parapet will be subject to shear and bending moment due to wave action, so wave loading according to Goda and a significant wave height (Hs) of 2.22m, results in:

- Shear at the base of the parapet: 38.21kN/m
- Bending Moment at the base of parapet: 30.29kNm/m

The parapet remains in the order of a 35% utilisation factor for both loads and is considered acceptable.

- **Section 5 (Rock Structures)**

Overturning and Sliding

Wave loads, for the storm scenario, as defined in the design approach, may vary significantly along the frontage. Wave loads adopted for the design section are, according to Goda (1974) with a significant wave height (Hs) of 2.72m - AEP 0.5% in 2070 (Table 5-10):

- Overturning moment: 73.06kNm/m
- Horizontal Load: 69.51kN/m

The structure remains stable for the given loads for overturning with a FoS greater than 1. In order to mitigate the risk of sliding, dowel bars (16mm dia) have been proposed. Other potential engineering solutions could also be considered during design, including increasing the promenade thickness, as required.

Structural Resilience of the Parapet

This parapet will be subject to shear and bending moment due to wave action, so wave loading according to Goda with a significant wave height (Hs) of 2.72m, results in:

- Shear at the base of the parapet: 49.77kN/m
- Bending Moment at the base of parapet: 24.21kNm/m

The parapet remains in the order of a 40% utilisation factor for both loads and is considered acceptable.

7.2 Global Stability

The global/slope stability of the revised sections with 9.81mAOD level backfill level has been undertaken using Slope W software. The design assumptions made used for the study are provided below:

- No ground investigation data available and the material properties assumed for the study are given below:

Name	Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
Concrete	High Strength	22				1
Fill	Mohr-Coulomb	20	0	30	0	1
Sand	Mohr-Coulomb	20	0	30	0	1
Stone Rubble Core	Mohr-Coulomb	22	0	40	0	1

Figure 7-2: Material Properties Assumed

- For the purpose of the assessment, soil underlying the coastal defences was assumed to be sand.
- Water Level assumed to be at beach level in each case analysed. Sensitivity analysis was carried out and the results were deemed to be acceptable.
- Geometry has been simplified and only typical cross sections were checked.
- Variable (i.e. traffic) characteristic surcharge of 10kpa has been applied on the retained side.

The summary of the study and the results are provided in Appendix A.

Based on the assumed material properties and the simplified geometry, the proposed works do not result in unacceptable Factors of Safety against global failure (i.e. sliding & slope stability). These results are only valid for outline masterplan consideration purposes only. Further calculations based on site-specific GI data are required in order to draw firm conclusions regarding the safety and feasibility of the development proposals.

8. Summary

The wave overtopping risk associated with the existing coastal defence structures has been evaluated by considering 5 no. of typical sections along the coastline as shown in Figure 2-2.

Overtopping rates were estimated for each of the 5 no. of sections using EurOtop Manual (2018). Based on the estimated overtopping rates, it is proposed to raise the existing walls by 1.2m in order to provide a 1:200 yr Standard of Protection to the development.

Additional raising and/or secondary defence measures are however likely to be required around the bunker / slipway to provide the required (SoP). Further development and appraisal work to confirm preferred defence options in this area is currently ongoing.

The structural and global stability of the selected sections along the coastline has been undertaken to understand the structural feasibility of the revised parapet walls with a 1.2m high crest level when compared to the existing levels.

The study shows that the new parapet wall structures are feasible and further modifications to the structure can be considered during the subsequent detailed design stage, as required.

8.1 Recommendations

Following the finalisation of the masterplan, the inundation model will need to be revised to include the updated coastline configuration and levels. Additionally, the details of any temporary works to be considered for any future development plans shall also be included.

9. References

AECOM (2020). First Tower to West Park -Coastal Defence Scheme Outline Design Report (Revision-5).

AECOM (2019a). Jersey Shoreline Climate Resilience Management Plan: Hydraulic Modelling Report: Wave Model Calibration (Appendix F).

AECOM (2019b). Jersey Shoreline Climate Resilience Management Plan: Hydraulic Modelling Report: Wave Transformation and Overtopping Modelling (Appendix G).

EurOTop (2018). Manual on wave overtopping of sea defences and related structures (Second Edition 2018)

Prime (2018). Jersey sea level and coastal conditions climate review. National Oceanography Centre.

EA/Defra (2005). Use of Joint Probability Methods in Flood Management: A Guide to Best Practice – R&D Technical Report FD2308/TR2, 2005).

HR Wallingford (2009). The effects of climate change on Jersey's Coastal defence structures. Report EX5964, Release 6.0.

HR Wallingford (1991), Jersey Coastal Management Study. HR Wallingford Report EX2490, December 1991.

Appendix A – Global Stability Calculations

CALCULATION SHEET

AECOM

Project:	South West St Helier	**Rev' No:**	**Originator:**	**Checker:**	**Reviewer:**	**Verifier:**	**Date:**
Project number:	60650295 - St Helier Waterfront Phase 1	01	AN	CTF	CTF	DD	10/03/21
Design Element:	Global Stability Assessment						
Document Number:							
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Purpose of calculation

Assess global stability of existing costal defences, under the client's development proposals.
Undertake slope stability calculations and sliding checks for three cross-sections based on available geometry drawings.

Design approach

Sliding Check without partial factors - Global FoS approach
Slope stability check SLS
Slope stability check DA1-C1 as per EC7
Slope stability check DA1 C2 as per EC7

Sliding check carried out using spreadsheet calculations.
Slope Stability analyses carried out using the software Slope/W 2021.

Main assumptions

No ground investigation data available. Material properties have been assumed as presented in the relevant calculation pages.
Soil underlying the coastal defences was assumed to be sand.
Water Level assumed to be at beach level in each case analysed. Sensitivity analysis were carried out and the results were deemed to be acceptable.
Geometry has been simplified and only typical cross sections were checked.
Variable (i.e. traffic) Characteristic surcharge of 10kpa has been applied on the retained side.

Conclusions:

Based on the assumed material properties and the simplified geometry, the proposed works do not results in unacceptable Factors of Safety against global failure (i.e. sliding & slope stability).
These results are only valid for masterplan consideration purposes only. Further calculations, based on site-specific GI data are required in order to draw safe conclusions regarding the safety of the development proposals.

