Appendix D

Technical Note: Breach Modelling at Priddy's Hard, Gosport

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Project: 2266 – Priddy's Hard, Gosport

1. Background Information

1.1. Herrington Consulting has been commissioned by Conservation Plus, on behalf of the Portsmouth Naval Base Property Trust (PNBPT), to undertake numerical flood modelling at Priddy's Hard, Gosport. At the time of commission, the Environment Agency (EA) had maintained an objection to the planning application made by PNBPT to develop the site (the first in a series of works), on the grounds of flood risk. The EA stated in their objection that the Flood Risk Assessment had not addressed the residual risk of flooding, which is primarily attributed to the breaching of the proposed defences.

1.2. Correspondence with the EA's Modelling and Forecasting Team has been undertaken to confirm the methodology for the numerical modelling of the breach scenarios. The EA agreed the methodology by letter, received on 27th February 2019 (a copy of which is appended to this document for reference).

1.3. Priddy's Hard is located in Gosport on the western shore inside Portsmouth Harbour, as shown in Figure 1. The eastern quayside of Priddy's Hard faces the greater body of Portsmouth Harbour, while the southern quayside faces Forton Lake.

Figure 1 – The sheltered location of Priddy's Hard inside Portsmouth Harbour.

2. Outline Methodology/Approach

2.1. The breach of the sea defences has been tested at two locations, referred to in this document as Breach A and Breach B (Figure 2). Breach A has been positioned in a section of concrete sea wall next to an existing building, referred to as Building U, which has been proposed to be converted to holiday letting accommodation. Breach B has been positioned in a section of a (proposed) concrete sea wall located at the southern end of Searle Drive, where additional residential properties have been proposed to be constructed.

2.2. The breaches have been modelled as the instantaneous collapse of a 20m length of the concrete sea walls, occurring when the surge water level reaches at least half way up the face of the walls. Each breach remains open for 18 hours before being closed to simulate its repair. Breaches A and B have not been modelled to occur simultaneously, as they each represent a separate residual risk event.

Figure 2 – Site location, breach locations and wave overtopping frontages.

2.3. Breach A and wave overtopping at Frontage C have been modelled simultaneously to simulate an event with north-easterly winds, while Breach B and overtopping at Frontage D and F have been modelled simultaneously to simulate an event generated by south-westerly winds.

2.4. The wave overtopping discharge rate along all frontages has been calculated using the HR Wallingford 'Bayonet' online wave overtopping calculation tool (overtopping.co.uk). Wind-wave hindcasting has been used to derive the wave inputs used in the overtopping calculator.

Calculations show zero overtopping discharge rates for the Frontage A, which is attributed to the very high sea wall. The buildings along Frontage B form a similarly high 'defence', which also prevent waves from overtopping the frontage at this location. Similarly, the infrastructure along Frontage E prevents overtopping along this section Therefore, wave overtopping is excluded from both of these frontages in the model. Further detail is provided in later sections.

3. Modelling Assumptions

3.1. A number of assumptions are required to be able to undertake breach modelling of Priddy's Hard to address the residual risk of flooding from a breach.

3.2. These assumptions are:

a) *The site is defended with a continuous perimeter sea wall, with a minimum crest of 4.85 m. This wall represents a defence with a 1 in 200 year standard of protection up to the year 2115;*

b) *The breaches occur almost instantaneously when the surge water level reaches at least half the height of the sea wall; and,*

c) *The breaches are unlikely to occur simultaneously due to their alignment. Similarly, wave overtopping cannot occur along the frontages facing the harbour and along the frontages facing Forton Lake simultaneously, as this would represent opposing wind and wave directions.*

4. Model Setup

4.1. The modelling has been undertaken using the current version of the TUFLOW 2D numerical flood modelling system, TUFLOW 2018-03-AD_w64, using double precision calculations for the best available accuracy.

4.2. The 2D Digital Elevation Model (DEM) of the floodplain uses a grid resolution of 2m so that the ground elevations and site features can be represented with a high level of detail. The ground elevations of the DEM are based upon the EA's LiDAR Digital Terrain Model (DTM) which has been verified and supplemented with site-specific topographic survey levels provided by the client. A comparison of the EA LiDAR DTM with the site-specific topographic survey measurements showed good correlation and therefore no adjustment of the LiDAR DTM levels have been undertaken when building the model DEM.

4.3. The friction, or the resistance to flood flow, of the model DEM is an important factor in determining model accuracy. Therefore, Ordnance Survey (OS) MasterMap data has been used to define all of the different types of land use across the model domain. This has been used in conjunction with the typical values of Manning's n roughness following Chow 1959.

4.4. The DEM of the model domain has been further adjusted when it is read into the model to include the features present on site that are likely to affect flooding, which are otherwise absent from the EA LiDAR. These adjustments have been implemented in the model by changing the DEM using GIS z-shapes.

4.5. The modification of the DEM includes the adjustment for buildings which have been modelled by increasing the ground level by 0.3 m over each building footprint. This 'stubby buildings' method increases the ground level of the DEM to represent the threshold of the building (at ground floor level). Combining an increased Manning's *n* roughness value of 0.3 over the raised footprint allows the model to replicate the reduced flow of water through the buildings.

4.6. Adopting a precautionary approach, the model excludes all of the surface water drainage system present on-site, which may otherwise reduce the predicted extent and depth of flooding. There are no watercourses or hydraulic control structures on-site which are required to be included in the model.

5. Surge Boundary Conditions

5.1. The extreme surge event has been modelled as a time-varying water level, with the peak of the surge timed to coincide with a Mean High Water Spring (MHWS) tide. The model simulates a total of four tides with the surge peak coinciding with the first tide. The MHWS tides upon which the surge has been superimposed originate from the water level records from Portsmouth Harbour tide gauge. A review of the water level records has been undertaken to find a high tide that matches MHWS in Portsmouth, which has a level of 4.7 m Chart Datum (CD), equivalent to 1.97 mAODN. This MHWS tide for Portsmouth is shown with a blue line in Figure 3.

