MVV Environment Devonport Ltd
Energy from Waste Combined Heat and Power Facility
North Yard, Devonport

Level 3 Flood Risk Assessment

Final Report
April 2011

Prepared for

Revision Schedule

Level 3 Flood Risk Assessment
April 2011

Rev	Date	Details	Prepared by	Reviewed by	Approved by
01	September 2010	D123356 Level 2 FRA Draft Report	**Gemma Hoad** Assistant Hydrologist	**Dr Rob Sweet** Senior Flood Risk Specialist	**Michael Timmins** Principal Consultant
02	October 2010	D123356 Level 2 FRA Final Report	**Gemma Hoad** Assistant Hydrologist	**Dr Rob Sweet** Senior Flood Risk Specialist	**Michael Timmins** Principal Consultant
03	February 2011	D123356 Level 3 FRA Draft Report	**Mark Crussell** Assistant Hydrologist	**Dr Rob Sweet** Senior Flood Risk Specialist	**Michael Timmins** Principal Consultant
04	April 2011	D123356 Level 3 FRA Final Report – including EA comments	**Mark Crussell** Assistant Hydrologist	**Dr Rob Sweet** Senior Flood Risk Specialist	**Michael Timmins** Principal Consultant

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1 Introduction

1.1 Background

1.1.1 Scott Wilson Ltd was commissioned by MVV to prepare a Level 2 Flood Risk Assessment (FRA) which was completed in October 2010. The Environment Agency review of the Level 2 report identified the requirement for further work. Therefore the Level 2 FRA has been updated to a Level 3 FRA with the addition of hydraulic modelling to quantify effects of structures and combined event modelling.

1.1.2 The Level 3 Flood Risk Assessment (FRA) is to be submitted with the planning application for the proposed Energy from Waste (EfW) Combined Heat and Power (CHP) facility. The Level 3 FRA includes combined event (tidal/fluvial) hydraulic modelling of the Weston Mill Stream.

1.1.3 MVV has been awarded the South West Devon Waste Partnership's (SWDWP) residual waste treatment and disposal contract. MVV's proposal is to construct and operate an EfW CHP facility on land situated in the North Yard of Her Majesty's Naval Base (HMNB) Devonport, Plymouth.

1.1.4 This FRA has been carried out in accordance with the requirements of Planning Policy Statement 25 (PPS25) – Development and Flood Risk[1] and the PPS25 Practice Guide[2].

1.1.5 An initial review of the Environment Agency Flood Map, provided in Appendix A, indicates that the built development area of the site lies in Flood Zone 1 of the Western Mill Stream, which is tidally influenced in the vicinity of the site. Land in Flood Zone 1 is classified as having less than a 0.1% (1 in 1000 year) annual probability of river and tidal flooding.

1.1.6 A section of the access road, where the road runs parallel to the Weston Mill Viaduct falls within Flood Zone 2. Land in Flood Zone 2 is classified as having between 0.1% and 1% (between 1 in 1000 and 1 in 100 year) annual probability of fluvial flooding or between 0.1% and 0.5% (between 1 in 1000 and 1 in 200 year) annual probability of tidal flooding in any year. Environment Agency Flood Zones show the flood extent without taking into account the presence of flood defences.

1.2 Methodology

1.2.1 The aim of FRAs is to assess the risks of all forms of flooding to and from a development. PPS25 emphasises the need for a risk-based approach to be adopted by planning authorities through the application of the Source-Pathway-Receptor model.

Source-Pathway-Receptor Model

1.2.2 Scott Wilson's approach to an FRA is based on the Source-Pathway-Receptor model, in accordance with the recommendations of Annex E of PPS25.

1.2.3 The Source-Pathway-Receptor model firstly identifies the causes or 'sources' of flooding to and from a development. The identification is based on a review of available information such as mapping, local conditions and consideration of the effects of climate change. The nature and

[1] Available online at:http://www.communities.gov.uk/documents/planningandbuilding/pdf/planningpolicystatement25.pdf
[2] Available online at:http://www.communities.gov.uk/documents/planningandbuilding/pdf/pps25practiceguide.pdf

likely extent of flooding arising from any one source is considered, e.g. whether such flooding is likely to be localised or widespread. As well as flooding from more obvious sources such as rivers and sea, FRAs include an assessment of other sources of flooding as required in PPS25 including groundwater flooding, surface water flooding and flooding from artificial sources.

1.2.4 The presence of a flood source does not always imply a risk. For example, the presence of a sewer does not necessarily increase the risk of flooding unless the sewer is local to the site and ground levels encourage surcharged water to accumulate. The exposure pathway or 'flooding mechanism' determines whether there is a risk of exposure to a flood source.

1.2.5 For Level 1 and Level 2 FRAs, the identification of flooding pathways is undertaken by considering the local and site topography, the proximity of the flood source to the receptor and the potential flood conveyance routes local to the site. For more detailed Level 3 FRAs, hydrological or hydraulic modelling may be required to quantify the flood risk and identify specific pathways, for the particular flood source.

1.2.6 If a flooding mechanism is considered not to be present, then the risk from the flood source is considered to be negligible.

Assessment of Flood Risk to Receptors

1.2.7 If a flood source and flooding pathway are identified, the assessment of the flood risk to the receptor is determined by combining the probability of the flood event occurring with the severity of impact (or consequences) if the flood event were to occur. Receptors include any people or buildings within the range of the flood source, which are connected to the source by a pathway.

1.2.8 For Level 1 and Level 2 FRAs, the probability of a flood event occurring is determined from historical records of events, available modelling information and the design standard and condition of any infrastructure associated with the flood source. For more detailed Level 3 FRAs, hydrological of hydraulic modelling may be used to determine the frequency of flood events occurring, for a particular flood source.

1.2.9 The potential severity of the impact is determined by considering a combination of the type of flood source, the flood mechanisms identified, the layout and design of the proposed receptor and the vulnerability of the proposed receptor.

1.2.10 The Scott Wilson FRA approach involves a desk-based review of available information to establish:

- Likely flooding sources;
- Potential flooding mechanisms;
- Probability of a flood event occurring; and,
- Severity of impact of a flood event for the site.

1.2.11 In summary, for there to be a flood risk all the elements of the Source-Pathway-Receptor model must be present. Furthermore, effective mitigation can be provided to reduce the magnitude of flood risk by removing one element of the model. For example, by removing the pathway, defending against the flood source, incorporating flood management or flood resilient measures into building receptors, or providing safe access and egress and flood evacuation plans for human receptors.

1.3 Aim and Objectives

Aim

1.3.1 The aim of this report is to provide a Level 3 FRA, to assess the flood risk to and arising from the proposed development. This report is intended to provide a detailed/quantitative assessment of fluvial and tidal flood risk through hydraulic modelling. Other sources of flood risk are also considered. In order to achieve this, the following objectives have been met:

Objectives

1.3.2 In order to achieve the above the following objectives have been met:

- Prepare tidal and fluvial Flood Zone 3, Flood Zone 3 including climate change allowance and Flood Zones 2 maps of the Weston Mill Stream;
- Predict peak tidal and fluvial water levels at locations of interest adjacent to the site and access roads and also further upstream to determine flood risk to the site and third parties;
- Investigate the upstream impact on flood levels resulting from the replacement of two existing site access bridges (over the Weston Mill Stream) with a single open span bridge;
- Identify the significance of other potential sources of flooding and determine whether potential pathways exist which may cause a flood risk to the proposed development site;
- Determine the surface water management requirements at the site in keeping with the principles of Paragraph F10 of PPS25;
- Propose mitigation measures to reduce the flood risk posed to, or arising from the site post development.

1.4 Environment Agency Liaison

1.4.1 Consultation between MVV, Scott Wilson and the Environment Agency has been ongoing since September 2009. Following the outcome of a meeting with the Environment Agency (3rd November 2010) and their subsequent comments (2nd December 2010) regarding the draft Level 2 FRA (dated October, 2010) it was identified that a more detailed Level 3 FRA was required. Environment Agency correspondence is provided in Appendix B.

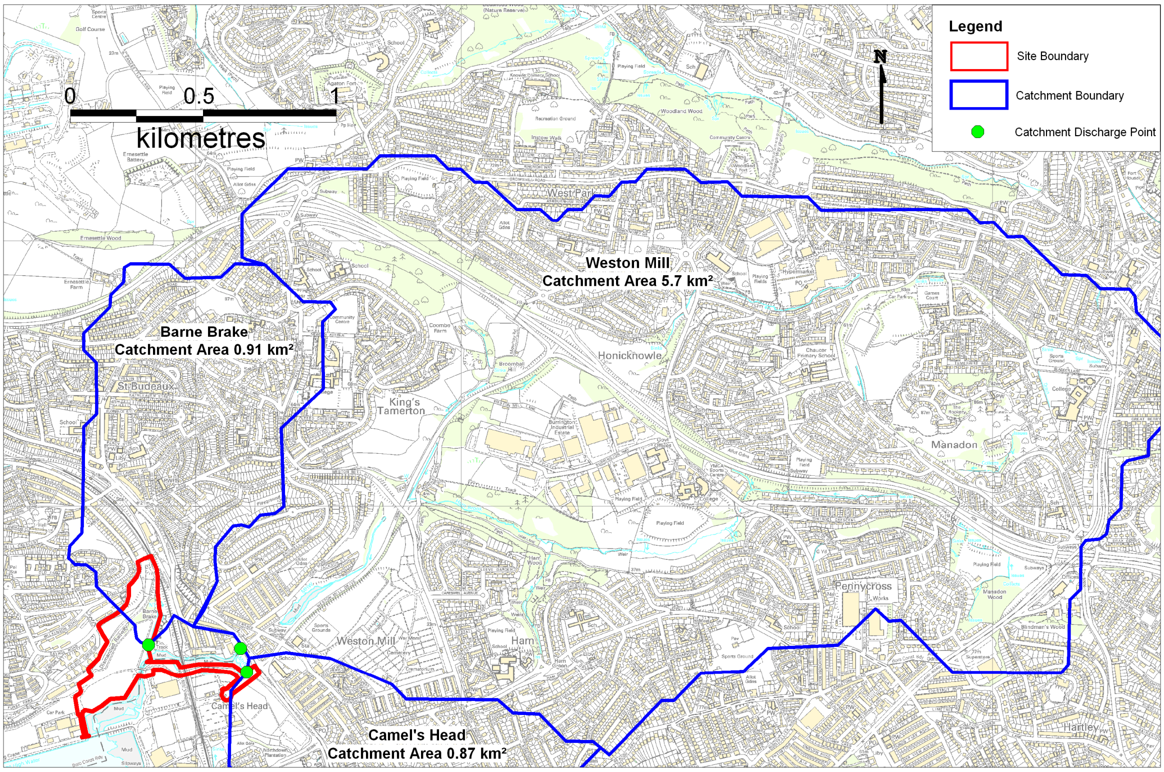
2 Site Description

2.1 Current Site

2.1.1 The proposed development site is located on land currently owned by the Ministry of Defence (MoD) within Her Majesty's Naval Base (HMNB) Devonport, Plymouth. The site location is identified in Figure 2-1.

