



Appendix J5 – Overtopping Report

Langstone FCERM

Coastal Partners

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Quality information

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1. Introduction

AECOM has been commissioned by Coastal Partners to assist in design and implementation of the Langstone Flood and Coastal Erosion Risk Management (FCERM) Scheme.

The scheme, which will introduce new defences, is required to provide improved protection against tidal flooding in Langstone's village and the A3023 road link to Hayling Island.

For ease of reference and reporting, the scheme has been split into frontages, as can be seen in Figure 1-1.



Figure 1-1: Scheme alignment split by frontages. The core scheme is represented by solid lines and the additional scheme represented by dotted lines. (Coastal Partners, 2020)

The design commission covers the entire potential scheme. This is comprised of two combinations – a core scheme whereby Frontages 1A, 2, 3, 4 and 5 are implemented, and an additional scheme where Frontages 1B and 6 are also implemented.

Concerns have been raised over the residual risk of wave overtopping flooding to the properties of Mill Lane and Harbourside at the intersection between Frontages 1A and 1B, particularly if only the core scheme is implemented. In addition, potential for residual wave overtopping under combined extreme events has also been identified at the end of the high street and Royal Oak area. Further assessment and definition of onset and potential scale of residual risk was therefore required to inform the drainage strategy and confirm any addition design mitigation.

In outline stages of design, and detailed design, precautionary design assumptions were applied based on conservative fetch based waves heights and wave/water level combinations. In order to refine and improve confidence in the levels of residual risk and any required mitigation, and ensure designs are lean and efficient based in sound science,

Coastal Partners commissioned AECOM to carry out a joint probability and wave overtopping assessment at:

- the intersection between Frontage 1A and B to ascertain the residual risk of flooding and make recommendations on the defence design in this location.
- Frontage 3, 4 and 5 where potential residual wave overtopping risk has been identified to inform the drainage strategy and any secondary mitigation / property level protection may be required to address residual risk; and
- Frontage 6 to refine design assumption and calculation for rock sizing and revetment design.

2. Base Data

2.1 Topographical and defence information

A ground model of the interface section was created using topographical survey information provided by Havant Borough Council (HBC) and LIDAR data downloaded from the Channel Coastal Observatory (CCO, 2018). Existing and proposed defence geometry and crest levels was obtained from the HBC lidar survey and the AECOM scheme design drawings (50% design level) respectively.

2.2 Local Climate Considerations

In the absence of wave data within the harbour, a conservative fetch-limited approach was used to determine the wave parameters at the various frontages (USACE, 2002).

2.3 Wind Data

The extreme value analysis wind data product was purchased from Met Office and provide extreme wind speeds divided into four wind direction sectors. The extreme values are calculated from wind speeds and directions measured between 1992 and 2021 on Thorney Island and are shown in Table 2-1 for the various calculated return periods.

Table 2-1: Extreme value wind speeds per directional sector (Met Office)

Return Period (Years)	Directional Sector			
	90° - 130°	130° - 180°	180° - 240°	240° - 270°
1	10.8m/s	16.8m/s	18.6m/s	17.7m/s
5	11.9m/s	18.5m/s	19.6m/s	19.1m/s
10	12.8m/s	19.8m/s	20.4m/s	20.1m/s
20	13.7m/s	21.1m/s	21.2m/s	21.2m/s
50	14.8m/s	22.8m/s	22.3m/s	22.5m/s
100	15.6m/s	24.0m/s	23.1m/s	23.5m/s
200	16.4m/s	25.3m/s	23.8m/s	24.5m/s

2.4 Fetch Lengths

Fetch lengths were determined using GIS mapping of Langstone Harbour as seen in Figure 2-1. Whilst it is noted a drying area exists in the centre of this harbour, for Frontage 1a/2 interface, and Frontage 6, the fetch analysis disregarded this mass, with there being no guarantee of the landmass remaining stable over the design life of the scheme and the longest fetch lengths typically coinciding with the deeper harbour channel. The longest

fetch¹ was measured to be 6,650m in a south-westerly direction for the western part of the bay. For the eastern part (Frontage 5), a maximum fetch length of 3,260m was measured from an easterly direction.

