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Staplefield ICW

Flood Risk Assessment

Ferbruary 2024

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Staplefield ICW

Flood Risk Assessment

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Issue and Revision Record

Revision	Date	Originator	Checker	Approver	Description
P01	05/05/23	T Luck	M Piggott	M Rickard	Flood modelling inputs to Flood Risk Assessment first issue
P02	25/01/24	V Vu	M Piggott	J Knightbridge	Flood Risk Assessment first issue
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This report has been prepared for the purposes of the Staplefield ICW Flood Risk Assessment January 2024 only. Climate change allowances have been defined by the Environment Agency (henceforth referred to as the EA) (Flood risk assessments: climate change allowances update 27 May 2022) and are correct at the time of writing this report.

The consultant will follow accepted procedure in providing the services but given the residual risk associated with any prediction and the variability which can be experienced in flood conditions, the consultant takes no liability for and gives no warranty against actual flooding of any property (client's or third party) or the consequences of flooding in relation to the performance of the service.

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1 Introduction

1.1 Purpose of this report

Mott MacDonald has been contracted by Southern Water to support the design of an Integrated Constructed Wetland (ICW) scheme located directly to the east of Staplefield Wastewater Treatment Works (WwTW), Cuckfield Road, Staplefield, West Sussex, RH17 6ES. This report covers the flood risk assessment for all flood risk sources to and from the proposed development. The report is structured as follows:

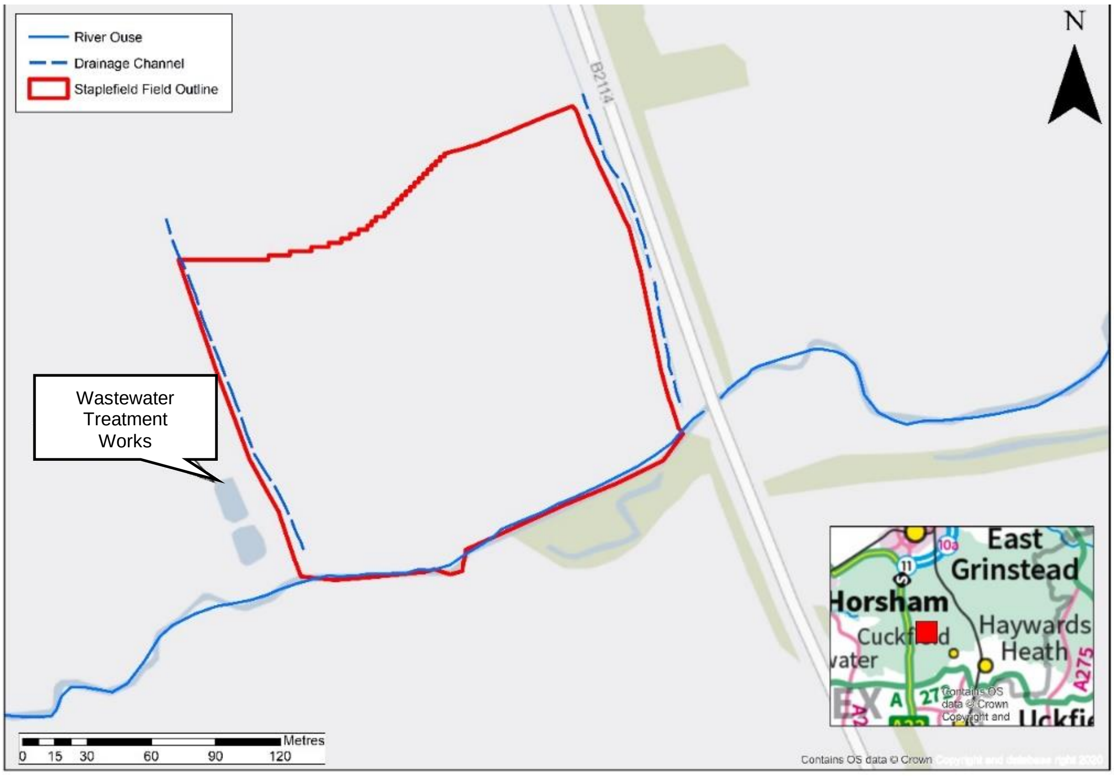
- Flood risk assessment approach taken for this site
  - Supported by site plans, flood estimation analysis, hydraulic model development, topographic survey and river channel survey in the Appendices.
- Assessment of the baseline flood risk from all sources
- Assessment of flood risk changes to and from the development from all sources of flooding and management of residual risk.
- Conclusions on flood risk and recommendations for future investigations in the catchment.