2.1.2 To the north and north-west of the site lies the residential area of Barne Barton. This area of housing is at a higher elevation to the proposed site. The Weston Mill Viaduct is close to the eastern boundary of the site. To the west of the site is a car park, whilst to the south lies Weston Mill Lake, beyond which lies the majority of the Dockyard facilities.

Figure 2-1: Site location map and sub-catchments of the Weston Mill Stream.



2.1.3 The main access road into the site is from the east, via the existing bridge over the Weston Mill Stream. The area of the site proposed for the EfW CHP facility itself consists of low lying land on which remains stockpiled earth and rubble left by the former tenants Ashcroft.

2.1.4 In the south of the site is an area of made-ground raised approximately 5 m above the land to the north. This area of land is known colloquially as 'Table Top Mountain'. It is understood that this made-ground consists of waste material from other parts of the dockyard. The made-ground area is currently used by the MoD as storage for heavy vehicles and machinery.

2.1.5 Appendix C provides photos taken during the site walkover, along with photos of the surrounding area, at both low and high tide.

2.2 Topography

2.2.1 The site topographic survey is provided in Appendix D. The general topography within the area proposed for the EfW CHP Facility is relatively flat with ground levels ranging between 7 and 8m AOD. 'Table Top Mountain' is also relatively flat with ground levels ranging between 11-13m AOD.

2.3 Existing Drainage

2.3.1 It is understood that there is currently no drainage network servicing the site. Therefore, surface water is thought to infiltrate directly into the ground, where ground conditions allow, or flow overland and drain into Weston Mill Stream.

2.4 Identification of Local Water Features

2.4.1 As shown in Figure 2-1 the Weston Mill Stream is the main watercourse in the vicinity of the site with a catchment area of 5.7 km^2. The Weston Mill Stream has two tributaries the Camels Head Creek and the Barne Brake Creek, which have catchment areas of 0.87 km^2 and 0.91 km^2 respectively. All three catchments are relatively steep, heavily urbanised catchments and respond rapidly to rainfall.

2.4.2 The Weston Mill Stream drains to Weston Mill Lake, which is connected to the dock and the Tamar Estuary, located to the south-west of the site, via a box culvert. All of these features are tidally influenced and are provided in the photos within Appendix C and the site layout in Appendix E.

3 Proposed Development and Policy Context

3.1.1 MVV proposes to construct and operate an EfW CHP facility at the site which will have the capacity to process under certain conditions up to 265,000 tonnes per annum of waste.

3.1.2 The development will comprise the following principal components:

- Tipping hall;
- Waste bunker hall;
- Bale store;
- Turbine / boiler house;
- Air pollution control system, including 95m high chimney;
- Bottom ash collection area;
- Air cooled condensers;
- Water treatment plant building;
- Central control room;
- Administration building;
- Workshop and stores building;
- Transformer compound for the export of electricity from the facility;
- Emergency diesel generator enclosure; and
- Electricity cables and steam and condensate pipework for connection to the relevant networks.

3.1.3 In addition to these principal components, there will also be access roads and trafficked areas for operational purposes; replacement of two existing crossings of Weston Mill Stream with a new clear-span bridge; weighbridges and a gatehouse; drainage and connections to infrastructure; hard and soft landscaping and an ecological mitigation area.. The site layout is provided in Appendix E.

3.1.4 The design life of the EfW CHP facility will be 30 years, and the life expectancy of the facility is approximately 40 years. Therefore when considering the impact of climate change on net sea level rise a design life of 60 years (up to 2071) has been used to ensure a conservative approach to flood risk is adopted.

3.1.5 The offsite access route for delivery of waste is largely dependent on its origin, however for waste generated outside of the Plymouth area the likely delivery route would be along the A38 via Weston Mill Drive.

3.2 Policy Context

Local Development Framework – Adopted Core Strategy

3.2.1 The Plymouth Core Strategy was adopted in April 2007 and provides a strategy and policy direction for development within Plymouth up to 2021 (Plymouth City Council, 2007). The Core Strategy does not include the site as an area for development. The core policies included within the Core Strategy relevant to flood risk reflect those stated in PPS25.

Plymouth City Council Strategic Flood Risk Assessment (SFRA)

3.2.2 Local Planning Authorities use SFRAs as part of the wider evidence base to inform the Sustainability Appraisal of the Local Development Framework. Evidence provided by the SFRA allows the application of the Sequential Test with regard to flood risk, as set out in PPS25 and the associated Practice Guide, in the allocation of development sites.

3.2.3 Four sites are allocated in the Plymouth Waste Development Plan Document (DPD) (Plymouth City Council, 2008) as being suitable for Strategic Waste Management Facilities.

- Coypool China Clay Works
- Chelson Meadow
- Moorcroft Quarry
- Land West of Ernesettle Lane

3.2.4 The SFRA examined the following four sites and their respective flood risks. The four sites in the SFRA do not have exactly the same boundaries as those in the Waste DPD, for reasons unknown, but for the purposes of this analysis we treat them here as being the same sites.

- Area 4: Coypool, Waste Site. Site in Flood Zone 1. No mitigation required. No risk from flooding.
- Area 5: Chelson Meadow Southwest Sector. Majority of site in Flood Zone 1 but southwest corner and boundaries of site within Flood Zones 2 and 3. Larger scale engineering is likely to be required for the shoreline defences. If mitigation advice is followed the significant risk from flooding will be reduced. Any development that is protected by shoreline defences will however still have a relatively high residual risk.
- Area 8: Moorcroft. Site in Flood Zone 1. No mitigation required. No risk from flooding.
- Area 29: Ernesettle. Site in Flood Zone 1. No mitigation required. No risk from flooding.

Sequential and Exception Tests

3.2.5 The site of the proposed EfW CHP facility at North Yard, Devonport, is not allocated in the Plymouth Waste DPD nor was it assessed in the SFRA. The significant majority of this site is within Flood Zone 1, except a small section of the proposed access road which falls within Flood Zone 2, for which mitigation measures are available (see Chapter 7).

3.2.6 The proposed development is classified as 'less vulnerable' according to PPS25, Table D.2. PPS25 Table D.3 indicates that 'less vulnerable' developments are considered appropriate in all Flood Zones except Flood Zone 3b. The Exception Test is therefore not required.

3.2.7 It is considered that in terms of the Sequential Test the site of the proposed development compares favourably with Coypool, Moorcroft and Ernesettle, and is preferential to Chelson Meadow, in flood risk terms. There are other factors involved in assessing the merits of alternative sites and determining site suitability and selection, but these will be dealt with elsewhere in the planning application.

4 Tidal and Fluvial Flood Hydraulic Modelling

4.1 Background

4.1.1 As part of this Level 3 FRA, combined tidal/fluvial event flood modelling has been undertaken to assess the flood regime of the Weston Mill Stream and its tributaries that flow adjacent to the site and access road. The rationale behind this was to investigate two main issues:

- The impact of a combined tidal/fluvial event on the Devonport EfW CHP facility and associated site access roads;
- The upstream impact on flood levels resulting from the replacement of two existing access bridges (over the Weston Mill Stream) with a single open span bridge.

4.1.2 Maximum flood levels at locations of interest adjacent to the site and access roads can be used to inform development design and provide information on site access road levels. Water levels in the Weston Mill Stream are dependent on downstream tidal levels associated with the Tamar Estuary, and upstream river flow, the relative importance of which may vary along the length of the watercourse.

4.1.3 The Environment Agency Flood Map does not adequately consider the flood risk from a combined event occurring in the Weston Mill Stream. Consequently to inform the understanding of the tidal/fluvial flood risk at the site a 1-dimensional (1D) hydrodynamic hydraulic model was developed using the ISIS (version 3.4) computational modelling package.

4.1.4 The ISIS hydraulic modelling has been undertaken in the following five stages, which are described within the following sections of the report:

1. Combined event analysis (Section 4.2 and Appendix F);
2. Model construction (Section 4.3);
3. Baseline model conditions (Section 4.4);
4. Review of the baseline results (Section 5.5); and
5. Sensitivity analysis (Section 5.6).

4.2 Combined Event Analysis

4.2.1 A combined event analysis desk based study has been undertaken to estimate the probability of extreme downstream tide levels occurring simultaneously with upstream river flows. The combined event analysis identifies the relevant return period for tide levels and river flows, which form the downstream and upstream boundaries of the 1d hydraulic model.

4.2.2 Details of the combined event analysis undertaken to determine tide levels, peak flows and return periods used within the model to represent the tidal and fluvial extents of Flood Zone 3 are provided within the Hydrology Briefing Note provided in Appendix F. The Environment Agency has confirmed that the information presented within the briefing note is acceptable (see Appendix B).