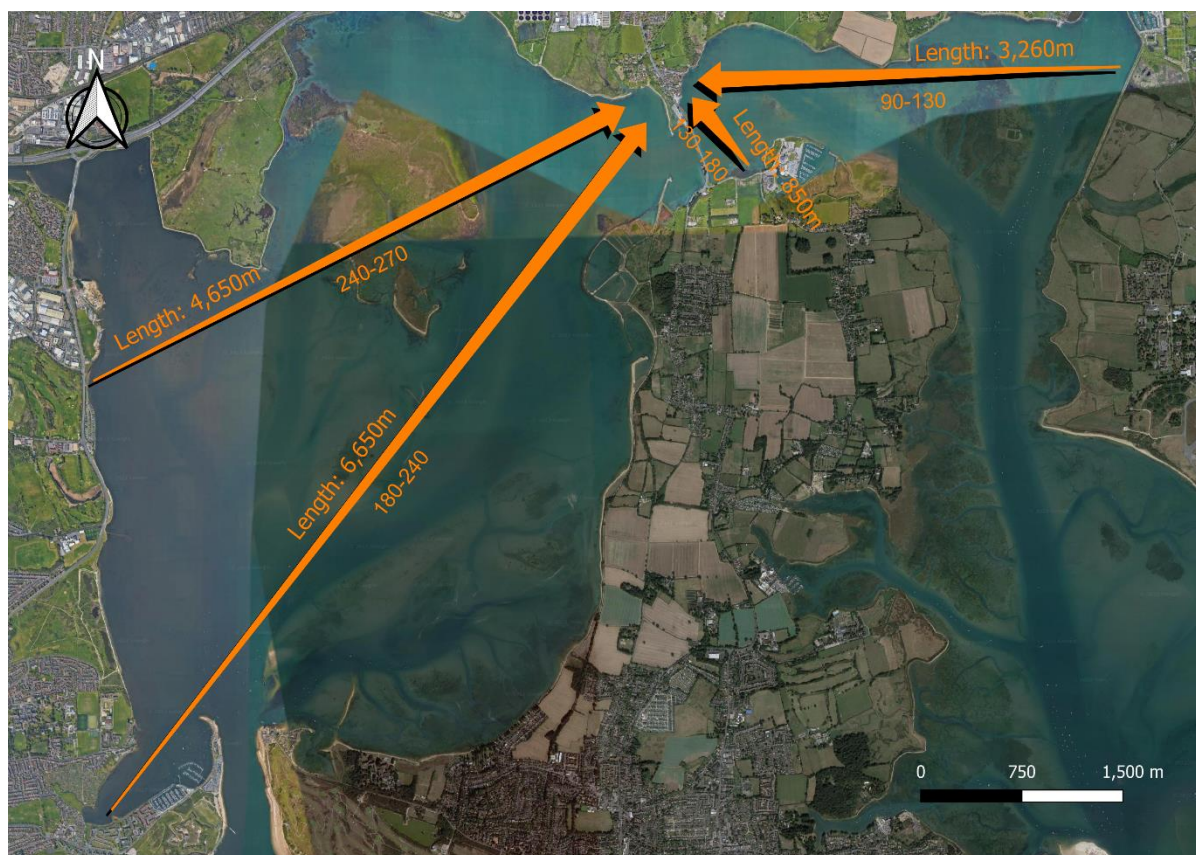


Figure 2-1: Fetch Lengths across Langstone Harbour.

2.5 Extreme Water Levels

The scheme provides protection against an extreme still water level event of 3.5m ODN, which equates approximately to a 1:200 yr (0.5% AEP) event at 2039. By the end of the scheme life, due to sea level rise, this Standard of Protection is anticipated to fall to a little below 1:20 yrs (5% AEP).

Therefore, when assessing the potential residual risk from overtopping from combined wave and water level events, the maximum water level applied in the joint probability is 3.5mODN (with the corresponding wave) as the SoP for the scheme in general is based on this design level.

The analysis therefore considers the calculated extreme water levels projected forwards from the year 2039 (mid scheme life) for a range of return periods². The extreme value analysis takes into account the UKCP18 (UK Climate Projections – Met Office) predicted sea level rise levels. The calculated extreme water levels, assuming the sea levels continue to rise, is shown in Table 2-2 (AECOM, 2022).

¹ The fetch length is the distance over which the wind blows. In general, for a semi-enclosed shallow embayment, a longer fetch length leads to a larger wind generated wave.

² The 2069 year sea level rise prediction by UKCP18 is applied to the present water levels and the extreme value calculations for the given return periods performed given the water level calculated for 2069.

Table 2-2: Calculated extreme maximum water levels projected from the year 2039.

Return Period (years)	Highest Water levels (projected for 2039) (mODN)
1	2.96
2	3.03
5	3.12
10	3.19
20	3.26
50	3.36
100	3.43
200	3.50

2.6 Fetch Limited Wave Parameters

The fetch-limited wave parameters, calculated from the extreme wind, can be seen tabulated in Table 2-3. The extreme wave heights are shown relative to the wind return period and for each directional sector. This analysis was based on the methods outlined in EM 1110-2-1100, Coastal Engineering Manual (Part II, Meteorology and Wave Climate).

Table 2-3: Fetch-Limited Wave Parameters

Return Period (Years)	90° - 130°		130° - 180°		180° - 240°		240° - 270°	
	Hm0 (m)	Tp (s)	Hm0 (m)	Tp (s)	Hm0 (m)	Tp (s)	Hm0 (m)	Tp (s)
1	0.35	1.9	0.30	1.4	0.96	3.0	0.76	2.6
5	0.39	2.0	0.34	1.5	1.02	3.0	0.82	2.7
10	0.43	2.0	0.37	1.5	1.07	3.1	0.88	2.7
20	0.46	2.1	0.40	1.6	1.12	3.1	0.93	2.8
50	0.51	2.1	0.43	1.6	1.18	3.2	1.00	2.8
100	0.54	2.2	0.46	1.6	1.23	3.2	1.05	2.9
200	0.58	2.2	0.48	1.7	1.27	3.3	1.09	2.9

It is assumed that projected climate change effects are incorporated into the extreme wind supplied by the Met Office.

From Table 2-3 it can be seen that:

- Eastern Sector (wind-waves from 90° to 180°): the maximum 1:200 year wave height wave height is given as $H_s = 0.58\text{m}$ with a peak wave period $T_p = 2.2\text{s}$.
- Western Sector (wind-waves from 90° to 180°): the maximum 1:200 year wave height wave height is given as $H_s = 1.27\text{m}$ with a peak wave period $T_p = 3.3\text{s}$.

For the overtopping analysis, Frontages 1, 2 and 6 will consider waves from the western sector while Frontages 5 will consider the 1:200 wave parameters from the eastern sector.

3. Joint Probability Analysis

A joint probability analysis (“JPA”) is performed on the probability of occurrence of a 1:200 year wave and the 1:200 year water level. The methodology employed in calculating the JPA is published in (Hawkes & Svensson, 2005).

