# Appendix G-1

Flood Risk Assessment - Part 1

States of Jersey

# Jersey Future Hospital

Flood Risk Assessment

17-9526

1 | 27 February 2018



Job number 237035

**Ove Arup & Partners Ltd**
4 Pierhead Street
Capital Waterside
Cardiff CF10 4QP
United Kingdom
www.arup.com

ARUP

# Document Verification

ARUP

| | | | | | |
|---|---|---|---|---|---|
| **Job title** | | Jersey Future Hospital | | | **Job number**<br>237035 |
| **Document title** | | Flood Risk Assessment | | | **File reference** |
| **Document ref** | | 17-9526 | | | |
| **Revision** | **Date** | **Filename** | Flood Risk Assessment.docx | | |
| Draft 1 | 24 May 2017 | **Description** | First draft | | |
| | | | Prepared by | Checked by | Approved by |
| | | Name | David Leak | John Smith | Peter Thomas |
| | | Signature | | | |
| Draft 2 | 13 Jun 2017 | **Filename** | Flood Risk Assessment.docx | | |
| | | **Description** | Updated following Department for Infrastructure meeting | | |
| | | | Prepared by | Checked by | Approved by |
| | | Name | David Leak | John Smith | Peter Thomas |
| | | Signature | | | |
| Draft 3 | 27 Feb 2018 | **Filename** | Flood Risk Assessment 2018 App.docx | | |
| | | **Description** | Updated to include Westaway proposal and reflecting the 2018 planning application | | |
| | | | Prepared by | Checked by | Approved by |
| | | Name | Carys Langsdale | David Leak | John Smith |
| | | Signature | | | |
| | | **Filename** | | | |
| | | **Description** | | | |
| | | | Prepared by | Checked by | Approved by |
| | | Name | | | |
| | | Signature | | | |

**Issue Document Verification with Document** ✓

# Contents

# 1 Introduction

This report considers flood risk for the Jersey Future Hospital (JFH), including Westaway, located in St Helier, Jersey. It considers flooding from tidal events, fluvial watercourses, groundwater, overland flow, surface water and foul drainage.

A planning application was previously submitted for the main JFH site. This revised application includes the proposed development of Westaway, a side 150m to the northeast of the main hospital, and proposes a large area of development on the main site.

The locations of the Main Build and Westaway are shown in *Figure 1*, below.


*Figure 1 – Location of Jersey Future Hospital Main Build and Westaway*

The Main Build is located approximately 275m from St Aubin’s Bay and within 450m of the Marina. There are no fluvial watercourses within 1km of the main site.

Westaway is located approximately 500m from St Aubin’s Bay and 750m from the Marina. The nearest fluvial watercourse is 900m from Westaway

Within the States of Jersey (SoJ), there is no dedicated body assigned to assessing flood risk. There is also no specific planning requirement for a Flood Risk Assessment (FRA) to be in place. However, through previous history of the site, flooding is known to be a risk. Therefore, it has been agreed with SoJ that an FRA will be undertaken, which will comply with the English National Planning Policy Framework (NPPF) as far as practicable, in order to outline the flooding risks to the site. These are discussed below.

There are no available flood maps for Jersey which show flood zones for different types of flooding. Existing anecdotal information on flooding in this area has been used in conjunction with available records to establish the risk of flooding from different sources.

# 2 Flooding Considerations

## 2.1 Tidal flooding

There have been incidents of the coastal defences being overtopped in recent years along Victoria Avenue, including a storm in 2008 where waves overtopped the coastal defences and caused flooding along Gloucester Street near the southern end of the proposed site, shown in *Figure 2* below.

*Figure 2 - Tidal flooding at the southern end of Gloucester Street*

The coastal defences along Victoria Avenue and the Marina consist of terraced revetments, sloped masonry and rock armour revetments.

Four structures were considered to be vulnerable to wave overtopping that could reach the sites:

- Vertical sea wall running along Victoria Avenue (Type 1)
- Slipway to the south of Victoria Park (Type 2)
- Concrete terrace sea wall adjacent to Les Jardins de la Mer (Type 3)
- Rock armour revetment to the south of the terraced wall section. (Type 4)

The location of these different wall types is shown in the *Figure 3*. This figure also shows the wave direction that was considered within the analysis. A wave direction of approximately 230° allows waves to enter St Aubin's Bay and reach Victoria Avenue without obstruction, resulting in the maximum wave overtopping potential.

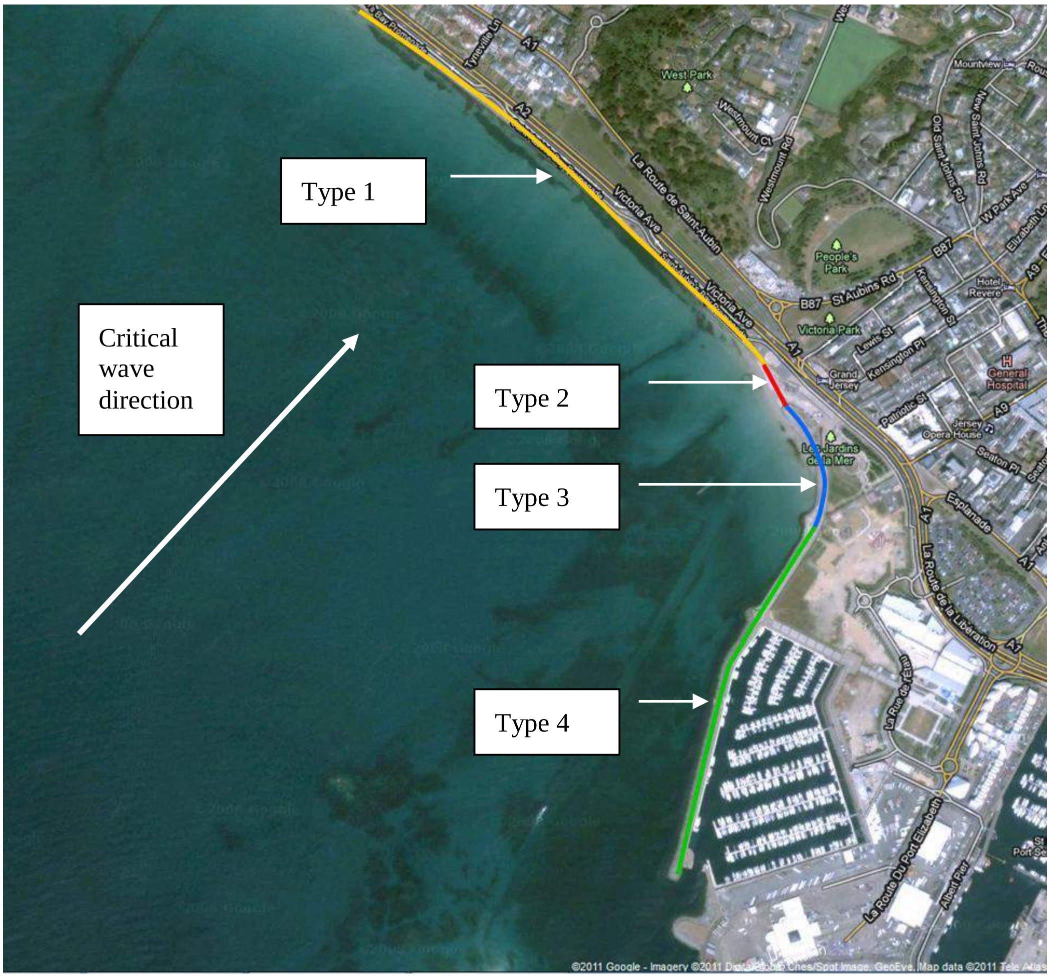



*Figure 3 - Wall section locations*

## 2.1.1 Victoria Avenue (Type 1)

The Victoria Avenue wall is a steeply sloped masonry structure with a curved lower section. Along some sections there is a stepped toe. *Figure 4* below is a photograph of this wall type.

*Figure 4 - Wall Type 1 photograph*

The wall geometry was taken from the drawing provided by TTS with the crest of the wall at +9.1mOD and the toe at +2.0mOD.

*Figure 5* below is an image from the BBC Jersey news website taken during the March 2008 floods showing the wave overtopped volume discharging into Victoria Avenue.

*Figure 5 - Overtopped discharge down Victoria Avenue (BBC Jersey)*

*Figure 6* below shows the partial collapse of the flood defences following the 2014 storms.

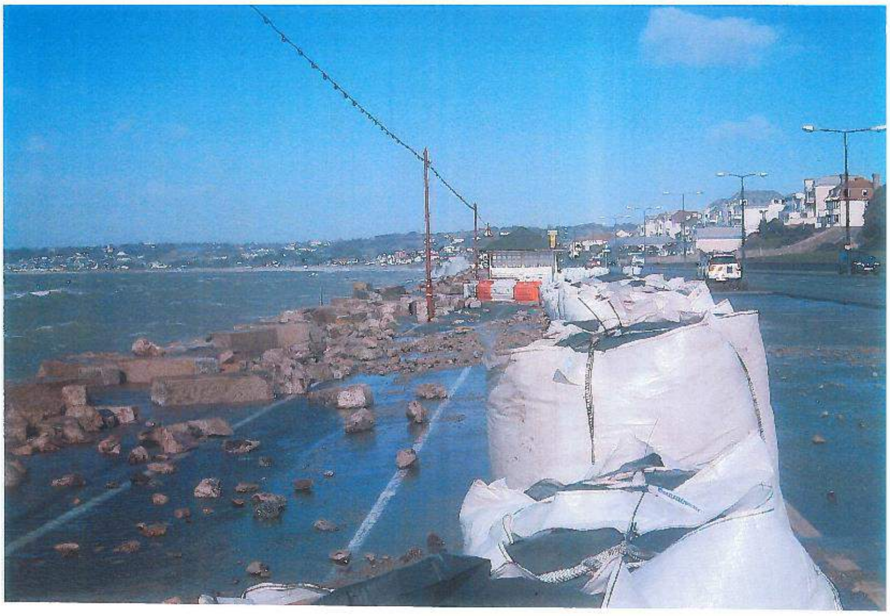


*Figure 6 - Partial collapse of defences in 2014 storms*

### 2.1.2 Slipway (Type 2)

*Figure 7* below shows a photograph of the slipway, wall Type 2.

*Figure 7 - Wall Type 2 - slipway*

The slipway has a width of 10m, a maximum slope angle of approximately 5% and a crest height of +8.0mOD. The surface is made up of blocks, resulting in a reduction factor to account for the increased surface roughness.

### 2.1.3 Terrace Revetment (Type 3)

This wall structure is made up of a sloped revetment of terraced blocks placed on a rock filter layer and central rubble core. It is understood that the toe of the wall is supported on a line of sheet piles driven to rock level. A photograph of this section of wall is shown in the *Figure 8* below.

*Figure 8 - Wall Type 3*

From the drawings provided by TTS the crest level and slope angle were determined to be 9.7mOD and 1:2 respectively.

### 2.1.4 Rock Armour Revetment (Type 4)

This section of wall is a rock revetment type structure with a double layer of 2 to 3 tonne rock armour placed on a stone rubble core. A photograph of the structure is shown in *Figure 9* below.

*Figure 9 - Wall Type 4*

From the drawings provided by TTS the crest level and slope angle were determined to be 9.7mOD and 1:2 respectively.

## 2.2 Fluvial flooding

There are no fluvial watercourses within 500m of the Main Build and Westaway developments.

The nearest watercourses to the Main Build are located approximately 1.1km to the northwest and 1.2km to the northeast. These watercourses are also the nearest to Westaway and are located 1.2km to the northwest and 800m to the northeast of the Westaway site. These watercourses will be unaffected by the proposals. There is a 'Brook Culvert' recorded on Department for Infrastructure (DfI) maps, which suggest that historically there may have been a watercourse within the vicinity of the site.

The watercourse to the west has a rise in terrain between it and the hospital, therefore reducing the risk of flooding to the proposed development. The watercourse to the north is culverted under St Helier approximately 1km away, and therefore minimises the risk of fluvial flooding from this impacting on the Main Build or Westaway.

Therefore, the risk of fluvial flooding to the developments is considered to be negligible and is not considered further.

## 2.3 Groundwater flooding

Ground Investigation has recently been undertaken for the proposed Main Build and Westaway sites. This data has not been reviewed for the purpose of the Flood Risk Assessment as it remains in draft form and is not yet complete. This is with the exception of Westaway, where the Ground Investigation has highlighted a different rock type to the historical records.

### 2.3.1 Main Build

The historical Ground Investigation shows that the groundwater is typically 1.5m deep. The ground conditions are made ground overlying blown sands and alluvium, with a bedrock of mudstone, shale and grit. As these materials have low permeability and there is permeable made ground at the surface this does not suggest that there would be a large fluctuation of groundwater, and there is no record of groundwater flooding in the area currently.

### 2.3.2 Westaway

The ground conditions, as noted in historical records and more recent Ground Investigation are made ground overlying blown sand and alluvium, similar to the main hospital site. The bedrock is andesite, which has moderate permeability. As there is permeable made ground at the surface this does not suggest that there would be a large fluctuation of groundwater, and there is no record of groundwater flooding in the area currently. The recent Ground Investigation indicates the groundwater to be 2m below ground level on Westaway, however, further monitoring is required to confirm this.

Due to the lack of recorded flooding and the low permeability of the alluvium, combined with the Ground Investigation, it is believed that the risk of groundwater flooding is negligible for both the Main Build and Westaway and is not considered further.

## 2.4 Overland flooding

### 2.4.1 Main Build

The existing JGH site falls approximately 3m between Kensington Place in the north and Gloucester Street in the south. The area around the site is drained by positive drainage, in the form of gullies, linear drainage channels, pipes and manholes which connect to the Department for Infrastructure (DfI) surface water drainage system described in Table 1.

There is no evidence of existing surface water flooding occurring on the site, and the gradients across the site are considered steep enough to avoid water from pooling within the site.

The area surrounding the existing hospital is developed with positive surface water drainage. Therefore, there are no dominant overland flow paths which would cause concern to the proposed Main Build.

Within the proposed site, all falls will be away from buildings into positive surface water drainage. Therefore, there will be no overland flow routes which will cause issues within the site and overland flooding is not considered further for the main hospital site.

### 2.4.2 Westaway

On Westaway, surface water flows from west to east, with a fall of approximately 1m across the site. The area around the site is drained by rainwater downpipes, gullies, pipes and manholes which are assumed to connect into the DfI combined sewer on Savile Street as detailed below.

Parade Gardens to the south east of Westaway allows for infiltration to the ground, and the remaining area around the site has positive surface water drainage which means there are no overland flow routes causing issues within the Westaway site.

Therefore, overland flooding is not considered an issue affecting either site and is not considered further.

## 2.5 Existing surface water drainage

### 2.5.1 Main Build

The existing drainage in the area of Jersey General Hospital is shown in Appendix A. There are DfI owned sewers on Kensington Place, Newgate Street, The Parade and Gloucester Street. Known information on these sewers from drawings provided by States of Jersey (SOJ) is summarised in Table 1 below.

| Sewer Location | Size | Description |
|---|---|---|
| Gloucester Street | Approximately 2.45m width at the widest point | Brook Culvert lined with PRC liners. Egg sewer inside brick sewer, with concrete infill |
| Gloucester Street | 1830mm segments lined down to 1525mm | Surface water tunnel with average cover of approximately 5.5m |
| Gloucester Street | 915mm x 760mm | Foul Brick Sewer with PRC liners. 230mm diameter spur connections |
| Gloucester Street | 600mm diameter | GRP Foul sewer concrete bed and surround – gradient of 1 in 73 |
| Kensington Place | 915mm x 710mm lined to 730mm x 530mm. | Combined brick sewer with PRC M196 liner approximately 4.1m to invert |
| Newgate Street | 230mm to 300mm diameter | Foul sewer at gradient of 1 in 76 |
| The Parade | 530mm diameter | Concrete foul sewer at a gradient of approximately 1 in 160 |

*Table 1 – Description of sewers around existing hospital site*

A CCTV drainage survey has been undertaken on the existing drainage within the JGH site, in addition to the Brook Culvert on Gloucester Street. Further survey is required to the main DfI sewers surrounding the development to determine their size and condition. Where required, the proposed development will include the upgrade or replacement of existing sewers.

Surface water flows from the existing hospital connect into the DfI surface water sewer on Gloucester Street, combined sewer on Kensington Place and appear to connect to the DfI foul sewers on Newgate Street and The Parade.

The development is proposed to include works to the Patriotic Street Multi-Storey Car Park. The surface water from the car park will therefore be accommodated within the proposed drainage system of the Main Build. Currently, the surface water from the area of the car park appears to discharge into a brick sewer, earthenware channel on Patriotic Street which flows down Patriotic Place. Historical drawings indicate that this sewer outfalls into the Brook Culvert on Gloucester Street.

The distribution of surface water flows between local sewers remains unclear, however the approximate surface water catchment areas connecting into each DfI sewer is shown in Appendix B.

To estimate existing surface water flows around the area of JGH, Jersey rainfall data and a rainfall intensity of 50mm/hour were used throughout the calculations. This was taken from the Jersey Bye Law Technical Guidance Document Part 6.