References

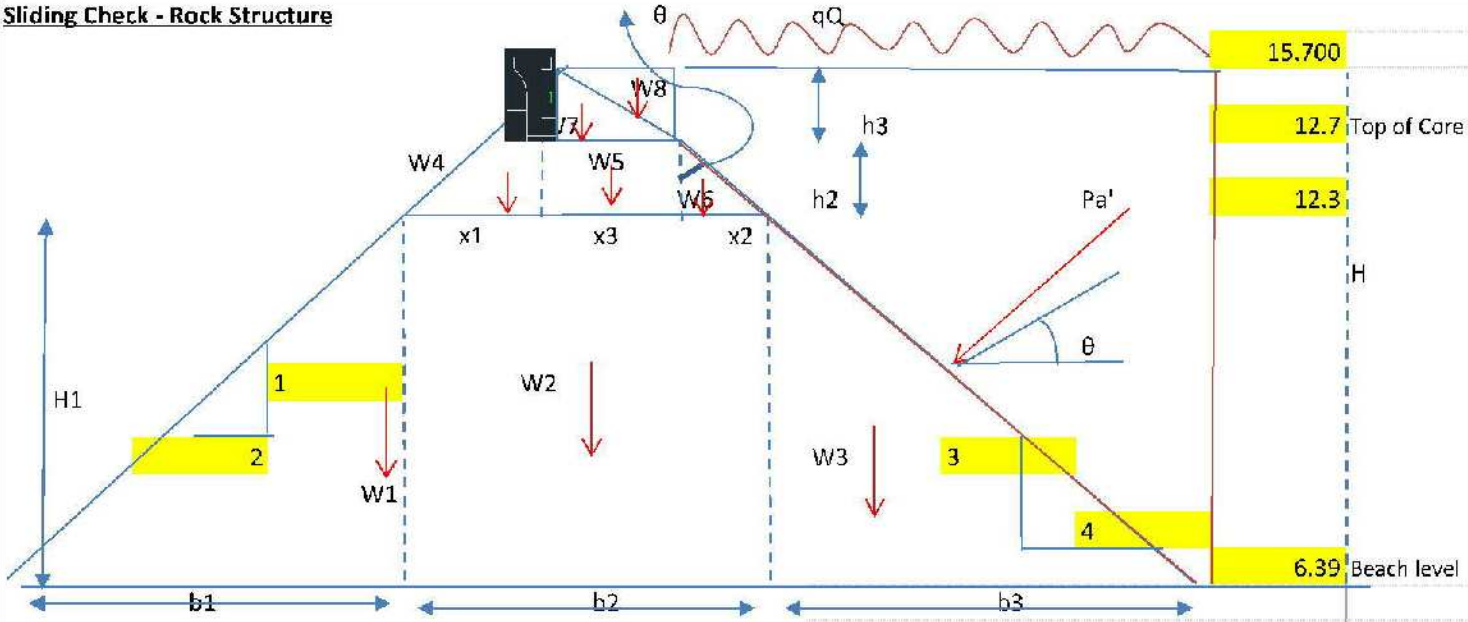
Available drawings of existing coastal defences and proposed Finished Ground Levels.

CALCULATION SHEET

AECOM

Project:	South West St Helier	**Rev' No:**	**Originator:**	**Checker:**	**Reviewer:**	**Verifier:**	**Date:**
Project number:	60650295 - St Helier Waterfront Pha:	01	AN	CTF	CTF	DD	10/03/21
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Sliding Check - Rock Structure



b2= 6.70

Parameters

Fill bulk density	γkstone	22	kN/m3	15.70	mCD	9.81	m OD
Concrete bulk density	γc.k	22	kN/m3	6.3900	mCD	0.50	m OD
Fill bulk density	γk	20	kN/m3				
Unit weight of water	γw	10	kN/m3				
internal friction angle	φ'	40	°				
Critical backfill friction angle	φb;cv	40	°				
internal friction angle	φ'	30	°				
Critical underlying soil friction angle	φu;cv	30	°				
Undrained shear strength	cu	0					
	δ/φ =	0.67					

Additional Loading

Characteristic Variable Surcharge	q	10	kPa

Sliding resistance

Favourable vertical action	2384.37	kN/m	Conservatively ignores the stabilising contribution of the backfill
Simplified sliding resistance	872.56	kN/m	Ignoring contribution from surcharge and backfill
Sliding Resistance	1491.82	kN/m	Taking contribution from surcharge and backfill

Thrust Horizontal

From backfill	Phad1=	432.52	kN/m
From surcharge	Phad2=	77.43	kN/m
Total horizontal thrust=	Pha=	509.94	kN/m

Sliding Resistance FoS	2.93	ok
Simplified sliding resistance FoS	1.71	ok

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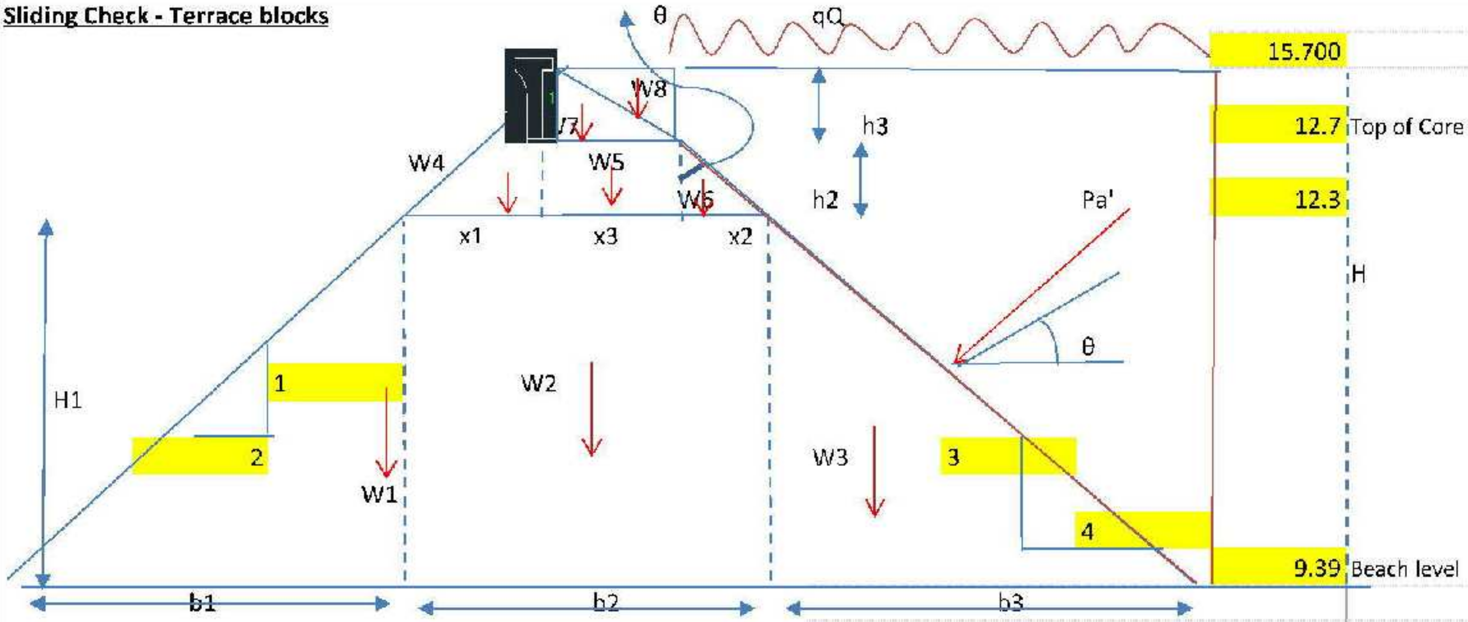
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Sliding Check - Terrace blocks



b2= 6.70

Parameters

Fill bulk density	γkstone	22	kN/m3	15.70	mCD	9.81	m OD
Concrete bulk density	γc.k	22	kN/m3	9.3900	mCD	3.50	m OD
Fill bulk density	γk	10	kN/m3				
Unit weight of water	γw	20	kN/m3				
internal friction angle	φ'	40	°				
Critical backfill friction angle	φb;cv	40	°				
internal friction angle	φ'	30	°				
Critical underlying soil friction angle	φu;cv	30	°				
Undrained shear strength	cu	0					
	δ/φ =	0.67					

Additional Loading

Characteristic Variable Surcharge q 10 kPa

Sliding resistance

Favourable vertical action	971.97	kN/m	Conservatively ignores the stabilising contribution of the backfill
Simplified sliding resistance	355.69	kN/m	Ignoring contribution from surcharge and backfill
Sliding Resistance	540.06	kN/m	Taking contribution from surcharge and backfill

Thrust Horizontal

From backfill	Phad1=	99.34	kN/m
From surcharge	Phad2=	52.48	kN/m
Total horizontal thrust=	Pha=	151.82	kN/m