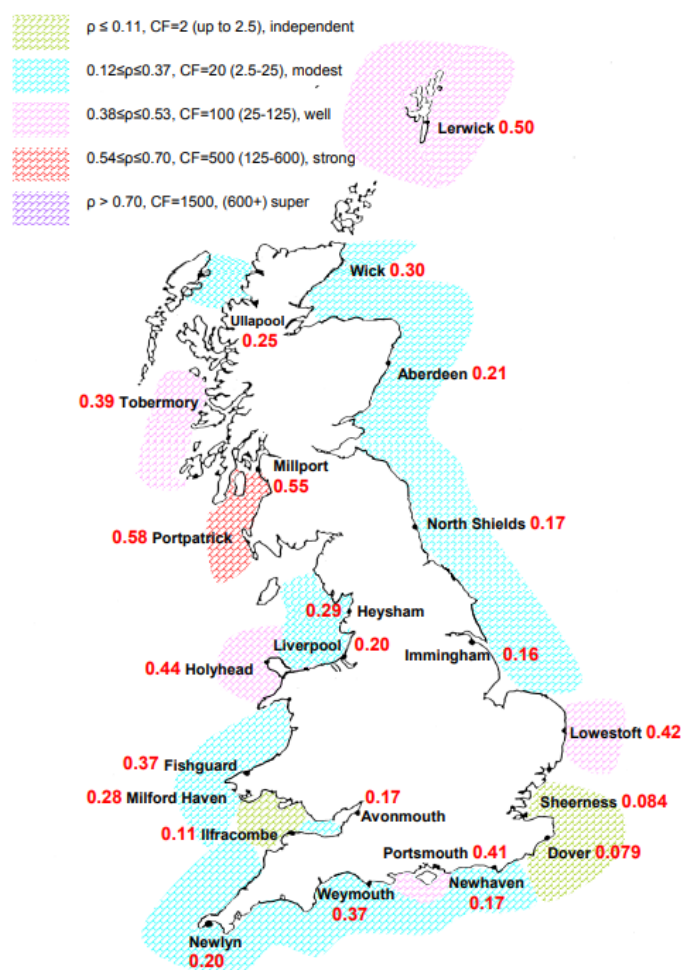


Figure 3-1. Correlation coefficient for wave height and water level. Joint Probability: Dependence Mapping and Best Practice: Technical report on dependence mapping R&D Technical Report FD2308/TR1 (2005)

In summary, for each return period, the combined probability of an extreme water level and the extreme wave height is calculated and the resulting wave height as a result of the correlation coefficient assumed for the site. Note that the report (see Figure 3-1) classifies the wave heights and water levels at Portsmouth as “well correlated” ($\rho = 0.41$ in Table 4.1 of the report). This means that extreme wave heights are correlated well with extreme water levels. The calculated JPA extreme wave heights for a given extreme water level are given in Table 3-1.

Table 3-1: Extreme water levels and calculated 1:200 year wave height joint probability analysis (well correlated).

Return Period (years)	Water Level (mODN)	1:200 Year Wave Height (m)			
		90° - 130°	130° - 180°	180°-240°	240°-270°
0.5	2.89	0.58	0.48	1.27	1.09
1	2.96	0.55	0.46	1.23	1.05
2	3.03	0.52	0.44	1.19	1.01
5	3.12	0.48	0.41	1.13	0.95
10	3.19	0.45	0.38	1.09	0.90
20	3.26	0.41	0.35	1.05	0.85
50	3.36	0.37	0.33	0.99	0.79
100	3.43	0.34	0.30	0.94	0.74
200	3.50	0.31	0.27	0.90	0.70

Note from Table 3-1 that the significant wave heights decrease with an increase in the extreme water levels. This means that a 1:200 year wave height (Table 2-3) has a significantly lower probability of occurring at the same time as a 1:200 year extreme water level. If the water level and wave heights were fully dependent, a 1:200 year extreme wave height will occur at the same time as a 1:200 year extreme water level.

Table 3-2 shows the correlated extreme 1:200 year combined water level and wave height for per directional sector, if the water levels and wave heights were indeed fully dependent.

Table 3-2: Extreme water levels and calculated 1:200 year wave height joint probability analysis if the wave heights and water levels were dependent

Return Period (years)	Water Level (mODN)	1:200 Year Wave Height (m)			
		90° - 130°	130° - 180°	180°-240°	240°-270°
200	3.5	0.58	0.48	1.27	1.09

The wave and water level combination given in Table 3-2 is highly unlikely to occur at the study site and is thus regarded as the theoretical potential upper limit of the combined extreme wind-wave and water level condition can occur at the study site.

4. Overtopping Assessment

4.1 Methodology

The overtopping assessment uses the methods outlined in the EuroTop Overtopping Manual. (EurOtop, 2018). The overtopping risk was assessed in accordance with the general limits for the property behind in the defence provided in the manual (Section 4.2).

The estimation of overtopping discharge rates for a given structure is a function of wave and water level conditions, the local bathymetry and the geometry of the coastal defence structure. As such, wave and water level combinations, calculated by the joint probability analysis for the 1:200 year joint probability (as per Table 3-1), are used in the overtopping calculations along with the proposed design elements at each frontage.

An additional overtopping calculation with the fully dependent 1:200 year water level and wave condition is sensitivity tested in order to determine the potential upper bound for overtopping at a particular site (as per Table 3-2).

For all the frontages, selected cross sections at various chainages are used as inputs into the overtopping calculations where it is deemed necessary for design purposes to assess the overtopping rates. Note all rates presented are maximum values for the instantaneous peak of the event. In reality overtopping will typically be fairly short lived events (15-60 mins) just over the high tide period and rates will ramp up, peak and reduce back down within this time frame.

4.2 Overtopping Criteria

The EurOtop II Manual recommends the following overtopping discharge rates relative to the facilities present:

Structural design: Overtopping design for the lee side of low crested breakwaters are important as significant overtopping can displace armour rock on the lee side and institute critical failure from the lee side (i.e. at frontage 6). Design overtopping rates of between 5 l/s/m and 10 l/s/m, and maximum overtopping volumes between 10,000 and 20,000 l/m are recommended for the lee side designed for overtopping (Table 4-1).

Personnel / Vehicles: During storms, human life can be in danger when large waves wash over defences that is accessible to humans and vehicles. Table 4-2 gives the recommended overtopping discharge rates for people and vehicles exposed to wave overtopping.