The total surface water flow from the existing hospital area and Kensington Place properties was estimated to be approximately 682l/s. A breakdown of these results is shown in Table 2.

| Sewer Location | Approximate storm flows entering sewer (l/s) |
|---|---|
| Newgate Street | 123 |
| Gloucester Street | 177 |
| The Parade | 118 |
| Kensington Place | 193 |
| Multi Storey Car Park | 71 |

*Table 2 – Existing surface water flows from the existing hospital area during a 50mm/hr storm.*

### 2.5.2 Westaway

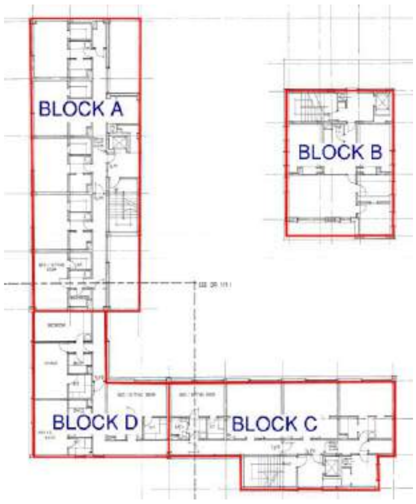



*Figure 10 – Westaway layout and block names*

Westaway consists of four blocks of accommodation (Blocks A-D), as shown in Figure 10. The existing drainage in the site is shown in Appendix C.

There are DfI sewers on Savile Street, Gloucester Street and in Parade Gardens. Information related to these sewers has been provided by SoJ and is summarised in Table 3.

| Sewer Location | Size | Description |
|---|---|---|
| Parade Gardens | 450mm diameter | VC surface water sewer at a gradient of 1 in 131 |
| Savile Street | 840mm diameter | Class H concrete combined sewer at a gradient of 1 in 166 |
| Savile Street | 380mm diameter | GVC foul sewer laid within brick sewer at a gradient of 1 in 164 |
| Gloucester Street | 1830mm segments lined down to 1525mm | Surface water tunnel with average cover of approximately 5.5m |
| Gloucester Street | 1000mm diameter | GRP foul sewer concrete bed and surround – gradient of 1 in 66 |

*Table 3 – Description of sewers around Westaway site*

A CCTV drainage survey has been undertaken on the existing foul and surface water drainage within the Westaway site. The main DfI sewers surrounding the site need further survey to determine their size and condition. Where required, the proposed development will include upgrades to existing sewers or replacement where existing damage means they are not suitable for use by the proposed development.

The surface water flows on the Westaway site have been estimated using the catchment area and a rainfall intensity of 50mm/hour. There are two connections from the site to the combined sewer in Savile Street, with the surface water around Blocks A and B discharging into the combined sewer approximately 15m upstream from the connection from the area around Blocks C and D. The indicative catchment areas are shown in Appendix E. Based on the area drained and the Jersey specific rainfall data, the estimated maximum flow from the site is approximately 69l/s across the two outfalls, with the distribution of flows at each connection point shown in Table 4.

| Sewer Location | Approximate storm flows entering sewer (l/s) |
|---|---|
| Savile Street (upstream) | 35 |
| Savile Street (downstream) | 34 |

*Table 4 – Existing surface water flows from Westaway during a 50mm/hr storm.*

## 2.6 Existing foul drainage

### 2.6.1 Main Build

The layout for the existing foul drainage of JGH is shown in Appendix A.

Existing foul flows from Jersey General Hospital connect into DfI sewers on Gloucester Street, Newgate Street, The Parade, and Kensington Place. In order to estimate the change in flows in the sewers around the site due to the new development, an estimate has been made of the flows discharging into the existing sewers.

The current flows from Jersey Hospital are unknown and therefore an assumption of 6 litres/m$^2$/day was used. This assumption was based on flow rates recorded from 122 hospitals across Wales, where flow rates varied from 0.3-7.1 litres/m$^2$/day. A peaking factor of 6 was used throughout all calculations, and it was assumed that the hospital facilities are used for 12 hours per day. This is a conservative design assumption, as the hospital facilities are likely to be used for more than 12 hours per day. The foul flow contribution from the existing hospital kitchen was based on the preparation of 1300 meals per day. This information was received from Jersey General Hospital. The total foul flow estimates were divided by hospital opening hours to produce flows per unit time. Therefore, assuming the hospital facilities were used for 12 hours per day resulted in a higher flow per second.

Using the existing drainage layout in Appendix A, foul flows from within the existing hospital were assigned to the different sewers based on the most likely connection points, this will need to be confirmed. Appendix C illustrates the approximate foul catchment areas that connect into either Gloucester Street and Newgate Street, The Parade, or Kensington Place. From this, it is assumed that all foul flows from the buildings to the north of Newgate Street discharge to Kensington Place.

The Newgate Street sewer connects into the Gloucester Street sewer, therefore the foul flow estimate in the Gloucester Street sewer includes the contributions from Newgate Street. Therefore it is assumed that 50% of the foul flows from the JGH site discharge to Gloucester Street, and that 25% discharges to The Parade and Kensington Place.

The foul flow estimate from the existing hospital and hotels/restaurants within the future hospital boundary is approximately 39 l/s. A summary of the estimated existing foul flows are shown in Table 5 below.

| **Sewer Location** | **Approximate foul flows entering sewers from existing hospital boundary ($l/s$)** |
|---|---|
| Gloucester Street | 13 |
| The Parade | 6 |
| Kensington Place | 12 |
| **Sewer Location** | **Approximate foul flows entering sewer from hotels and restaurants within future hospital boundary ($l/s$)** |
| Kensington Place | 8 |

*Table 5 – Foul flow estimates from the existing hospital, and hotels/restaurants within the future hospital boundary.*

### 2.6.2 Westaway

There are a total of 56 units, with a mixture of one and two bedroom apartments. Foul drainage from the site outfalls from two 150mm diameter sewers to the 380mm diameter foul sewer laid within a brick sewer on Savile Street. Indicative catchment areas are shown in Appendix F.

To determine whether the proposed change of use will increase flows to the foul sewer, the existing foul flows have been estimated using the population method. Using an assumed occupancy of 1.8 persons for a one-bedroom apartment and 2.3 for a two-bedroom apartment, and estimating that the apartments were in operation for 10 hours a day, average flow from the site is 0.56l/s. Using a peaking factor of 6, the peak foul flow is estimated as 3.33l/s. This is based on standard flow and population rates appropriate to accommodation.

# 3 Assessing Flood Risk

## 3.1 Tidal flooding

### 3.1.1 Main Build

A flood modelling report has been prepared which considers wave overtopping of the existing coastal defences discussed above. There were two main events considered, a 1 in 200 year event plus climate change, and a 1 in 1000 year present day event. The flood modelling report is included in Appendix D. It has been assumed for the flood modelling that the coastal defences will be maintained in a reasonable state of repair, and that they will not breach in a large storm event.

The model shows the 1 in 200 year event plus climate change is more onerous and the results of this are shown in Figure 11. There is some inundation at the south west corner of the main site. The peak level of the flooding in this area is 8.2m AD.

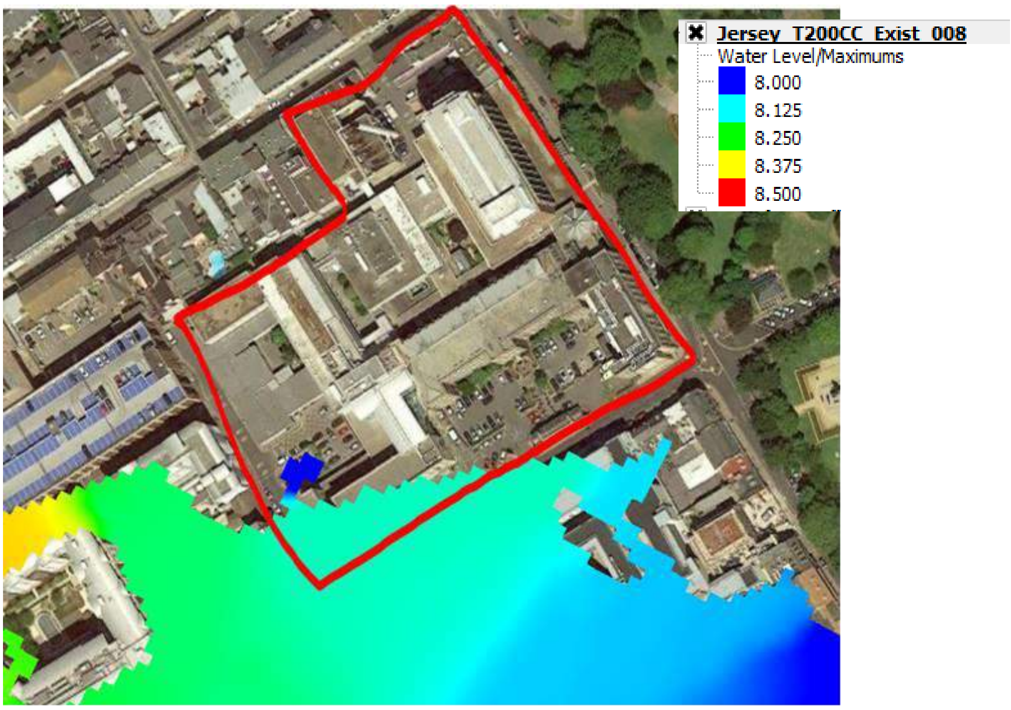



*Figure 11 - 1 in 200 year plus Climate Change flood levels*

The results of the 1 in 1000 year event present day are shown in Figure 12. Similarly to the 1 in 200 year event plus climate change, there is some inundation at the south west corner of the site, however the peak level is lower at 7.853m AD.

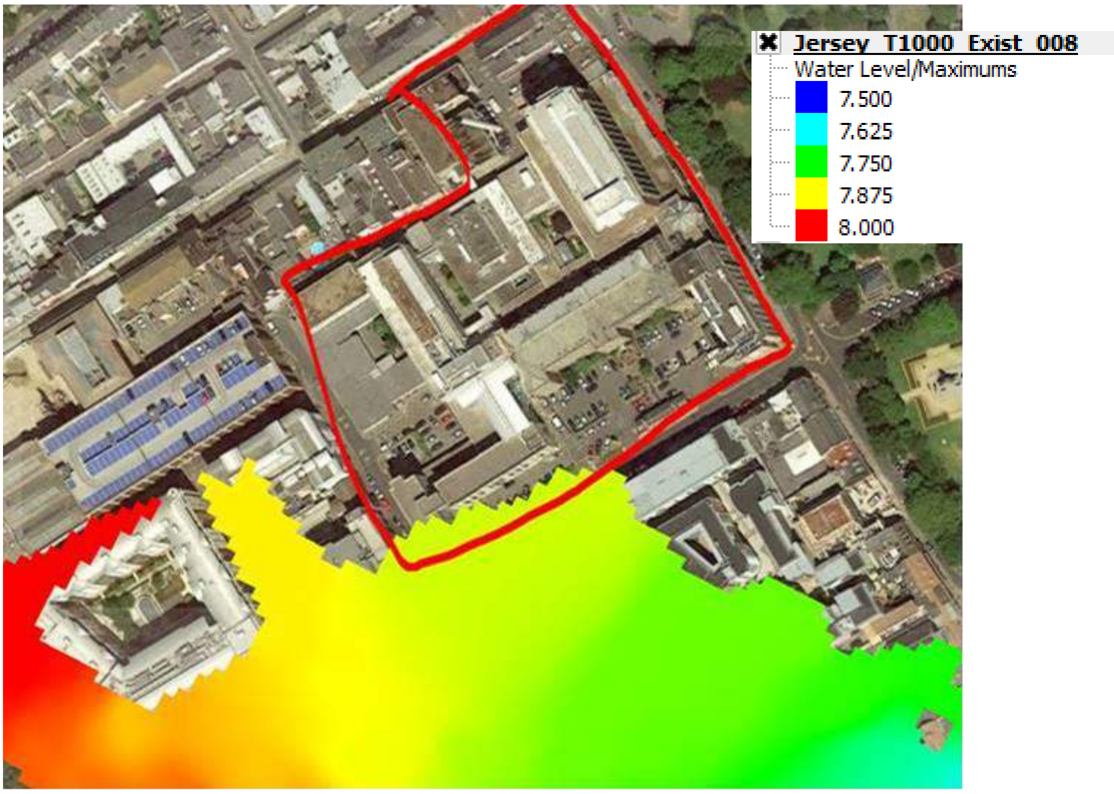



*Figure 12 - 1 in 1000 year present day flood levels*

NPPF recommends that 0.6m freeboard should be allowed above the 1 in 200 year plus climate change event. This would mean that a minimum level of 8.8m AD should be used for the hospital threshold. The threshold level for the building is currently proposed to be at 10m AD. This will reduce risk of flooding in an extreme event.

The basement level will be below the extreme flood level. Therefore, the basement should be sealed, with no openings below the threshold level to ensure water cannot enter the basement.

Therefore, the residual risk from tidal flooding on the Main Build is considered negligible.

### 3.1.2 Westaway

The existing ground level on the Westaway site is greater than 11m AD. This is considerably higher than the peak flood levels of the 1 in 200 year plus climate change event and the 1 in 1000 year present day event. Therefore, the risk of tidal flooding on Westaway is not considered further.

### 3.1.3 Main Build tidal flooding exception test

NPPF recommend that the exception test is done on the proposed Main Build, as it is assumed to be located within Flood Zone 3.

*Part 1*

The exception test states that the sustainability benefits to the community should outweigh the flood risk. The existing hospital is currently at capacity and is in need of renewal. There has already been a significant exercise to confirm the most appropriate location for the hospital, from a range of different options. This has been approved by the Council of Ministers. Therefore, it is considered that the sustainability benefits outweigh the flood risk.

*Part 2*

This requires the developer to show that the proposed development would be safe from residual flood risk. This is done through consideration of several different criteria if the site flooded. The relevant risks are discussed below.

*Access and egress*

Access and egress would not be possible from Gloucester Street in the case of a flood. However, access is still possible to the hospital from Kensington Place and Newgate Street. The accident and emergency ambulance drop off would also be above the 8.8m AD.

*Operation and maintenance*

The basement contains much of the plant for the hospital. This would be protected by sealing the basement to stop the ingress of water from flooding. Therefore, the hospital would be able to operate. Maintenance would be possible from the access on Kensington Place.

*Design of development to manage and reduce flood risk where possible.*

As discussed above, the threshold level of the building will be 0.6m above the 1 in 200 year plus climate change event to manage flood risk. Basements will be sealed to stop the ingress of water.

*Flood warning and evacuation procedures*

SoJ have an Emergency Measure Plan which considers flooding incidents. It is recommended that evacuation procedures are developed for JFH in case of an extreme flood event.

## 3.2 Justification for Location of Development

### 3.2.1 Main Build

Tidal flood modelling shows that the main hospital is at risk of flooding in a 1 in 200 year event plus climate change. There is some uncertainty as to whether this would be within Flood Zone 2 or 3. However, as a conservative approach, the English NPPF guidance for a Flood Zone 3 area has been applied.

There has already been a significant exercise to confirm the most appropriate location for the hospital, from a range of different options. This has been approved by the Council of Ministers. Therefore, it is considered that the sequential test does not apply.

As it has been assumed that the site is located within Flood Zone 3, and that a hospital is considered to be a ‘more vulnerable’ development, in accordance with Table 3 of the NPPF, the exception test applies. This will be discussed below.

### 3.2.2 Westaway

The tidal flood modelling shows a maximum water level of 8.0m in the 1000 year tidal event. The existing level of the Westaway site is greater than 11m AD, and therefore, as there would be no flooding of Westaway during this event, it lies within Flood Zone 1. Whilst the site is classed as ‘more vulnerable’, the exception test does not apply because it lies within Flood Zone 1.

## 3.3 Proposed surface water drainage

The DfI have stipulated that the proposed surface water drainage system should accommodate flows within the network without flooding for up to a 1 in 30 year return period. A 30% increase in surface water flow was used for climate change considerations in accordance with UK best practice.

Following consultations with DfI, it is proposed that surface water flows should outfall into a dedicated surface water sewer which is currently not the case for the main hospital or Westaway.

### 3.3.1 Main Build

The construction of the Main Build will not affect surface water volumes as the impermeable areas across the hospital site remain the same. However, the distribution of surface water flows between each sewer will change.

Surface water from the existing hospital area, Kensington Place properties and the multi-storey car park will be separated from foul drainage, and where possible discharge into the dedicated DfI surface water tunnel in Gloucester Street.

Due to the phasing of the construction and the proposed basement connection underneath Newgate Street, it may be required to have a connection to the combined sewer on Kensington Place.

In accordance with UK best practice, the sewers will be designed to discharge a 1 in 30 year + 30% climate change storm event. It is therefore estimated that the proposed surface water flow from the main site will be approximately 800l/s.

### 3.3.2 Westaway

The impermeable area on Westaway is not proposed to change, and therefore the surface water flows will not be affected by the proposed development. The proposed sewers will, however, be designed to accommodate the 1 in 30 year + 30% climate change storm event.

The flow from the site in a 1 in 30 year storm event, including a 30% allowance for climate change, is estimated to be approximately 93l/s.

There is not a dedicated surface water sewer in close proximity to Westaway, and therefore various options will need to be considered in order to discharge into a dedicated sewer.

To align with the DfI proposal, a new surface water sewer would be required from Westaway to the surface water tunnel on Gloucester Street. This could be via Savile Street or Parade Gardens, but would require 170-230m of new sewer to be laid.

Alternatively, the surface water flows from the site could be attenuated to Greenfield Runoff Rate or another reduced flow deemed suitable by SoJ. The attenuated flow could outfall into the low capacity surface water sewer in Parade Gardens or the combined sewer on Savile Street. This proposal is subject to agreement with States of Jersey and the Parish of St Helier.

## 3.4 Proposed foul drainage

### 3.4.1 Main Build

To estimate the foul flows from the main building following the proposed development, architectural plans received 7th March 2018 have been used. These plans detail the proposed area of each floor within the building.