Sliding Resistance FoS	3.56	ok
Simplified sliding resistance FoS	2.34	ok

Project:	South West St Helier	Rev' No:	Originator:	Checker:	Reviewer:	Verifier:	Date:
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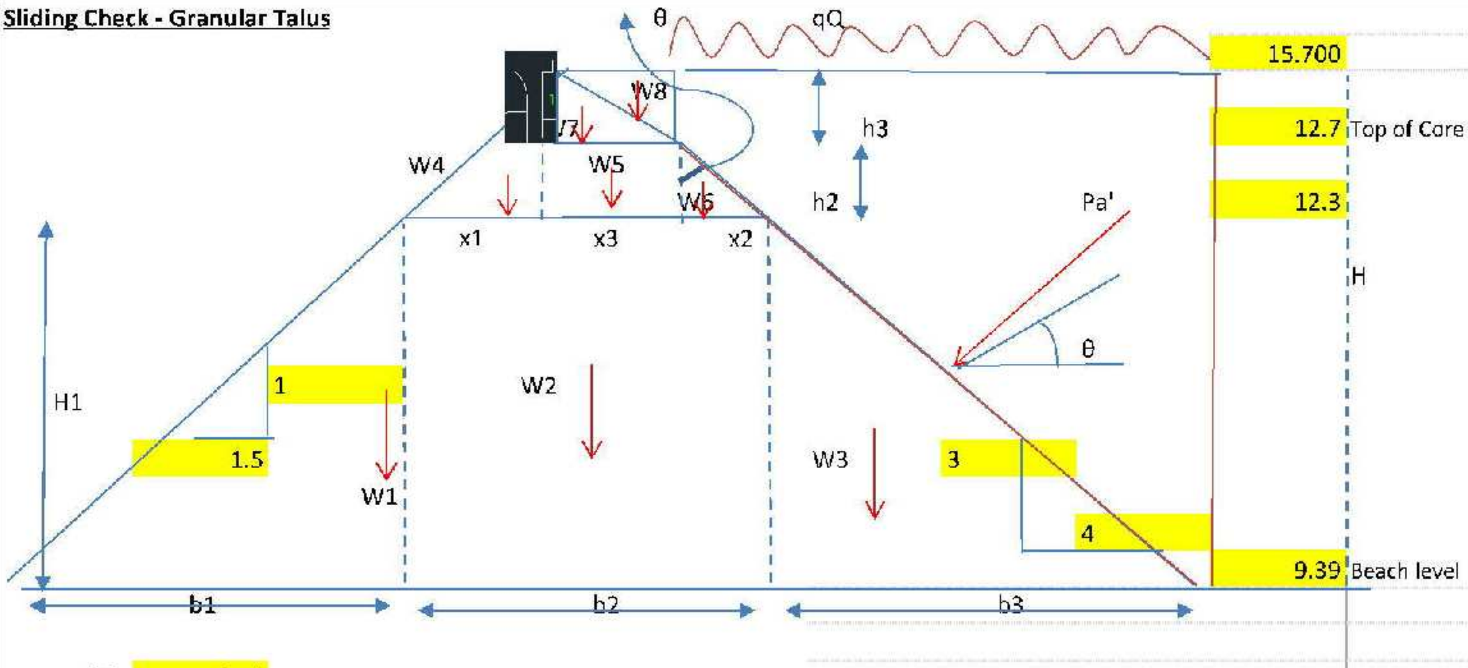
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Sliding Check - Granular Talus



b2= 6.70

Parameters

Fill bulk density	γkstone	22	kN/m3	15.70	mCD	9.81	m OD
Concrete bulk density	γc.k	22	kN/m3	9.3900	mCD	3.50	m OD
Fill bulk density	γk	20	kN/m3				
Unit weight of water	γw	10	kN/m3				
internal friction angle	φ'	40	°				
Critical backfill friction angle	φb;cv	40	°				
internal friction angle	φ'	30	°				
Critical underlying soil friction angle	φu;cv	30	°				
Undrained shear strength	cu	0					
	δ/φ =	0.67					

Additional Loading

Characteristic Variable Surcharge q 10 kPa

Sliding resistance

Favourable vertical action	932.88	kN/m	Conservatively ignores the stabilising contribution of the backfill
Simplified sliding resistance	341.38	kN/m	Ignoring contribution from surcharge and backfill
Sliding Resistance	646.39	kN/m	Taking contribution from surcharge and backfill

Thrust Horizontal

From backfill	Phad1=	198.68	kN/m
From surcharge	Phad2=	52.48	kN/m
Total horizontal thrust=	Pha=	251.16	kN/m

Sliding Resistance FoS	2.57	ok
Simplified sliding resistance FoS	1.36	ok

CALCULATION SHEET

AECOM

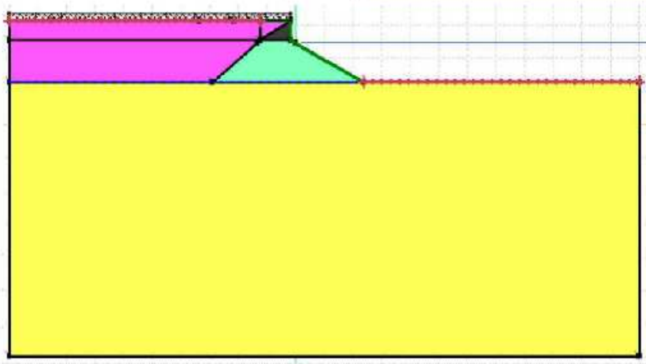
Project:	South West St Helier	Rev' No:	Originator:	Checker:	Reviewer:	Verifier:	Date:
Project number:	60650295 - St Helier Waterfront Pha:	01	AN	CTF	CTF	DD	10/03/21
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Slope Stability Check - Rock Structur

Geometry and parameters

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Concrete	High Strength	22				1
	Fill	Mohr-Coulomb	20	0	30	0	1
	Sand	Mohr-Coulomb	20	0	30	0	1
	Stone Rubble Core	Mohr-Coulomb	22	0	40	0	1