5.2. The extreme water level for a 1 in 200 year tidal surge event of 3.1 mAODN (for the base year 2008) has been taken from node 1924 of the Environment Agency's Coastal Flood Boundary (CFB) database. The difference between this extreme water level and MHWS is 1.13 m and is known as the 'surge residual' and has been used to scale Donor Surge Shape 13 (for Portsmouth) following the methodology found in EA guidance on design sea levels. The resulting tidal surge residual has been superimposed upon the MHWS tidal curve to yield the water level time series applied in the model, before any adjustments are made to allow for future climate change. The surge residual and the adjusted tidal surge water levels are shown in Figure 3 as the red and yellow lines, respectively.

5.3. Future climate change has been accounted for in the model by superimposing future Sea Level Rise (SLR) of 1.13 m[1] upon the extreme surge water level curve (the green line in Figure 3). This increase represents the effect of 100 years of future climate change upon mean sea level for the south east of England between the years 2008 and 2115 (following the method set out in the NPPF). The peak of the extreme surge water level curve including sea level rise for the year 2115 reaches 4.23 mAODN. This extreme event is referred to as the 'Design Event'.

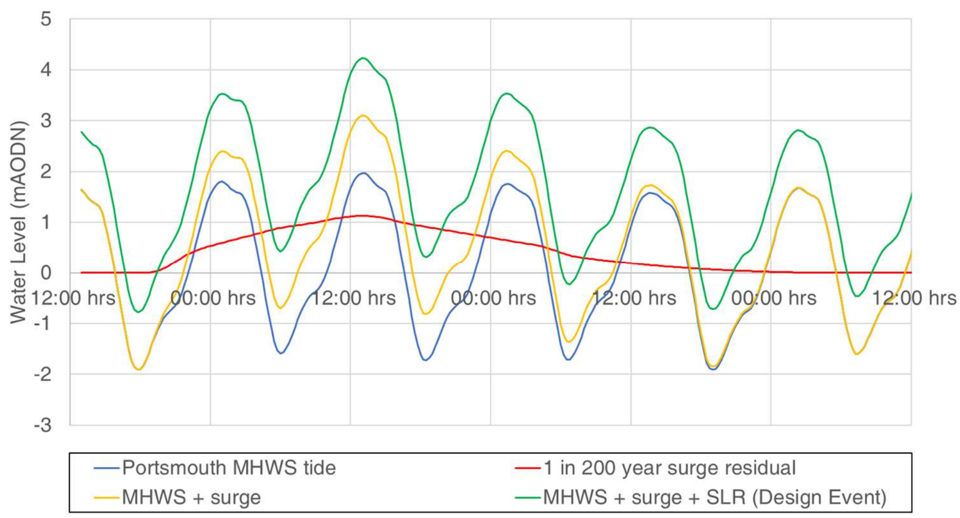

Figure 3 – The MHWS tide, surge residual and design event water level curve (including SLR) at Portsmouth.

6. Wave Overtopping Boundary Conditions

6.1. The wave overtopping associated with the Design Event has been modelled as a time-varying discharge onto the ground on the landward side of the sea walls. Breach A has been modelled with simultaneous overtopping at Frontage C, while Breach B has been modelled with simultaneous overtopping at Frontage D and F. The exclusion of wave overtopping at Frontages A, B and E are discussed in following sections.

6.2. Wave overtopping discharge rates have been estimated using HR Wallingford's 'Bayonet' online wave overtopping calculation tools[2] which relies upon hindcast estimates of wind-wave growth over the fetches of Portsmouth Harbour and Forton Lake. Wind-wave hindcast has been undertaken using the methods of the Shoreline Protection Manual (SPM)[3] and is discussed in subsequent sections.

[1] coincidentally the adjustment for SLR is equal to the extreme surge height residual of the 1 in 200 year water level
[2] overtopping.co.uk
[3] Coastal Engineering Research Center (1984). *Shore Protection Manual.* U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg Mississippi

6.3. Estimating the winds associated with surge water levels at Portsmouth requires an appreciation of the meteorological mechanisms which cause surges on the south coast of the England. Surges in this area are caused by low pressure weather systems travelling either down the North Sea and through the Strait of Dover, or into the English Channel from the North Atlantic. Surges approaching from the North Sea have the potential to drive strong winds from the north and north east at Portsmouth, which may result in waves in the harbour coinciding with, or near to, the surge in water level. However, strong winds from the south west would require a low pressure system from the North Atlantic to be centred over England, Scotland or Ireland, rather than travelling directly along the English Channel. Therefore, winds from this direction are significantly less likely to coincide with surge water levels at Portsmouth.

6.4. To represent the conditions discussed in 6.3, a conservative approach has been taken to represent a 'worst case' scenario, whereby a gale-force wind has been applied to the wind-wave hindcast. The mean gale-force wind speed of 42.5 mph (18.95 m/s) has been assumed to be an overly-conservative estimate of surge-coincident winds from both north-westerly and south-westerly sectors. This mean gale-force wind has been increased by 10% to 46.75 mph (20.85 m/s) to represent future climate change allowances, following the guidance of the NPPG, before being applied to the wind-wave hindcast.

6.5. The SPM method of wind-wave hindcast requires the fetch length at high water over which wave growth can occur, which in Portsmouth Harbour and Forton Lake are 2.0 km and 0.35 km, respectively. The maximum fetch in Portsmouth Harbour at high water is to the north-north-east and would result in a wave approaching the sea wall almost perpendicular to the frontage. The longest fetch length at high water in Forton Lake is approximately 0.35 km (at an angle to the creek) and would result in a wave that approaches the sea wall at an oblique angle of around 60° (waves approaching perpendicular to the sea wall would have a significantly reduced fetch length of approximately 0.1 km). This is assumption is also considered to be an overly-conservative estimate of wave heights in Forton Lake, as the SPM method is 1-dimensional and cannot account for wave spreading, assuming an infinitely wide waterbody for wave growth. In essence, the adopted approach leads to an over-estimate of the wave heights in Forton Lake to present the most conservative estimate of wave overtopping discharge rates along this frontage.