Using a flow assumption of 6 litres/m$^2$/day, the proposed peak foul flow from the new site is estimated to be 39l/s. It is estimated that the peak foul flow produced by Phase A of the works will be approximately 14l/s and similarly, from Phase B will be 25l/s.

The proposed foul flows following the completion of the development is estimated to be similar to the existing flows from the site. It is proposed to discharge surface water to a dedicated sewer and therefore the demand for capacity in the existing foul sewer on Gloucester Street and combined sewer in Kensington Place will be reduced. It is therefore proposed to discharge the proposed foul flows in a similar manner to the existing.

### 3.4.2 Westaway

To estimate the proposed foul flows from Westaway, architectural plans received 8th March 2018 have been used. These plans detail the proposed area of each floor within the building.

Using a flow assumption of 6 litres/$m^2$/day, the proposed peak foul flow from the new site is estimated to be approximately 3.3l/s, remaining the same as the existing site. As the surface water is proposed to discharge to a dedicated sewer, the demand for capacity on the existing foul sewer in Savile Street will be reduced.

# 4 Conclusion

This assessment has considered the different types of flooding that could occur as a result of Jersey Future Hospital Main Build and Westaway. After assessment, the main flood risks to the site are considered to be from tidal flooding and surface water drainage.

Westaway is located within a Flood Zone 1 area and the existing ground levels are considerably higher than the peak flood level of the 1 in 200 year flood which accounts for climate change. Therefore, tidal flooding is not considered to be an issue on this site.

The Main Build is considered a ‘more vulnerable’ site, and is assumed to be located within a Flood Zone 3 area. Therefore, in accordance with the NPPF, a sequential test has been undertaken. With the recommendations which have been set out within the report, it is considered an appropriate development.

The measures discussed above will be used to protect the hospital in the event of extreme flooding. These will ensure that the risk due to tidal flooding is minimised.

The proposed JFH will contain flows within the drainage system for a 1 in 30 year + climate change event. Further details will be provided to SoJ to confirm the capacity of the existing surface water sewers to allow these connections.

# Appendix A


**Legend:-**

- Existing Foul Drainage
- Existing Storm Drainage
- Existing HV Electricity
- Existing LV Electricity
- Existing Gas
- Existing Water
- Existing Telecoms

**Notes:-**

1. These drawings are produced for RIBA Stage 2 and are subject to change in subsequent design phases.
2. The positions of the existing services on this drawing have been interpreted from information received from the statutory local authorities and a site visit. It is subject to confirmation by CCTV survey.
3. Contractor to identify any unknown services prior to commencement of works.
4. Contractor to consult with appropriate statutory authorities prior to commencement of works on their assets.

# Appendix B

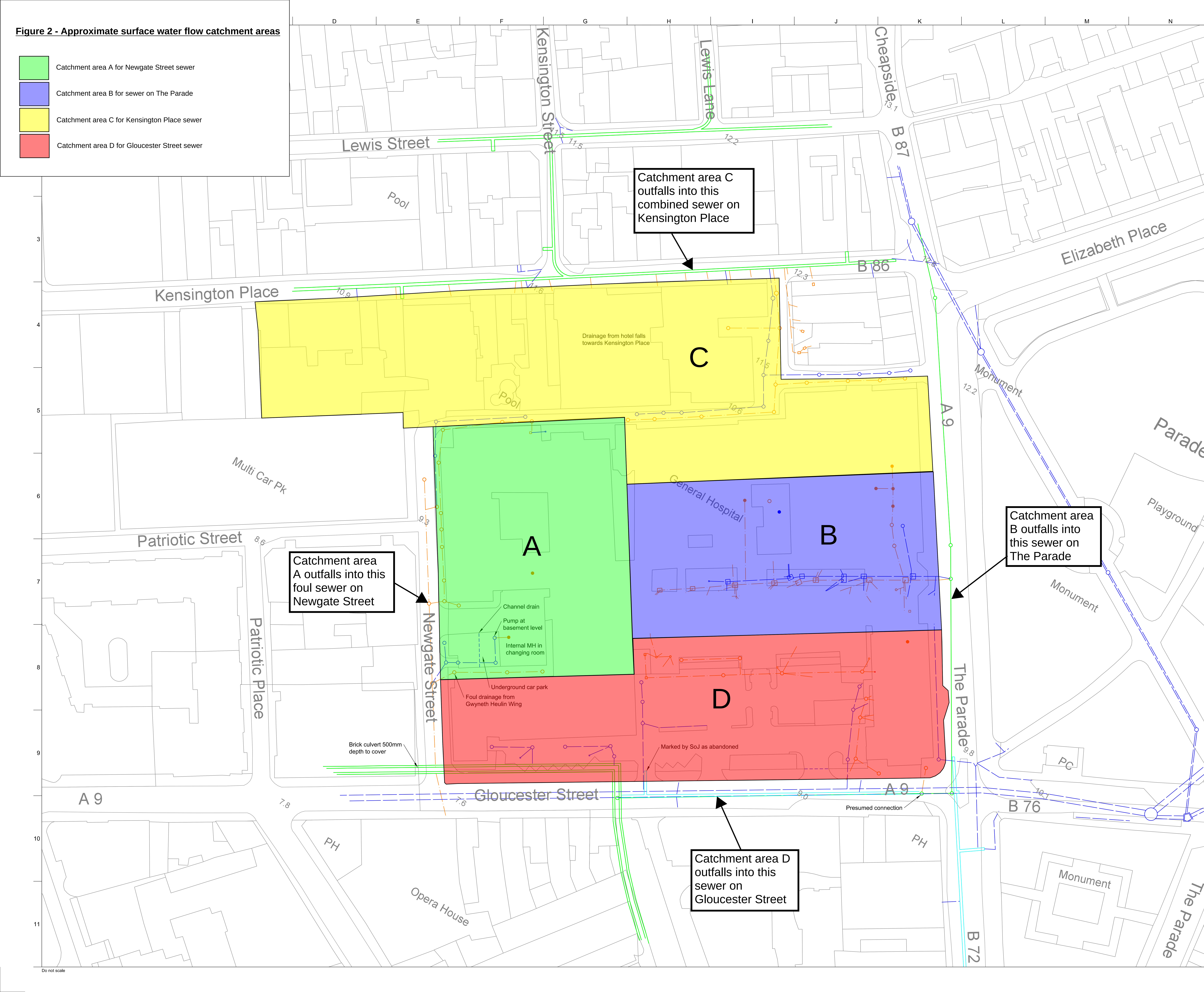



Figure 2 - Approximate surface water flow catchment areas

# Appendix G-1

Flood Risk Assessment - Part 2

# Appendix C

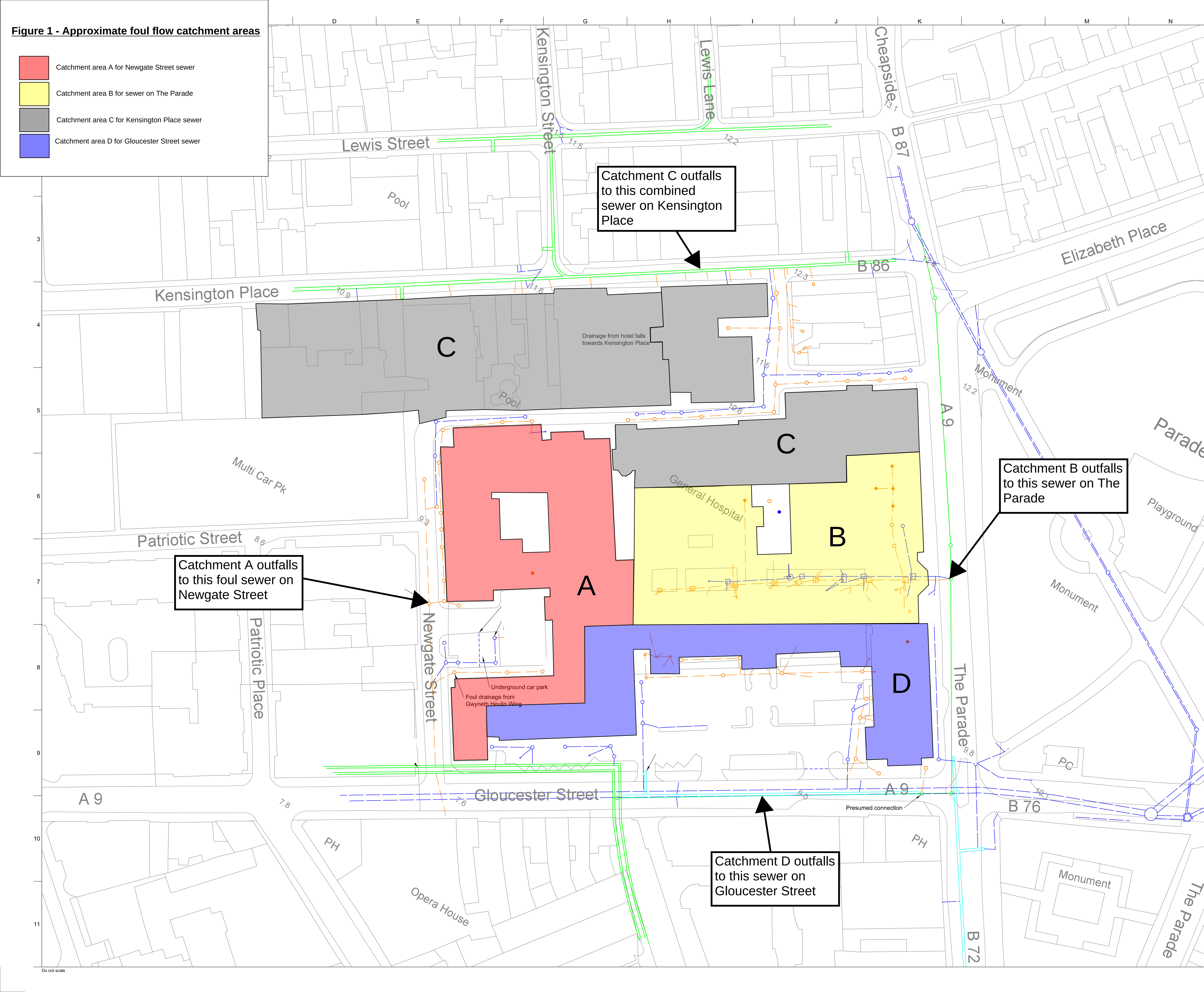


**Figure 1 - Approximate foul flow catchment areas**

# Appendix D

# St Helier Coastal Risk Assessment

Draft Report

May 2017

Arup
4 Pierhead Street
Capital Waterside
Cardiff  CF10 4QP
United Kingdom

# JBA Project Manager

Daryl Taylor
JBA Consulting
Bradley House
Park Five
Harrier Way
Exeter
EX2 7HU

# Revision History

| Revision Ref / Date Issued | Amendments | Issued to |
|---|---|---|
| 1.0 DRAFT / 01.06.2017 | | PT, DL |
| | | |

# Contract

This report describes work commissioned by David Leak, on behalf of Arup, by an e-mail dated 13th April 2017. Arup's representative for the contract was David Leak. Pamela Wong, Adrian Kolander and Lachlan Attard of JBA Consulting carried out this work.

Prepared by .................................................. Pamela Wong BEng (Civil)
Civil Engineer

Prepared by .................................................. Lachlan Attard BEng(Civil)(Hons)
Civil Engineer

Reviewed by ................................................. Adrian Kolander BSc MSc (Civil)
Civil Engineer

Approved by .................................................. Daniel Rodger BSc MEng CEng CMarEng MIEAust MICE IMarEST
Director

# Purpose

This document has been prepared as a Draft Report for Arup. JBA accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

JBA has no liability regarding the use of this report except to Arup.

## Acknowledgements

JBA would like to acknowledge Jamie Mason, Fiona Glover, Laura Hopley, John Searson of the Department of the Environment, Aaron Gavey of the Ports of Jersey, Dr Thomas Prime of the UK National Oceanography Centre, and Andrew Blogg of Future Aerial Innovations Ltd for their assistance during this project.

## Copyright

# Executive Summary

This assessment was undertaken by JBA Consulting on behalf of Arup to consider the impacts of extreme sea levels, waves, overtopping and the resulting inundation at a proposed site in St Helier, Jersey. The aim of this study is to assess the impacts of inundation from coastal extreme events by assessing the joint probability of coincident offshore waves and extreme sea levels, wave transformation and breaking, wave overtopping, and resulted inundation through St Helier and at the proposed site.

A range of joint probability extreme wave and water level scenarios have been established offshore of Jersey. These have been simulated through a wave transformation model to the nearshore zone, adjacent to four difference coastal defences. The rate of wave overtopping during an extreme event at each defence was estimated using the EurOtop Neural Network tool, considered the most suitable approach for the composite defences at St Helier. Wave overtopping rates were estimated for return periods between 1 in 1-year and 1 in 20-year, and matched against anecdotal information and images of the March 2008 event to validate predictions. The Neural Network was then used to estimate extreme overtopping for a range of 1 in 1000-year joint probability events under present day conditions, and 1 in 200-year joint probability events under climate change conditions. The worst-case joint conditions for each return periods was identified and used to assess the potential flood risk within St Helier.

Inundation modelling was undertaken using the TUFLOW hydrodynamic model. The model was established for a 30-hour period with two tides, the second equalling the worst-case peak extreme sea level, waves and overtopping rates for each return period. The resulting flood conditions were were observed to spread throughout the low lying foreshore, along roadways and into the port areas of St Helier. At the proposed site the worst-case conditions arose from the 1 in 200-year plus climate change event, where peak flood levels were estimated to be 8.2mAOD.

# Contents

# List of Figures



# List of Tables

# Abbreviations

AOD ................................ Above Ordnance Datum
DEM ............................... Digital Elevation Model
DSM ............................... Digital Surface Model
ESL ................................ Extreme Sea Level
GIS ................................. Geographic Information System
JBA ................................ Jeremy Benn Associates
LiDAR ............................. Light Detection and Ranging
MHWS ............................ Mean High Water Spring
NOC ............................... National Oceanography Centre
SWAN ............................ Simulating WAves Nearshore (wave model)
TSL ................................ Total Seal Level

# 1 Introduction

## 1.1 Project background

This assessment was undertaken by Jeremy Benn Associates (JBA), on behalf on Arup, to prepare a coastal risk assessment to consider the impact of extreme sea levels, waves, overtopping and the resulting inundation at a site in St Helier, Jersey, as shown in Figure 1-1.

The aim of this study is to assess the impacts of inundation from coastal extreme events by assessing the joint probability of coincident offshore waves and extreme sea levels, wave transformation and breaking, wave overtopping, and resulted inundation through St Helier and at the proposed site. This has been undertaken to understand the risk posed by two different extreme events; a 1 in 1000-year event, under present day conditions, and a 1 in 200-year event, under a 2117 climate change condition.


Figure 1-1: Location of proposed site - existing Jersey General Hospital

## 1.2 Report structure

In addition to this introductory chapter, this report is summarised into the following sections:

- **Chapter 2 (Drivers for coastal risk**) describes the processes that lead to a coastal risk, such as wave transformation, breaking, overtopping and inundation.
- **Chapter 3 (Wave modelling)** outlines the approach to calculate extreme wave conditions at the study site.
- **Chapter 4 (Wave overtopping modelling)** outlines the approach to calculate the overtopping resulting from the nearshore wave conditions.
- **Chapter 5 (Inundation modelling)** outlines the approach to calculate the inundation due to wave overtopping at the study site.
- **Chapter 6 (Summary of assessment)** summarises the modelling undertaken due to combined wave and extreme water levels at the study site.

## 2 Drivers for coastal risk

### 2.1 Background

Coastal flooding is a complicated process, affected by a number of dependant and independent variables. Figure 2-1 illustrates the main components of sea-level variation that contribute to coastal flooding during a storm event. The base sea-level, often referred to as either the still water sea-level or total sea-level, is comprised of the underlying astronomical tide and the passage of a large-scale storm surge. These two components determine the average sea-level for a particular location at a particular time. Whilst this variable is very important in terms of coastal flooding, still water-induced flooding is normally limited to sheltered locations such as tidal rivers and harbours. Not surprisingly, the sea is not still during a storm event and for more exposed locations, such as the shoreline of St Helier, most flooding occurs through wave action, rather than still water flooding.

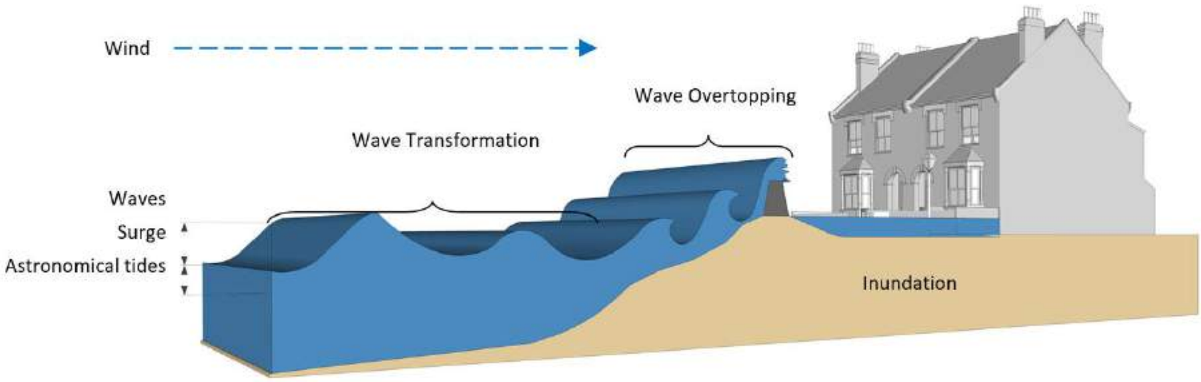



Figure 2-1: Components of sea-level variation that lead to typical coastal flooding

Wave action is a complex process controlled by a number of factors. The manner in which these factors combine determines the magnitude of any wave induced flood impacts. Waves generate in deep water and then propagate towards land. As they do so, they enter shallower bathymetry where wave transformation processes occur, including shoaling, diffraction, refraction, depth limitation and breaking. These waves are also subject to additional influence from wind. The consequence of these processes is that the properties of the waves, when they reach the base of flood defences, are quite different to the waves in deep water. It is these nearshore waves that are of most importance because they interact with beaches and defences and lead to wave overtopping.