Characteristic Variable Surcharge: 10kPA



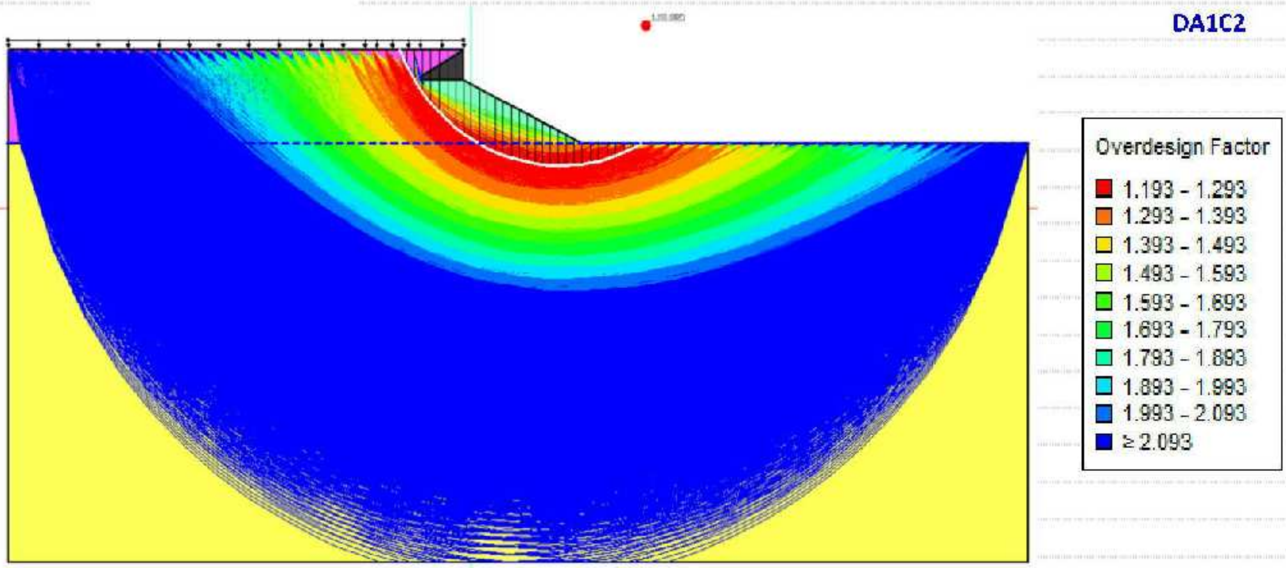
15.7mCD Top of Parapet lvl

12.7 mCD Top of Core lvl

6.39 Assumed Beach level

	Internal Stability	External Stability	
DA1C1	1.455	1.850	>1.0 OK
DA1C2	1.193	1.660	>1.0 OK
SLS	1.502	2.100	>1.3 OK

DA1C2



CALCULATION SHEET

AECOM

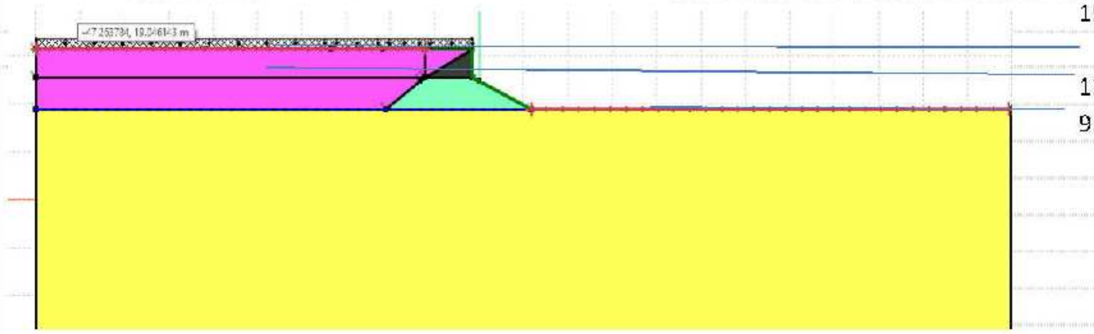
Project:	South West St Helier	Rev' No:	Originator:	Checker:	Reviewer:	Verifier:	Date:
Project number:	60650295 - St Helier Waterfront Pha:	01	AN	CTF	CTF	DD	10/03/21
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Slope Stability Check - Terrace Block

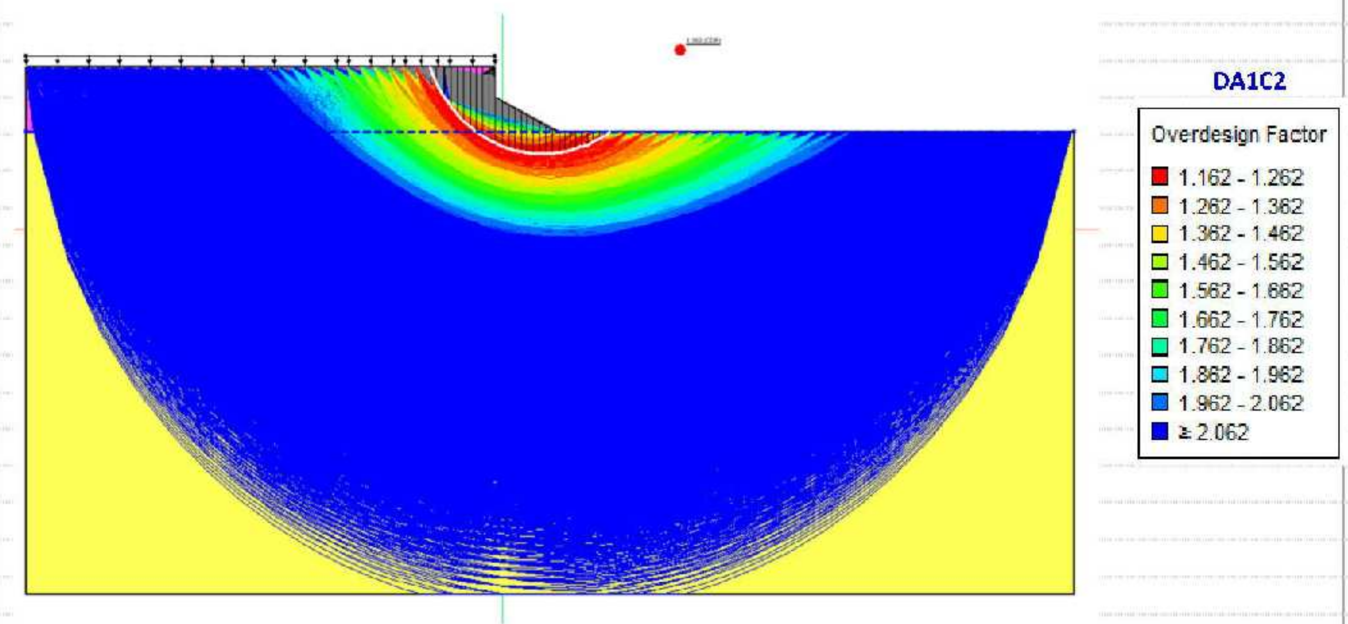
Geometry and parameters

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Concrete	High Strength	22				1
	Fill	Mohr-Coulomb	20	0	30	0	1
	Sand	Mohr-Coulomb	20	0	30	0	1
	Stone Rubble Core	Mohr-Coulomb	22	0	40	0	1

Characteristic Variable Surcharge: 10kPA



	Internal Stability	External Stability	
DA1C1	1.405	1.800	>1.0 OK
DA1C2	1.162	1.600	>1.0 OK
SLS	1.467	2.000	>1.3 OK



CALCULATION SHEET

AECOM

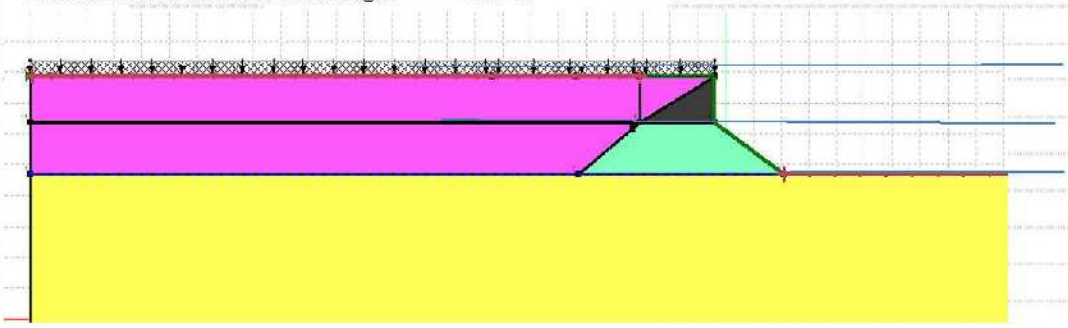
Project:	South West St Helier	Rev' No:	Originator:	Checker:	Reviewer:	Verifier:	Date:
Project number:	60650295 - St Helier Waterfront Pha:	01	AN	CTF	CTF	DD	10/03/21
Design Element:	Global Stability Assessment						
Document Number:	0						
Calculation Title:	**Global Stability assessment - Pre-Feasibility Stage**						