4.2.3 Due to the long return period associated with Flood Zone 2 (1 in 1000 year) the combined event analysis desk based study is considered unsuitable. Therefore, the parameters used to determine the tidal and fluvial extents of Flood Zone 2 use the 1 in 1000 year fluvial/tidal return period with a similar combination used to define the Flood Zone 3 extents.

4.2.4 Table 4-1 provides a summary of the tide levels, peak flows and return periods used within the model to determine the fluvial and tidal extents of Flood Zone 3, Flood Zone 3 accounting for climate change (up to 2071) and Flood Zone 2.

Table 4-1: Flood Zone Return Periods River Flow and Sea Level Combinations

Flood Zone	Return Period and ReFH (Winter Method) Peak Flow Estimate (m^3/s)*	Tidal Return Period and Sea Level (m AOD)
Flood Zone 3 Tidal (2011)	5 Year (5.90)	200 Year (3.71)
Flood Zone 3 Fluvial (2011)	100 Year (13.00)	2 Year (3.09)
Flood Zone 2 Tidal (2011)	5 Year (5.90)	1000 Year (3.99)
Flood Zone 2 Fluvial (2011)	1000 Year (24.90)	2 Year (3.06)
Flood Zone 3 Tidal (Climate Change 2071)	5 Year (7.08)	200 Year (4.18)
Flood Zone 3 Fluvial (Climate Change 2071)	100 Year (15.6)	2 Year (3.56)

*River flows shown are for Weston Mill Stream

4.2.5 Extreme tidal still water levels (Year 2002) for Devonport (SX455540) have been drawn directly from the Environment Agency South West Region Report on Regional Extreme Tide Levels (Environment Agency, 2003 – See Appendix B). Due to the anticipated effects of climate change an allowance for net sea level rise has been calculated for Year 2011 and Year 2071 to represent present day and future extreme tide levels over the developments lifetime (PPS25 Table B-1).

4.2.6 Flow estimation has been undertaken for each watercourse using both the Revitalised Flood Hydrograph (ReFH) Method and WINFAP-FEH v3 (2009) Statistical Method to allow comparison of flow estimates to ensure conservative flow estimates are adopted within the hydraulic modelling study. The ReFH winter design rainfall parameters peak flow estimates were shown to be the more conservative flow estimates and have therefore been adopted (as hydrographs – Appendix G) for use to represent the upstream fluvial boundaries of the model. The Environment Agency has confirmed that the flow estimation technique and peak flows adopted within the model are acceptable (see Appendix B). Full details of the hydrological analysis undertaken with further reasoning to support the chosen flow estimation technique are provided in Appendix F.

4.2.7 Due to the anticipated affects of climate change a 20% increase in peak river flow has been calculated for the future climate change scenarios (PPS25 Table B-2).

4.3 Model Construction

Model Software and Approach

4.3.1 The Weston Mill Stream and its tributaries (Camels Head Creek and Barn Brake Creek) are a tidally influenced system located within the downstream reaches of the valley. The ISIS 1D hydrodynamic hydraulic model approach has incorporated extended cross sections to represent the channel and floodplain.

4.3.2 A location map indicating the upstream and downstream extents of the hydraulic model, key cross section location and in channel structures is provided in Appendix H.

Topographic Information

4.3.3 Cross sections to represent the channel and floodplain have been extracted from LiDAR data (1m resolution) using the Geographical Information System (GIS) software MapInfo.

4.3.4 Scott Wilson undertook a site visit to identify specific cross section locations of the Weston Mill Stream and its tributaries to allow an accurate representation of the watercourse and floodplain geometry. The dimensions of other key structures in the model reach were also identified.

4.3.5 The topographic survey was also used to measure the dimensions of the structures associated within the two existing access bridges due to be replaced.

4.3.6 It is noted that a full channel survey has not been undertaken to inform this modelling study. For sections of the channel and floodplain covered by the topographic survey (see Appendix D), the survey was reviewed to ensure that the LiDAR data provides accurate level information within the model.

4.3.7 Furthermore, the downstream boundary water level within the model is based on a stage height (m AOD) and therefore modelled water levels are independent of the channel bed level and any potential inaccuracies in bed level, resulting from LiDAR data are unlikely to influence model results.

Hydraulic Model Extent

4.3.8 The upstream extent of the hydraulic model is located immediately downstream of the culvert adjacent to the Weston Mill Hill ‘fly over’ (SX457579). The box culvert entering Weston Mill Lake located approximately 1.5 km downstream (SX446571) represents the downstream extent of the hydraulic model. The open channel extents of Camels Head Creek and Barne Brake Creek are also included within the hydraulic model. A model schematic is provided in Appendix H.

Hydrological Representation

4.3.9 Inflows for the Weston Mill Stream, Camels Head Creek and Barne Brake Creek are represented as a point source hydrograph, at the upstream extent of each watercourse.

4.3.10 Water levels to represent the tidal downstream boundary during a tidal flood event were generated by a summation of the astronomical tide levels and the storm surge residual. An example of the tidal water levels used for the downstream boundary is shown in Figure 4-1.

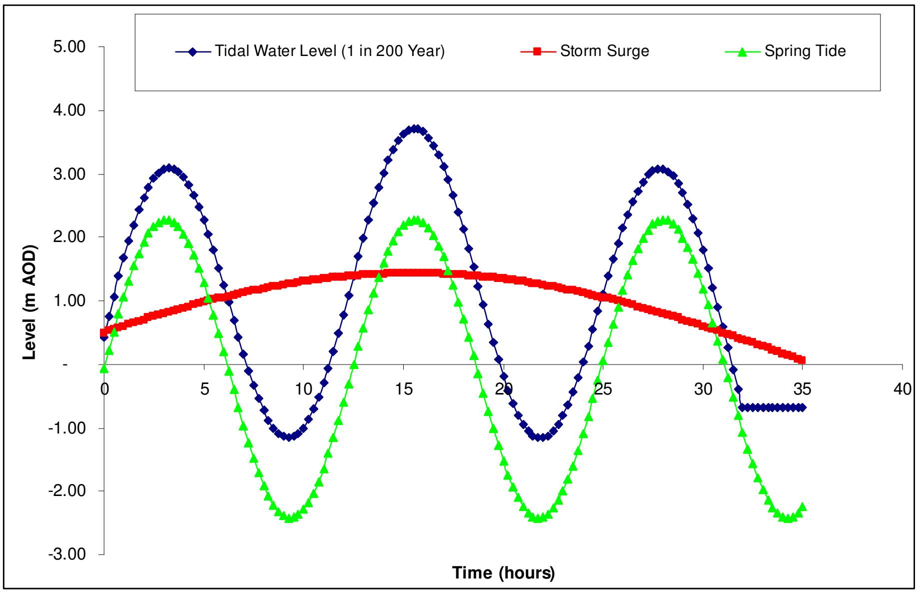


Figure 4-1: Extreme tidal curve with tidal and surge components

Structures

4.3.11 Key structures considered to have an impact on local water levels, especially during flood flows were identified and included in the hydraulic model. Table 4-2 identifies the key structures included within the model. Appendix H provides a location map of the structures within the model.

Table 4-2: Description of in-line structures

Map Reference	Location	Details
A	Entrance to Weston Mill Lake	Box culvert – concrete structure. Width 4.2m, height 6.1m, length 12m
B	Site access bridge (downstream)	4 x 2.2m diameter culvert. Corrugated metal structure
C	Site access bridge (upstream)	Box culvert – concrete structure, with sheet piling headwalls and security screen. Width 5m, height 5.59m, length 5.6m
D	Wolseley Road	1 x 1.6m diameter culvert. Concrete structure, with trash screen. Length 170m. A 10% debris factor was included within the model based on observations.

Proposed Development

4.3.12 The proposed development plans include the replacement of the two existing site access bridges (B and C) and associated in channel structures (see Table 4-2) with a single open span bridge.

4.4 Baseline Model Conditions

4.4.1 The baseline model conditions consist of various parameters that will exert a control upon the calculated water levels within the hydraulic model. The baseline model forms the key scenario which is tested as part of the sensitivity analysis, as discussed in Section 4.7, to ensure the model is robust and reliable.

4.4.2 The selected baseline model is the Flood Zone 3 tidal dominant scenario. See Table 4-1 for the relevant return periods, tide levels and river flows, which form the downstream and upstream boundaries for this selected baseline scenario.

Roughness Coefficient (Manning's 'n' Value)

4.4.3 The roughness coefficients used in the hydraulic model are represented by Manning's 'n' values. Values of Manning's 'n' for the channel and floodplain were estimated from visual inspection of the channel and floodplain and with reference to Open Channel Hydraulics (Chow, 1973 Table 5-6). Manning's values adopted for the channel bed, channel sides and floodplain were 0.035, 0.045 and 0.065 respectively. Manning's values adopted for in channel concrete structures were 0.02.

Model Run Parameters

4.4.4 Model run parameters were retained for all modelled scenarios. Table 4-3 lists the model run parameters applied.

Table 4-3: Model run parameters

Parameter	
Run time	36 hours
Timestep	1 second
Maximum iterations	15
All other parameters	Default

4.5 Model Results

Peak Flood Levels

4.5.1 Peak flood levels from selected cross sections along the model reach have been extracted from the ISIS model for each of the modelled scenarios. The selected node points and their locations are provided in Table 4-4 and the peak flood level for each scenario modelled is provided in Table 4-5. Cross section locations are also shown on the mapping provided in Appendix H.