1.2 Site description

Staplefield WwTW is located 500m south of the village of Staplefield, West Sussex, at National Grid Reference 528093E, 127401N. The proposed location for ICW is directly to the east of the existing WTW and 1km south of Staplefield village on 3 hectares of arable farmland (Figure 1.1). The River Ouse flows from west to east along the south of the site. There are two drainage channels on the eastern and western perimeters of the site that both flow southward to the River Ouse. The western drain drains a small area of the local field and upslope area. The eastern drains the highway area and part of Staplefield village upstream.

The site is predominantly agricultural land bounded by a mixture of hedges and trees. The site can be accessed from Cuckfield Road, located on the eastern boundary of the site. The access track which runs around the southern perimeter of the field to the WTW is unpaved and narrow in places. The site can also be accessed via a small road bridge over the River Ouse on the B1124 directly south of the entrance to the WTW.

Figure 1.1: Staplefield site area



1.3 Proposed Integrated Constructed Wetland design

The main works at the site include the development of a 1.3ha ICW to achieve phosphorus permits as proposed by the Environment Agency.

The wetland site will be accessed via the existing access track to the WTW on the northern bank of the River Ouse, a new entrance track may be required to connect from the existing field access at the west of the wetland site to the WTW and surrounding fields.

A flood mitigation feature is proposed directly to the north of the River Ouse, allowing outfall of water directly to the water course and drainage within 10.5 hours before the next storm. This drain down time was assumed to align with the critical storm for the site which goes beyond the CIRIA SuDS guidance for attenuation ponds.

Numerous bunds are proposed to be constructed, separating the four individual wetland cells, as well as a larger bund between the wetland cells and the flood mitigation feature. It is proposed that the bunds will be lined; this will either be by the Weald Clay Formation, or by an imported fill material, to be determined based on the information obtained from the ground investigation.

The proposed general arrangement of the ICW is presented in Appendix A.

2 Flood risk assessment approach

2.1.1 Approach

A full site-specific flood risk assessment approach has been taken since the proposed Staplefield ICW lies partly within Flood Zone 3.Existing flood data for the site relied on national mapping, and there was no detailed river modelling for the upper Ouse catchment. Therefore, flood flows were estimated using the latest Flood Estimation Guidance (Appendix B), and a 1D-2D FMP-TUFLOW model was developed for the River Ouse affected by the proposed Staplefield ICW (Appendix C). The results from this analysis have been used to update the assessment of existing fluvial flood risk and understand the flood risk impacts of the design in Sections 3.2 and 4.1 of this report.

The hydraulic mdoel developed for this FRA has been run for flood events with a 50%, 10%, 3.3% , 2%, 1%, 1%+ climate change, and 0.1% annual exceedance probability (%AEP). This refers to the probability of a flood event occurring in any year. The probability is expressed as a percentage. For example, a large flood which is calculated to have a 1%chance of occurring in any one year, is described as 1% AEP.A volume for volume approach was taken to any required floodplain compensation given the limited space on site and presence of the B2114 bridge that strongly dictates water level upstream and flows passed on downstream. The results of this mitigation design on third parties can be found in Section 4.2 of this report. A foul and surface water drainage plan has been produced to assess any surface water outside the Flood Zone and management of this to greenfield runoff rates post-development (See Section 4.3 of this report).

An operations and maintenance strategy has been developed to manage residual flood risk and maintain full capacity of the flood mitigation features in Section 4.4.

2.1.2 Flood vulnerability classification and the Sequential Approach

In agreement with the Environment Agency, the ICW was deemed to be "water compatible" land use according to Annex 3 Flood Vulnerability Classification[1] under water and sewage transmission infrastructure and pumping stations.

Table 3: Flood risk vulnerability and flood zone 'compatibility' [2] sets out that no sequential test is required for water compatible development.