Slope Stability Check - Granular Talu

Geometry and parameters

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Phi-B (°)	Piezometric Line
	Concrete	High Strength	22				1
	Fill	Mohr-Coulomb	20	0	30	0	1
	Sand	Mohr-Coulomb	20	0	30	0	1
	Stone Rubble Core	Mohr-Coulomb	22	0	40	0	1

Characteristic Variable Surcharge: 10kPA



	Internal Stability	External Stability	
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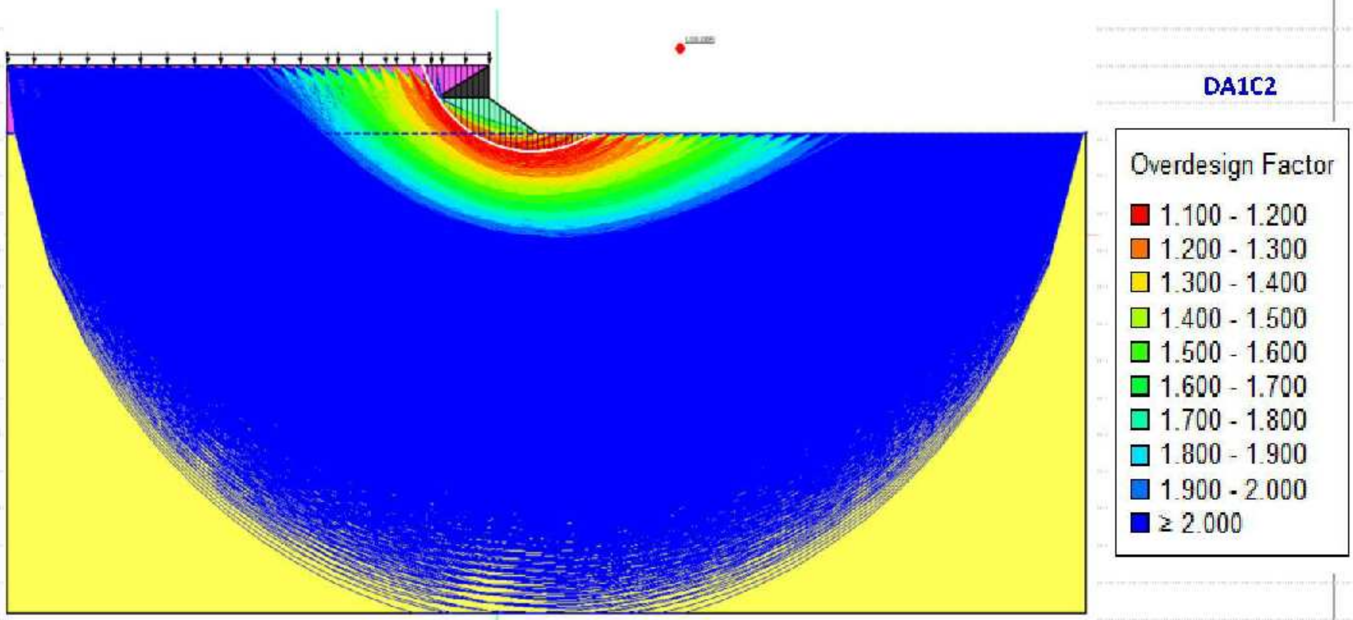
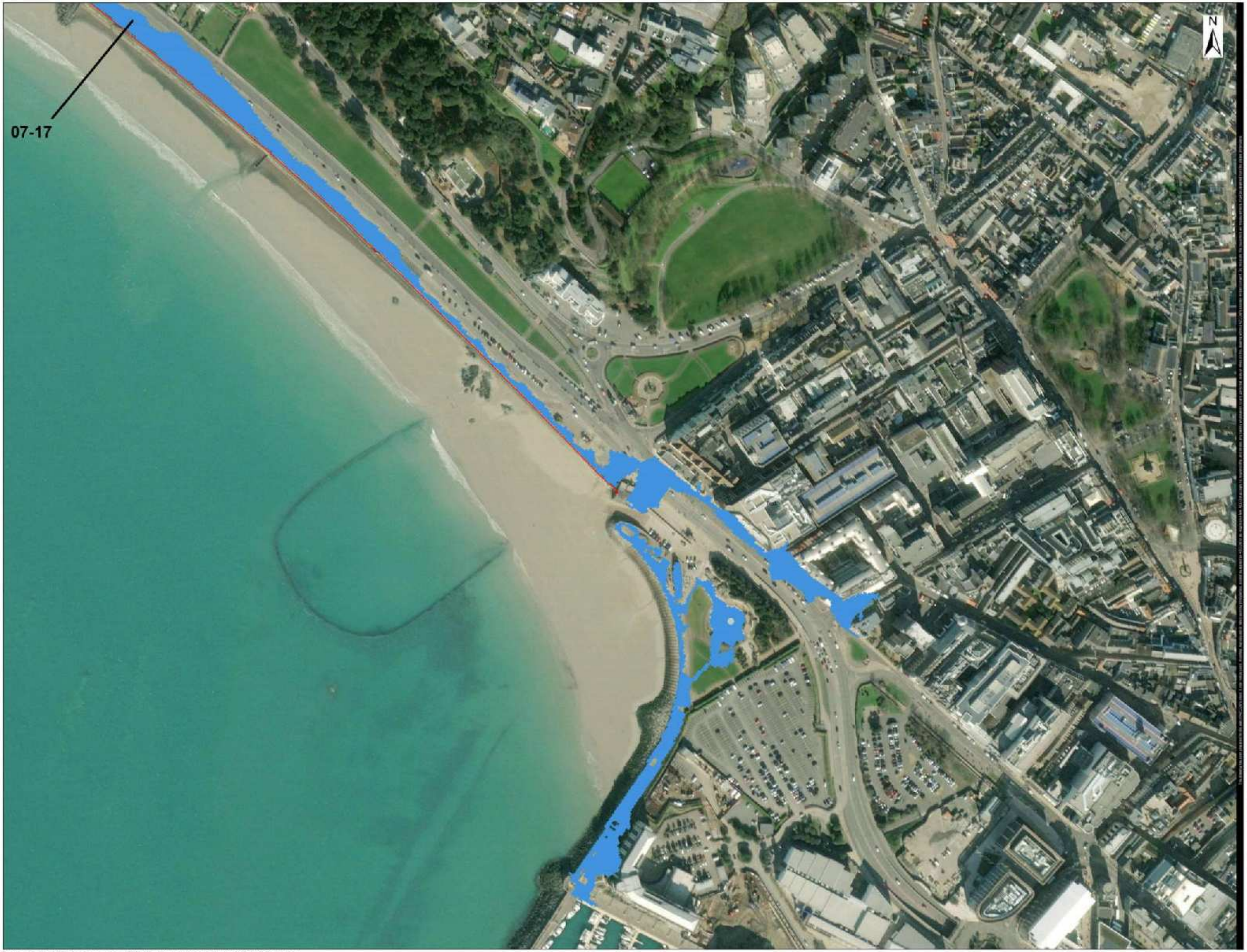
DA1C2	1.100	1.550	>1.0 OK
SLS	1.338	2.000	>1.3 OK



Appendix B – Inundation Model Results



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Project Title:

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT

Client:

Government of JERSEY

LEGEND

Defences Used in Overtopping Analysis

2020 200yr

Copyright:

AECOM Internal Project No:

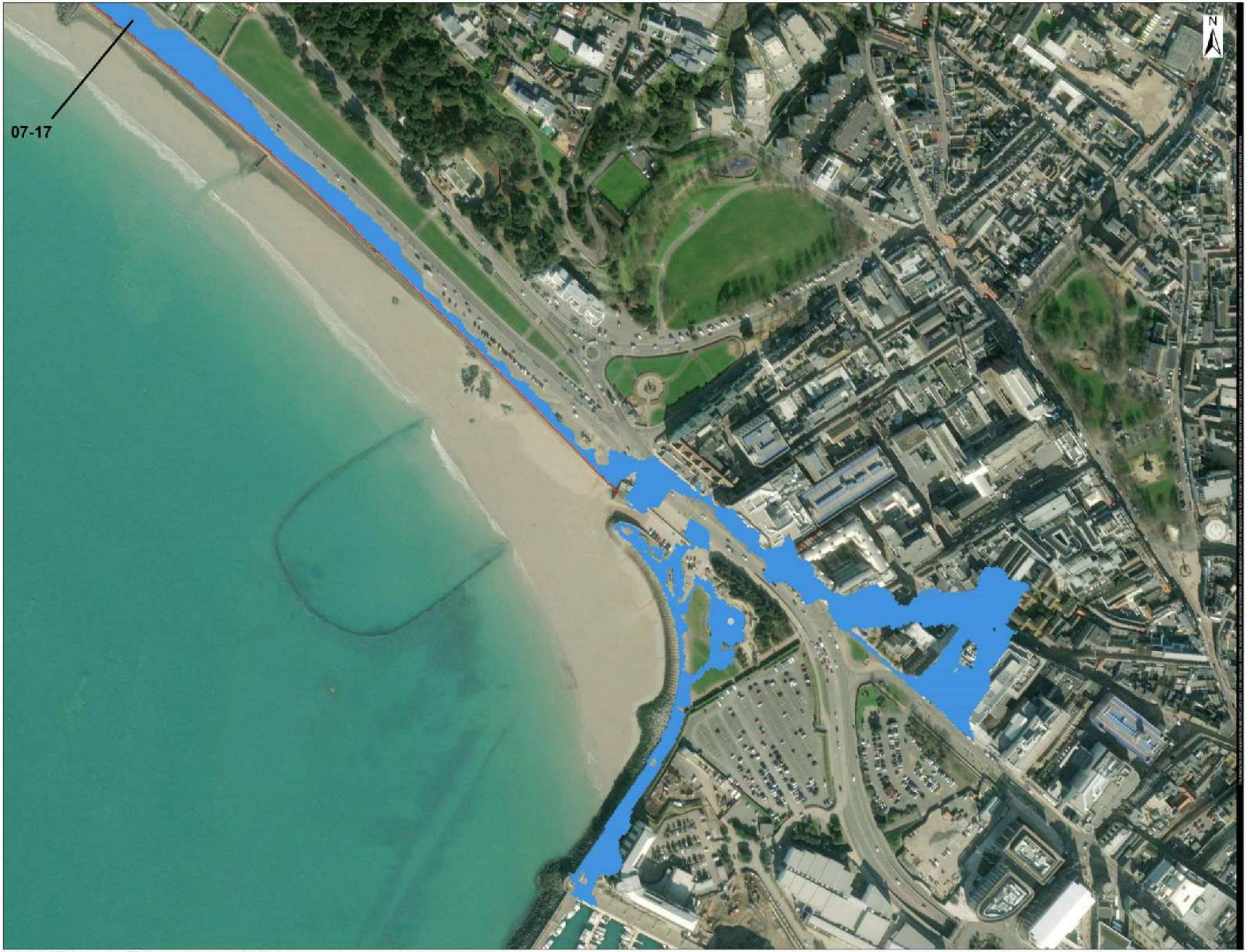
60541614

Drawing Title:

0.5% JOINT PROBABILITY AEP (1 IN 200 YR RETURN PERIOD) PROJECTED FOR 2020

Scale at A1: 1:1,500

Drawing No:			Rev:
			01
Drawn:	**Chk'd:**	**App'd:**	**Date:**
RH/BT	VD	JG	12/04/21



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Project Title:

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT

Client:

Government of JERSEY

LEGEND

Defences Used in Overtopping Analysis

2070 200yr

AECOM Internal Project No:

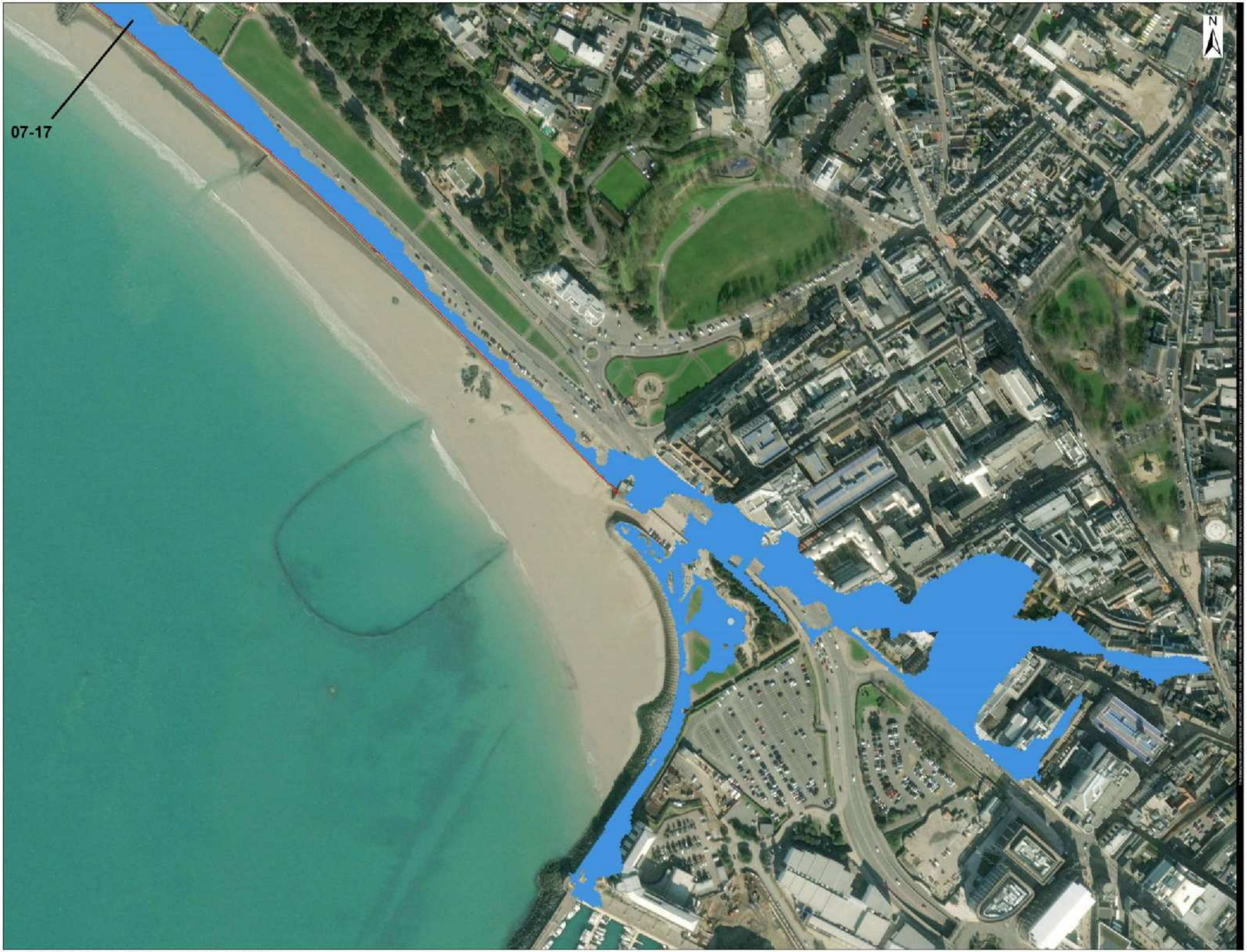
60541814

Drawing Title:

0.5% JOINT PROBABILITY AEP (1 IN 200 YR RETURN PERIOD) PROJECTED FOR 2070

Scale at A1: 1:1,500

Drawing No:			Rev:
			01
Drawn:	**Chk'd:**	**App'd:**	**Date:**
RH/BT	VD	JG	12/04/21



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Project Title:

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT

Client:

Government of JERSEY

LEGEND

Defences Used in Overtopping Analysis

2120 200yr

Copyright:

AECOM Internal Project No:

60541614

Drawing Title:

0.5% JOINT PROBABILITY AEP (1 IN 200 YR RETURN PERIOD) PROJECTED FOR 2120

Scale at A1: 1:1,500

Drawing No:	Rev:
	01

Drawn:	Chk'd:	App'd:	Date:
RH/BT	VD	JG	12/04/21

SAFETY, HEALTH & ENVIRONMENTAL INFORMATION

IN ADDITION TO THE HAZARDS/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING:

CONSTRUCTION

WORKS WILL TAKE PLACE WITHIN THE INTERTIDAL ZONE

BEACH DEPOSITS MAY BE LOOSE AND COULD THEREFORE LEAD TO CONSTRUCTION PLANT INSTABILITY.

MAINTENANCE/CLEANING

N/A

DECOMMISSIONING/DEMOLITION

N/A

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.

0 25 50 m

1:1000

PROJECT

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT CONCEPT DESIGN

CONSULTANT

AECOM Limited
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Hampshire
RG21 7PP
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www.aecom.com

NOTES

1. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED. DO NOT SCALE.
2. ALL LEVELS SHOWN ARE IN METRES AND mAOD UNLESS OTHERWISE NOTED.
3. REFER TO D1247-00-003-CL-C-01 FOR SITE LEVELS AND READING.

KEY PLAN

PROJECT NUMBER

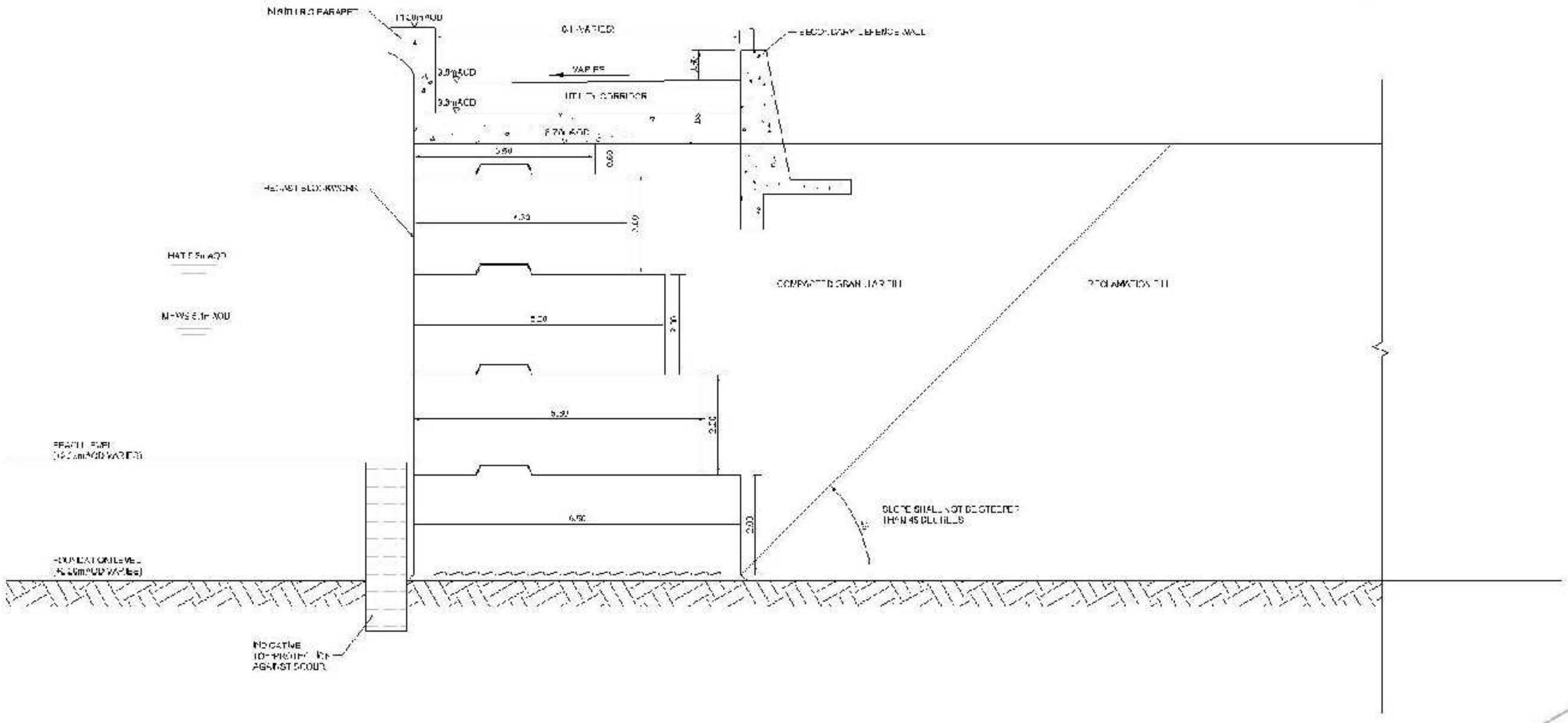
60650295

SHEET TITLE

PLAN

SHEET NUMBER

000



SAFETY, HEALTH & ENVIRONMENTAL INFORMATION

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CONSTRUCTION

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MAINTENANCE/CLEANING

N/A

DECOMMISSIONING/DEMOLITION

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PROJECT

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT CONCEPT DESIGN

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3. DETAILS ARE DEVELOPED TO A CONCEPT STAGE

KEY PLAN

PROJECT NUMBER

60650295

SHEET TITLE

CROSS SECTION 1

SHEET NUMBER

001

SAFETY, HEALTH & ENVIRONMENTAL INFORMATION

IN ADDITION TO THE HAZARDS/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING:

CONSTRUCTION

WORKS WILL TAKE PLACE WITHIN THE INTERTIDAL ZONE

BEACH DEPOSITS MAY BE LOOSE AND COULD THEREFORE LEAD TO CONSTRUCTION PLANT INSTABILITY.