Table 4-4: Key cross sections within model domain used to compare water levels

Node	Location
WM_0329	Weston Mill Stream immediately downstream of site access bridge A
WM_0390	Weston Mill Stream immediately upstream of site access bridge B
BB_0054	Barne Brake Creek adjacent to eastern site boundary
WM_0512	Weston Mill Stream immediately downstream of viaduct
WM_0563	Weston Mill Stream immediately upstream of viaduct
WM_0716	Weston Mill Stream downstream of Wolseley Road Culvert
WM_0986	Weston Mill Stream upstream of Wolseley Road Culvert
WM_1128	Weston Mill Stream adjacent to Weston Mill Drive
WM_1518	Weston Mill Stream upstream extent of model

Table 4-5: Peak flood levels for each modelled scenario for selected node points

Node	Flood Zone 2 Fluvial 2011 (mAOD)	Flood Zone 2 Tidal 2011 (mAOD)	Flood Zone 3 Fluvial 2011 (mAOD)	Flood Zone 3 Tidal 2011 (mAOD)	Flood Zone 3 Fluvial 2071 (mAOD)	Flood Zone 3 Tidal 2071 (mAOD)
WM_0329	3.17	3.99	3.10	3.71	3.58	4.18
WM_0390	3.17	3.99	3.10	3.71	3.58	4.19
BB_0054	3.18	3.99	3.11	3.72	3.58	4.19
WM_0512	3.18	4.00	3.11	3.72	3.58	4.19
WM_0563	3.18	4.00	3.11	3.72	3.58	4.19
WM_0716	3.19	4.00	3.11	3.72	3.58	4.19
WM_0986	5.25	4.54	4.90	4.30	5.09	4.90
WM_1128	5.25	4.54	4.90	4.30	5.09	4.90
WM_1518	5.49	4.56	4.92	4.34	5.18	4.91

Flood Mapping

4.5.2 Maximum water levels were exported from ISIS to MapInfo and assigned to a relevant cross section polyline. A maximum water surface grid was generated using the Triangular Irregular Network (TIN) 5th order solution interpolation method with a 1 m cell size equivalent to the LiDAR Digital Terrain Model (DTM) data. The maximum water surface grid was then subtracted from the DTM to produce the maximum flood depth grid.

4.5.3 Flood extent mapping and flood depth mapping for all the modelled scenarios shown in Table 4-1 are provided in Appendix I.

Indicative Flood Hazard

4.5.4 Indicative flood hazard information based on 'Flood Risk Assessment Guidance for New Developments: Phase 2, FD2320/TR2[3]' (specifically Table 13.1 of the document) has been shown for critical locations on the flood depth mapping provided in Appendix I.

4.5.5 The indicative flood hazard information takes into account flood depth and velocity to provide an indication of the likely hazard posed to people (see Table 4-6). Flood depth information is available from the ISIS 1D model (as shown in Appendix I), however the 1D model outputs do not include velocity information, this type of information is normally generated using a 2D model. Therefore, in liaison with the Environment Agency an alternative approach using Manning's equation[4] has been agreed to assume an indicative velocity at each of the critical locations.

4.5.6 Based on the flood depth and the assumed velocity an average indicative flood hazard (based on the average depth) and the maximum indicative flood hazard (based on the maximum depth) within the flood extent at critical locations during the flood peak have been provided in Appendix I. The flood depth value and assumed velocity value, calculated using Manning's equation is also shown in Appendix I. The flood hazard matrix and categories are shown below in **Error! Reference source not found.**.

Table 4-6: Danger to people for different combinations of depth and velocity. (Table 13.1 from 'Flood Risk Assessment Guidance for New Developments: Phase 2, FD2320/TR2).

		Depth (m)											
		0.05	0.10	0.20	0.30	0.40	0.50	0.60	0.80	1.00	1.50	2.00	2.50
Velocity ($m\,s^{-1}$)	0.00												
	0.10												
	0.25												
	0.50												
	1.00												
	1.50												
	2.00												
	2.50												
	3.00												
	3.50												
	4.00												
	4.50												
	5.00												

Key:

	Danger for some – includes children, the elderly and the infirm
	Danger for most – includes the general public
	Danger for all – includes emergency services

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[3] Available online: http://evidence.environment-agency.gov.uk/FCERM/Libraries/FCERM_Project_Documents/FD2320_3364_TRP_pdf.sflb.ashx

[4] Manning's equation determines velocity using flood depth, channel (flow pathway) dimensions, channel slope and Manning's 'n' value. A Manning's 'n' value of 0.020 has been used to represent the road surface.

Baseline Model: Flood Zone 3 Tidal

4.5.7 The baseline Flood Zone 3 tidal flood extent scenario consists of the 200 year tidal flood peak coinciding with the 5 year fluvial flood peak.

4.5.8 The flood mapping provided in Appendix I indicates that the Flood Zone 3 tidal extent remains predominantly within channel upstream of the Wolseley Road Culvert. Downstream of this culvert some flooding is observed on the right bank upstream of the Weston Mill Viaduct with flood depths predominantly less than 0.5 m. No flooding is observed within the site boundary including the access roads both within and beyond the site boundary along Weston Mill Drive.

4.5.9 No flood hazard exists at critical locations identified in Appendix I.

Flood Zone 3 Fluvial

4.5.10 The Flood Zone 3 fluvial flood extent scenario consists of the 100 year fluvial flood peak coinciding with the 2 year tidal flood peak.

4.5.11 The flood mapping provided in Appendix I indicates that the Flood Zone 3 fluvial extent also remains predominantly within channel upstream of the Wolseley Road Culvert. However, overbank flows occur at the inlet to the Wolseley Road Culvert.

4.5.12 A review of ground levels indicates floodwaters would flow into the subway, before flowing down Weston Mill Drive. The model indicates that the volume of water flowing into the subway (649 m^3), would be contained within the void provided by the subway (approximately 1200 m^3). Therefore, no floodwaters would flow down Weston Mill Drive during this scenario.

4.5.13 Downstream of the Wolseley Road Culvert the Flood Zone 3 fluvial extent predominantly remains within channel. No flooding is observed within the site boundary, which includes the proposed site access routes within the site boundary.

4.5.14 No flood hazard exists at critical locations identified in Appendix I.

Flood Zone 2 Tidal

4.5.15 The Flood Zone 2 tidal flood extent scenario consists of the 1000 year tidal flood peak coinciding with the 5 year fluvial flood peak.

4.5.16 The flood mapping provided in Appendix I indicates that the Flood Zone 2 tidal extent remains predominantly within channel upstream of the Wolseley Road Culvert. Downstream of this culvert flooding is observed on the right bank upstream of the Weston Mill Viaduct with flood depths predominantly less than 0.75 m.

4.5.17 Flooding is also observed on the left bank running parallel to the Weston Mill Viaduct, which partially covers the proposed site access route. Flood depths in this area are predominantly less than 0.3 m. No flooding is observed elsewhere within the site boundary.

4.5.18 With the exception of the onsite access road, where mitigation measures are proposed (see Chapter 7), no flood hazard exists at critical locations identified.

Flood Zone 2 Fluvial

4.5.19 The Flood Zone 2 fluvial flood extent scenario consists of the 1000 year fluvial flood peak coinciding with the 2 year tidal flood peak.

4.5.20 The flood mapping provided in Appendix I indicates that the Flood Zone 2 fluvial extent covers Weston Mill Drive and Wolseley Road in the vicinity of Camel's Head Junction. The model results indicate that overbank flows occur in the vicinity of the Wolseley Road Culvert inlet flooding the subway before flowing down Weston Mill Drive to Wolseley Road.

4.5.21 The model results shown in Figure 4-2 demonstrates that inundation occurs at the site entrance for less than 4 hours during the modelled event. During this period flood waters return to the channel (Camel's Head Creek) adjacent to the site entrance, after which, the flood level would decrease sufficiently to ensure dry access to the site is available. Figure 4-2 also indicates that floodwater at the site entrance would only exceed 0.3 m in depth for approximately 1.5 hours.

4.5.22 With the exception of the subway, flood depths along Weston Mill Drive are predominantly less than 0.3 m. Flood depths along Wolseley Road (including the site entrance) are predominantly less than 0.3 m, however, depths up to 0.5 m are experienced during the flood peak.

4.5.23 Downstream of the Wolseley Road Culvert the Flood Zone 2 fluvial extent remains predominantly within channel with only a small area of flooding observed on the right bank upstream of the Weston Mill Viaduct. No flooding is observed within the site boundary including site access routes within the site boundary.

4.5.24 No flood hazard exists on the onsite access road. Average and max flood Hazard at site entrance with Wolseley Road is considered 'danger for most'. However, it is important to note that period of inundation for this extreme scenario occurs for less than 4 hours over the tidal peak, before dry access to the site would again be available.

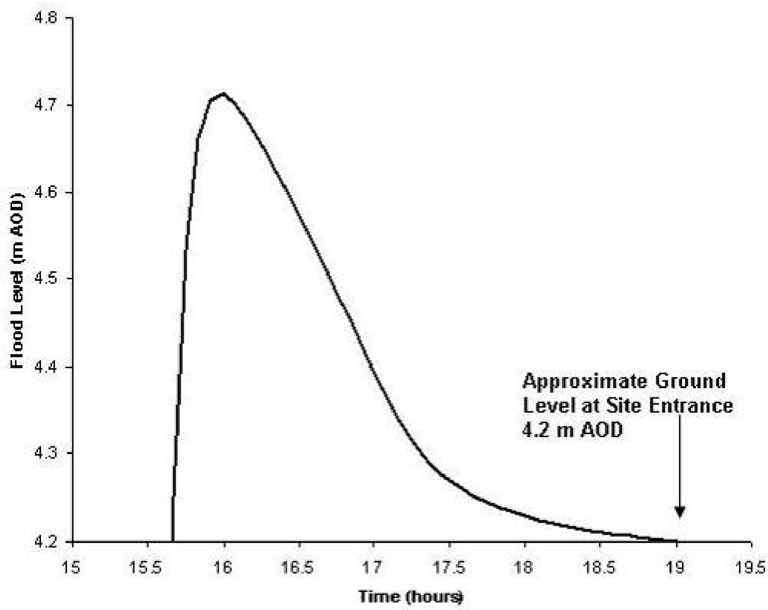

Figure 4-2: Time period during which fluvial Flood Zone 2 flood level exceeds ground level at site entrance

Flood Zone 3 Tidal - Climate Change 2071

4.5.25 The Flood Zone 3 climate change tidal flood extent scenario consists of the 200 year tidal flood peak (accounting for predicted sea level rise up to 2071) coinciding with the 5 year fluvial flood peak (including a 20% increase in peak flows).