Wave overtopping itself is also a complex process controlled by the state of the sea (depth, wave properties), the geometry of the beach and local flood defences. The impact of all of the above flood risk drivers during a particular storm is also heavily dependent upon the location and orientation of the defences with respect to the sea. This means that while one location may be flooded during a storm event another, just a short distance away, may have lesser impacts due to its orientation with respect to the dominant wind/wave direction.

At present there is no one numerical model or calculation approach able to replicate all of these processes. Instead, they are represented through a suite of numerical models as shown in Figure 2-2.


Figure 2-2: Modelling procedure for coastal flooding assessments

# 3 Wave modelling

## 3.1 Introduction

The extend and severity of coastal inundation is dependent on the nearshore wave and water level conditions. These have been considered in terms of the joint-probability of offshore extreme waves coinciding with extreme sea levels, and the subsequent wave transformation processess as they propagate towards the coastline. A range of offshore joint probability scenarios have then used, each simulated through the surf zone to estimate the nearshore conditions, which have been used in subsequent wave overtopping calculations.

## 3.2 Joint probability simulations

The coastal flood risk at the proposed site has been assessed for two return period and climate change events. They are:

- 1 in 1000-year event, under present day conditions
- 1 in 200-year event, under a 2117 climate change condition.

Scenarios were developed using the following datasets.

1. The National Oceanographic Centre (NOC) Jersey Sea level and coastal conditions climate review report[1], completed in March 2017. This provides a range of joint probability extreme sea level and offshore was scenarios for return periods between 1 and 200-years, under present day conditions. These are provided for a location representing the Jersey Wave Buoy (49° 04' 90N, 02° 13' 00W). The report also provides 'marginal' extreme values for sea levels alone, and sea level rise assumptions under a range of climate change scenarios.
2. The Jersey Wave Buoy data has been provided by the UK NOC. This contains significant wave height, maximum wave height, and periodic wave period data between 1996 and 2012. The record has several gaps, including the March 2008 extreme event.
3. GIS analysis of potential swell wave directions originating from the Atlantic Ocean.

This data was used to develop joint probability boundary conditions for the following two scenarios.

A 1 in 1000-year event, under present day conditions: Has been developed by modifying the joint probability scenarios provided in the NOC report. The report presents 25 potential 1 in 200-year offshore events. For each scenario, the sea levels were increased by 0.17m to represent 1 in 1000-year levels, which is the difference between the 1 in 200 and 1000-year marginal extreme sea level estimates. The wave conditions were increased by a nominal 10%. The wave period was estimated based on a relationship between wave height and the 90th percentile wave period at the Jersey Wave Buoy (e.g. cases where longer period waves were observed). The wave direction was set at 240$^{o}$ deg/N, which is considered a conservative assumption of the 'worst case' and likely to over-estimate the resulting nearshore wave conditions. Wind conditions of 17m/s (force 7) from 255$^{o}$ deg/N were applied, based on typical conditions in the leadup to the 2008 storm surge[2].

A 1 in 200-year event, under climate change conditions: Has also been based on the 1 in 200-year offshore joint probability scenarios provided in the NOC report. For each scenario, the present day sea levels were adjusted to 2117 by using extrapolated sea level rise estimates. The median emissions scenario, 95th percentile confidence interval was applied, represented by AR5_4.5RCP in the NOC report (Table 4), which extends to 2100. The rate of sea level between 2100 to 2117 was extrapolated using an annual rate of 0.007m/year, based on the latter years of the presented data, resulting in a total sea level rise of 0.75m. Wave periods were assigned using the relationship established at the Jersey Wave Buoy, the wave direction was set at 240$^{o}$ deg/N, and wind conditions of 17m/s (force 7) from 255$^{o}$ deg/N were applied.

The two sets of offshore wave model boundary conditions are presented in Table 3-1.

1 NOC (2017). Jersey seal level and coastal conditions climate review.

2 NOC (2017). Jersey seal level and coastal conditions climate review. Chapter 5.1: Details of 2008 surge provided by Jersey Met Office.

Table 3-1: Offshore joint probability scenarios

| Scenario | 1 in 1000 year, present day | | 1 in 200 year, climate change | |
|---|---|---|---|---|
| Combination | Sea level (mAOD) | Hs (m) | ESL (mAOD) | Hs (m) |
| 1 | 1.2 | 8.1 | 1.8 | 7.4 |
| 2 | 1.7 | 8.1 | 2.3 | 7.4 |
| 3 | 2.2 | 8.1 | 2.8 | 7.4 |
| 4 | 2.7 | 8.0 | 3.3 | 7.3 |
| 5 | 3.2 | 8.0 | 3.8 | 7.3 |
| 6 | 3.7 | 8.0 | 4.3 | 7.3 |
| 7 | 4.0 | 7.7 | 4.5 | 7.0 |
| 8 | 4.2 | 7.6 | 4.8 | 6.9 |
| 9 | 4.7 | 7.6 | 5.3 | 6.9 |
| 10 | 5.1 | 7.2 | 5.7 | 6.5 |
| 11 | 5.2 | 7.0 | 5.8 | 6.4 |
| 12 | 5.7 | 6.7 | 6.3 | 6.1 |
| 13 | 5.8 | 6.6 | 6.4 | 6.0 |
| 14 | 6.0 | 6.1 | 6.6 | 5.5 |
| 15 | 6.2 | 5.8 | 6.8 | 5.3 |
| 16 | 6.2 | 5.5 | 6.8 | 5.0 |
| 17 | 6.3 | 5.0 | 6.9 | 4.5 |
| 18 | 6.4 | 4.4 | 6.9 | 4.0 |
| 19 | 6.4 | 3.9 | 7.0 | 3.5 |
| 20 | 6.6 | 3.3 | 7.1 | 3.0 |
| 21 | 6.6 | 2.8 | 7.2 | 2.5 |
| 22 | 6.6 | 2.2 | 7.2 | 2.0 |
| 23 | 6.7 | 1.7 | 7.2 | 1.5 |
| 24 | 6.7 | 1.1 | 7.2 | 1.0 |
| 25 | 6.7 | 0.6 | 7.2 | 0.5 |

## 3.3 Wave model development

Nearshore wave conditions have been estimated using a spectral wave model, which simulates the development and transformation of waves as they propagate from deep-water to the shoreline. The industry-standard SWAN (Simulating Waves Nearshore) model was used, it is a third-generation wave model capable of simulating wave-wave interactions, shoaling, refraction, wave breaking, and energy distribution.

The model grid, with which SWAN performs its calculations was designed using an unstructured mesh generation method, with a spatially vary resolution ranging from 500m at the offshore boundary and 10m in the nearshore adjacent to the defence structures. Wave data is estimated from the Jersey wave buoy (Ref: 62027), located 8km offshore of St Helier. The offshore model boundary followed the depth contour of the buoy, which is located in 34m of water. The model mesh is shown in Figure 3-1.

The model bathymetry has been sourced from a number of surveys and previous models. For areas close to the shoreline 0.25m Light Detection and Ranging (LiDAR) data was used, flown in 2017 by Future Aerial[3]. This was merged with data provided by the Jersey Government[4], which included processed bathymetry from a previous HR Wallingford MIKE21 Spectral Wave model, that combined harbour swath bathymetry surveys. The model bathymetry is shown in Figure 3-2.

3 LiDAR flow on 30-31 March 2017 by Future Innovations Ltd, 50 Ashbourne Road London, W5 3DJ,
4 Geophysical Data supplied by States of Jersey Chief Minister's Department – Information Services, Jubilee Wharf

Model outputs have been specified in the nearshore zone adjacent to coastal defences. Model outputs have included the significant wave height, Hmo (m), spectral mean period, Tm-10 (s), and peak direction (θ Deg/N).

The wave model has not been calibrated, due to the lack of quantifiable data between the offshore buoy and the nearshore zone. Instead, a more pragmatic approach was undertaken where the results of both the wave model and wave overtopping model were compared to historic events to assess their performance as a whole.

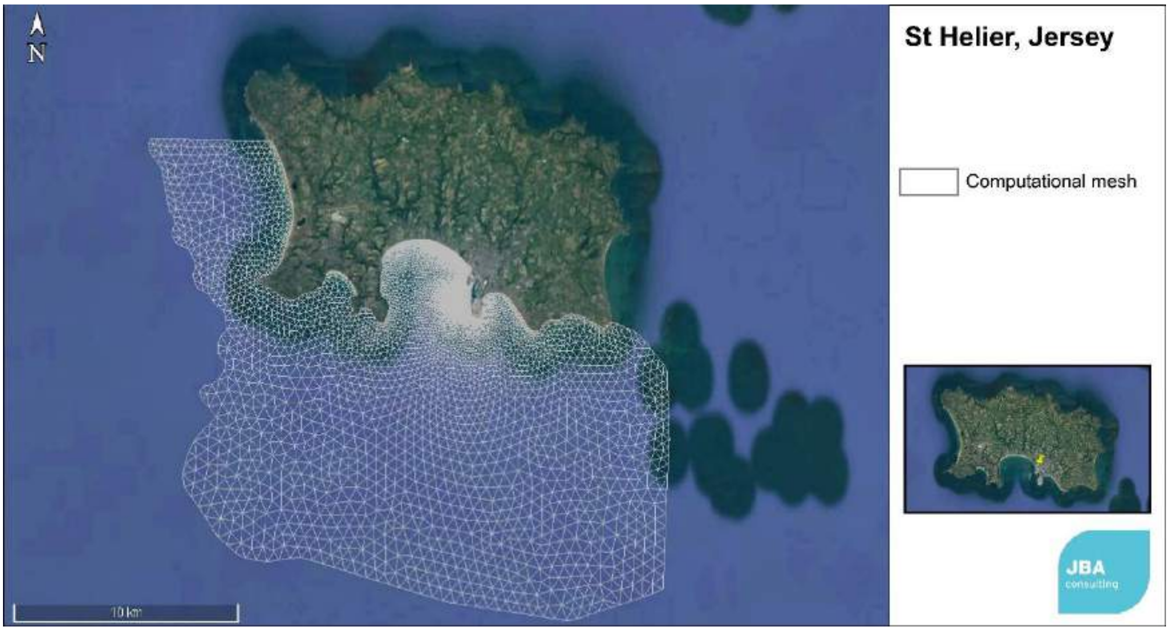



Figure 3-1: Computational model mesh

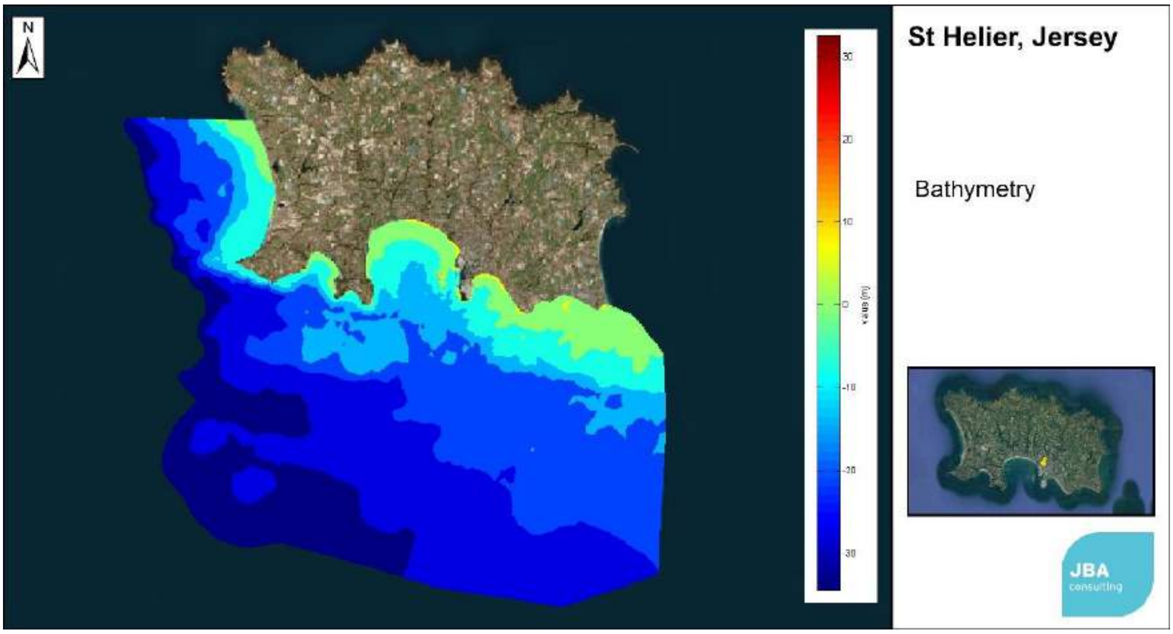



Figure 3-2: Wave model bathymetry (mAOD)

# 4 Wave overtopping modelling

## 4.1 Approach

The complexity of the physical processes leading to wave overtopping introduces a high degree of uncertainty into its quantification. As a result, the overtopping caused by individual waves is not typically calculated; instead the average overtopping rate for a particular sea-state is estimated using empirical or physical models. An example is the Neural Network tool, which was used for this study. The overtopping estimates undertaken for this assessment are described in the following sections:

- Observed wave overtopping
- Validation of wave and overtopping models using observed events
- Extrapolation to higher return periods
- Modelling of worst-case wave overtopping.

## 4.2 Observed wave overtopping

Wave overtopping occurring at St Helier can be categorised into three mechanisms. The first occurs as strong onshore winds carry spray over the coastal defences. Whilst a nuisance, this mechanism does not lead to coastal inundation and is not typically included in wave overtopping calculations. The second is inundation due to sea levels alone, which occur if the total sea level (tide plus surge) exceeds the crest level of a defence. This has been reported to occur around the harbour areas of St Helier, and needs to be included in any inundation assessment. The final mechanism occurs as waves break directly on the seaward face of a defence, which can cause large volumes of water to overtop the structure and inundate the surrounding land.

Anecdotal information and photographs have been provided on the nature and frequency of coastal inundation at St Helier. This has provided a conceptual understanding of five high-risk defences (see Figure 4-1) that has been used to validate wave and overtopping models. Information on each of the defences is summarised below, with a full description provided in Appendix A.

Responses to survey:

- Has wave overtopping been observed in and around St Helier? Yes
- How frequently does it overtop: Seasonally (winter months).
- What is the typical magnitude of overtopping:
    - Spray: monthly, during winter months
    - Minor splashes: monthly, during winter months
    - Moderate overtopping (limited to wetting the first 5m behind the defence): every two months during winter months
    - Major overtopping causing running water behind the defence: Annually.

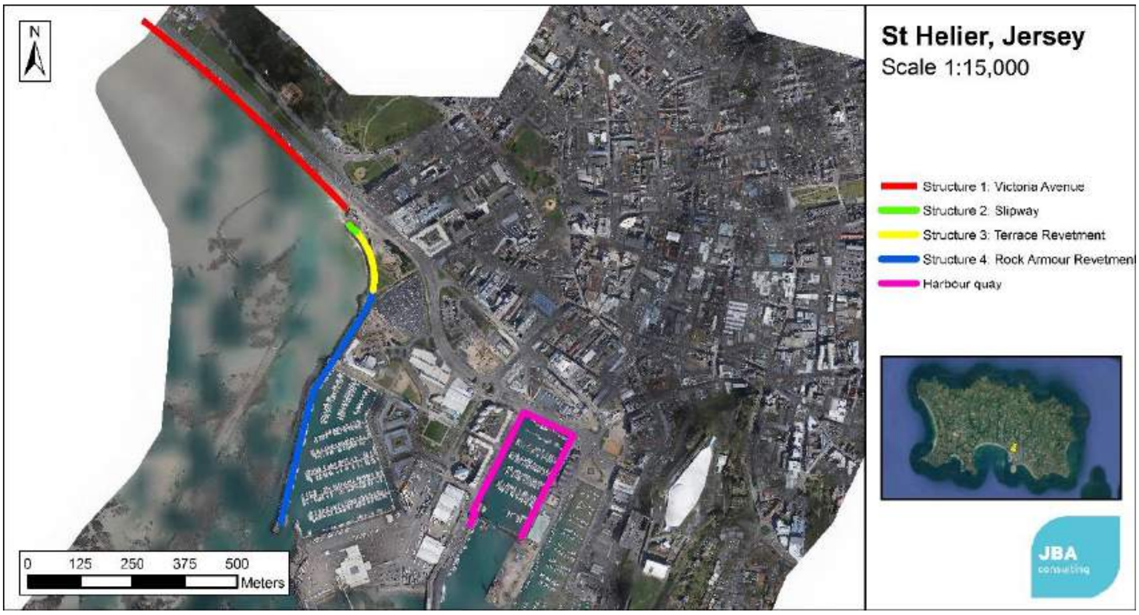



Figure 4-1: Key overtopping locations in St Helier

### 4.2.1 Structure 1: Victoria Avenue seawall

The Victoria Avenue seawall is a steep sloped masonry structure with a mixed of curved lower sections and stepped toe. Its exposed face is oriented towards 224º deg/N. It extends around 1km west of the slip way at St Helier Beach, fronting Victoria Avenue. It was heavily damaged during the March 2008 event, although structural engineering reports suggest it was in a poor condition prior to the event and its subsequent damage was not due to extreme wave overtopping alone.