The sequential approach has been applied to locate the wetland cells in the areas of lowest flood risk and provide volume for volume flood mitigation in the areas of greatest flow risk in accordance with the guidance for water compatible as follows:

- Stay safe and operational during a flood
- Avoid blocking water flows or increasing flood risk elsewhere
- Avoid loss of floodplain storage (i.e. loss of land where flood waters used to collect)

[1] DEFRA (2012) Annex 3: Flood risk vulnerability classification to the National Policy Planning Framework. Accessed via https://www.gov.uk/guidance/national-planning-policy-framework/annex-3-flood-risk-vulnerability-classification

[2] DEFRA (2012) Technical Guidance to the National Planning Policy Framework. Accessed via https://assets.publishing.service.gov.uk/media/5a79a6a6e5274a684690b1b3/2115548.pdf

2.1.3 Climate change allowance

The central value of climate change was used for the 1% AEP plus climate change scenario. This equates to an allowance of 37% for the Adur and Ouse Management Catchment, in which the River Ouse is located[3]. The central value for the 2080s was used as the wetland development is deemed to be water compatible and has a design life of over 50 years[4].

[3] Climate change allowances for peak river flow in England (data.gov.uk)

[4] National Planning Policy Framework - Annex 3: Flood risk vulnerability classification - Guidance - GOV.UK (www.gov.uk)

3 Existing flood risk

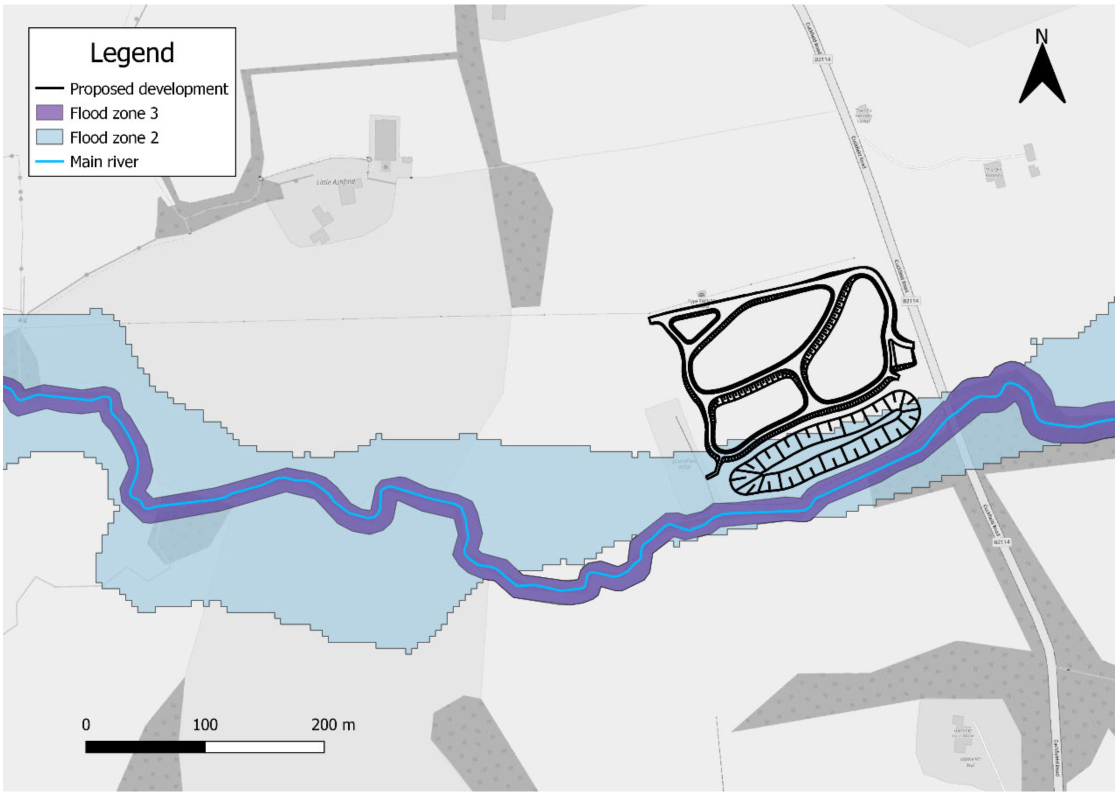
3.1 Flood Zones

The existing site lies partly within Flood Zone 2 and 3 (After discussions with the Environment Agency, it was discovered that the Flood Zone mapping is based on national JFLOW modelling, which did not account for the impact of the B2114 bridge, the latest hydrology, or detailed channel topography. As a result, a more detailed fluvial model was necessary to enhance the understanding of local fluvial flood risk.

Figure 3.1). The southern third of the site lies within Flood Zone 2 near to the River Ouse and the southern boundary lies within Flood Zone 3. The remaining two thirds of the site to the north lies in Flood Zone 1.