4.5.26 The flood mapping provided in Appendix I indicates that the Flood Zone 3 tidal climate change extent comes out of bank upstream of the Wolseley Road Culvert flooding the subway. Similar to the Flood Zone 3 Fluvial scenario, the volume of water spilling out of bank (820 m^3) would be contained within the subway void and therefore, no floodwaters would flow down Weston Mill Drive during this scenario.

4.5.27 Downstream of this culvert, flooding is observed on the right bank upstream of the Weston Mill Viaduct with flood depths predominantly between 0 and 1 m.

4.5.28 Flooding is also observed on the left bank running parallel to the Weston Mill Viaduct, which partially covers the proposed site access route. Flood depths in this area are predominantly less than 0.5 m. No flooding is observed elsewhere within the site boundary.

4.5.29 With the exception of the onsite access road, where mitigation measures are proposed (see Chapter 7), no flood hazard exists at critical locations identified.

Flood Zone 3 Fluvial - Climate Change 2071

4.5.30 The Flood Zone 3 fluvial flood extent scenario consists of the 100 year fluvial flood peak (including a 20% increase in peak flows) coinciding with the 2 year tidal flood peak (accounting for predicted sea level rise up to 2071).

4.5.31 This scenario is similar to the present day Flood Zone 2 fluvial scenario. The flood mapping provided in Appendix I shows that the Flood Zone 3 climate change fluvial extent comes out of bank upstream of the Wolseley Road Culvert flooding the subway before flowing down Weston Mill Drive to Wolseley Road.

4.5.32 The model results shown in Figure 4-3 demonstrates that inundation at the site entrance for a period of approximately 3 hours occurs at the flood peak. During this period flood waters return to the channel (Camel's Head Creek) adjacent to the site entrance, after which, the flood level would decrease sufficiently to ensure dry access to the site is available. Figure 4-3 also indicates that floodwater at the site entrance is less than 0.3 m for the majority of the flood peak.

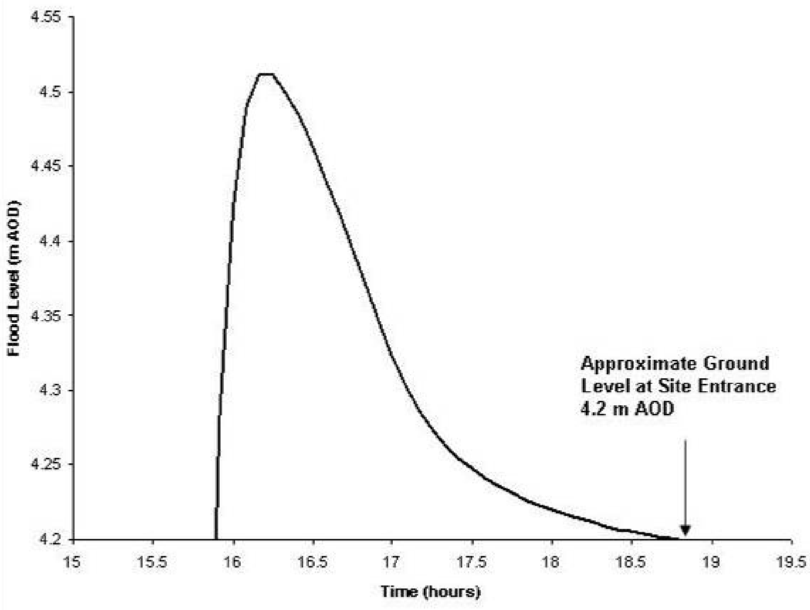

Figure 4-3: Time period during which fluvial Flood Zone 3 including climate change flood level exceeds ground level at site entrance

4.5.33 With the exception of the subway, flood depths along Weston Mill Drive are predominantly less than 0.3 m. Flood depths along Wolseley Road (including the site entrance) are predominantly less than 0.3 m, however depths up to 0.5 m are experienced during the flood peak.

4.5.34 Downstream of the Wolseley Road Culvert the Flood Zone 3 climate change fluvial extent predominantly remains within channel with the exception of an area of flooding on the right bank upstream of the Weston Mill Viaduct. No flooding is observed within the site boundary, which includes the proposed site access routes within the site boundary.

4.5.35 No flood hazard exists on the onsite access road. Average and max flood Hazard at site entrance with Wolseley Road is considered 'danger for most'. However, it is important to note that inundation for this extreme scenario would only occur for approximately 3 hours over the tidal peak, before dry access to the site would again be available.

4.6 Sensitivity Analysis

4.6.1 Sensitivity testing has been undertaken on the hydraulic model to assess the impact of altering key parameters within the model and observing the change in output. This provides an indication of the robustness of the hydraulic model. The key parameters adjusted are:

- Upstream boundary conditions (i.e. fluvial flows);
- Downstream boundary condition (i.e. tide level);
- Manning's 'n' values (roughness coefficient); and
- Removal of existing site access bridges A and B (including associated structures).

4.6.2 All of these model runs were variations of the baseline model (i.e. Flood Zone 3 Tidal), consistent with that discussed previously within this report. Table 4-4 identifies nodes selected along the model reach to compare peak water levels for each of the sensitivity test model runs.

4.6.3 Where peak water levels deviate by 0.05m or less from the corresponding baseline peak water level, the particular cell in the table has been shaded in blue. This has been undertaken for each of the sensitivity tests and helps to identify the more sensitive nodes (i.e. non-shaded cells) and aids comparison of individual sensitivity tests.

Upstream Boundary Condition (Inflows)

4.6.4 Inflow sensitivity has been tested by running the hydraulic model with a ±20% for the length of the hydrograph. A summary of the results of the sensitivity analysis for inflow is presented in Table 4-6.

Table 4-6: Model inflow sensitivity testing results

Node/Cross Section	Baseline Water Level (mAOD)	Baseline +20% Inflow Flood Level (mAOD)	Baseline -20% Inflow Flood Level (mAOD)
WM_0329	3.71	3.71	3.71
WM_0390	3.73	3.73	3.72
BB_0054	3.73	3.73	3.72
WM_0512	3.73	3.73	3.72
WM_0563	3.73	3.73	3.72
WM_0716	3.73	3.73	3.72
WM_0986	4.31	4.49	4.14
WM_1128	4.31	4.49	4.14
WM_1518	4.34	4.51	4.18

4.6.5 Table 4-7 indicates that nodes further upstream within the model are more sensitive to changes in inflows, which is illustrated by the non-shaded cells further down the table. The average impact on maximum water level in the upper extent of the model is approximately ±0.18 m.

4.6.6 The flows used within the hydraulic modelling apply the ReFH Winter Method and are considered to be conservative flow estimates.

Downstream Boundary Conditions (Extreme Tide Levels)

4.6.7 The downstream boundary conditions within the model simulate a downstream extreme tidal water level. To investigate the potential effects of this upon maximum flood levels, the downstream boundary condition has been increased by 0.5 m within the hydraulic model. A summary of the results of the sensitivity analysis for downstream tidal level is presented in Table 4-7.

Table 4-7: Downstream boundary conditions sensitivity testing results

Node/Cross Section	Baseline Water Level (mAOD)	Baseline Flood Level +0.5m Boundary Condition(mAOD)
WM_0329	3.71	4.21
WM_0390	3.73	4.22
BB_0054	3.73	4.22
WM_0512	3.73	4.22
WM_0563	3.73	4.22
WM_0716	3.73	4.22
WM_0986	4.31	4.73
WM_1128	4.31	4.73
WM_1518	4.34	4.74

4.6.8 Table 4-8 indicates that the model is extremely sensitive to changes in the downstream boundary tide level, which is illustrated by all the non-shaded cells. Table 4-8 demonstrates how a 0.5 m increase in the downstream water level results in a 0.5 m increase throughout the extent of the model. This is expected as the Weston Mill Stream is tidally influenced throughout.

Roughness Coefficient (Manning's 'n' Value)

4.6.9 Channel and floodplain roughness sensitivity has been tested by running the hydraulic model with ±20% Manning's 'n' value. The impact observed on peak water level as a result of the varying Manning's 'n' values are shown in Table 4-9.

Table 4-8: Manning's 'n' values sensitivity testing results

Node/Cross Section	Baseline Water Level (mAOD)	Baseline +20% Manning's Flood Level (mAOD)	Baseline -20% Manning's Flood Level (mAOD)
WM_0329	3.71	3.71	3.71
WM_0390	3.73	3.72	3.73
BB_0054	3.73	3.73	3.73
WM_0512	3.73	3.73	3.73
WM_0563	3.73	3.73	3.73
WM_0716	3.73	3.73	3.73
WM_0986	4.31	4.35	4.27
WM_1128	4.31	4.35	4.27
WM_1518	4.34	4.40	4.29

4.6.10 The majority of peak water levels are unchanged compared to the baseline for both Manning's 'n' value sensitivity tests, which is illustrated by the majority of shaded cells. Therefore the hydraulic model is not considered to be sensitive to Manning's 'n' values.

Removal of Site Access Bridges

4.6.11 One of the main objectives of the modelling study is to assess the upstream impact on flood levels resulting from the replacement of the two existing access bridges A and B with a single open span bridge.

4.6.12 The sensitivity of the hydraulic model to these two in channel structures has been tested by running the Flood Zone 3 tidal model without the structures in place and comparing the results with the baseline Flood Zone 3 tidal model. A summary of the results of the sensitivity analysis for these in channel bridges structures is presented in Table 4-9.