### 4.2.2 Structure 2: Slipway

The slipway is located at the east of the Victoria Avenue seawall. It inlcudes a stone masonry sloped revetment, at a 1(v):2(h) gradient, with a small vertical rear wall. Its exposed face is oriented towards 227º deg/N. The structure allows additional protection to the slipway, used for beach access by vehicles towing water craft. In terms of wave overtopping, the majority of water breaking over the structure will be returned to the sea.

### 4.2.3 Structure 3: Terrace Revetment

The terraced revetment extends south from the slipway. The structure is made up of a sloped revetment of terraced blocks placed on a rock filter layer and central rubble core. The toe of the wall is supported on a line of sheet piles driven to rock level. It follows a long, curved shape, with an average direction of 254º deg/N. No information of overtopping has been provided at this location.

### 4.2.4 Structure 4: Rock armour revetment

The rock armour revetment forms the outer breakwall of the Elizabeth Marina. Design drawings show the revetment to have a double layer of 2 - 3 tonne rock armour, placed on a stone rubble core. Its exposed face is oriented towards 303º deg/N. No photographed overtopping has been supplied, however the majority of overtopped water is expected to fall into the marina.

### 4.2.5 Harbour quay and walls

Within the St Helier marina there is the potential for coastal flood inundation. Whilst protected from waves, the crest levels of the adjacent quay walls and access points allow a potential source of extreme water level flooding. The crest elevation of the quay walls are approximately 6.5m AOD, with a rear wall present to prevent further inundation if water levels breach the promenade.

## 4.3 Validation of wave and overtopping models for observed events

Evaluating the performance of wave overtopping models is the most complex element of a coastal flood risk assessment that involves wave overtopping. This stems from the fact that recorded overtopping data do not exist. A formal quantitative evaluation of the performance of overtopping models is therefore not generally possible and a more qualitative approach must be taken. Nevertheless, this element of the performance evaluation and optimisation process is arguably the most important for St Helier, where the most significant source of flood risk is from wave overtopping. The development and validation of wave overtoppping included three steps:

- Development of overtopping models using the Neural Network
- Selection of an event with observed wave overtopping, and estimating the peak rate of overtopping
- Calibration to match observations.

### 4.3.1 Development of overtopping models using the Neural Network

The Neural Network has been used to estimate the rate of overtopping. This empirical-based model is described in the industry standard EurOtop manual as the most suitable methodology for evaluating wave overtopping for composite defences such as seawall structures and armour. Even so, as with all calculation approaches, the Neural Network tool has limitations. Estimates are given based on a dataset of small-scale physical model tests which are affected by model and scale effects, the accuracy of measurement equipment and wave generation techniques. As a result, it is important that the results of the Neural Network should be supported by observations and anecdotal information to provide an evidence-base to its performance in regular, more frequent events (say 1 in 1-year to 10-year return periods) before results are extrapolated to larger return period events.

The Neural Network uses wave and geometric properties to estimate wave overtopping. As shown in Figure 4-2, this include 15 parameters such as crest height (Rc); armour height (Ac); armour width (Gc); berm elevation (hb); berm width (B); upper slope (αu); lower slope (αd); and roughness (γf). Each of the four coastal defences subject to overtopping (defences 1-4) were schematised for use by the Neural Network, and are shown in Appendix B. Defence 5 (harbour walls) was represented directly with the flood inundation model.

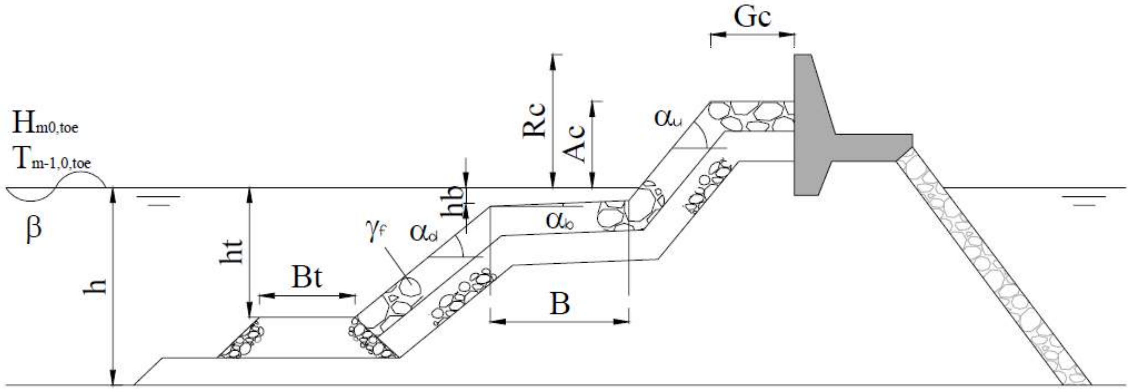



Figure 4-2: Neural Network coastal defence profile schematisation

### 4.3.2 Modelling of a significant event:

The preferred event for validation occurred in March 2008. However, data from the Jersey wave buoy or other nearby gauges was not available for calibration, which was assumed to be damaged during the storm. Instead, multiple joint probability events were simulated and the worst-case result compared against the return period of the March 2008 event.

The NOC Jersey sea level and coastal conditions climate review report[5] estimated the event to have a return period 1 in 10-years. However, it is believed this has been assigned based on sea levels only. If large waves occurred simultaneously (which are the likely cause of the gauge malfunction) the return period may have been higher, e.g. 1 in 20-years.

Multiple joint probability scenarios were simulated for a range of return periods. Lower return periods (e.g. 1 in 1 to 1 in 20-years) were used for model validation, and higher return periods (up to 1 in 1000-years) used to understand the response for larger events. The Neural Network schematisations and SWAN wave model outputs were altered in an iterative process to achieve a suitable overtopping rate that matched anecdotal observations, where expected overtopping rates have been assigned (see below). This was made in conjunction with descriptions within the EurOtop manual, which relates hazardous situations to overtopping rates and volumes. The full EurOtop tables are provided in Appendix C for pedestrians, vehicles and structures, which were used to generate the following assumptions:

- Defence 1 and 2 is expected to have the higher rate of overtopping.
- The protection from rocky reefs suggests Defence 4 will have the lowest rate of overtopping.
- Based on anecdotal information, overtopping is expected for a 1 in 1 year event, that should be limited to 10 l/s/m for Defence 1 and 2.
- The March 2008 event had observed damage to the seawall which would typically mean the peak overtopping rate was larger than 200 l/s/m. However, structural reports indicate seawall was in poor condition, and did not fail due to wave overtopping alone. Wave overtopping was observed and photographed, so the rate of overtopping is expected to be upwards of 50 l/s/m at Defence 1 and 2.

The results of the qualitative wave overtopping validation are provided in Table 4-1. A balance had to be made between achieving the observations at both 1 in 1-year and 20-year return periods, e.g.

5 NOC (2017). Jersey seal level and coastal conditions climate review.

iterations could be made to increase the overtopping rate for a 1 in 20-year event, however they also increased the overtopping for the 1 year event. The model results match the observations, and reflect low-level overtopping in a 1 in 1-year (beyond the limit of trained staff) and higher overtopping that is likely to cause damage to a lightly protected promenade (or a defence in poor condition) in a 1 in 20-year. As such it was considered to have a suitable performance for this assessment.

Table 4-1: Worst-case overtopping rate for all defence sections (l/s/m)

| Return period | Defence 1 | Defence 2 | Defence 3 | Defence 4 |
|---|---|---|---|---|
| 1-year | 3.2 | 12.9 | 0.1 | 0.0 |
| 10-year | 11.2 | 42.0 | 0.4 | 0.1 |
| 20-year | 13.9 | 49.7 | 0.6 | 0.1 |
| 50-year | 18.6 | 63.1 | 0.9 | 0.2 |
| 100-year | 20.1 | 70.2 | 1.1 | 0.3 |
| 200-year | 24.8 | 82.0 | 1.5 | 0.4 |
| 1000-year | 43.3 | 122.8 | 3.4 | 0.9 |

## 4.4 Modelling of worst-case wave overtopping

The validated wave and Neural Network model was used to simulate the worst-case joint probability events for the two design events (as described in Section 3.2):

- 1 in 1000-year event, under present day conditions
- 1 in 200-year event, under a 2117 climate change condition.

Each of the 25 potential scenarios were simulated through the wave and overtopping model for both events, and the worst-case runs identified. For both events the SWAN wave model was then used to simulate a 36-hour storm event. Here offshore wave conditions remain constant, reflecting the peak wave conditions, and a varying sea level was applied based on a Mean High Water Spring (MHWS) tidal profile for St Helier, plus a 36-hour tidal surge to increase water levels to the required worst-case joint probability total sea level. The wave overtopping was estimated at each hour of the simulation, as shown in Figure 4-3 and Figure 4-4 for the 1000-year and 200-year (climate change) scenarios. Defence two resulted in the highest rate of overtopping, peaking at 123 l/s/m for the 1000-year event, and 273 l/s/m for the 200-year (climate change) event.

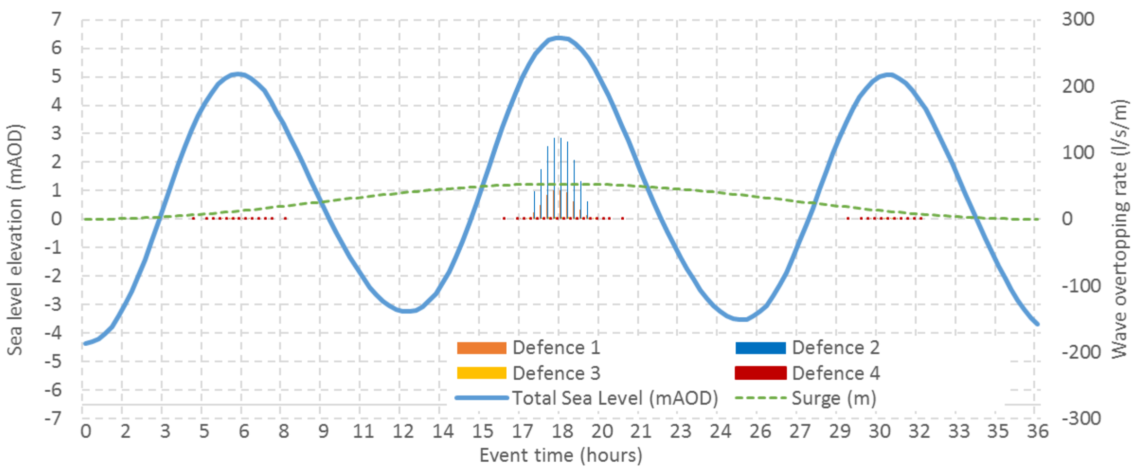



Figure 4-3: Estimated wave overtopping rates for 36-hour storm event: 1 in 1000-year event, under present day conditions

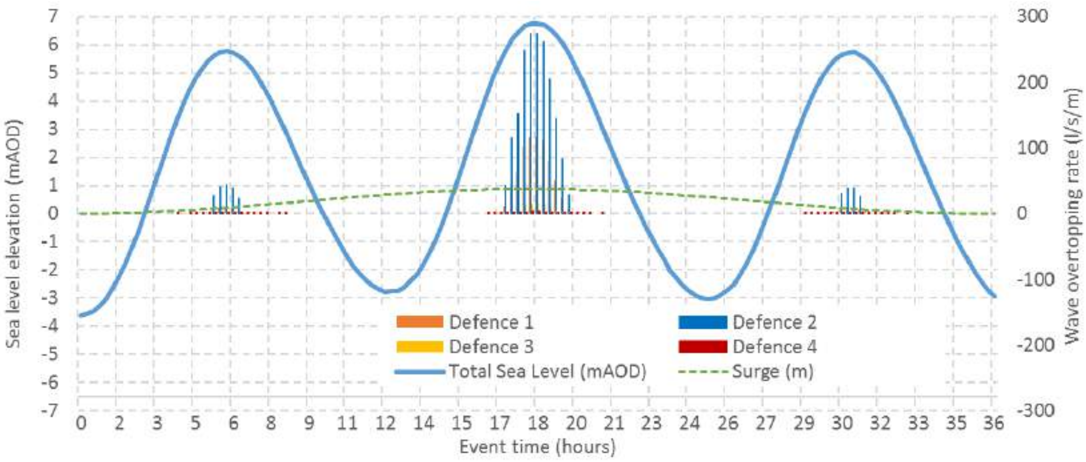



Figure 4-4: Estimated wave overtopping rates for 36-hour storm event: 1 in 200-year event, under climate change conditions

# 5 Inundation modelling

## 5.1 Introduction

During an extreme coastal event both the total sea level plus waves overtopping the coastline may inundate the surrounding coastal floodplain, and have the potential to cause widespread flooding. This has been estimated using a hydrodynamic model coupled with both a time-varying total sea level boundary plus the estimated overtopping during the event. The approach to develop and use the model to estimate flood inundation is outlined in the following sub-sections.

## 5.2 Model overview

### 5.2.1 Summary of model setup

Flood inundation modelling for this study was undertaken using a 2D hydrodynamic model constructed using TUFLOW[6]. The model was used to estimate the coastal inundation extent for a design 1 in 1000-year, present day event and a 1 in 200-year, climate change event.

The model extends midway along Victoria Avenue seawall, where the coastal strip narrows due to the local topography. It extends to the southern end of the St Helier harbour, covering an area of 2.75km$^2$ as shown in Figure 5-1. The model used a 4.0m resolution with a timestep of two seconds.


Figure 5-1: TUFLOW flood inundation model domain

### 5.2.2 Topography and roughness

A varying hydraulic roughness was used across the 2D model domain, established based on aerial photography. An appropriate Manning's n value was applied to each of these classifications derived from Hicks and Mason (1998)[7] and cross-checked with Chow (2009)[8]. The values used within the model are shown in Table 5-1.

6 TUFLOW version 2013-12-AB-w64, 64bit. http://www.tuflow.com/.

7 Hicks, D.M. & Mason, P.D., Roughness Characteristics of New Zealand Rivers, NIWA, Christchurch, (1998), 329pp.

8 Chow, V.T. (1959). Open Channel Hydraulics. New York, NY: McGraw-Hill Book Co.

Table 5-1: Land use descriptions and applied Manning's n Values.

| Land use description | Manning's n |
|---|---|
| Buildings | 0.300 |
| Inland and coastal water | 0.030 |
| Natural surface and gardens | 0.070 |
| Manmade surface roads and paths | 0.025 |
| Trees, rough land and scrub | 0.100 |
| Marsh, reeds or saltmarsh | 0.046 |

### 5.2.3 Modifications to the Digital Terrain Model

TUFLOW requires a topographic grid, or Digital Elevation Model (DEM), to represent the land surface. Ground level information was derived from Light Detection and Ranging (LIDAR) data, flown for this project, and combined with nearshore bathymetry where needed (refer to Section 3.3). The two datasets were smoothed together in ArcGIS to minimised transitions which could cause model instabilities. Other changes to the model domain are as follows:

1. Filtering issues. The DEM was reviewed for remaining irregularities due to influences of vegetation, structures and other objects, and edited where necessary.
2. Bathymetry. Unnatural high water levels and velocities within the harbour basins were initially predicted. This was rectified by altering the bathymetry to prevent the simulation of large water columns. The approach is appropriate as the modelled tide is practically a long wave with infinite volume and flood impacts are not defined by offshore water depths.
3. Defences. Coastal and flood defences were added into the model as a 3D breaklines to ensure accurate description of the defence crest. Point elevations were extracted from LiDAR Digital Surface Model (DSM) data and added to the 3D breaklines.
4. Identification of flow paths. The LiDAR data provided did not reflect realistic flow paths at the tunnel adjacent to the harbour quay. The DEM was edited to introduce a flow path. Invert levels of the tunnel were set according to LiDAR elevations adjacent to the tunnel inlets.
5. Initial water level. Initial water levels were set in the model domain to represent mid tide (e.g. nearshore was wet at the start of the model simulation).
6. Roughness patches. A manning's n value of 0.15 was introduced as roughness patch to slow water rushing around the edge of a defence and flowing back into the sea. This was applied at the Structure 2: slipway and the harbour quay. Impacts on simulated peak water levels has not been detected.

### 5.2.4 Model boundaries and simulation

Two types of model boundaries have been used for this study. A tidal boundary has been established in the nearshore, which runs parallel to the coastline and ties into the topographic squeeze point along Elizabeth Avenue and the southern harbour. Four discharge boundaries have been applied to represent the time-varying wave overtopping conditions that would have been experienced during the extreme event. These have been applied landward of the coastal defences (represented as 3D breaklines).

#### 5.2.4.1 Tidal boundary

The tidal boundary applies a time-varying sea level which includes the underlying astronomical tide and a component of surge to make the overall extreme sea level. The underlying tide is based on a MHWS tidal signature for the St Helier port. The surge and final extreme sea level was based on the worst-case wave overtopping joint probability scenario, which was the scenario that produced the highest rate of overtopping from Table 3-1 (reproduced in Table 5-2 below). The peak conditions for the worst-case scenarios are:

- 1 in 1000-year event, under present day conditions: Scenario 18, TSL: 6.4mAOD, Offshore Hs: 4.4m, resulting peak overtopping rate: 122.80 l/s/m.
- 1 in 200-year event, under 2117 climate change conditions. Scenario 15, TSL: 6.8mAOD, Offshore Hs: 5.3m, resulting peak overtopping rate: 273.20 l/s/m.