After discussions with the Environment Agency, it was discovered that the Flood Zone mapping is based on national JFLOW modelling, which did not account for the impact of the B2114 bridge, the latest hydrology, or detailed channel topography. As a result, a more detailed fluvial model was necessary to enhance the understanding of local fluvial flood risk.

Figure 3.1: Flood Zone Map for Planning at the proposed site of Staplefield ICW



3.2 Updated fluvial flood risk

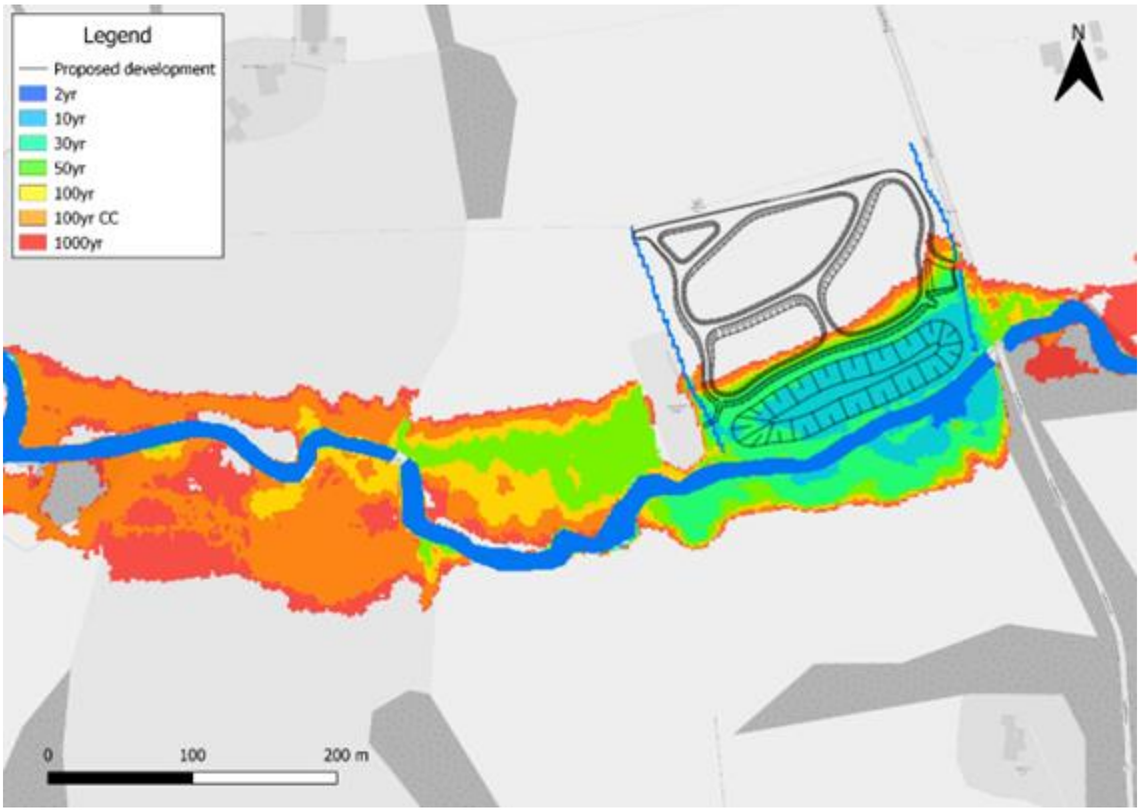
A 1D-2D FMP-TUFLOW detailed fluvial model was developed specifically for this site to update the existing flood risk mapping. See Appendix B and C for more details on how the flood hydrology and modelling were developed. The following sections detail the updated assessment of fluvial flood risk that refine the Flood Zone Map for Planning in Section 3.1.

3.2.1 Flood extents and mechanisms

The flood extents from all simulated baseline events are shown in Figure 3.2. The site first floods during a 10% AEP event (or 1 in 10-year event). During the lower order events, the flooding occurs from the two drainage channels that flow either side of the site as the flow is impeded by the culverts at their downstream extent. During the larger events, the B2114/Cuckfield Road bridge becomes the controlling factor and results in significant out of bank flooding as described in Section 4.2. Flooding first overtops the B2114 road during the 1% AEP event.