6.6. The wave conditions output from the hindcast have been applied in the Bayonet wave overtopping calculator, along with water levels conditions and the dimensions of each frontage. The results of the wind-wave hindcast and wave overtopping calculations are shown in Table 1. For application in the model, these overtopping rates have been converted to discharge rates (m^3/s) which represent the total overtopping for the length of each frontage where; Frontage C is 187 m long, and Frontage D and F are 50 m and 103 m long, respectively. The wave overtopping is modelled such that it increases from $0m^3/s$ (around two hours before high water), reaching a peak overtopping rate which coincides with the peak surge water level. The rate diminishes as the water level drops following high water, until it reaches 0m³/s once more. Reduced wave overtopping discharge rates due to diminishing fetch length and water depth in the harbour and Forton Lake (i.e. as the water levels drop) have not been considered and therefore represent the most conservative estimate of wave overtopping discharge rates at these locations.

Table 1 – Results of the wind-wave SPM hindcast and Bayonet wave overtopping calculations for the design event conditions.

Sector	Wind speed (m/s)	Fetch length (m)	Wave height (m)	Wave period (s)	Wave angle (°)	Overtopping rate (l/s/m)
NE	20.85	2,000	0.637	2.315	0	3.46
SW	20.85	350	0.280	1.339	60	0.03

6.7. Wave overtopping has been excluded from the model simulations at Frontages A, B and E. Frontage A has a steeply sloped foreshore and a slightly set-back brick sea wall, as shown in Figure 4, which has a crest elevation of 6.0 mAODN, resulting in no wave overtopping.

Figure 4 – View of Frontage A

6.8. Frontage B is shown in Figure 5, and not only affords protection from the Camber Basin, but is also backed by buildings which form a continuous barrier over which waves cannot overtop the frontage. Consequently, it is assumed that the wave overtopping discharge rate is zero for Frontage B.

Figure 5 - View of Frontage B

6.9. Frontage E has a number of features positioned at the quayside, which will all lead to significant wave attenuation prior to the wave reaching a set-back sea wall. *Figure 6* shows Frontage E and the features, which include a bridge-deck, a pier-deck, a piled platform (which extends over the foreshore) and a shore-parallel wall (~10 m forward of the quayside). Therefore, it is assumed that the wave overtopping discharge rate is zero for Frontage E.

Figure 6 - View of Frontage E

7. Sensitivity Tests

7.1. A number of sensitivity tests have been undertaken to determine the effects of the various input parameters on the model results. These are detailed as follows:

a) ***Manning's n ±20%.*** *Two sensitivity tests have been included whereby the standard surface roughness values of Manning's n have been varied by ±20% and applied to the Breach A test. The site is mainly urban, with little vegetation other than gardens, and therefore varying the roughness values is used to simulate the potential variation in the estimates of surface roughness by Chow (1959), which may occur for different types of man-made surfaces to assess the potential effects on flows;*

b) ***Surge level confidence interval.*** *The EA CFB indicates a confidence interval in the estimate of the 1 in 200 year surge level of ±0.3 m and therefore, a sensitivity test has been undertaken of Breach A, where an additional 0.3 m has been added to the water level boundary. The overtopping rate has been recalculated for this event where the higher water level results in a rate of 17.4 l/s/m at Frontage C; and,*

c) ***Wave overtopping rate confidence interval****. A test has been undertaken where the overtopping rate at Frontage C is increased to 12 l/s/m to match the 95% confidence interval of the estimates of overtopping for the Breach A simulation. The rate of 12 l/s/m represents an increase of more than 300% of the mean overtopping discharge rate for the Design Event and is intended to test the sensitivity of the model to wave overtopping rates within the bounds of error for the estimates of overtopping.*

8. Model Results

8.1. The Figures in this section show the results of the numerical flood modelling of Breach A and Breach B with wave overtopping, including the maximum extent and depth of flooding for the Design Event. The maximum hazard rating across the site, for the post-development scenario, is also included. The maximum predicted depth of flooding extracted from the model is calculated for each 2D grid cell at any point in time (throughout the entire model duration). Therefore, these outputs should not be interpreted as a single snapshot in time. Full size figures showing the model results from all modelled scenarios are available in Appendix 2.

8.2. Figure 7 shows that for the Breach A scenario (Design Event), the depth of flooding can reach more than 1.0 m in the lower-lying areas of the site, including near to Building U and in the areas of non-residential use. Floodwater reaches Searle Drive where residential dwellings are located, however, the depth of flooding here is predicted to be less than 0.5 m, with only a very small area in the centre of the road which exceeds this depth.