Table 5-2: Peak overtopping for worst-case return period scenarios

| Scenario | 1 in 1000 year, present day | | | 1 in 200 year, climate change | | |
|---|---|---|---|---|---|---|
| Combination | Sea level (mAOD) | Hs (m) | Peak overtopping rate (l/s/m) | ESL (mAOD) | Hs (m) | Peak overtopping rate (l/s/m) |
| 15 | | | | 6.8 | 5.3 | 273.20 |
| 18 | 6.4 | 4.4 | 122.80 | | | |

#### 5.2.4.2 Wave overtopping boundary

Four water inflow lines were used to represent overtopping into the model, to represent the defences shown in Figure 4-1. The wave overtopping was calculated using the Neural Network and injected into the model landward of the coastal defence to simulate overtopping water.

### 5.2.5 Model simulation

The model was run for consecutive tides, i.e. approximately 30 hours, with wave overtopping simulated during both two high tides. The additional time then allowed the maximum water extent to be mapped, allowing the water to spread through the town, port and coastal floodplain.

## 5.3 Model results

The result of the flood inundation modelling has been assessed to identify the peak flood conditions at the proposed site. The peak water levels occurred within the 1 in 200-year simulation, including climate change, where they peaked at 8.2mAOD.

The full set of results are provided in Appendix D for both simulations. They include:

1 in 1000-year event, under present day conditions:

- Appendix D.1: Peak depth (at site)
- Appendix D.2: Peak water level
- Appendix D.3: Peak velocity

1 in 200-year event, under climate change conditions

- Appendix D.4: Peak depth (at site)
- Appendix D.5: Peak water level
- Appendix D.6: Peak velocity

## 5.4 Model assumptions and limitations

There remains uncertainty in the estimated inundation extents due to several factors. Unfortunately, there is no single model capable of simulating all the processes occurring as waves propagate towards and overtop a coastal defence. Therefore, a suite of numerical models was used for this assessment (wave model, overtopping model and inundation model) with the results used concurrently. Due to these limitations, and as appropriate in all complex modelling studies, the model results have been used in conjunction with a wider range of supporting information (e.g. anecdotal reports, photographs, surveys, etc.) to estimate inundation extents.

The models have been used to simulate a sequence of events; first transforming offshore wave conditions to nearshore, before calculating overtopping and inundation. As such, any uncertainty in the offshore conditions and joint probability assessments will be present throughout the entire process. It was noted that 1 in 100-year conditions, or any wave directions and periods for all return periods were not assigned within the NOC joint probability scenarios, which had to be estimated or inferred from available buoy data.

The event conditions from the March 2008 event could not be used for calibration, as they were not captured within the wave buoy record. Subsequently a qualitative approach was used, where hundreds of joint probability scenarios were simulated and the worst-case results compared to anecdotal information.

The inundation model only accounts for flooding from coastal and tidal sources. Other sources of inundation may occur from surface water flooding and sewer surcharge.

Topography roughness values used in the model were derived from available descriptions, e.g. Hicks and Mason (1998) and Chow (2009). However, there is no definitive guidance on defining roughness values for 2D hydraulic models. It is assumed that the values used are representative.

The wave overtopping input boundary is applied landward of the coastal defences. Difficulties arise where a defence is at an angle to the model grid, which introduces a degree of 'staircasing' into the model. This has been identified and minimised where possible.

# 6 Summary of assessment

This assessment was undertaken by JBA Consulting on behalf of Arup to consider the impacts of extreme sea levels, waves, overtopping and the resulting inundation at a proposed site in St Helier, Jersey. The study considered joint probability of coincident offshore waves and extreme sea levels, wave transformation and breaking processes, extreme water levels and a storm surge occuring over several days, and the wave overtopping to estimate the potential coastal inundation through St Helier and at the proposed site.

A range of joint probability extreme wave and water level scenarios have been established offshore of Jersey. These have been simulated through a wave transformation model to the nearshore zone, adjacent to four difference coastal defences. The rate of wave overtopping during an extreme event at each defence was estimated using the EurOtop Neural Network tool, considered the most suitable approach for the composite defences at St Helier. Wave overtopping rates were estimated for return periods between 1 in 1-year and 1 in 20-year, and matched against anecdotal information and images of the March 2008 event to validate predictions. The Neural Network was then used to estimate extreme overtopping for a range of 1 in 1000-year joint probability events under present day conditions, and 1 in 200-year joint probability events under climate change conditions. The worst-case joint conditions for each return periods was identified and used to assess the potential flood risk within St Helier.

Inundation modelling was undertaken using the TUFLOW hydrodynamic model. The model was established for a 30-hour period with two tides, the second equalling the worst-case peak extreme sea level, waves and overtopping rates for each return period. The resulting flood conditions were were observed to spread throughout the low lying foreshore, along roadways and into the port areas of St Helier. At the proposed site the worst-case conditions arose from the 1 in 200-year plus climate change event, where peak flood levels were estimated to be 8.2mAOD.

# Appendices

# A Appendix - Description of overtopping sites

## A.1 Structure 1: Victoria Avenue seawall

The Victoria Avenue seawall is a steep sloped masonry structure with a mixed of curved lower sections and stepped toe. Its exposed face is oriented towards 224º deg/N. It extends around 1km west of the slip way at St Helier Beach, fronting Victoria Avenue. It was heavily damaged during the March 2008 event, although structural engineering reports suggest it was in a poor condition prior to the event and its subsequent damage was not due to extreme wave overtopping alone.

Figure 6-1 shows (clockwise from top left): Location of defence, photographed wave overtopping, photograph of defence, and available cross section information.

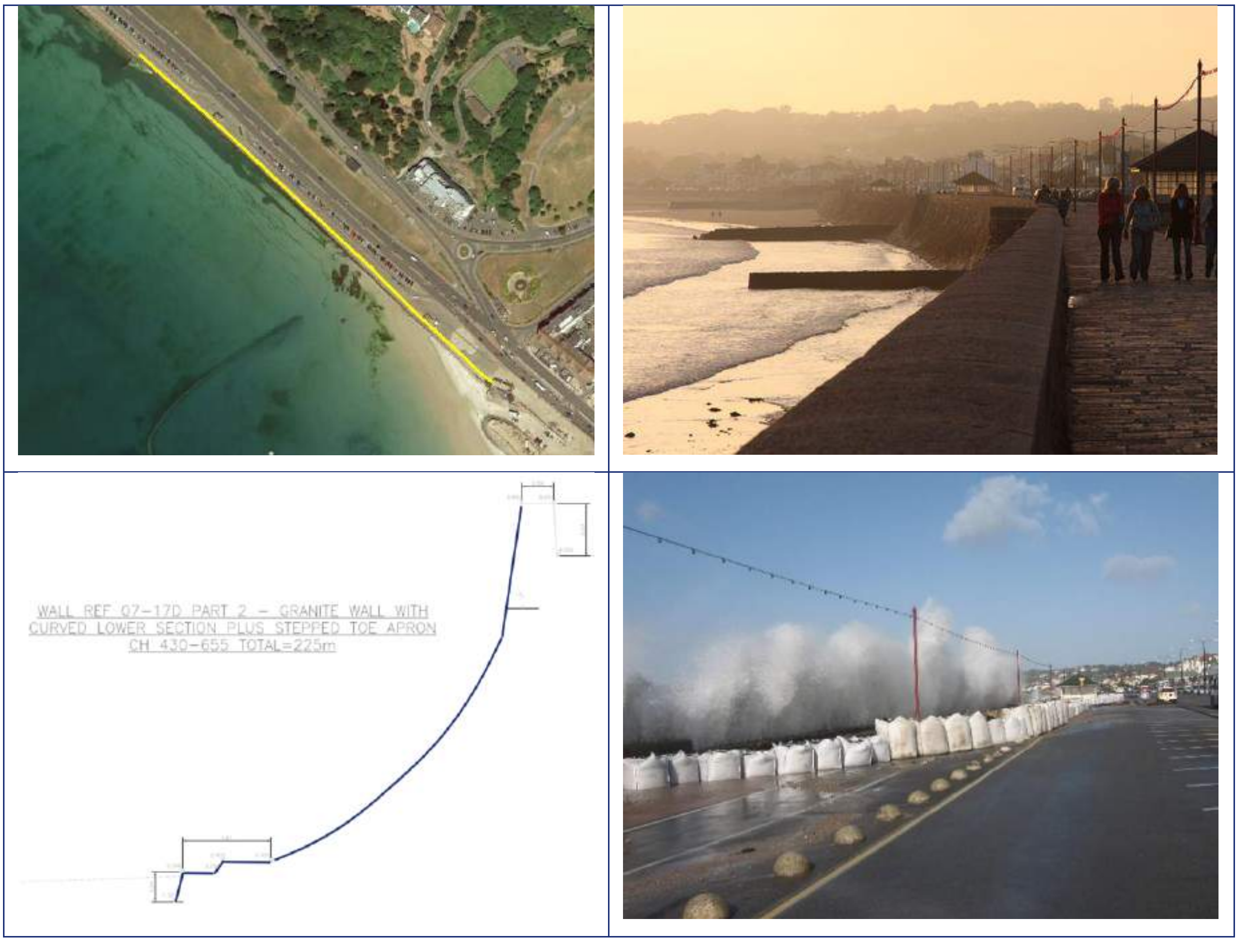
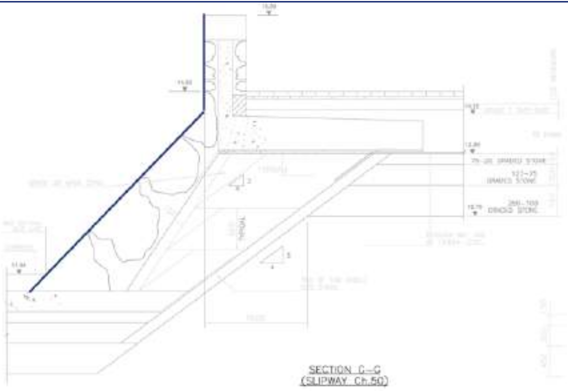



Figure 6-1: Structure and overtopping evidence at Structure 1: Victoria Avenue seawall

## A.2 Structure 2: Slipway

The slipway is located at the east of the Victoria Avenue seawall. It is a stone masonry sloped revetment, at a 1(v):2(h) gradient, with a small vertical rear wall. Its exposed face is oriented towards 227º deg/N. The structure allows additional protection to the slip way, used for beach access by vehicles towing water craft. In terms of wave overtopping, the majority of water breaking over the structure will be returned to the sea via the slipway itself.

Figure 6-2 shows (clockwise from top left): Location of defence, photographed wave overtopping, photograph of defence, and available cross section information.



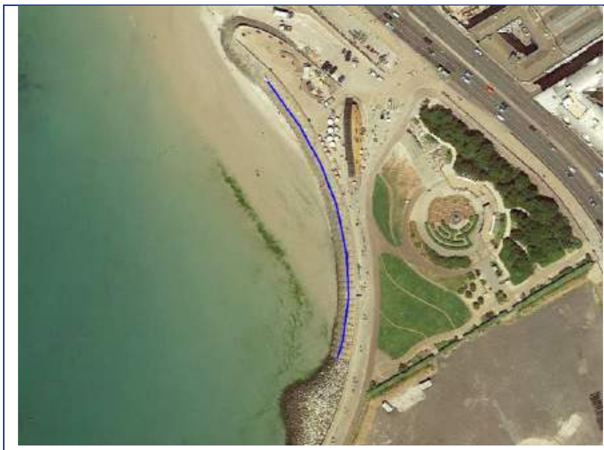

No photographed overtopping

Figure 6-2: Structure and overtopping evidence at Structure 2: Slipway

## A.3 Structure 3: Terrace Revetment

The terraced revetment extends south from the slipway. The structure is made up of a sloped revetment of terraced blocks placed on a rock filter layer and central rubble core. The toe of the wall is supported on a line of sheet piles driven to rock level. It follows a long, curved shape, with an average direction of 254º deg/N. No information of overtopping is provided, however, given its orientation it is expected to have been subject to a high rate during the March 2008 event.

Figure 6-3 shows (clockwise from top left): Location of defence, photographed wave overtopping, photograph of defence, and available cross section information.



No photographed overtopping

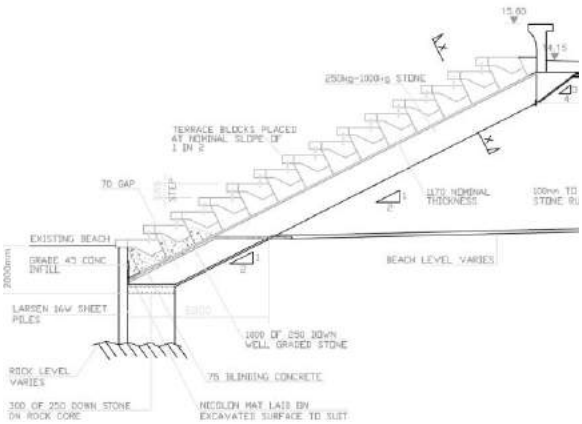




Figure 6-3: Structure and overtopping evidence at Structure 3: Terrace revetment

## A.4 Structure 4: Rock armour revetment

The rock armour revetment forms the outer breakwall of the Elizabeth Marina. Design drawings show the revetment to have a double layer of 2 - 3 tonne rock armour, placed on a stone rubble core. Its exposed face is oriented towards 303$^{o}$ deg/N. No photographed overtopping has been supplied, however the majority of overtopped water is expected to fall into the marina itself.

Figure 6-4 shows (clockwise from top left): Location of defence, photographed wave overtopping, photograph of defence, and available cross section information.

No photographed overtopping

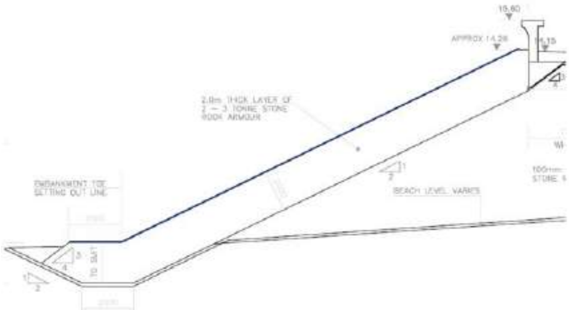



Figure 6-4: Structure and overtopping evidence at Structure 4: Rock armour revetment

## A.5 Harbour quay and walls

Within the St Helier marina there is the potential for coastal flood inundation. Whilst protected from waves, the crest levels of the adjacent quay walls and access points allow a potential source of extreme water level flooding. The crest elevation of the quay walls are approximately 6.5m AOD, with a rear wall present to prevent further inundation if water levels breach the promenade.

Figure 6-5 shows (clockwise from top left): Location of defence, photographed wave overtopping, photograph of defence, and available cross section information.

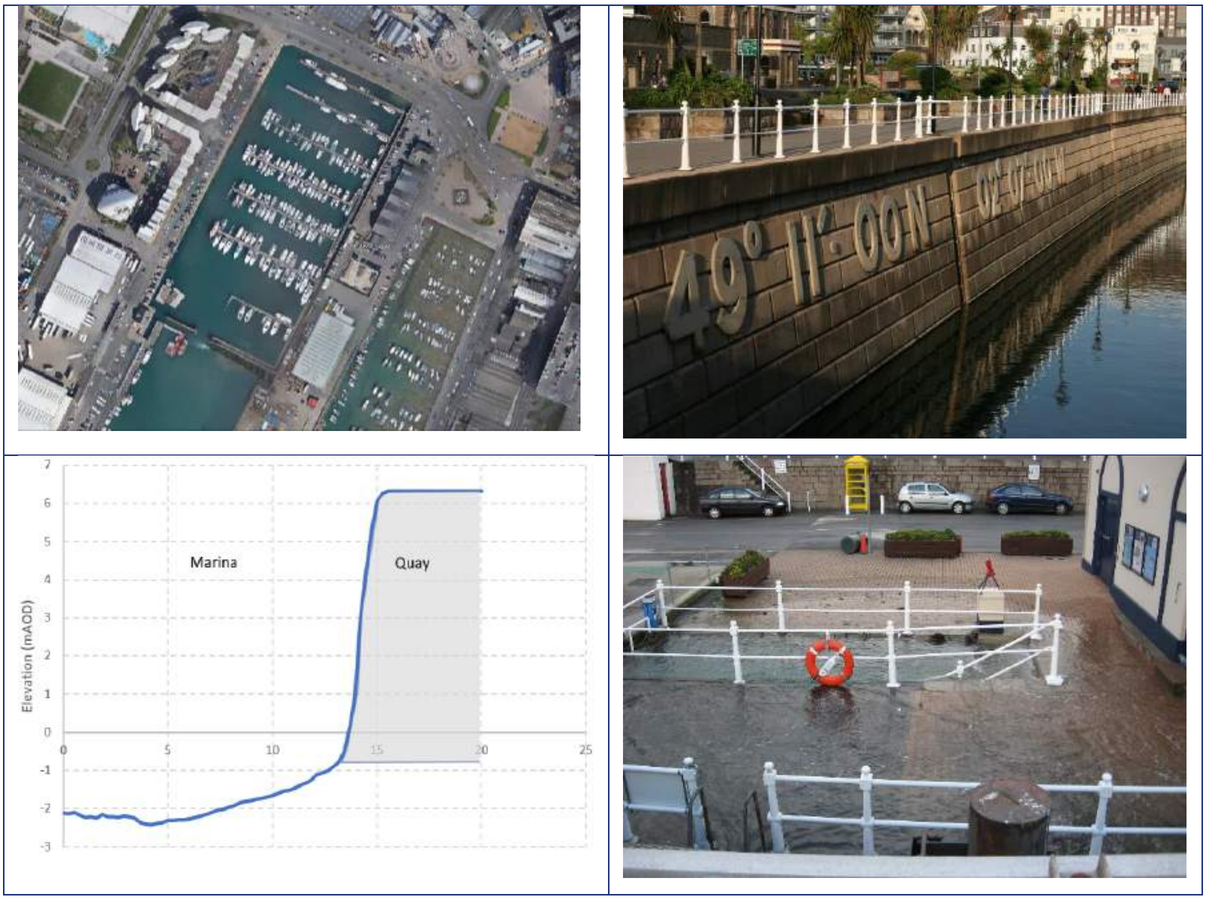



Figure 6-5: Structure and overtopping evidence at Structure 5: Harbour quay and walls