Table 4-1: Limits for wave overtopping for structural design of breakwaters, seawalls, dikes and dams (EurOtop II - Table 3.1, p47)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
Rubble mound breakwaters; $H_{m0} > 5$ m; no damage	1	2,000-3,000
Rubble mound breakwaters; $H_{m0} > 5$ m; rear side designed for wave overtopping	5-10	10,000-20,000
Grass covered crest and landward slope; maintained and closed grass cover; $H_{m0} = 1 - 3$ m	5	2,000-3,000
Grass covered crest and landward slope; not maintained grass cover, open spots, moss, bare patches; $H_{m0} = 0.5 - 3$ m	0.1	500
Grass covered crest and landward slope; $H_{m0} < 1$ m	5-10	500
Grass covered crest and landward slope; $H_{m0} < 0.3$ m	No limit	No limit

Table 4-2: Overtopping limit rates for people and vehicles (EurOtop II - Table 3.3, p54)

Hazard type and reason	Mean discharge q (l/s per m)	Max volume V_{max} (l per m)
People at structures with possible violent overtopping, mostly vertical structures	No access for any predicted overtopping	No access for any predicted overtopping
People at seawall / dike crest. Clear view of the sea.		
$H_{m0} = 3$ m	0.3	600
$H_{m0} = 2$ m	1	600
$H_{m0} = 1$ m	10-20	600
$H_{m0} < 0.5$ m	No limit	No limit
Cars on seawall / dike crest, or railway close behind crest		
$H_{m0} = 3$ m	<5	2000
$H_{m0} = 2$ m	10-20	2000
$H_{m0} = 1$ m	<75	2000
Highways and roads, fast traffic	Close before debris in spray becomes dangerous	Close before debris in spray becomes dangerous

Note that the extreme wave heights at the study site vary between $H_{m0} = 0.48$ m and $H_{m0} = 1.27$ m (Table 2-3).

4.3 Frontage 2

An overtopping assessment for Frontage 2 is done at chainage 0m (section 1-1 on drawing LANG-ACM-ZZ-F2-DR-ZZ-000001). This section has the lowest and flattest foreshore slope from the coastal defence to the water line. As such, a higher average overtopping rate is potentially foreseen in this area. Figure 4-1 shows an extract from drawing LANG-ACM-ZZ-F2-DR-ZZ-000001 zooming in on frontage 2 exposed to the wind-waves.

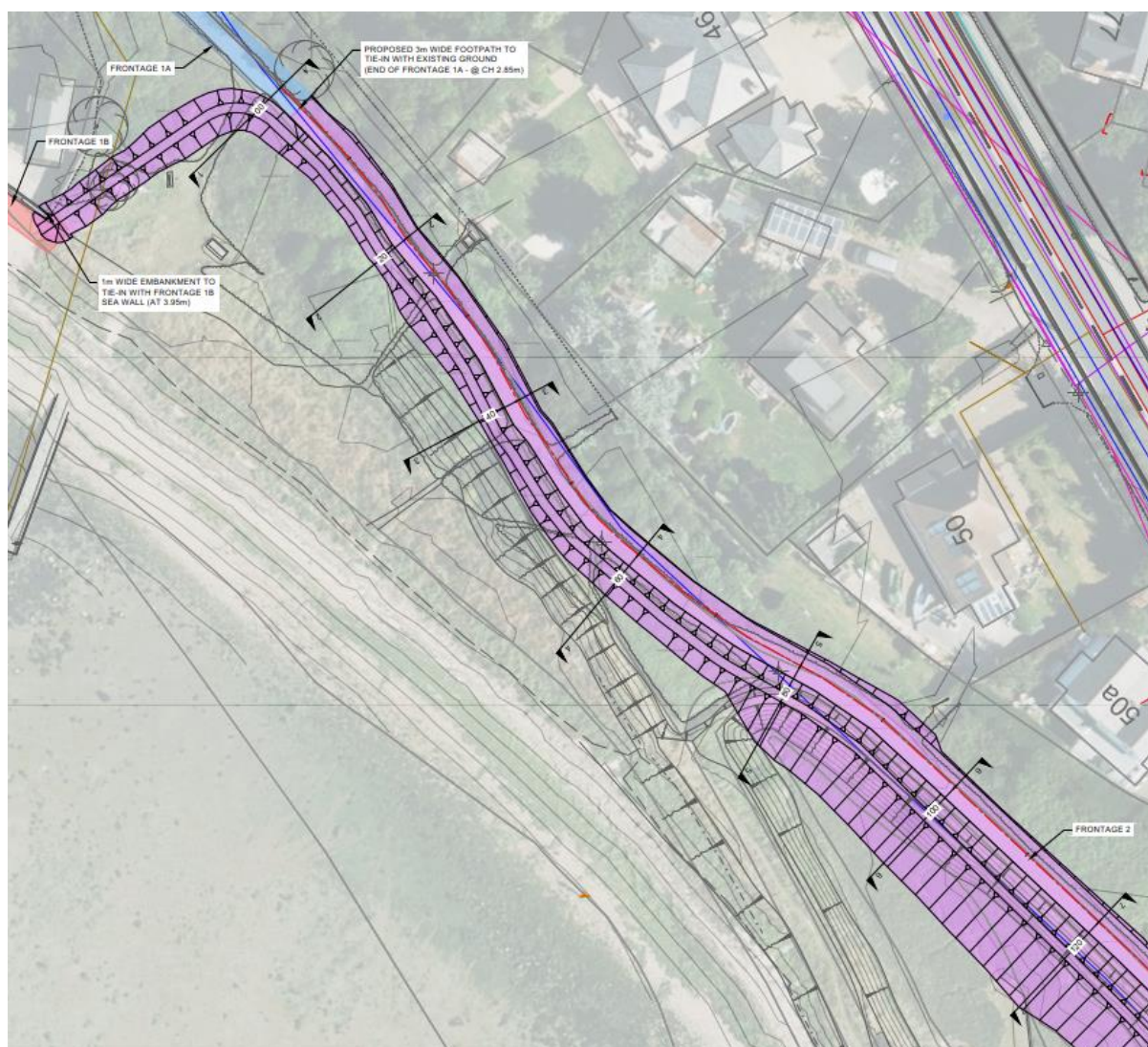


Figure 4-1: Plan view of frontage 2 between chainage 0m and 120m

The results from this overtopping assessment can be seen in Table 4-3 for the combination of wave heights and water levels given in Table 3-1 and Table 3-2 for wind-waves from the westerly sector. Note that the full range of results are given in 0.