Table 4-9: Removal of site access bridges sensitivity testing results

Node/Cross Section	Baseline Water Level (mAOD)	Flood Level (mAOD) Flood Zone 3 Tidal (without bridge structures A and B)
WM_0329	3.71	3.71
WM_0390	3.73	3.71
BB_0054	3.73	3.72
WM_0512	3.73	3.72
WM_0563	3.73	3.72
WM_0716	3.73	3.72
WM_0986	4.31	4.30
WM_1128	4.31	4.30
WM_1518	4.34	4.34

4.6.13 Table 4-10 demonstrates that there is a negligible change in peak water levels when the Flood Zone 3 tidal model with bridges structures A and B removed is compared to the baseline model, which is illustrated by the shaded cells. On average the removal of the bridges results in a reduction of 0.01 m in peak water levels.

4.6.14 As the hydraulic model is not considered to be sensitive to the removal of the two existing access site bridges the impact on flood levels upstream is considered to be negligible.

4.7 Summary

4.7.1 The combined analysis hydraulic modelling study has demonstrated that the proposed EfW CHP facility is located on land outside of the fluvial and tidal extents for Flood Zone 3, Flood Zone 2 and Flood Zone 3 including climate change. Therefore the built development area of the site should be considered as Flood Zone 1.

4.7.2 The proposed access road is located outside the fluvial extent for the Flood Zone 3, Flood Zone 2 and Flood Zone 3 including climate change and therefore any proposed level changes to the access route would not diminish the fluvial floodplain. This access route is also located outside of the tidal Flood Zone 3 extent.

4.7.3 Where considering the tidal extents for Flood Zone 2 and Flood Zone 3 including climate change scenarios parts of the onsite access road in the vicinity of the Weston Mill Viaduct experience flooding with depths less than 0.3 m and 0.5 m respectively. Mitigation measures are proposed (see Chapter 7) to ensure that there is no flood hazard along the onsite access road at this location.

4.7.4 Where considering site access and egress beyond the site boundary, dry safe access and egress is achievable via Wolseley Road to the north-west during all tidal scenarios considered and the present day fluvial Flood Zone 3 scenario. With the exception of a period less than 4 hours over the flood peak (where the flood hazard is 'danger for most') dry safe access and egress is also achievable along this route during the fluvial Flood Zone 2 and fluvial Flood Zone 3 including climate change scenarios.

4.7.5 Floodwaters coming out of bank upstream of the Wolseley Road Culvert during the fluvial Flood Zone 3 and the tidal Flood Zone 3 including climate change scenarios are contained within the subway void and therefore no floodwaters would flow down Weston Mill Drive.

4.7.6 Flooding is observed during the Flood Zone 2 and Flood Zone 3 including climate change scenarios along Weston Mill Drive, the more likely access route for delivery of waste generated outside the City. Flood depths associated with these scenarios are predominantly less than 0.3 m, however flood depths of 0.5 m are experienced for limited periods of time. The associated flood hazard is 'danger for some' and 'danger for most'. It should be noted that these peak flood depths and associated hazards would only be experienced for a short duration (less than 4 hours) over the tidal peak, after which flood waters would recede.

4.7.7 A sensitivity analysis of the hydraulic model has been undertaken. As expected the model is most sensitive to inflows and changes to the tidal downstream boundary. The model is not particularly sensitive to changes to Mannings 'n' values. The sensitivity analysis also suggests that the removal of the two existing access bridges onsite has a negligible impact on peak flood levels throughout the model.

4.7.8 It should be noted that there is always a level of uncertainty within the model results and an appropriate freeboard should be used when using model outputs to inform development design.

5 Flood Risk – To Development

5.1 PPS25 Policy

5.1.1 PPS25 requires that all potential sources of flooding that could affect the proposed development are considered. The section considers all potential sources as listed in Annex C (Forms of Flooding) of PPS25.

5.1.2 A combined analysis fluvial/tidal hydraulic modelling study has been undertaken to assess the flood risk to the site and third parties and to define the Flood Zones (see Chapter 4). The associated flood mapping outputs are presented in Appendix I. A risk based approach for the assessment of the other potential flood sources is presented below.

5.2 Tidal Flooding

Built Development Area

5.2.1 The Environment Agency Flood Map (see Appendix A), together with the modelling undertaken identifies that the built development area of the site is located within Flood Zone 1 (low probability of tidal flooding; i.e. land assessed as having less than 1 in 1000 annual probability of flooding in any year).

5.2.2 Proposed finished floor levels for the built development area are 9 m AOD, approximately 4.80 m above the modelled peak flood level for the tidal Flood Zone 3 including climate change scenario (4.19 m AOD) (see Table 4-5).

Onsite Access Route

5.2.3 The modelling undertaken indicates that parts of the access road, where the road runs parallel to the Weston Mill Viaduct fall within the tidal Flood Zone 2 extent and the Flood Zone 3 climate change (up to 2071) extent. Tidal flood depths during the Flood Zone 2 scenario are predominantly less than 0.3 m, increasing to between 0.3 m and 0.5 m during the Flood Zone 3 climate change (up to 2071) scenario.

5.2.4 During the flood peak the average flood depth and assumed velocity indicates no associated hazard within the flood extent during the tidal Flood Zone 2 extent and the Flood Zone 3 climate change (up to 2071), however the max hazard is ‘danger for most’.

Site Entrance

5.2.5 The modelling undertaken identifies that the site entrance at Wolseley Road is located within Flood Zone 1 for all tidal scenarios considered.

Replacement of Existing Access Bridges

5.2.6 The upstream impact on flood levels resulting from the replacement of two existing access bridges (over the Weston Mill Stream) with a single open span bridge has been assessed during the hydraulic modelling. The results shown in Table 4-9 indicate that on average a 10 mm reduction in peak water levels is experience throughout the model extent when the bridges are removed from the model. Therefore the removal of the two access bridges is considered to have a negligible impact on flood levels upstream or downstream of the site.

Flood Defences

5.2.7 The Environment Agency has confirmed that the site is in not within an area classified as benefiting from defences.

Historic Flood Event

5.2.8 The Environment Agency historic flood records indicate that there is no history of tidal flooding around the site.

5.2.9 Taking the above information into account the tidal flood risk posed to the built development area of the site is considered low. The tidal flood risk posed to the access road is considered medium.

5.2.10 Chapter 7 presents the proposed flood mitigation measures to ensure flood risk posed to the built development remains low and to ensure safe access and egress is maintained along identified access routes within the site boundary. Mitigation measures are also provided to inform the design of the proposed single open span bridge to ensure the channel is not impeded during a flood event.

5.3 Fluvial Flooding

Built Development Area

5.3.1 As stated in Section 5.4, the Environment Agency Flood Map (see Appendix A), together with the modelling undertaken identifies that the built development area of the site is located within Flood Zone 1 (low probability of fluvial flooding; i.e. land assessed as having less than 1 in 1000 annual probability of flooding in any year).

5.3.2 The overall modelled peak fluvial flood level (3.58 m AOD) adjacent to the site is during the Flood Zone 3 including climate change (up to 2071) scenario, which is considerably less then the peak tidal flood level (see Table 4-5).

Onsite Access Route

5.3.3 The modelling undertaken identifies that the onsite access road is located within Flood Zone 1 for all fluvial scenarios considered.

Site Entrance

5.3.4 The modelling undertaken identifies that the site entrance at Wolseley Road is located within Flood Zone 1 during the fluvial Flood Zone 3 scenario. During the fluvial Flood Zone 2 and fluvial Flood Zone 3 including climate change scenarios the entrance to the site at Wolseley Road is inundated for less than 4 hours over the flood peak, after which dry access to the site would again be available (see Figure 4-2 and 4-3). During this 4 hour period, if required, alternative dry emergency access remains available via the dockyard to the south.

5.3.5 At the flood peak, during the fluvial Flood Zone 2 extent and the fluvial Flood Zone 3 climate change (up to 2071) scenario both the average and max flood depth and assumed velocity within the flood extent results in a flood hazard of ‘danger for most’.

5.3.6 However, as indicated in Figures 4-2 and 4-3 flood depths greater than 0.3 m would only be experienced for limited periods of time over the flood peak (1.5 hours during the fluvial Flood Zone 2 scenario and 0.5 hours during the fluvial Flood Zone 3 including climate change scenario).

Offsite Access Route

5.3.7 The modelling undertaken identifies that during the fluvial Flood Zone 3 event floodwaters, overtopping at the upstream Wolseley Road Culvert inlet would be contained within the subway void. Therefore, no floodwaters would flow down Weston Mill Drive during this scenario.

5.3.8 The modelling indicates that flooding is observed during the fluvial Flood Zone 2 scenario, and to a slightly lesser extent the Flood Zone 3 including climate change. Floodwaters flow down Weston Mill Drive to Wolseley Road to the east of Camel’s Head Junction and then inundate the site entrance for a limited period of time over the flood peak (less than 4 hours), before dry access to the site is again available.

5.3.9 As previously mentioned in Chapter 4 in the above sections, peak flood depths and the associated hazard would only be experienced for a short duration (less than 4 hours) over the tidal peak, after which flood waters would recede. Furthermore, the probability of a 1 in 1000 year fluvial event occurring over the 40 year life expectancy of the development is less than 4%

5.3.10 Taking the above information into account the fluvial flood risk posed to the built development area of the site and the onsite access road is considered low and is not considered further within this assessment.

5.4 Overland Flow

5.4.1 Due to the urban nature of the catchment, overland flow generated within the catchment drains to the public and private sewer network. During events exceeding the design capacity of the system, sewers may become surcharged and overland flow may occur.

5.4.2 At the local scale, land to the east and the south of the site is relatively low lying, with ground levels in the region of 6m AOD. To the north and north-west of the site boundary in the vicinity of Barne Barton residential area the land rises steeply to a maximum elevation of around 40m AOD.

Flood Map for Surface Water

5.4.3 The Environment Agency have provided extracts from their Flood Map for Surface Water (FMfSW), which show areas in the vicinity of the site potentially affected by surface water flooding during the 1 in 30 year and 1 in 200 year rainfall events. These FMfSW are provided in Appendix B.