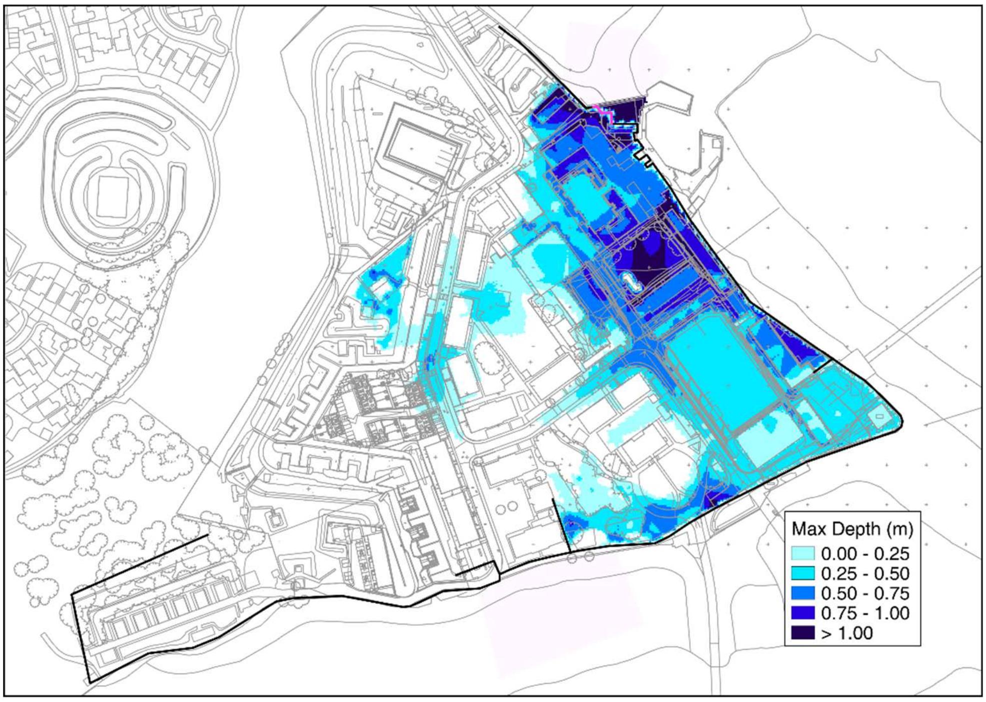

Figure 7 – Maximum depth of flooding resulting from Breach A for the Design Event, including wave overtopping.

8.3. Figure 8 shows a significant hazard rating for the non-residential area of the site, increasing to an extreme hazard rating near to Breach A itself. A moderate hazard is predicted in the residential areas on Searle Drive, with small isolated areas of significant hazard.

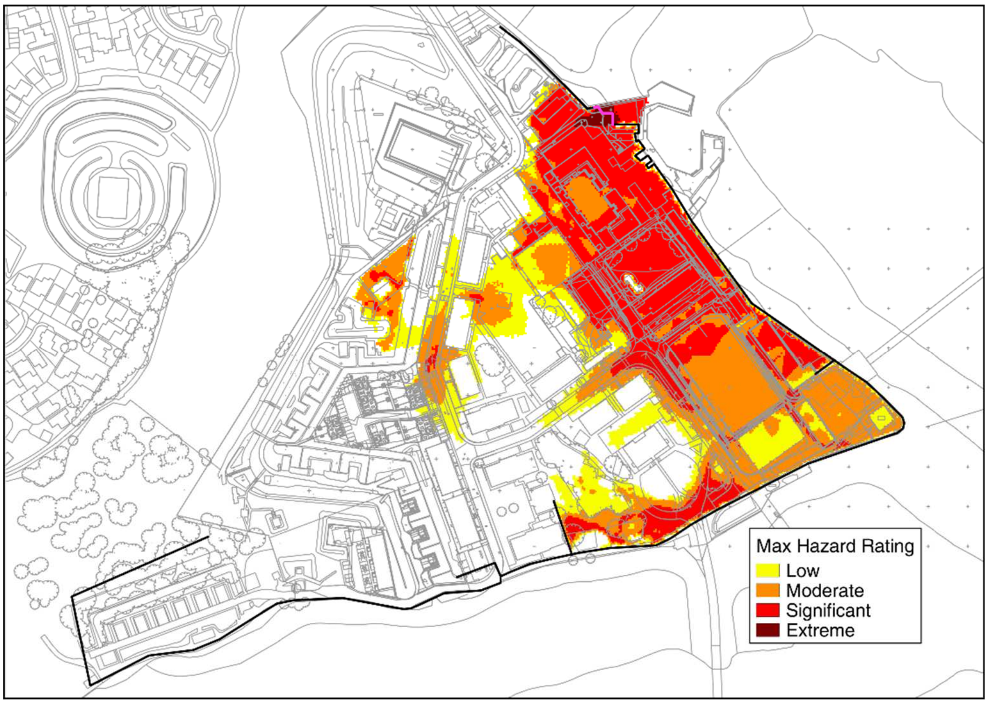

Figure 8 – Maximum hazard rating resulting from Breach A for the Design Event, including wave overtopping.

8.4. Figure 9 shows that the depth of flooding ranges between 0.5 m to 0.75 m as a result of Breach B and wave overtopping under the Design Event. High ground levels and the arrangement of sea defences limit the spread of floodwater from the Breach B, generally confining it to Searle Drive alone. A small amount of flooding is observed, less than 0.25 m, in the non-residential areas which is due only to overtopping.

Figure 9 - Maximum depth of flooding resulting from Breach B for the Design Event, including wave overtopping.

8.5. Figure 10 shows a significant hazard rating within an area located approximately 50 m of Breach B, however, this rapidly diminishes to a low hazard rating for other affected areas.