From Table 4-3 it can be seen that, for waves approaching the frontages head on, the average overtopping discharge is potentially just above the recommended limits specified in Table 4-2 which could be dangerous for pedestrians and animals behind the structure during storm events coinciding with high water levels.

Table 4-3: Range of overtopping rates calculated for frontage 2

Coastal Defence

Average Overtopping Discharge (l/s/m)

**Joint Probability
Assessment**

**1:200 Year Fully
dependent Scenario**

Vegetated slopes along Frontage 2: Sect 1-1 Ch0

9 - 27

52

4.4 Frontage 6

An overtopping assessment for Frontage 6 is done at chainage 40m (section 5-5 on drawing LANG-ACM-ZZ-F6-DR-ZZ-000001). This section is deemed be the most exposed frontage to the approaching wind-waves. Figure 4-2 shows an extract from drawing LANG-ACM-ZZ-F6-DR-ZZ-000001 zooming in on chainage 40m

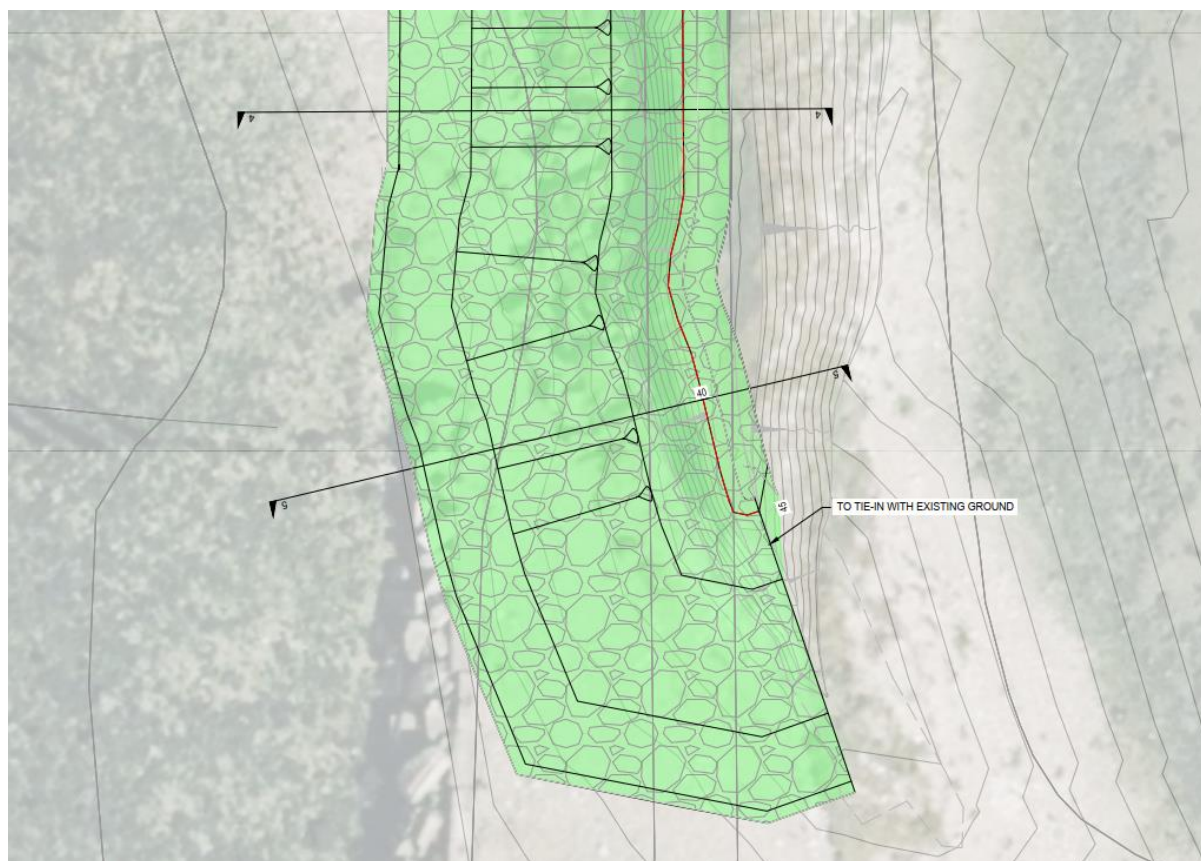


Figure 4-2: Plan view of frontage 6 between chainage 30m and 40m

The results from this overtopping assessment can be seen in Table 4-4 for the combination of wave heights and water levels given in Table 3-1 and Table 3-2 for wind-waves from the westerly sector. Note that the full range of results are given in Appendix 0.

Table 4-4: Range of overtopping rates calculated for frontage 6

Coastal Defence	Average Overtopping Discharge (l/s/m)	
	Joint Probability Assessment	1:200 Year Fully dependent Scenario
New revetment Frontage 6: Sect 5-5 Ch40	5 - 10	26

From Table 4-4 it can be seen that, for waves approaching Frontage 6 head on, the average overtopping discharge is slightly above the recommended limits specified in Table 4-1 for waves between $1\text{m} < H_{m0} < 3\text{m}$.

4.5 Frontage 3

An overtopping assessment for frontage 3 is done at chainage 105m and 125m (section 6-6 and sect 7-7 respectively on drawing LANG-ACM-ZZ-F3-DR-ZZ-000001). In addition, the proposed mound defence located in the carpark is assessed for overtopping given the existing frontage of the carpark.