5.4.4 The 1 in 30 year and 1 in 200 year FMfSW indicates that land to the north-west, north and north-east of the built development area of the site may be affected by surface water flooding. A drainage channel has been identified during the site walkover at this location. Surface water runoff from the surrounding land drains to the channel, which is located immediately north-west of the built development area. This drainage channel is thought to infiltrate into the subsoil or eventually drain into Weston Mill Stream via the Barne Brake Creek.

5.4.5 This drainage channel is located approximately 4m below the proposed finished floor levels of the built development. Therefore the risk posed by this flood source is considered low due to absence of a flow pathway.

5.4.6 The 1 in 30 year and 1 in 200 year FMfSW also indicates that parts of the access road, where the road runs parallel to the Weston Mill Viaduct may also be affected by surface water flooding. This area is a natural low point where surface water is thought to pond after significant heavy rainfall. A small area of the access road at the junction with Wolseley Road is also shown to be affected by surface water flooding during the 1 in 200 year rainfall event.

5.4.7 The FMfSW indicate that flood depths along these access routes are greater than 0.1 m but less than 0.3 m. Therefore the flood risk posed by surface water flooding to site access and egress routes is considered low.

Critical Drainage Area

5.4.8 The Environment Agency has confirmed that the site is located within an area considered to be a Critical Drainage Area (Appendix B). This also highlights a series of reported incidents to the east, beyond the site boundary, where Weston Mill Stream passes under the railway line, through the Weston Mill Viaduct. However, there are no reported flooding incidents at the site.

5.4.9 Taking into account the above information, flooding on site from surface water is considered to pose a low flood risk. However the mitigation proposed to reduce the tidal flood risk to the built development and access roads described in Chapter 7 also incorporates mitigation for surface water flooding. Proposed plans to enhance the existing drainage channel identified to the north and north-west of the site are also presented in Chapter 7.

5.5 Sewers

5.5.1 Sewer flooding occurs when the sewer capacity becomes exceeded or where a blockage occurs causing the sewer to surcharge and flood. South West Water Internet Mapping has been consulted and identifies that no SWW assets are present within the area of the site proposed for built development, however a foul sewer crosses the far north west of the site, located approximately 80 – 100 m from the proposed built development area.

5.5.2 All drainage and water supply in the dockyard is dealt with by Kelda Water Services. A review of the asset plan indicates that no surface water or foul drainage networks exist within the site boundary.

5.5.3 Taking the above information into account the flood risk posed by sewer flooding is considered to be low and is not considered further within this assessment.

5.6 Artificial Sources

5.6.1 Artificial flood sources include raised channels such as canals or storage features such as ponds and reservoirs. There are no artificial flood sources close to the site. Therefore the flood risk posed by this flood source is considered to be negligible.

5.7 Groundwater

5.7.1 Groundwater flooding can occur when groundwater levels rise above the surface of the site. The Environment Agency has not provided any information on groundwater levels or history of flooding in the area.

5.7.2 The British Geological Survey (BGS) Solid and Drift Geology Map (Sheet 348), indicates that beneath the made ground the site is underlain by salt marsh/alluvium deposits over Upper Devonian Slates comprising the Saltash Formation. The Saltash Formation typically compromised of mudstone, siltstone and fine sandstone.

5.7.3 A review of the Environment Agency Groundwater Aquifer map[5] of the site indicates that the site does not lie within a Source Protection area. The superficial deposits are also not designated an aquifer, however, the bedrock has been designated as Secondary B bedrock. This is defined as 'predominantly lower permeability layers which may store and yield limited amounts of groundwater due to localised features such as fissures, thin permeable horizons and weathering. These are generally the water-bearing parts of the former non-aquifers'.

5.7.4 It should be noted that the groundwater regime at the site will be artificially altered due to the made ground on site.

5.7.5 Groundwater levels were recorded during the ground investigation undertaken by Geotechnics and are between 5 and 6m below ground level and are tidally influenced.

5.7.6 Therefore, taking the above information into account, the risk posed by groundwater flooding is considered low and is not considered further within this assessment.

[5] www.environment-agency.gov.uk

6 Flood Risk – From Development

6.1 Surface Water Management

6.1.1 PPS25 states that as well as assessing risk to a development, the risk of flooding arising from a development should be considered. In general, site development reduces the permeability of the site, increasing the volume and rate of water running off the site, potentially increasing flood risk to third parties. Therefore appropriate drainage arrangements are required for new developments to ensure that flood risk to others is not increased.

6.1.2 Annex F of PPS25 promotes the use of Sustainable Drainage Systems (SUDS) in new developments to ensure volumes and peak flow rates of surface water leaving a developed site are no greater than the rates prior to the proposed development, unless specific off-site arrangements are made and result in the same net effect.

6.1.3 Paragraph 5.54 of the PPS25 Practice Guide indicates that this should be achievable up to and including the 1% annual exceedance probability (1 in 100 years) event, including an appropriate allowance for climate change. For residential development climate change should be considered for a minimum of 60 years.

6.1.4 The following SUDS could be used within the site to restrict site runoff rates:

- **Filter strips and swales** - which are vegetated features that hold and drain water mimicking natural drainage patterns; and
- **Basins and ponds** - to hold excess water after rain and allow controlled discharge that avoids flooding.

6.1.5 Further guidance on SUDS selection, design and operation is provided in the following reference documents:

- **CIRIA C609 (2004)** Sustainable Drainage Systems, Hydraulic, Structural and Water Quality Advice. Wilson et al. Department of Trade and Industry;
- **CIRIA C635 (2006)** Designing for Exceedance in Urban Drainage - Good Practice. Digman et al. Department of Trade and Industry; and,
- **CIRIA C697 (2007)** The SUDS Manual. Woods et al. Department of Trade and Industry.

6.2 Existing Site Drainage

6.2.1 As discussed in Section 5.5, it is understood that there is currently no drainage network servicing the site. Therefore, surface water is thought to infiltrate directly into the ground, where ground conditions allow, or flow overland and drain into Weston Mill Stream.

6.2.2 Existing runoff rates from the site are likely to increase post development, as there will be an increase in hard standing area.

6.2.3 However, to ensure surface water is managed appropriately on site and does not exacerbate localised flooding issues as identified in Chapter 5, an appropriate surface water management strategy is required. The surface water management strategy will ensure that surface water is positively drained from the development and will account for anticipated effects of climate change on rainfall intensity over the development's lifetime (20% increase in rainfall intensity, to account for the anticipated effects of climate change over the lifetime of the development (PPS25, Table B2)).

6.3 Proposed Drainage Strategy

6.3.1 The proposed drainage strategy provided below has been developed by GHA Livigunn. A drainage layout provided by GHA Livigunn is provided in Appendix J.

Hardstandings

6.3.2 Positive drainage will be provided to all hardstanding areas through the use of a combination of gullies, linear drains or channels and hard pipe. The surface water will pass through a class 1 by-pass petrol interceptor (estimated size at this stage NSB20 - to be confirmed at detailed design stage) prior to being discharged to the tidal estuary of the river Tamar. An outfall structure complete with adequate flow calming measures and scour protection will be provided at the point of discharge. This new outfall structure will be located within the foot print of the site, the invert level of the outfall pipe at the point of discharge will be set such that it is above the maximum tidal water level for a 1 in 200 years return period (i.e 4.48 m AOD - note that this level already includes an allowance for climate change and a 300 mm freeboard). Consequently the design of the surface water system will be based on free discharge flow conditions.

6.3.3 Please note that it is intended to provide an emergency cut-off valve immediately upstream of the outfall such as to prevent any water discharging to the environment in the event of an accidental spill on site.

Roof and walls

6.3.4 It is proposed to provide a drainage system to drain the run-off roof and wall rain water to an infiltration system. It is intended that the main building roof and wall surfaces will be drained to an infiltration basin whereas the workshop building, due to its size, will be drained to an infiltration trench.

Design conditions

6.3.5 The design of the drainage system will be based on the following performance criteria:

- Design return period of 1 in 30 years: No surcharge in the system is allowed;
- Design return period of 1 in 30 years including an allowance of 20% for climate change: Surcharge of pipe work is allowed but no surcharge of the manhole and no flood risk;
- Design return period of 1 in 100 years including an allowance of 20% for climate change: Surcharge of the manhole is allowed and flooding is allowed locally.

6.3.6 At this stage, preliminary sizing calculations have been carried out for both system and the results are presented below:

Roof and Walls Run-off Rain Water

6.3.7 Within these preliminary calculations, a rate of infiltration of 0.7 m/hr has been considered. The description of the made ground material tends to indicate a gravel/sand type of ground. Typical infiltration values for these type of soils range between 0.1 m/hr to 10 m/hr for sands and 10 m/hr to 1000 m/hr for gravels which tend to indicate that the infiltration value used is realistic and could be viewed as conservative. A regime of infiltration tests will be carried out on site to confirm the value to be used at detailed design stage. The ground water table is significantly affected by the tides, however, the anticipated top ground water level is circa 5 m below ground level. This is considered sufficiently low to allow infiltration to be considered.

Infiltration basin

6.3.8 The maximum volume of stored water in the basin in a 1 in 30 years return period and 20% climate change is approximately 381 m^3 (254 m^2 plan area). It is intended that during the 1 in 100 event, the basin with a coping level set slightly below the average level of the site would overspill in to the neighbouring land (the excess volume is in the order of 134 m^3) or an overflow will be provided which will direct all flows to the outfall (in this instance, an additional peak flow of 40 l/s will be discharged at the outfall in addition to the peak outfall flow from the hardstanding, see below).

Infiltration trench (workshop)

6.3.9 The infiltration trench required is 0.6 m wide, 2 m depth and 50 m long. Porous material is provided over a depth of 1.5 m. This preliminary sizing (to be confirmed at detailed design stage) should be sufficient for a 1 in 100 years return period event (including 20% climate change).