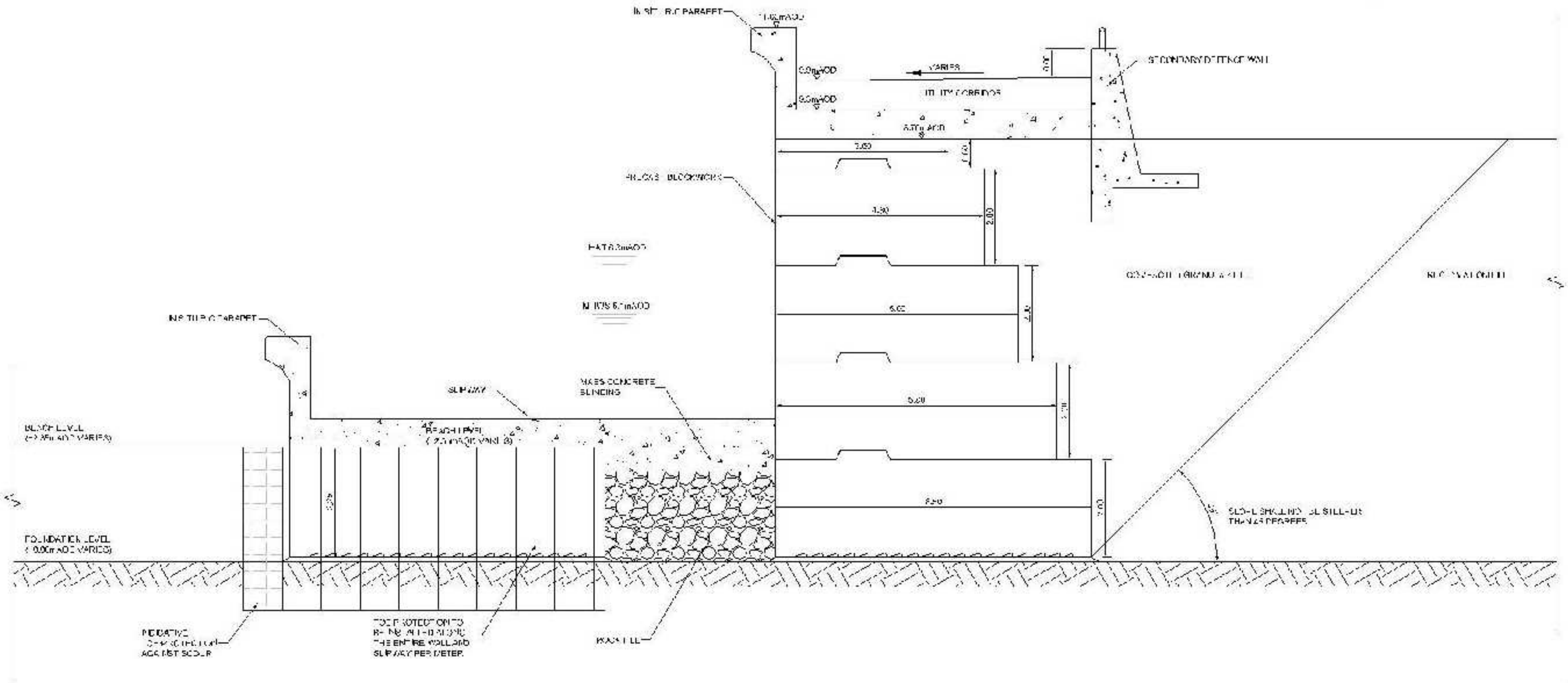
MAINTENANCE/CLEANING

N/A

DECOMMISSIONING/DEMOLITION

N/A

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.



NOT FOR CONSTRUCTION

1:50

PROJECT

SOUTH WEST ST HELIER WATERFRONT DEVELOPMENT CONCEPT DESIGN

CONSULTANT

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NOTES

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2. ALL LEVELS SHOWN ARE IN METRES AND IN AOD UNLESS OTHERWISE NOTED.
3. DETAILS ARE DEVELOPED TO A CONCEPT STAGE

KEY PLAN

PROJECT NUMBER

60650295

SHEET TITLE

CROSS SECTION 2

SHEET NUMBER

002

Filename: \\na.aecomnet.com\lfs\EMEA\Basingstoke-UKBAS1\Legacy\UKBAS1PFILW001\WPPi5 - Projects\60650295 - St Helier Waterfront, Jersey\900_CAD_GIS\920_GIS\Modelling Outputs_TG.aprx

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Project Title:

South West St Helier Waterfront Development Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Areas of Overtopping Flooding from a 0.5% AEP (1 in 200yr return period) projected for 2020

Final Master Plan

Copyright:

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

AREAS OF OVERTOPPING FLOODING FROM A 0.5% AEP (1 IN 200YR RETURN PERIOD) PROJECTED FOR 2020

Scale at A3: 1:6,000

Drawing No:	**Rev:**
	V1

Drawn:	**Chk'd:**	**App'd:**	**Date:**
RH/TG	VD	PC	26/08/21

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Project Title:

South West St Helier Waterfront Development Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Areas of Overtopping Flooding from a 0.5% AEP (1 in 200yr return period) projected for 2070

Final Master Plan

Copyright:

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

AREAS OF OVERTOPPING FLOODING FROM A 0.5% AEP (1 IN 200YR RETURN PERIOD) PROJECTED FOR 2070

Scale at A3: 1:6,000

Drawing No:	**Rev:**
	V1

Drawn:	**Chk'd:**	**App'd:**	**Date:**
RH/TG	VD	PC	26/08/21

Filename: \\na.aecomnet.com\lfs\EMEA\Basingstoke-UKBAS1\Legacy\UKBAS1PFILW001\WPP\5 - Projects\60650295 - St Helier Waterfront, Jersey\900_CAD_GIS\920_GIS\Modelling Outputs_TG.aprx

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www.aecom.com

Project Title:

South West St Helier Waterfront Development Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Areas of Overtopping Flooding from a 0.5% AEP (1 in 200yr return period) projected for 2120

Final Master Plan

Copyright:

Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS,

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

AREAS OF OVERTOPPING FLOODING FROM A 0.5% AEP (1 IN 200YR RETURN PERIOD) PROJECTED FOR 2120

Scale at A3: 1:6,000

Drawing No: **Rev:** V1

Drawn:	Chk'd:	App'd:	Date:
NB	VD	PC	26/08/21

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Project Title:

South West St Helier Waterfront Development Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Final Master Plan

Max. 2020 Flood Depth (m)

- < 0.05
- 0.05 - 0.10
- 0.10 - 0.50
- 0.50 - 1.00
- >1.00

Copyright:

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

FLOOD DEPTH
FROM A 0.5% AEP
(1 IN 200YR RETURN PERIOD)
PROJECTED FOR 2020

Scale at A3: 1:6,000

Drawing No: **Rev:** V1

Drawn:	Chk'd:	App'd:	Date:
RH/TG	VD	PC	26/08/21

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www.aecom.com

Project Title:

South West St Helier Waterfront Development
Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Final Master Plan

Max. 2070 Flood Depth (m)

- < 0.05
- 0.05 - 0.10
- 0.10 - 0.50
- 0.50 - 1.00
- > 1.00

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

FLOOD DEPTH
FROM A 0.5% AEP
(1 IN 200YR RETURN PERIOD)
PROJECTED FOR 2070

Scale at A3: 1:6,000

Drawing No: **Rev:** V1

Drawn:	Chk'd:	App'd:	Date:
RH/TG	VD	PC	26/08/21

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Project Title:

South West St Helier Waterfront Development Concept Design of Coastal Defences and EIA Support

Client:

JERSEY DEVELOPMENT COMPANY

Final Master Plan

Max. 2120 Flood Depth (m)

- < 0.05
- 0.05 - 0.10
- 0.10 - 0.50
- 0.50 - 1.00
- > 1.00

Issue/Revision:

I/R	Date	Description

AECOM Internal Project No:

60650295

Drawing Title:

FLOOD DEPTH
FROM A 0.5% AEP
(1 IN 200YR RETURN PERIOD)
PROJECTED FOR 2120

Scale at A3: 1:6,000

Drawing No: **Rev:** V1

Drawn:	Chk'd:	App'd:	Date:
RH/TG	VD	PC	26/08/21

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