Figure 4-3 shows an extract from drawing LANG-ACM-ZZ-F3-DR-ZZ-000001 zooming in on frontage 3 exposed to the wind-waves primarily from the eastern sector.

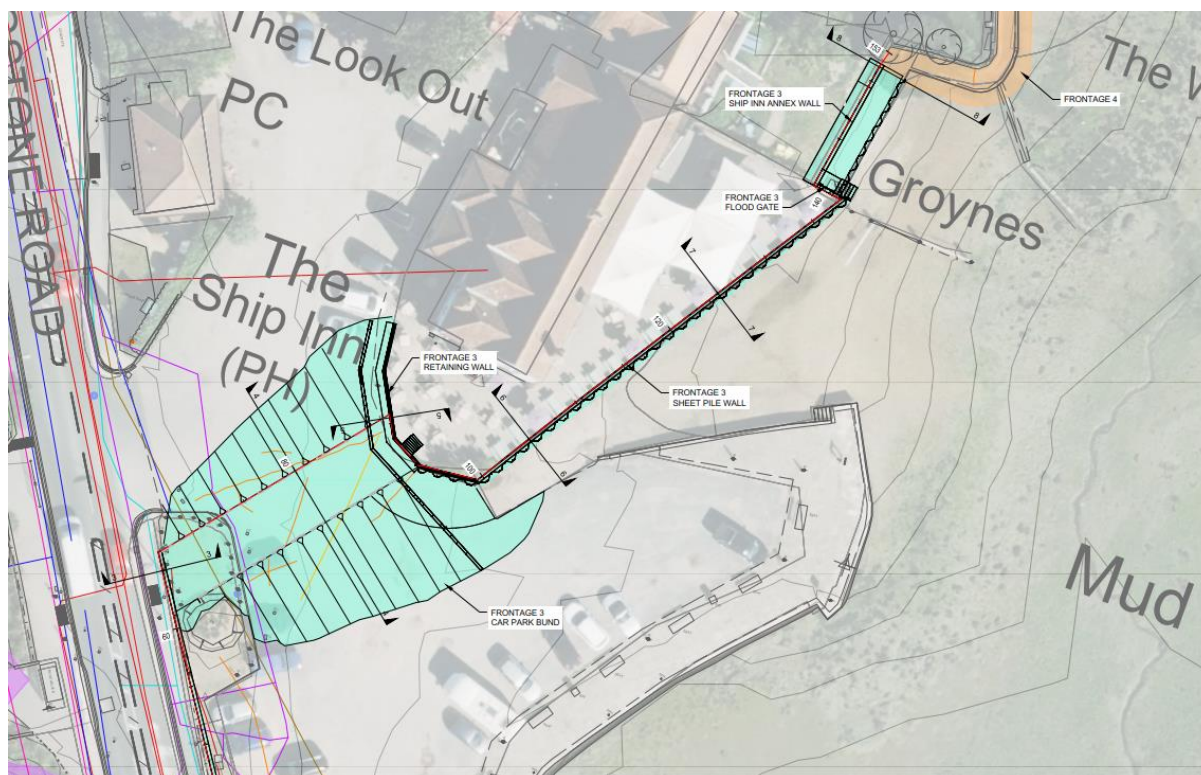


Figure 4-3: Plan view of frontage 3

The results from this overtopping assessment can be seen in Table 4-5 for the combination of wave heights and water levels given in Table 3-1 and

Table 3-2 for wind-waves from the easterly sector. Note that the full range of results are given in Appendix 0.

Table 4-5: Range of overtopping rates calculated for frontage 3

Coastal Defence	Average Overtopping Discharge (l/s/m)	
	Joint Probability Assessment	1:200 Year Fully dependent Scenario
Frontage 3: CH105 sect 6-6: Simple wall: vertical or steep structures without a significant mound	1 - 2	16
Frontage 3: CH125 sect 7-7: Simple wall: vertical or steep structures without a significant mound	1 - 2	16
Frontage 3: Carpark: Sect 1-1 at Ch15 Composite wall with mound.	1 - 11	12

From Table 4-5 it can be seen that, for waves approaching the frontages head on, the average overtopping discharges are generally within recommended limits specified in Table 4-2 where pedestrians and vehicles have a clear view of the sea, given maximum wave heights less than 1m.

4.6

4.6 Frontage 5

An overtopping assessment for Frontage 5 was carried out at all cross sections between chainage 0m and 80m (between section 1-1 and sect 5-5 on drawing LANG-ACM-ZZ-F5-DR-ZZ-000001). The orientation of the coastal defences, the varying crest heights and foreshores of the defence structures and the varying bathymetry along the structure warrants an overtopping assessment at 20m intervals.