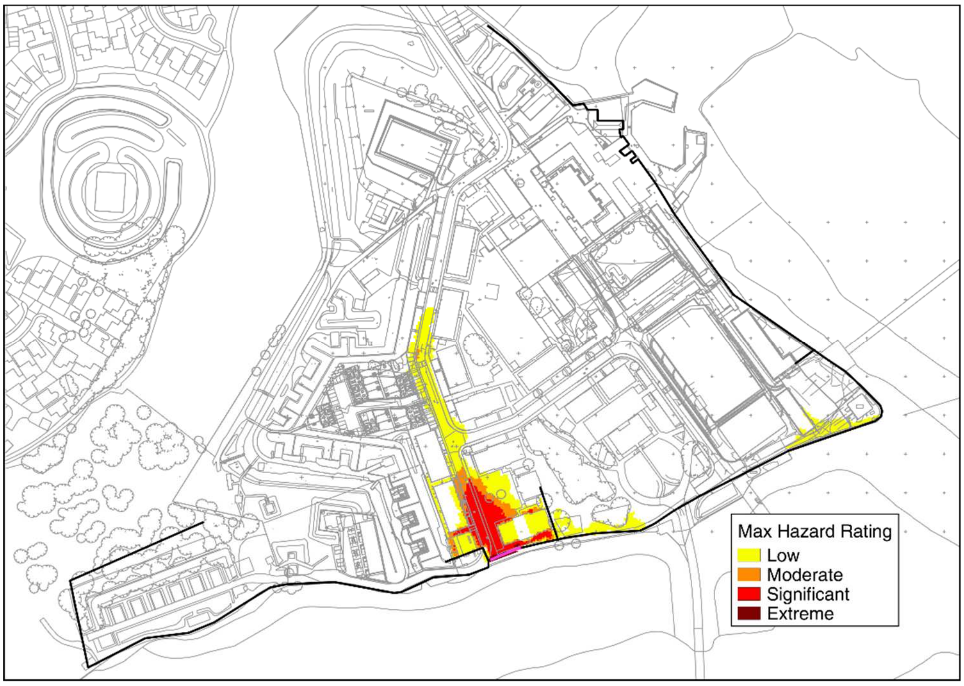

Figure 10 – Maximum hazard rating resulting from Breach B for the Design Event, including wave overtopping.

8.6. The maximum depth of flooding and maximum flood level has been tabulated for three locations on site as shown in Figure 11. Model Extraction Location (MEL) A is selected due to its proximity to Building U and Breach A. MEL B is on Searle Drive, near to the existing and proposed new residential dwellings, in an area shown to be affected during Breach A. MEL C is on Searle Drive, in close proximity to Breach B, near to the existing and proposed residential dwellings.

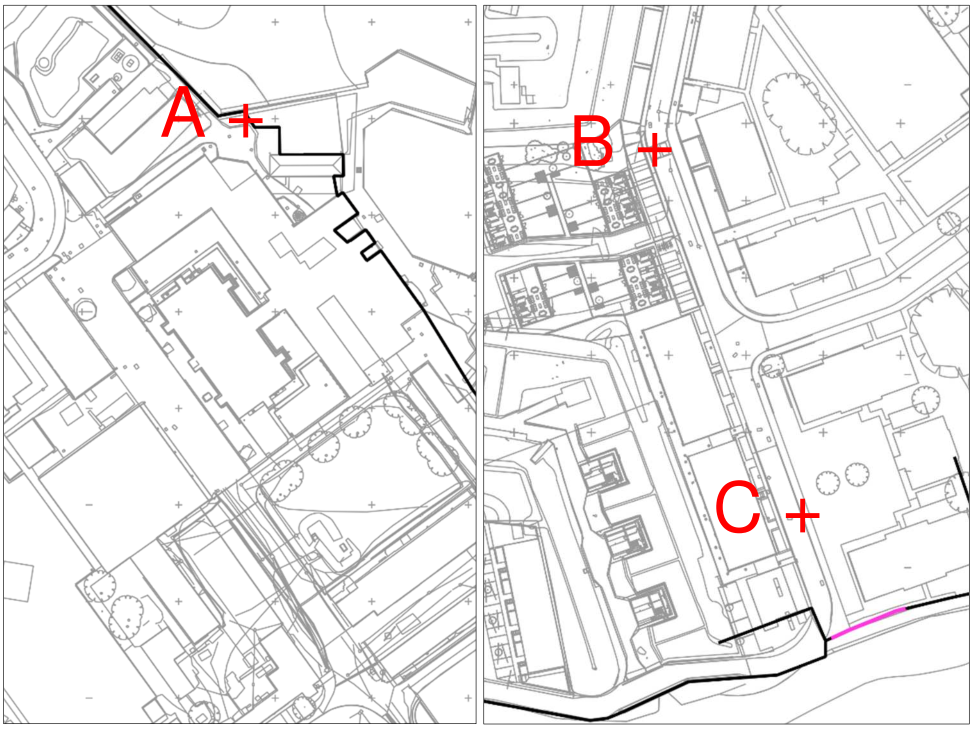

Figure 11 – Model result extraction locations.

8.7. Table 2 shows the maximum depth of flooding and maximum flood level at MEL A, B and C (see Figure 11) during Breach A and Breach B, including wave overtopping. These results show that the flood level close to each breach is generally very close to the level of the extreme surge water level, with a small influence from the wave overtopping near Breach B.

Table 2 – Maximum depth of flooding (m) and flood level (mAODN) for Breach A and Breach B (including wave overtopping) at MEL A, B and C.

Model Extraction Location	Breach A: Design Event		Breach B: Design Event	
	Depth (m)	Flood Level (mAODN)	Depth (m)	Flood Level (mAODN)
A	0.97	4.22	Dry	
B	0.36	4.07	0.06	3.79
C	Dry		0.64	4.27

9. Sensitivity Test Results

9.1. The following Figures and Table show the results of the four sensitivity tests. Each Figure shows the maximum depth of flooding. Table 3 shows the maximum depth of flooding, maximum flood level and the difference compared to the Design Event at MEL A, B and C (see Figure 11). The sensitivity tests have been run for the Breach A scenario only (including wave overtopping), as this represents the greatest extent of flooding for the Design Event conditions.