# B Appendix - Neural Network defence schematisations

## B.1 Defence 1: Victoria Avenue seawall

**CALCULATION RECORD**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project Code:** | Q17-7344 | **Page** | 1 | of | 4 | | |
| | | | | | | ***Initials*** | ***Date*** |
| | | | | ***Designer:*** | | PW | 21/05/17 |
| **Project Title:** | Jersey Wave Overtopping | | | ***Checker:*** | | DWR | 22/05/17 |
| **Subject:** | Overtopping<br>Defence 1: Victoria Avenue seawall | | | ***Approver:*** | | DWR | 29/05/17 |
| | | | | ***Office:*** | | Brisbane | |

| Input Data | |
|---|---|
| X | Y |
| 0 | 8 |
| 0.9 | 8 |
| 1.7 | 8 |
| 2.6 | 8 |
| 3.4 | 7.9 |
| 4.3 | 7.9 |
| 5.1 | 7.9 |
| 6 | 7.9 |
| 6.8 | 7.9 |
| 7.7 | 7.9 |
| 8.5 | 7.9 |
| 9.4 | 7.9 |
| 10 | 7.9 |
| 11 | 7.9 |
| 12 | 7.8 |
| 13 | 7.2 |
| 14 | 6.2 |
| 15 | 5.2 |
| 15 | 3.7 |
| 16 | 3.7 |
| 17 | 3.7 |
| 18 | 3.7 |
| 19 | 3.7 |
| 20 | 3.6 |
| 21 | 3.5 |
| 21 | 3.5 |
| 22 | 3.5 |
| 23 | 3.4 |
| 24 | 3.4 |
| 25 | 3.3 |
| 26 | 3.3 |
| 26 | 3.3 |
| 27 | 3.2 |
| 28 | 3.2 |
| 29 | 3.2 |
| 30 | 3.1 |
| 31 | 3.1 |
| 32 | 3.1 |
| 32 | 3 |
| 33 | 3 |
| 34 | 3 |
| 35 | 2.9 |
| 36 | 2.9 |
| 37 | 2.9 |

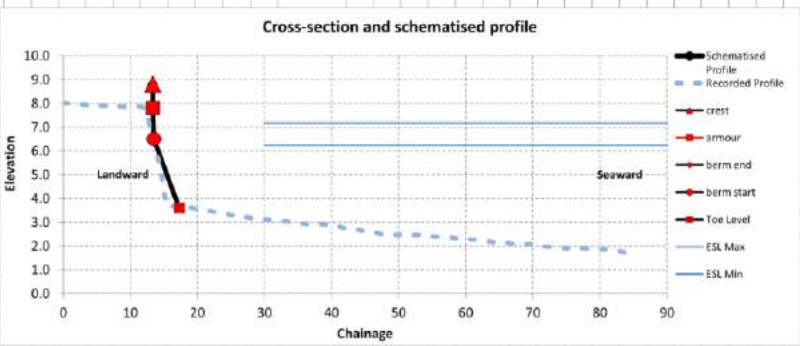
| Schematisation Inputs in Green - Automated Functions in Blue. | | |
|---|---|---|
| ***TOE Data*** | ***X*** | ***Y*** |
| Front_of_toe_level_(mAOD) | 17.3 | 3.6 |
| Toe_level_(mAOD) | 17.3 | 3.6 |
| Width_of_toe_(m), B | 0.00 | |
| | | |
| ***BERM Slope Data (Angle Check)*** | ***X*** | ***Y*** |
| Slope_downward_of_Berm_(cotangent), cot α | 1.30 | 37.57 |
| Slope_upward_of_Berm_(cotangent), cot α | 0.12 | 83.04 |
| Berm_level_(mAOD) | 6.50 | |
| ***BERM Calcs*** | ***X*** | ***Y*** |
| Start | 13.5 | 6.5 |
| End | 13.5 | 6.5 |
| Slope_of_Berm_(tangent), tan aB | 0.00 | |
| Width_of_Berm_(m), B | 0.00 | |
| Berm level check | 7.74 | 8.43 |
| ***CREST Data*** | ***X*** | ***Y*** |
| Armour_crest_level_(mAOD) | 13.3 | 7.8 |
| Crest_level_(mAOD) | 13.3 | 8.8 |
| Width_of_Armour_crest_(m), Gc | 0.00 | |
| | | |
| ***ROUGHNESS Data*** | ***X*** | ***Y*** |
| Upper Break in Roughness 1 | 4.0 | |
| Lower Break in Roughness 2 | 6.0 | |
| Roughness 1 - Rock armour | 1.0 | |
| Roughness 2 - Rock armour | 1.0 | |
| Roughness 3 - Rock armour | 1.0 | |
| | | |
| ***OTHER Data*** | | |
| Wave return wall (If at crest then; yes=1, No=0) | 0.0 | |
| Normal_angle_of_defence_(degrees from North) | 224.0 | |

| Neural Network Inputs | Value |
|---|---|
| Front_of_toe_level_(mAOD) | 3.60 |
| Toe_level_(mAOD) | 3.60 |
| Width_of_toe_(m), B | 0.00 |
| Slope_downward_of_Berm_(cotangent), cot α | 1.30 |
| Berm_level_(mAOD) | 6.50 |
| Width_of_Berm_(m), B | 0.00 |
| Slope_of_Berm_(tangent), tan α | 0.00 |
| Slope_upward_of_Berm_(cotangent), cot α | 0.12 |
| Armour_crest_level_(mAOD) | 7.80 |
| Width_of_Armour_crest_(m), Gc | 0.00 |
| Crest_level_(mAOD) | 8.80 |
| Normal_angle_of_defence_(degrees from North) | 224.00 |
| Wave return wall (yes=1, No=0) | 0.00 |
| Break in Roughness 1 | 4.00 |
| Break in Roughness 2 | 6.00 |
| Roughness 1 | 1.00 |
| Roughness 2 | 1.00 |
| Roughness 3 | 1.00 |

Entered

Calculated

**Review Notes**




W:\2017\Projects\Q17-7344 - JBA - Wave modelling Jersey\Calculations\Overtopping\2017s6170_JBP_Overtopping_Schematisation_QA_Sheet_v1.0.xlsx 1: 29/05/2017

JBA Consulting
www.jbaconsulting.co.uk

## B.2 Defence 2: Slipway

### CALCULATION RECORD

| | | | | | |
|---|---|---|---|---|---|
| **Project Code:** | Q17-7344 | **Page** 2 of 4 | | ***Initials*** | ***Date*** |
| **Project Title:** | Jersey Wave Overtopping | | ***Designer:*** | PW | 21/05/17 |
| **Subject:** | Overtopping<br>Defence 2: Slipway | | ***Checker:*** | DWR | 22/05/17 |
| | | | ***Approver:*** | DWR | 29/05/17 |
| | | | ***Office:*** | Brisbane | |

**Input Data**

| X | Y (JBA St) |
|---|---|
| 0 | 8.5 |
| 1.1 | 8.5 |
| 2.1 | 8.5 |
| 3.2 | 8.5 |
| 4.2 | 8.4 |
| 5.3 | 8.4 |
| 6.3 | 8.4 |
| 7.4 | 8.3 |
| 8.5 | 8.3 |
| 9.5 | 8.3 |
| 11 | 8.3 |
| 12 | 8.3 |
| 13 | 8.3 |
| 14 | 8.3 |
| 15 | 8.2 |
| 16 | 8 |
| 17 | 7.8 |
| 18 | 7.4 |
| 19 | 6.9 |
| 20 | 6.4 |
| 21 | 5.8 |
| 22 | 5.2 |
| 23 | 4.7 |
| 24 | 4.3 |
| 25 | 3.9 |
| 26 | 3.7 |
| 27 | 3.7 |
| 29 | 3.7 |
| 30 | 3.6 |
| 31 | 3.6 |
| 32 | 3.5 |
| 33 | 3.5 |
| 34 | 3.4 |
| 35 | 3.4 |
| 36 | 3.3 |
| 37 | 3.3 |
| 38 | 3.3 |
| 39 | 3.3 |
| 40 | 3.2 |
| 41 | 3.2 |
| 42 | 3.1 |
| 43 | 3.1 |
| 44 | 3 |

**Schematisation Inputs (Green) Automated Functions (Blue)**

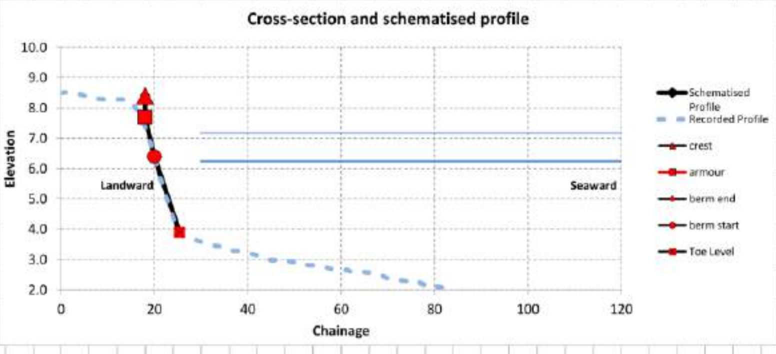
| *TOE Data* | *X* | Y |
|---|---|---|
| Front_of_toe_level_(mAOD) | 25.4 | 3.9 |
| Toe_level_(mAOD) | 25.4 | 3.9 |
| Width_of_toe_(m), $B_t$ | 0.00 | |
| | | |
| ***BERM Slope Data (Angle Check)*** | *X* | Y |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 2.16 | 24.84 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 1.54 | 33.02 |
| Berm_level_(mAOD) | 6.40 | |
| ***BERM Calcs*** | *X* | Y |
| Start | 20.0 | 6.4 |
| End | 20.0 | 6.4 |
| Slope_of_Berm_(tangent), tan aB | 0.00 | |
| Width_of_Berm_(m), B | 0.00 | |
| Berm level check | 7.74 | 8.43 |
| ***CREST Data*** | *X* | Y |
| Armour_crest_level_(mAOD) | 18.0 | 7.7 |
| Crest_level_(mAOD) | 18.0 | 8.4 |
| Width_of_Armour_crest_(m), Gc | 0.00 | |
| | | |
| ***ROUGHNESS Data*** | *X* | Y |
| Upper Break in Roughness 1 | 5.0 | |
| Lower Break in Roughness 2 | 6.0 | |
| Roughness 1 - Earth Embankment | 1.0 | |
| Roughness 2 - Earth Embankment | 1.0 | |
| Roughness 3 - Concrete Revetment | 1.0 | |
| | | |
| ***OTHER Data*** | | |
| Wave return wall (If at crest then; yes=1, No=0) | 0.0 | |
| Normal_angle_of_defence_(degrees from North) | 227.0 | |

| Neural Network Inputs | Value |
|---|---|
| Front_of_toe_level_(mAOD) | 3.90 |
| Toe_level_(mAOD) | 3.90 |
| Width_of_toe_(m), $B_t$ | 0.00 |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 2.16 |
| Berm_level_(mAOD) | 6.40 |
| Width_of_Berm_(m), B | 0.00 |
| Slope_of_Berm_(tangent), tan $\alpha_B$ | 0.00 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 1.54 |
| Armour_crest_level_(mAOD) | 7.70 |
| Width_of_Armour_crest_(m), Gc | 0.00 |
| Crest_level_(mAOD) | 8.40 |
| Normal_angle_of_defence_(degrees from North) | 227.00 |
| Wave return wall (yes=1, No=0) | 0.00 |
| Break in Roughness 1 | 5.00 |
| Break in Roughness 2 | 6.00 |
| Roughness 1 | 1.00 |
| Roughness 2 | 1.00 |
| Roughness 3 | 1.00 |

Entered

Calculated

**Review Notes**




N:\2017\Projects\Q17-7344 - JBA - Wave modelling Jersey\Calculations\Overtopping\2017s6170_JBP_Overtopping_Schematisation_QA_Sheet_v1.0.xlsx2: 29/05/2017

JBA Consulting
www.jbaconsulting.co.uk

## B.3 Defence 3: Terrace Revetment

### CALCULATION RECORD

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | Initials | Date |
| Project Code: | Q17-7344 | Page | 3 | of | 4 | Designer: | PW | 21/05/17 |
| Project Title: | Jersey Wave Overtopping | | | | | Checker: | DWR | 22/05/17 |
| Subject: | Overtopping | | | | | Approver: | DWR | 29/05/17 |
| | Defence 3: Terrace Revetment | | | | | Office: | Brisbane | |

JBP scientists and engineers

| Input Data X | Input Data Y |
|---|---|
| 0 | 8.4 |
| 0.2 | 8.4 |
| 0.4 | 8.4 |
| 0.6 | 8.5 |
| 0.8 | 8.5 |
| 1 | 8.5 |
| 1.2 | 8.5 |
| 1.4 | 8.5 |
| 1.6 | 9 |
| 1.9 | 9.1 |
| 2.1 | 9.1 |
| 2.3 | 9.1 |
| 2.5 | 9.1 |
| 2.7 | 8.8 |
| 2.9 | 8.8 |
| 3.1 | 8.8 |
| 3.3 | 8.8 |
| 3.5 | 8.8 |
| 3.7 | 8.2 |
| 3.9 | 8.2 |
| 4.1 | 8.2 |
| 4.3 | 8.2 |
| 4.5 | 8.2 |
| 4.7 | 7.8 |
| 4.9 | 7.8 |
| 5.1 | 7.8 |
| 5.3 | 7.8 |
| 5.6 | 7.8 |
| 5.8 | 7.3 |
| 6 | 7.3 |
| 6.2 | 7.3 |
| 6.4 | 7.3 |
| 6.6 | 6.8 |
| 6.8 | 6.8 |
| 7 | 6.8 |
| 7.2 | 6.8 |
| 7.4 | 6.8 |
| 7.6 | 6.3 |
| 7.8 | 6.3 |
| 8 | 6.3 |
| 8.2 | 6.3 |
| 8.4 | 6.3 |
| 8.6 | 5.9 |
| 8.8 | 5.9 |

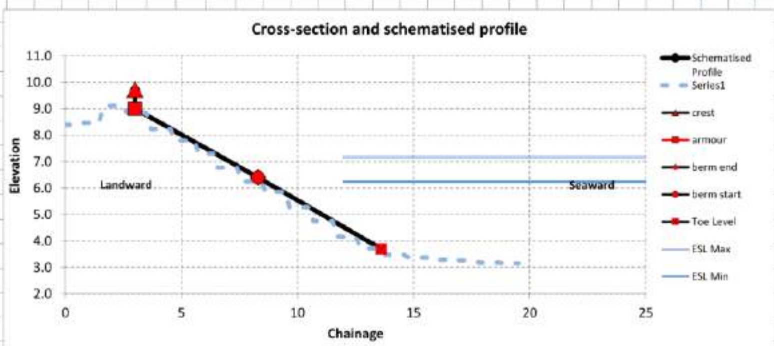
| Schematisation Inputs in Green - Automated Functions in Blue. | | |
|---|---|---|
| **TOE Data** | **X** | **Y** |
| Front_of_toe_level_(mAOD) | 13.6 | 3.7 |
| Toe_level_(mAOD) | 13.6 | 3.7 |
| Width_of_toe_(m), $B_t$ | 0.00 | |
| **BERM Slope Data (Angle Check)** | **X** | **Y** |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 1.96 | 27.00 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 2.04 | 26.13 |
| Berm_level_(mAOD) | 6.40 | |
| **BERM Calcs** | **X** | **Y** |
| Start | 8.3 | 6.4 |
| End | 8.3 | 6.4 |
| Slope_of_Berm_(tangent), tan aB | 0.00 | |
| Width_of_Berm_(m), B | 0.00 | |
| Berm level check | 7.74 | 8.43 |
| **CREST Data** | **X** | **Y** |
| Armour_crest_level_(mAOD) | 3.0 | 9.0 |
| Crest_level_(mAOD) | 3.0 | 9.7 |
| Width_of_Armour_crest_(m), Gc | 0.00 | |
| **ROUGHNESS Data** | **X** | **Y** |
| Upper Break in Roughness 1 | 4.0 | |
| Lower Break in Roughness 2 | 8.3 | |
| Roughness 1 - Seawall | 1.0 | |
| Roughness 2 - Stepped Revetment | 0.8 | |
| Roughness 3 - Stepped Revetment | 0.8 | |
| **OTHER Data** | | |
| Wave return wall (If at crest then; yes=1, No=0) | 0.0 | |
| Normal_angle_of_defence_(degrees from North) | 254.0 | |

| Neural Network Inputs | Value |
|---|---|
| Front_of_toe_level_(mAOD) | 3.70 |
| Toe_level_(mAOD) | 3.70 |
| Width_of_toe_(m), $B_t$ | 0.00 |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 2.00 |
| Berm_level_(mAOD) | 6.40 |
| Width_of_Berm_(m), B | 0.00 |
| Slope_of_Berm_(tangent), tan $\alpha_B$ | 0.00 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 2.00 |
| Armour_crest_level_(mAOD) | 9.00 |
| Width_of_Armour_crest_(m), Gc | 0.00 |
| Crest_level_(mAOD) | 9.72 |
| Normal_angle_of_defence_(degrees from North) | 254.00 |
| Wave return wall (yes=1, No=0) | 0.00 |
| Break in Roughness 1 | 4.00 |
| Break in Roughness 2 | 8.27 |
| Roughness 1 | 1.00 |
| Roughness 2 | 0.80 |
| Roughness 3 | 0.80 |