Figure 3.2 shows the flood extents from all simulated baseline events. The site first floods during a 10% AEP event (or 1 in 10 year event). In higher probability events, flooding occurs from the two drainage channels that flow on either side of the site, as the flow is impeded by the culverts at their downstream extent. In lower probability, larger events, the B2114/Cuckfield Road bridge becomes the controlling factor and results in significant out-of-bank flooding. The B2114 road first overtops during the 1% AEP event, adding to backwater along the eastern perimeter drain.

Figure 3.2: Summary of the baseline flood extents for the modelled scenarios

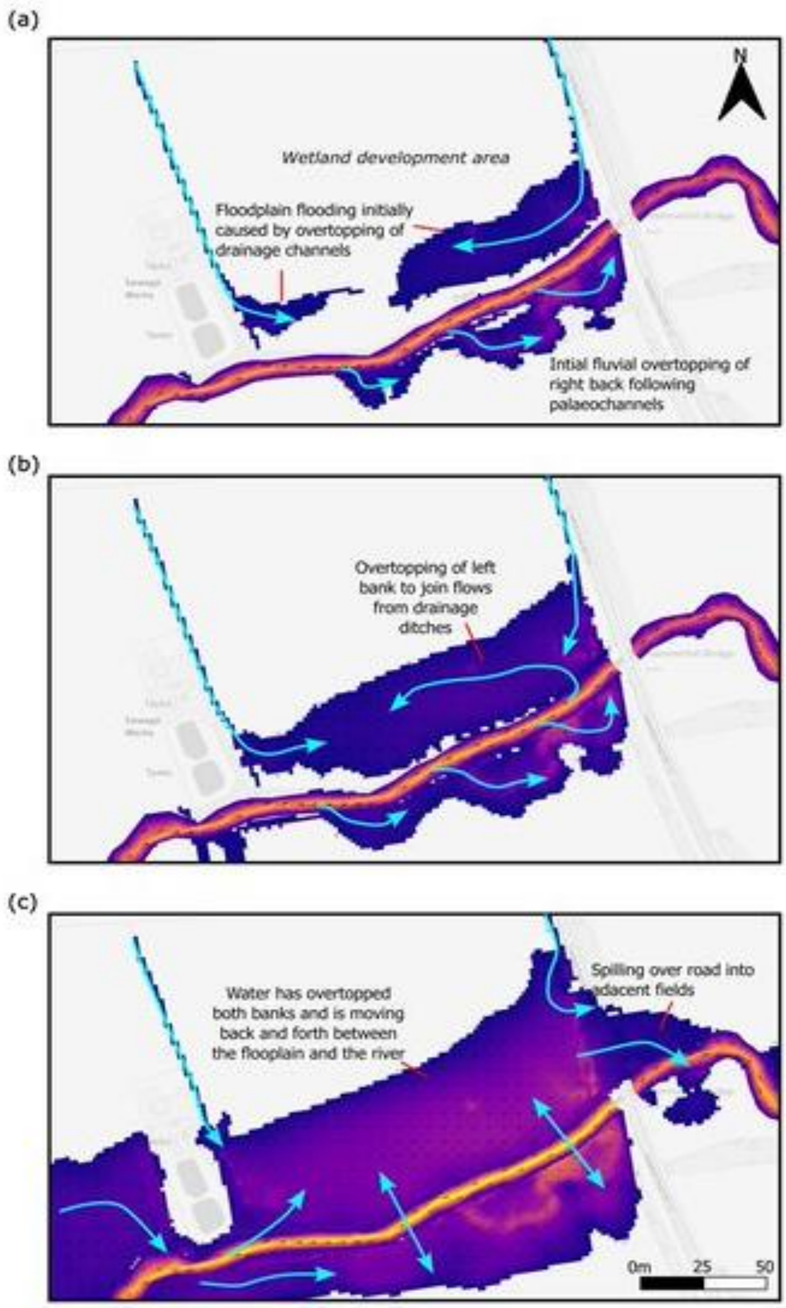


In lower probability, large magnitude events, the site floods as follows (see Figure 3.3):

- Overland runoff is directed along drains adjacent to the sewage treatment works and Cuckfield Road. These tributaries react quickly and cause initial local flooding to the area south of the proposed development. The flooding is caused by the flow constriction of the two culverts at the downstream extents of the drainage channels. The culverts are too small to deal with the flow and become surcharged.
- River flow is impeded by bridge structure at Cuckfield Road. The right bank is initially overtopped early in the event. Flow paths follow a depression likely caused by a former paleochannel of the River Ouse (Figure 4.1a).