Hardstanding

6.3.10 The total contributing area is in the order of 11000 m^2. The expected peak flows are 249 l/s for a 1 in 30 (including 20% climate change) and 298 l/s for a 1 in 100 (including 20% climate change).

7 Proposed Flood Mitigation

7.1 Mitigation of Flood Risk – To Development

7.1.1 The PPS25 Practice Guide[6] provides advice on the management of flood risk by design. This advice has been taken into account during the preparation of this FRA to inform the design of the proposed development at the site to reduce the level of flood risk posed to the development.

7.1.2 Minimum finished floor levels of the building, access road and the soffit level of the new open span bridge should be set above the modelled tidal Flood Zone 3 including climate change (up to 2071) flood level adjacent to the site (see Table 4-5).

7.1.3 Due to the uncertainty inherent with climate change predictions, the Environment Agency requires a 300 mm freeboard to be added to this flood level. Table 7-1 indicates the required minimum finished floor level for the built development, access road and soffit level of the new bridge.

Table 7-1: Minimum finished floor level

Modelled Event	Flood Level (m AOD)	Freeboard Allowance (m)	Minimum Finished Floor Level (mAOD)
Tidal Flood Zone 3 including climate change up to 2071	4.19	0.3	4.49

Onsite Access Road

7.1.4 The proposed onsite access route is located outside of the fluvial flood extent for all scenarios modelled. Therefore any proposed level changes to the onsite access road would not diminish the fluvial floodplain, and therefore no floodplain compensatory storage is required.

Replacement of Existing Access Bridges

7.1.5 The modelling study demonstrates that the removal of the two access bridges will have a negligible impact on flood levels upstream or downstream of the site. However, in addition to the minimum bridge soffit level (see Table 7-1) the proposed single open span bridge abutments will be positioned to allow a 15 m wide channel, this channel capacity provides considerable betterment compared to the existing bridge structures. A drawing of the bridge design is provided in Appendix K.

Enhancement of Drainage Ditch

7.1.6 Enhancement of the existing drain ditch to the west of the site will be carried out in order to collect and convey surface water (generated on higher ground to the north-west of the site) to the Barne Brake Creek. The drainage plan provided in Appendix J includes an indicative route for the drainage channel enhancement.

[6] Available online at:http://www.communities.gov.uk/documents/planningandbuilding/pdf/pps25practiceguide.pdf

7.2 Mitigation of Flood Risk – From the Development

Drainage Strategy

7.2.1 As outlined in Section 6.3, a drainage strategy has been provided by GHA Livigunn.

7.2.2 Surface water generated on all hardstanding areas will be drained through the use of a combination of gullies, linear drains or channels and hard pipe. Surface water will be discharged to the Weston Mill Stream via an outfall structure located within the foot print of the site.

7.2.3 The invert level of the outfall pipe at the point of discharge will be set such that it is above the maximum tidal water level for a 1 in 200 years return period, including an allowance for climate change and freeboard to ensure tide locking does not occur.

7.2.4 An emergency cut-off valve immediately upstream of the outfall will be provided to prevent any water discharging to the environment in the event of an accidental spill on site.

7.2.5 Surface water draining from the roof and walls of the main building will drain to an infiltration basin, whereas the workshop building, due to its size, will be drained to an infiltration trench.

7.2.6 To ensure the development does not pose a flood risk to itself from sewer flooding the drainage system has been designed to ensure that no surcharge in the system occurs for events up to and including the 1 in 30 year return period (in accordance with Paragraph 5.50 PPS25 Practice Guide[7]).

[7] PPS25 Practice Guide Updated December 2009 accessed online:
http://www.communities.gov.uk/documents/planningandbuilding/pdf/pps25guideupdate.pdf

8 Conclusions

8.1.1 Following the completion of this Level 3 Flood Risk Assessment, in line with the recommendations of PPS25, the following conclusions can be made:

8.2 Flood Risk – To Development

- The proposed EfW CHP facility at the North Yard, Devonport site is classified as 'less vulnerable' development according to PPS25 Table D.3. Less vulnerable developments are considered appropriate in all Flood Zones except Flood Zone 3b;
- The combined analysis hydraulic modelling study has demonstrated that the proposed EfW CHP facility is located on land outside of the fluvial and tidal extents for Flood Zone 3, Flood Zone 2 and Flood Zone 3 including climate change. Therefore the built development site is considered to be within Flood Zone 1;
- Areas of the proposed onsite access road in the vicinity of the Weston Mill Viaduct fall within the modelled Flood Zone 2 and Flood Zone 3 including climate change tidal flood extents with depths up to 0.3 m and 0.5 m respectively. The average depth an assumed velocity within these scenarios indicate no associated flood hazard, however the max indicative flood hazard is 'danger for most'. This FRA proposes mitigation measures to ensure no flood hazard exists along the onsite access road at this location;
- The upstream impact on flood levels resulting from the replacement of two existing access bridges (over the Weston Mill Stream) with a single open span bridge has been assessed within the modelling study. The removal of the two access bridges is considered to have a negligible impact on flood levels upstream or downstream of the site;
- The probability of a 1 in 1000 year fluvial event occurring over the 40 year life expectancy of the development is less than 4%. However, in the unlikely event of fluvial flooding, with the exception of 3 to 4 hours over the flood peak, dry safe access is achievable west via Wolseley Road. During this flood peak the average and max indicative flood hazard at the site entrance is 'danger for most'. During this 4 hour period, if required, alternative dry emergency access remains available via the dockyard to the south.
- Localised flood risk from surface water, sewers and artificial flood sources has been reviewed and are considered to pose a low risk to the site and access roads.

8.3 Flood Risk – From the Development

- There is currently no drainage network servicing the site. Therefore, surface water is thought to infiltrate directly into the ground, where ground conditions allow, or flow overland and drain into Weston Mill Stream;
- Existing runoff rates from the site are likely to increase post development, as there will be an increase in hard standing area;
- An appropriate surface water management strategy is required to ensure surface water is positively drained from the development and will account for anticipated effects of climate change on rainfall intensity over the development's lifetime (20% increase in rainfall intensity PPS25, Table B2).

8.4 Flood Mitigation Measures

Development Mitigation Measures

- Minimum finished floor levels of the building, access road and the soffit level of the new open span bridge should be set above the modelled tidal Flood Zone 3 including climate change (up to 2071) flood level, including freeboard (4.49 m AOD);
- The proposed onsite access road (with the exception of the site entrance) is located outside of the fluvial flood extent for all scenarios modelled. The proposed level changes along the onsite access road would not diminish the fluvial floodplain, and therefore no floodplain compensatory storage is required;
- In addition to the minimum soffit level recommended for the proposed single open span bridge, to ensure the bridge does not increase flood risk to the site or third parties the bridge will be designed to ensure that the abutments are positioned outside of the channel to avoid any in channel obstruction;
- Enhancement of the existing drainage ditch to the west of the site will be carried out in order to collect and convey surface water (generated on higher ground to the north-west of the site) to the Barne Brake Creek.

Drainage Strategy

- As outlined in Section 6.3, a drainage strategy has been provided by GHA Livigunn;
- Surface water generated on all hardstanding areas will discharge to the Weston Mill Stream via a combination of gullies, linear drains or channels and hard pipe;
- The invert level of the outfall pipe at the point of discharge will be set such that it is above the maximum tidal water level for a 1 in 200 years return period, including an allowance for climate change and freeboard to ensure tide locking does not occur;
- An emergency cut-off valve immediately upstream of the outfall will be provided to prevent any water discharging to the environment in the event of an accidental spill on site;
- Surface water draining from the roof and walls of the main building will drain to an infiltration basin, whereas the workshop building, due to its size, will be drained to an infiltration trench.
- To ensure the development does not pose a flood risk to itself from sewer flooding the drainage system has been designed to ensure that no surcharge in the system occurs for events up to and including the 1 in 30 year return period (in accordance with Paragraph 5.50 PPS25 Practice Guide[8]).

[8] PPS25 Practice Guide Updated December 2009 accessed online: http://www.communities.gov.uk/documents/planningandbuilding/pdf/pps25guideupdate.pdf

9 References

- British Geological Society 'Solid and Drift Geology Map, Sheet 348, 1:50,000;
- CIRIA Publication C697 (2007) The SUDS Manual;
- Department for Communities and Local Government (2006) Planning Policy Statement 25: Development and Flood Risk (PPS25), The Stationery Office, London;
- Department for Communities and Local Government (2009) Planning Policy Statement 25: Development and Flood Risk Practice Guide, Communities and Local Government Publications, Wetherby;
- Environment Agency (2003) South West Region Report on Regional Extreme Tide Levels;
- Pell Frischmann (2006) Plymouth City Council Strategic Flood Risk Assessment;
- Plymouth City Council (2007) Local Development Framework Core Strategy, Adopted April 2007.

10 Appendices

Appendix A – Environment Agency Flood Map

Appendix B – Environment Agency Correspondence

Appendix C - Site Photos

Plate 1: Table Top Mountain – the proposed location for the construction compound. The Barne Barton residential area on the steeper ground to the west can be seen in the background.

Plate 2: The site, including and to the north of the existing access road. The Barne Barton residential area on the steeper ground to the north west can be seen in the background.

Plate 3: Weston Mill Stream (left of picture), flows into the Weston Mill Lake along the site's eastern boundary.

Plate 4: Weston Mill Lake connects to the Tamar Estuary, beyond the site's southern boundary, via a box culvert.

Appendix D – Site Topographic Survey

Appendix E – Site Layout

Appendix F – Hydrology Briefing Note

Appendix G – ReFH Hydrographs

Appendix H – Map of Hydraulic Model Extent

Appendix I – Modelled Flood Mapping Outputs

Appendix J - Drainage Layout undertaken by GHA Livigunn

Appendix K – Proposed Access Bridge Design