Entered

Calculated

Review Notes




W:\2017\Projects\Q17-7344 - JBA - Wave modelling Jersey\Calculations\Overtopping\2017s6170_JBP_Overtopping_Schematisation_QA_Sheet_v1.0.xlsx 3 29/05/2017

JBA Consulting
www.jbaconsulting.co.uk

## B.4 Defence 4: Rock Armour Revetment

### CALCULATION RECORD

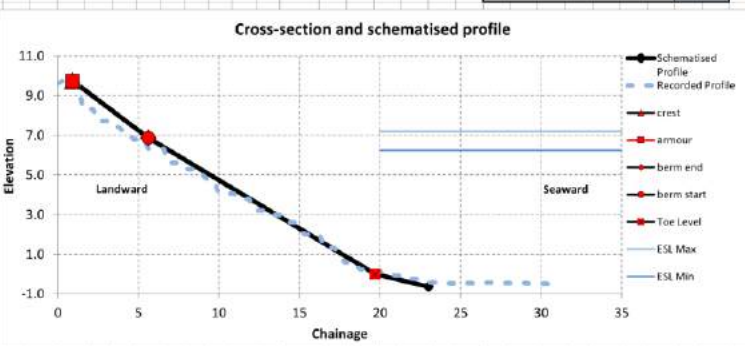
| | | | | | | Initials | Date |
|---|---|---|---|---|---|---|---|
| **Project Code:** | Q17-7344 | **Page** | 4 | **of** | 4 | | |
| | | | | | ***Designer:*** | PW | 21/05/17 |
| **Project Title:** | Jersey Wave Overtopping | | | | ***Checker:*** | DWR | 22/05/17 |
| **Subject:** | Overtopping | | | | ***Approver:*** | DWR | 29/05/17 |
| | Defence 4: Rock Armour Revetment | | | | ***Office:*** | Brisbane | |

| Input Data X | Input Data Y |
|---|---|
| 0 | 9.62 |
| 0.31 | 9.75 |
| 0.62 | 9.75 |
| 0.94 | 9.75 |
| 1.25 | 9.66 |
| 1.56 | 8.32 |
| 1.87 | 8.32 |
| 2.19 | 8.32 |
| 2.5 | 7.72 |
| 2.81 | 7.72 |
| 3.12 | 7.72 |
| 3.44 | 7.72 |
| 3.75 | 7.41 |
| 4.06 | 7.22 |
| 4.37 | 7.22 |
| 4.69 | 6.8 |
| 5 | 6.8 |
| 5.31 | 6.8 |
| 5.62 | 6.32 |
| 5.93 | 6.32 |
| 6.25 | 6.32 |
| 6.56 | 6.32 |
| 6.87 | 5.61 |
| 7.18 | 5.61 |
| 7.5 | 5.61 |
| 7.81 | 5.29 |
| 8.12 | 5.29 |
| 8.43 | 5.29 |
| 8.75 | 5.29 |
| 9.06 | 4.93 |
| 9.37 | 4.69 |
| 9.68 | 4.69 |
| 10 | 4.06 |
| 10.3 | 4.06 |
| 10.6 | 4.06 |
| 10.9 | 4.06 |
| 11.3 | 3.78 |
| 11.6 | 3.78 |
| 11.9 | 3.78 |
| 12.2 | 3.21 |
| 12.5 | 3.21 |
| 12.8 | 3.21 |
| 13.1 | 3.21 |
| 13.4 | 2.98 |

| Schematisation Inputs in Green - Automated Functions in Blue. | | |
|---|---|---|
| ***TOE Data*** | ***X*** | ***Y*** |
| Front_of_toe_level_(mAOD) | 23.0 | -0.6 |
| Toe_level_(mAOD) | 19.7 | 0.0 |
| Width_of_toe_(m), B, | 3.30 | |
| ***BERM Slope Data (Angle Check)*** | ***X*** | ***Y*** |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 2.05 | 25.99 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 1.64 | 31.44 |
| Berm_level_(mAOD) | 6.87 | |
| ***BERM Calcs*** | ***X*** | ***Y*** |
| Start | 5.6 | 6.9 |
| End | 5.6 | 6.9 |
| Slope_of_Berm_(tangent), tan aB | 0.00 | |
| Width_of_Berm_(m), B | 0.00 | |
| Berm level check | 7.74 | 8.43 |
| ***CREST Data*** | ***X*** | ***Y*** |
| Armour_crest_level_(mAOD) | 0.9 | 9.8 |
| Crest_level_(mAOD) | 0.9 | 9.8 |
| Width_of_Armour_crest_(m), Gc | 0.00 | |
| ***ROUGHNESS Data*** | ***X*** | ***Y*** |
| Upper Break in Roughness 1 | 4.0 | |
| Lower Break in Roughness 2 | 8.0 | |
| Roughness 1 - Concrete Seawall | 1.0 | |
| Roughness 2 - Rock Armour | 0.6 | |
| Roughness 3 - Rock armour | 0.6 | |
| ***OTHER Data*** | | |
| Wave return wall (If at crest then; yes=1, No=0) | 0.0 | |
| Normal_angle_of_defence_(degrees from North) | 303.0 | |

| Neural Network Inputs | Value |
|---|---|
| Front_of_toe_level_(mAOD) | -0.64 |
| Toe_level_(mAOD) | 0.00 |
| Width_of_toe_(m), B, | 3.30 |
| Slope_downward_of_Berm_(cotangent), cot $\alpha_d$ | 2.05 |
| Berm_level_(mAOD) | 6.87 |
| Width_of_Berm_(m), B | 0.00 |
| Slope_of_Berm_(tangent), tan $\alpha_B$ | 0.00 |
| Slope_upward_of_Berm_(cotangent), cot $\alpha_u$ | 1.64 |
| Armour_crest_level_(mAOD) | 9.75 |
| Width_of_Armour_crest_(m), Gc | 0.00 |
| Crest_level_(mAOD) | 9.75 |
| Normal_angle_of_defence_(degrees from North) | 303.00 |
| Wave return wall (yes=1, No=0) | 0.00 |
| Break in Roughness 1 | 4.00 |
| Break in Roughness 2 | 8.00 |
| Roughness 1 | 1.00 |
| Roughness 2 | 0.55 |
| Roughness 3 | 0.55 |

| |
|---|
| Entered |
| Calculated |
| **Review Notes** |




W:\2017\Projects\Q17-7344 - JBA - Wave modelling Jersey\Calculations\Overtopping\2017s6170_JBP_Overtopping_Schematisation_QA_Sheet_v1.0.xlsx4: 29/05/2017

JBA Consulting
www.jbaconsulting.co.uk

# C Appendix - EurOtop hazard estimates

Table 6-1: Limits for overtopping for pedestrians

| Hazard type and reason | Mean discharge | Max volume |
|---|---|---|
| | Q (l/s/m) | Vmax (l/m) |
| Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower level only, no falling jet, low danger of fall from walkway. | 1-10 | 500<br>at low level |
| Aware pedestrian, clear view of sea, not easily upset or frightened, able to tolerate getting wet, wider walkway. | 0.1 | 20-50<br>at high level or velocity |

Table 6-2: Limits for overtopping for vehicles

| Hazard type and reason | Mean discharge | Max volume |
|---|---|---|
| | Q (l/s/m) | Vmax (L/m) |
| Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed. | 10 - 50[9] | 100 – 1,000 |
| Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets. | 0.01 – 0.05[10] | 5 – 50 at high level or velocity |

Table 6-3: Limits for overtopping for property and damage to the defence

| Hazard type and reason | Mean discharge |
|---|---|
| | Q (l/s/m) |
| Damage to building structural elements | 1[11] |
| Damage to equipment set back 5-10m | 0.4[12] |
| No damage to embankment/seawalls if crest and rear slope are well protected | 50-200 |
| No damage to embankment / seawall crest and rear face of grass covered embankment of clay | 1-10 |
| Damage to paved or armoured promenade behind a seawall | 200 |
| Damage to grassed or lightly protected promenade | 50 |

9 Note: These limits relate to overtopping defined at highways.
10 Note: These limits relate to overtopping defined at the defence, assumes the highway is immediately behind
11 Note: This limit relates to the effective overtopping defined at the building
12 Note: This limit relates to overtopping defined at the defence

# D Appendix - Coastal flood mapping

**1 in 1000-year event, under present day conditions**

D.1 1 in 1000-year event, under present day conditions: Peak depth

D.2 1 in 1000-year event, under present day conditions: Peak water level

D.3 1 in 1000-year event, under present day conditions: Peak velocity

**1 in 200-year event, under climate change conditions**

D.4 1 in 200-year event, under climate change conditions: Peak depth

D.5 1 in 200-year event, under climate change conditions: Peak depth

D.6 1 in 200-year event, under climate change conditions: Peak depth

Offices at

Coleshill

Doncaster

Edinburgh

Haywards Heath

Limerick

Newcastle upon Tyne

Newport

Saltaire

Skipton

Tadcaster

Thirsk

Wallingford

Warrington

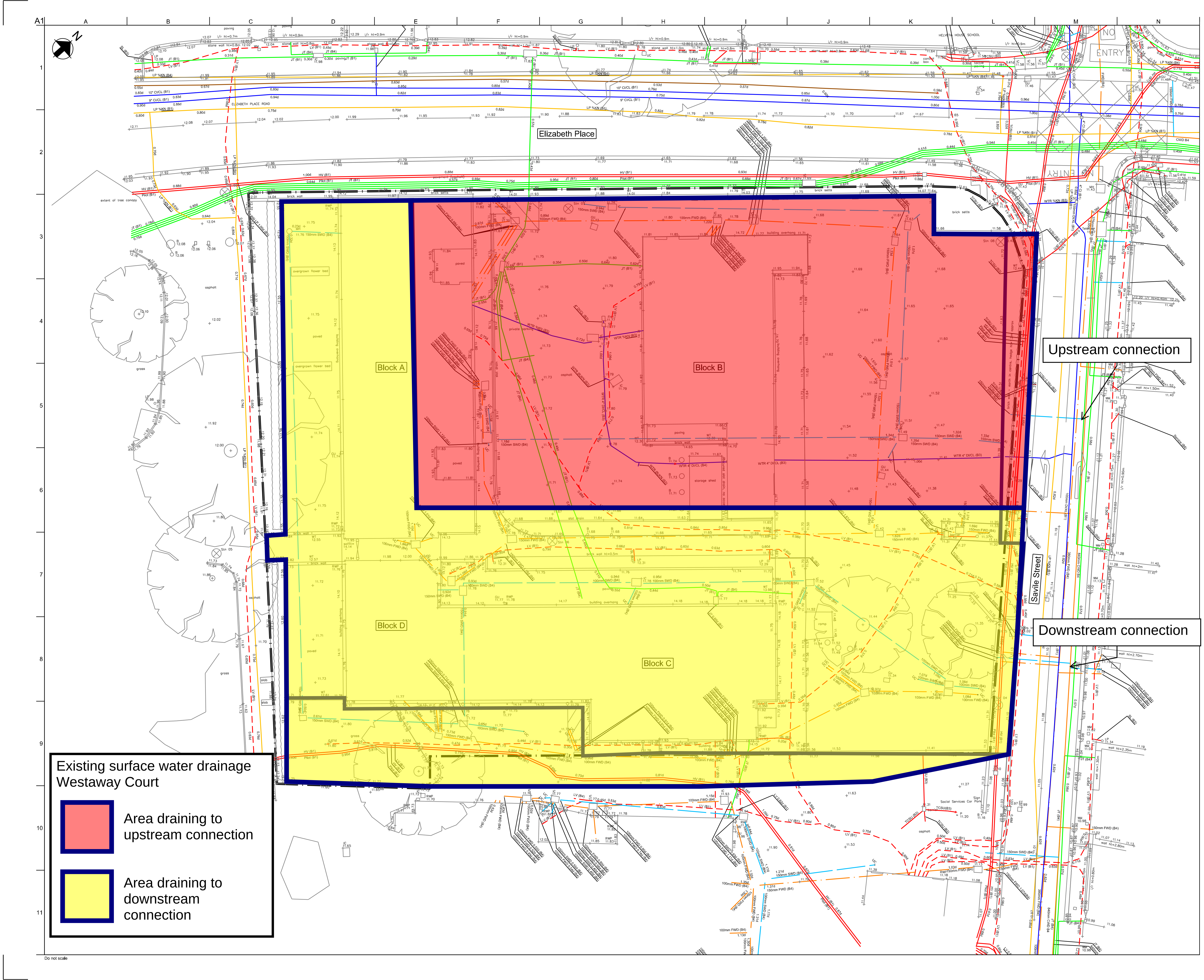
Registered Office
South Barn
Broughton Hall
SKIPTON
North Yorkshire
BD23 3AE

t:+44(0)1756 799919
e:info@jbaconsulting.com

Jeremy Benn Associates Ltd
Registered in England
3246693

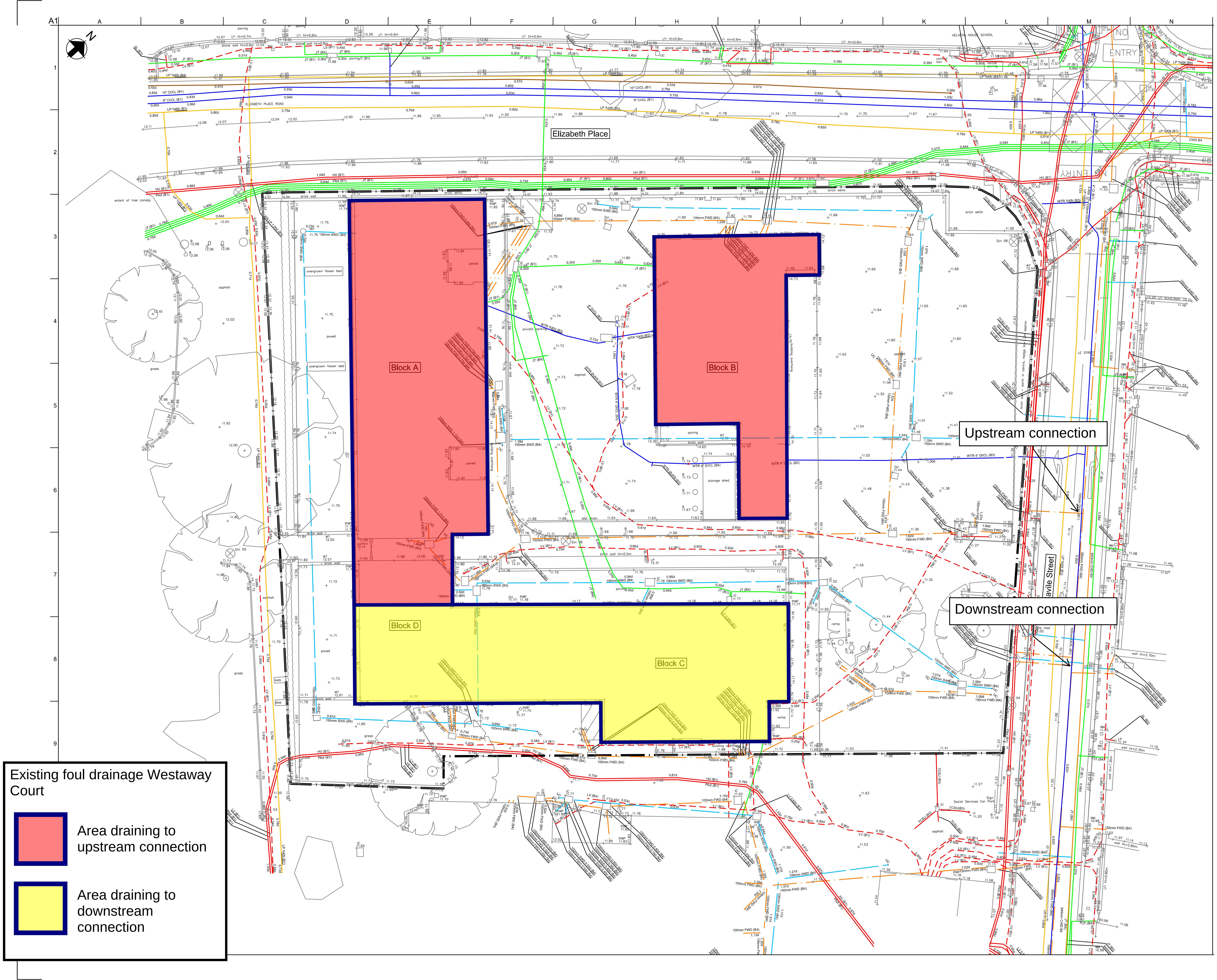
Visit our website
www.jbaconsulting.com

# Appendix E



Elizabeth Place
Savile Street
Block A
Block B
Block C
Block D
Upstream connection
Downstream connection
NO ENTRY
Existing surface water drainage
Westaway Court
Area draining to upstream connection
Area draining to downstream connection
Do not scale

# Appendix F


Elizabeth Place
Block A
Block B
Block C
Block D
Upstream connection
Downstream connection
Existing foul drainage Westaway Court
Area draining to upstream connection
Area draining to downstream connection