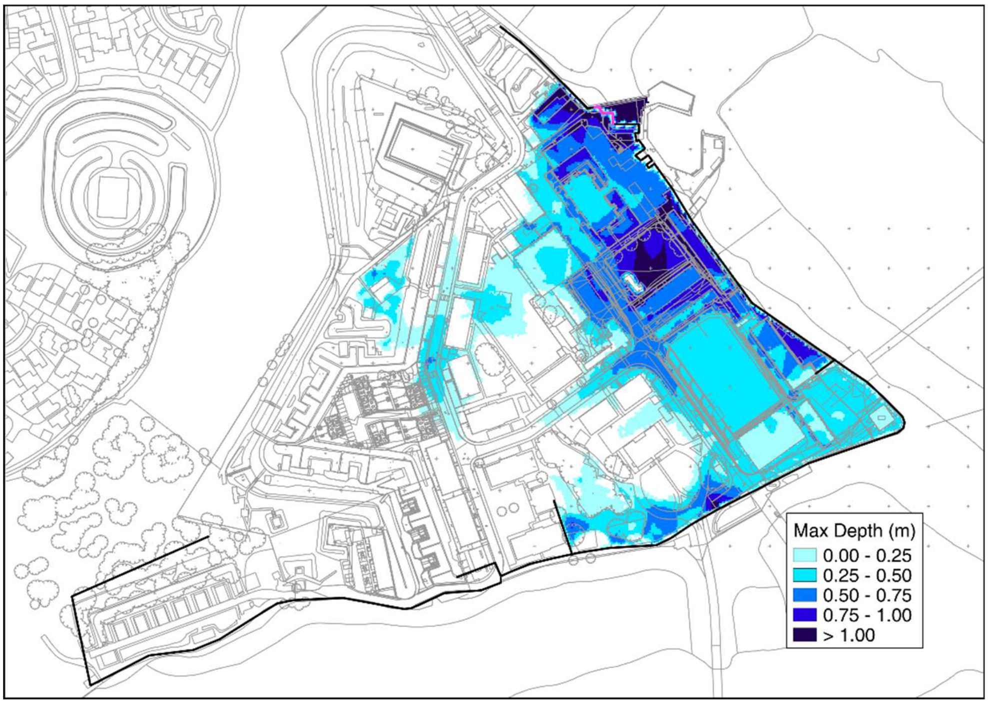

Figure 12 – Maximum depth of flooding due to Breach A (including wave overtopping) with Manning's n roughness increased by 20%.

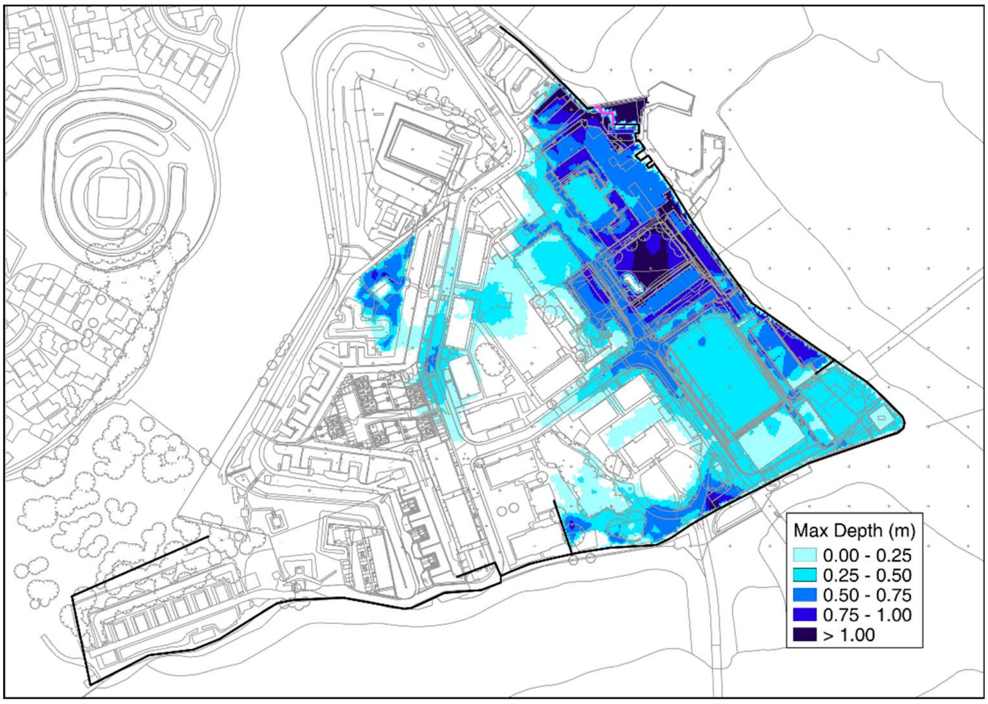

Figure 13 – Maximum depth of flooding due to Breach A (including wave overtopping) with Manning's n roughness decreased by 20%.

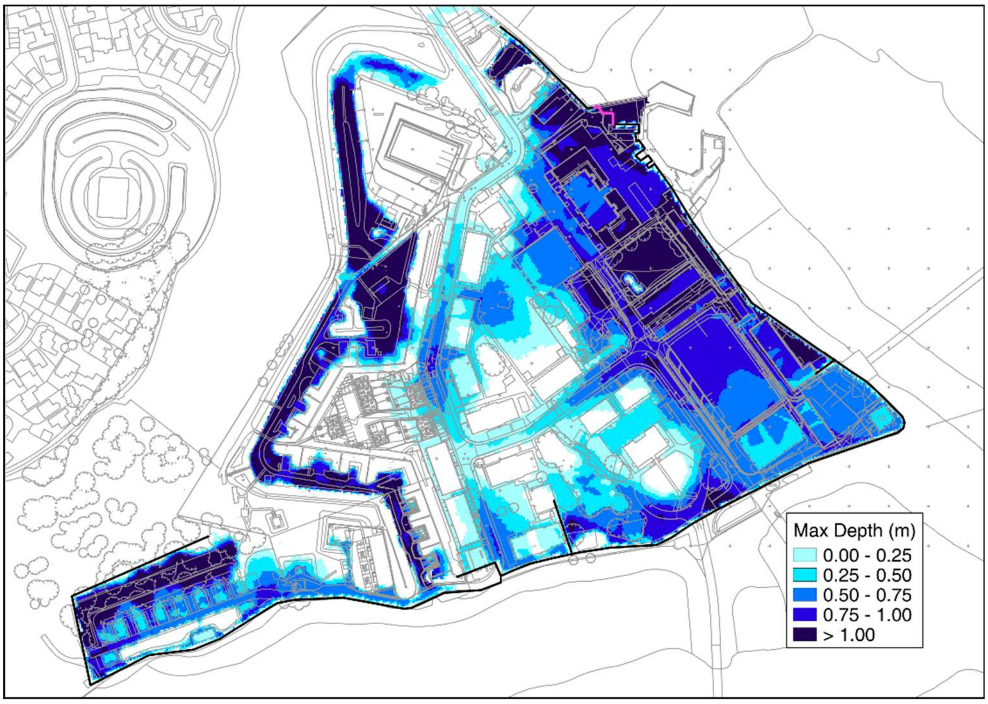

Figure 14 – Maximum depth of flooding due to Breach A (including wave overtopping) with the 1 in 200 year extreme surge level increased by 0.3 m.