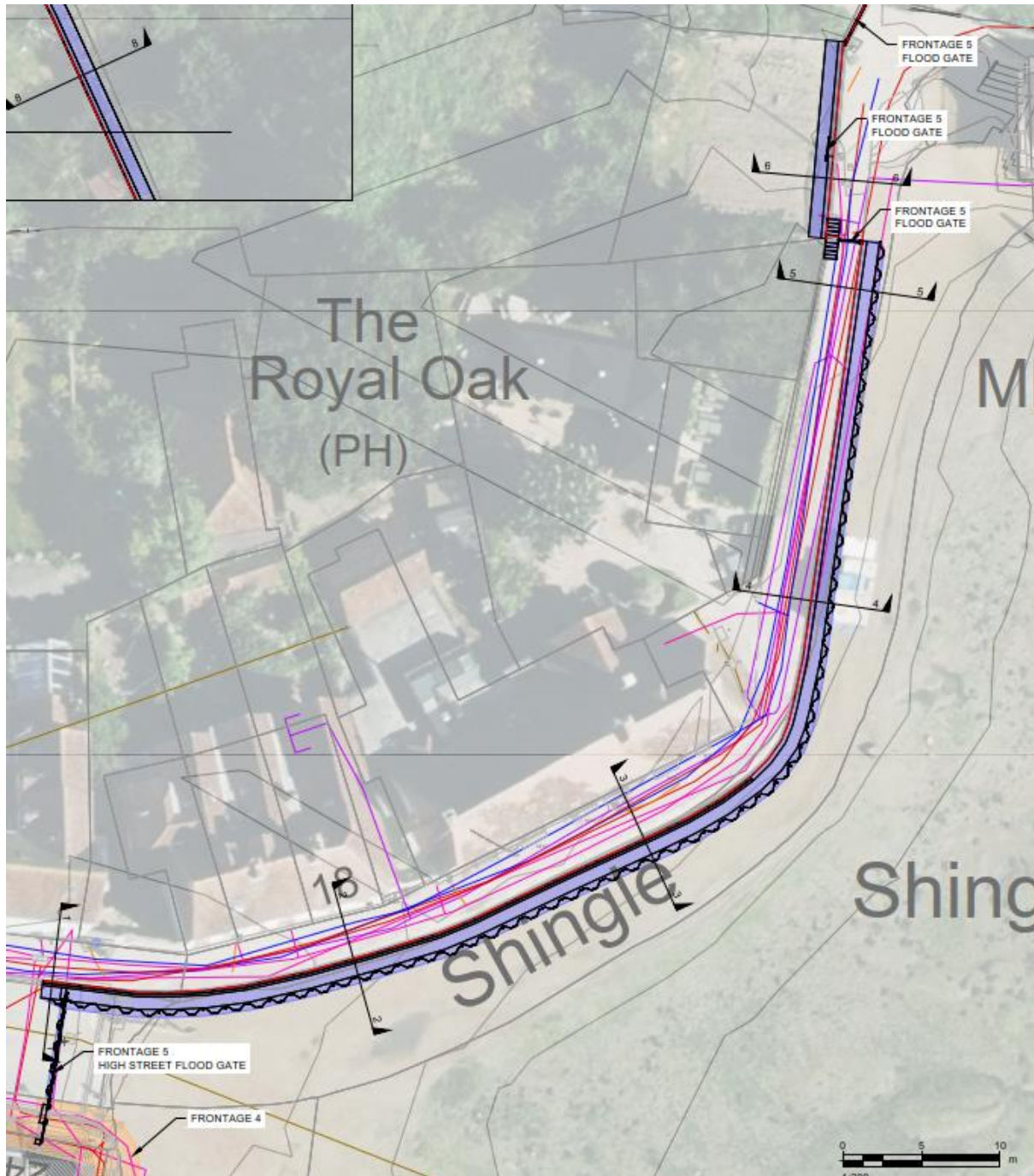


Figure 4-4: Plan view of frontage 5 between chainage 0m and chainage 80m

Figure 4-4 shows an extract from drawing LANG-ACM-ZZ-F5-DR-ZZ-000001 zooming in on frontage 5 exposed to the wind-waves primarily from the eastern sector.

Table 4-6: Range of overtopping rates calculated for Frontage 5

Coastal Defence	Average Overtopping Discharge (l/s/m)	
	Joint Probability Assessment	1:200 Year Fully dependent scenario
Frontage 5: Sect 1-1 at Ch0 Composite wall without significant mound.	1 - 2	16
Frontage 5: Sect 2-2 at Ch20 Composite wall without significant mound.	1 - 2	16
Frontage 5: Sect 3-3 at Ch40 Composite wall without significant mound.	1 - 2	16
Frontage 5: Sect 4-4 at Ch60 Composite wall without significant mound.	1 - 2	16
Frontage 5: Sect 5-5 at Ch80 Composite wall without significant mound.	1 - 2	16

The results from this overtopping assessment can be seen in

Table 4-6 for the combination of wave heights and water levels given in Table 3-1 and Table 3-2 for wind-waves from the easterly sector. Note that the full range of results are given in Appendix 0.

From Table 4-6 it can be seen that, for waves approaching the frontages head on, the average overtopping discharge is fairly consistent in the calculated results along the whole frontage. These overtopping values are within the recommended limits specified in Table 4-2 which means that pedestrians along the frontage might get wet from the overtopped water but poses lower threat to health and safety during overtopping events. Note that during the 1:200 year fully dependent wave / water level event overtopping in the order of 16 l/s/m is calculated which is at the higher end of recommended limits specified in Table 4-2 which could be dangerous for pedestrians and animals behind the structure during storm events coinciding with high water levels.

5. Conclusions

5.1 Wave Overtopping

The overtopping assessment uses the methods outlined in the EuroTop Overtopping Manual. (EurOtop, 2018). The overtopping risk was assessed in accordance with the general limits for the property behind in the defence provided in the manual. The wave parameters used as input were calculated from extreme wind data statistics supplied by the Met Office calculated from a 30year wind record at Thorney Island based on the longest fetch lengths at the study site.

The extreme water levels used in the overtopping assessment considers the calculated extreme water levels projected forwards from the year 2039 (mid scheme life) for a range of return periods (up to the 1:200 year extreme water level event – design SoP).

A joint probability analysis (“JPA”) is performed on the probability of occurrence of extreme wave heights and extreme water levels for each return period, the combined probability of an extreme water level and the extreme wave height is calculated and the resulting wave height as a result of the correlation coefficient assumed for the site.

- Frontage 2:

The calculated average overtopping discharge (9 – 27 l/s/m) is slightly above the recommended limits specified in EuroTop II which would be dangerous for pedestrians and animals behind the structure during storm events coinciding with high water levels.

- Frontage 6:

The calculated average overtopping discharge (5 – 10l/s/m for chainage 40) is slightly above the recommended limits specified in EuroTopII for wave heights ranging between $1\text{m} < H_{m0} < 3\text{m}$.

- Frontage 3: The average overtopping discharge within the recommended limits specified in EuroTopII which would be dangerous for pedestrians and animals behind the structure during storm events coinciding with high water levels.

- Frontage 5:

The calculated average overtopping discharge is fairly consistent in the calculated results along the whole frontage. The overtopping values (1 – 2l/s/m) are within the recommended limits specified in EuroTopII which means that pedestrians along the frontage might get wet from the overtopped water but poses lower threat to health and safety during overtopping events.

Note that during the 1:200 year fully dependent wave / water level event overtopping in the order of 16 l/s/m is calculated which is at the upper end of the recommended limits specified in Table 4-2 which would be dangerous for pedestrians and animals behind the structure during storm events coinciding with high water levels.

Generally the proposed defences significantly reduce wave overtopping risk to the Frontages. Varying levels of residual overtopping have been estimated up to the design So . The results will be used to inform the drainage design for the scheme and where discharges remain in excess of tolerable limits, additional mitigation measures such as properties behind applying the property level protection devices as they do now, or restricting access to pedestrians in a storm event may be considered. It should be noted that as sea level rise increases, greater residual risk from overtopping than estimated here will be experienced, and additional measures may be required to manage the risks later in the design life of the scheme.

6. References

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Appendix A Appendix

Appendix A

A.1 Frontage 2

A 1: Vegetated slopes along Frontage 2: Sect 1-1 Ch0

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Max Ave Discharge	Design Discharge	Max Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m	l/s/m	l/s/m
0.5	1.27	3.3	3.95	2.89	0	0	0	0
1	1.23	3.3	3.95	2.96	0	0	0	0
2	1.19	3.3	3.95	3.03	0	0	0	0
5	1.13	3.3	3.95	3.12	9	109	13	145
10	1.09	3.3	3.95	3.19	11	111	16	146
20	1.05	3.3	3.95	3.26	13	114	19	148
50	0.99	3.3	3.95	3.36	18	117	25	151
100	0.94	3.3	3.95	3.43	22	118	29	150
200	0.90	3.3	3.95	3.5	27	121	35	152
Dependent 1:200 year	1.27	3.3	3.95	3.5	52	260	65	316

A.2 Frontage 6

A 2: New revetment Frontage 6: Sect 5-5 Ch40

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Max Ave Discharge	Design Discharge	Max Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m	l/s/m	l/s/m
0.5	1.27	3.3	4.72	2.89	5	2	8	5
1	1.23	3.3	4.72	2.96	5	2	9	5
2	1.19	3.3	4.72	3.03	6	2	9	5
5	1.13	3.3	4.72	3.12	6	2	10	4
10	1.09	3.3	4.72	3.19	7	2	11	4
20	1.05	3.3	4.72	3.26	8	2	12	4
50	0.99	3.3	4.72	3.36	9	2	13	4
100	0.94	3.3	4.72	3.43	9	2	14	4
200	0.90	3.3	4.72	3.5	10	2	15	4
Dependent 1:200 year	1.27	3.3	4.72	3.5	26	18	36	30

A.3 Frontage 3

A 3: Frontage 3: CH105 sect 6-6: Simple wall: vertical or steep structures without a significant mound

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	2	3
1	0.55	2.22	3.80	2.96	2	3
2	0.52	2.22	3.80	3.03	2	2
5	0.48	2.22	3.80	3.12	2	2
10	0.45	2.22	3.80	3.19	2	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 4: Frontage 3: CH125 sect 7-7: Simple wall: vertical or steep structures without a significant mound

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	1	2
1	0.55	2.22	3.80	2.96	1	1
2	0.52	2.22	3.80	3.03	1	1
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 5: Frontage 3: Carpark: Sect 1-1 at Ch15 Composite wall with mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.60	2.89	1	1
1	0.55	2.22	3.60	2.96	1	1
2	0.52	2.22	3.60	3.03	1	1
5	0.48	2.22	3.60	3.12	1	2
10	0.45	2.22	3.60	3.19	1	2
20	0.41	2.22	3.60	3.26	2	2
50	0.37	2.22	3.60	3.36	6	8
100	0.34	2.22	3.60	3.43	8	10
200	0.31	2.22	3.60	3.5	11	14
Dependent 1:200 year	0.58	2.22	3.60	3.5	12	17

A.4 Frontage 5

A 6: Frontage 5: Sect 1-1 at Ch0 Composite wall without significant mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	2	2
1	0.55	2.22	3.80	2.96	2	2
2	0.52	2.22	3.80	3.03	2	2
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 7: Sect 2-2 at Ch20 Composite wall without significant mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	1	1
1	0.55	2.22	3.80	2.96	1	1
2	0.52	2.22	3.80	3.03	1	1
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 8: Sect 3-3 at Ch40 Composite wall without significant mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	1	1
1	0.55	2.22	3.80	2.96	1	1
2	0.52	2.22	3.80	3.03	1	1
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 9: Sect 4-4 at Ch60 Composite wall without significant mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	1	1
1	0.55	2.22	3.80	2.96	1	1
2	0.52	2.22	3.80	3.03	1	1
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

A 10: Sect 5-5 at Ch80 Composite wall without significant mound.

Return Period	Significant wave height (Hm0)	Peak Wave Period (Tp)	Defence Crest Level	Design Water Level	Average discharge	Design Discharge
Years	m	s	mODN	mODN	l/s/m	l/s/m
0.5	0.58	2.22	3.80	2.89	1	1
1	0.55	2.22	3.80	2.96	1	1
2	0.52	2.22	3.80	3.03	1	1
5	0.48	2.22	3.80	3.12	1	2
10	0.45	2.22	3.80	3.19	1	2
20	0.41	2.22	3.80	3.26	1	2
50	0.37	2.22	3.80	3.36	1	2
100	0.34	2.22	3.80	3.43	1	2
200	0.31	2.22	3.80	3.50	2	3
Dependent 1:200 year	0.58	2.22	3.80	3.50	16	22