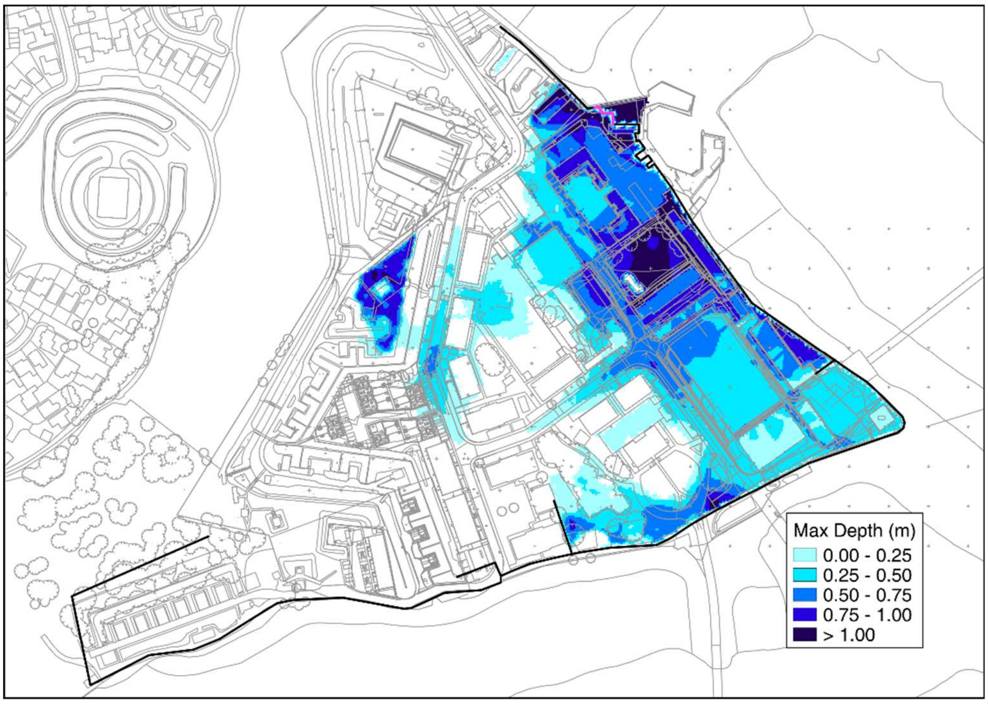

Figure 15 – Maximum depth of flooding due to Breach A (including wave overtopping) with the overtopping rate increased from 3.46 l/s/m to 12 l/s/m.

Table 3 – Maximum flood level [mAODN] and difference compared to the Design Event at model extraction locations A, B and C for all sensitivity tests.

Model Extraction Location	Manning's *n* +20%	Manning's *n* -20%	Surge Confidence interval	Overtopping Confidence interval
A	4.22 (No change)	4.22 (No change)	4.52 (+0.30)	4.23 (+0.01)
B	4.06 (-0.01)	4.09 (+0.02)	4.36 (+0.29)	4.11 (+0.04)
C	Dry	Dry	4.34 (n/a)	Dry

9.2. The results of the sensitivity tests show the following:

a) ***Manning's n ±20%****: The greatest effect of varying the Manning's n roughness is observed with a reduction in the depth of flooding at MEL B by 0.02 m when values of n have been increased by 20%. This is due to the effect of the increased friction on the shallow water depths in the flow path that flood water must take across the site to reach MEL B. Lowering the Manning's value does not result in an equivalent increase in the depth of flooding at MEL B. Little change in the depth of flooding is observed at MEL A which is likely to be due to its close proximity to the breach. Review of Figure 12 and Figure 13 further* *indicates little change in the extent of flooding during either roughness sensitivity scenario and therefore, it is concluded that the model is insensitive to any uncertainty in the estimates of surface roughness.*

b) ***Surge level confidence interval**: Increasing the extreme surge water level by 0.3 m results in an almost equivalent increase in the depth of flooding at MEL A and B. Reference to Figure 14 shows an increase in the extent of flooding commensurate with this greater extreme surge water level, including the flooding of the historic defence ditches which surround the site such that they become a conduit by which flood water can reach the otherwise well-defended south-western corner of the site. The depth of flooding in these areas reaches more than 1.0 m. Similarly, MEL C floods in this sensitivity test to a depth of 0.12 m, where during Breach A it remained dry under Design Event conditions. The results of this test indicate that the model is sensitive to the level of uncertainty associated with the offshore extreme water levels. Although not tested, this would include the lower confidence interval of the extreme surge level during which areas on-site would remain dry, likely including MEL B.*

c) ***Wave overtopping rate confidence interval**: Increasing the wave overtopping discharge rate to 12 l/s/m (an increase of almost 350%) results in only a very small increase (0.04 m) in the depth of flooding at MEL B and minimal increase in the extent of flooding (Figure 15). Therefore, it is concluded that the model is generally insensitive to the wave overtopping discharge rate within estimated confidence limits.*

10. Summary

10.1. A numerical flood model has been constructed to determine the residual risk of flooding, including wave overtopping, to the proposed development at Priddy's Hard, Gosport due to breaches in the defences near Building U and Searle Drive.

10.2. The model shows that **Building U** may be subject to a depth of flooding of up to 0.97 m during the Design Event. Model sensitivity testing has shown that this may increase by no more than 0.02 m as a result of the variation in surface roughness, or due to the uncertainty in the predictions of wave overtopping rates. However, the depth of flooding has the potential to increase by 0.30 m up to 1.28 m in line with the potential uncertainty (±0.3 m) associated with the extreme surge water level estimates of the CFB database.

10.3. The residential development proposed on **Searle Drive** may fall within the flood extent during a breach at either of the tested locations (Breach A near Building U or Breach B at the end of Searle Drive). The southern end of Searle Drive is next to Breach B which results in a potential depth of flooding at that location of up to 0.63 m during the Design Event. However, the increasing land levels in the middle of Searle Drive limit the depth of flooding there to just 0.06 m during Breach B. However, Breach A may lead to a depth of flooding in the middle of

Searle Drive of 0.36 m for the same Design Event. Sensitivity tests have shown that the depth of flooding may increase to 0.76 m at the centre of Searle Drive in line with the potential uncertainty associated with the extreme surge water level estimates of the CFB database.

Appendix 1

Correspondence with the Environment Agency with respect to the breach modelling methodology.

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Environment Agency

Ms Kirsty Thomas
Herrington Consulting Ltd.
Barham Business Park (Unit 6)
Elham Valley Road
Barham
Canterbury
CT4 6DQ

Our ref: HA/2019/121057/02-L01
Your ref: ENVPAC/1/SSD/00068

Date: 27 February 2019

Dear Ms Thomas

Detailed advice - Review modelling specification

Priddys Hard, Heritage Way, Gosport, Hampshire, PO12 4LE

Thank you for accepting our offer to provide detailed planning advice. We have reviewed the following documents:

- 2266_Model_Specification_Table_v2_[Feb19]

We are providing this advice under Agreement No. ENVPAC/1/SSD/00068. Please note we have taken 6 hours to review and provide our advice on these documents which is within our Programme of Works. The total will be £600 plus VAT which is payable on receipt of our invoice.

Environment Agency Advice

We have reviewed the submitted model specification in relation to the proposed development and have the following detailed comments:

<u>Available Environment Agency Data</u>
The Environment Agency has recently completed a new coastal model for the East Solent, which includes Gosport. If required, this model can be supplied as part of a data request and modified to assess the risk to the site.

<u>Model review</u>
We have reviewed the model specification for two breach scenarios for a development at Priddy's Hard, Gosport. The model specification was supplied by Herrington Consulting Limited. No additional data or previously submitted information was supplied for review.

The document provides comment on each of the sections from the model specification.

<u>Description</u>

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No wave overtopping is considered in this model specification – this needs to be included. If wave overtopping is to form part of a bespoke model then we recommend a further review of the amended model specification.

Software
Acceptable use of TuFlow version.

Domain
Grid size of 2m is sufficient. The following points should be followed:

1. Grid orientation should align with the predominant flood route
2. Time step (seconds) should be no longer than 0.5 to 0.25 times the grid size
3. Ensure domain is large enough to accommodate flood extent and prevent 'glass walling'

Base DEM
Acceptable use of up to date LiDAR at sufficient resolution complimented by site survey.

Boundary Source Data
Coastal Flood Boundaries 2011 (CFB) is proposed for use. This is an Environment Agency dataset not a JBA one as described in the model specification. The use of this dataset is correct however attention should be paid to the phasing of the surge and tidal peaks as outlined in the published guidance on https://www.gov.uk/government/publications/coastal-flood-boundary-conditions-for-uk-mainland-and-islands-design-sea-levels.

Any present day model runs must uplift the CFB extreme sea levels from the baseline year (2008) up to the present day using the 'Flood risk assessments: climate change allowances 2016' https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances.

Use of the above climate change guidance for future epochs is proposed correctly but the uplift from 2008 to 2115 is 1.13m not 0.94m.

Climate change allowances for wind speed and wave height are not required unless wave overtopping is being modelled.

Event Scenario
The assessment of a 0.5% AEP event (1 in 200) is acceptable. Baseline and post development scenarios are recommended. The use of three successive tides for model run duration is sufficient (we recommend tidal models to be run between 30-60 hours).

Breach Location
Little information is provided on the rationale for locating the breach. This should be at a location of high risk either where defences are known to be particularly exposed or in poor condition and where a breach would lead to the largest flood risk impact.

Breach timing
Timing of the breach in the model is instantaneous at the peak of the tide. EA internal breach guidance (Breach of Defences Guidance 2017) stipulates that breach should start at least when some loading is applied to the defence:

'In a river or 'non wave' tidal situation this can be considered to be a water level at ¾ of the defence height (consider datum is toe level from depth of breach consideration). Where there is a wave loading we can consider the breach starts either when still water level reaches half defence height or when any overtopping begins whichever is first.'

The current proposal will limit the volumes of water entering the site as the tide will recede once the breach is initiated. A more precautionary method is recommended in line with the above guidance.

<u>Breach width</u>
The guidance for breach width for Estuary/tidal river locations is 50m for earth banks and 20m for reinforced concrete. The breach of 10m at 'Building U' should be either increased or justified.

<u>Grid modifications</u>
Correct use of building footprints and landscape features.

<u>Model Boundaries</u>
Correct use of HT boundary for extreme water levels.

<u>Buildings Method</u>
Correct use of raised footprint and increased Manning roughness values of 0.3.

<u>Roughness</u>
Correct use of Mastermap polygon data and application of mannings roughness following Chow 1959.

<u>Hydraulic Structures</u>
Not applicable.

<u>Model Proving</u>
Sensitivity testing using increased roughness 20% is good. Testing of boundary conditions is also recommended using the CFB interval data.

I hope the above advice is helpful. Should you have any further queries please do not hesitate to contact me.

Yours sincerely

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Disclaimer
Our opinion is based on the information available to us at the time of the enquiry. When the formal planning application is submitted, our position may change if there have been changes to environmental risk or evidence, and/or planning policy.

Appendix 2

Full set of detailed model results including Depth of Flooding, Hazard Rating, Velocity and Flood Level, for all tested scenarios.