- The left bank is overtopped approximately 4 hours before the peak of the flood, connecting with the overland flow paths and surface water flooding, and causing more extensive flooding of the left bank (Figure 4.1b).
- As the flood progresses towards its peak (at 10.8 hours), extensive overtopping of both banks occurs, and water moves freely between the floodplains on either side of the channel. Flooding occurs within southern boundaries of the proposed wetland site.
- At the flood peak, there is potential for overtopping of the road, and spilling on to areas east of the bridge.
- Flooding also occurs from the field to the west via flow through the access track to the south of the sewage treatment works. The treatment works itself is protected by raised ground embankments that are approximately 1m higher than the surrounding ground level.

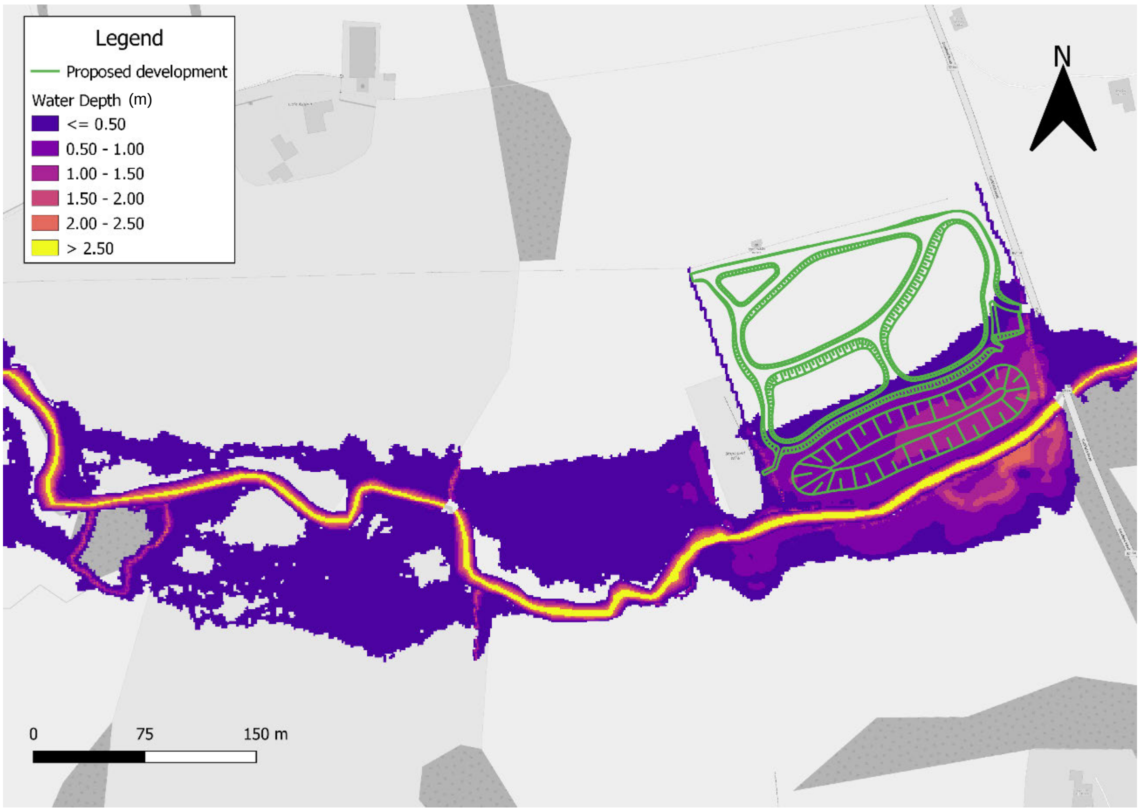
Figure 3.3: Schematic of flood mechanisms during the 100yr+cc baseline scenario



3.2.2 Flood depths

The B2114 bridge acts as a barrier to flow due to its constriction and raised elevation, containing flood water within the field for the proposed development (Figure 3.4). This leads to high flood depths of over 1m accumulating within the field before it can spill over the road and re-join the river downstream. The outline of the proposed wetland is shown in green and indicates that approximately 1240m^3 of the 1%AEP + climate change flood event could be obstructed by the ICW embankments.

Figure 3.4: Modelled water depths in the 1% AEP plus climate change baseline scenario



3.2.3 Flood velocity

Flood velocities are generally low (< 0.5m/s) (Figure 3.5). Water ponding behind the road in extreme floods causes this. Higher velocities up to 1m/s are found over the access track where the eastern drainage channel joins the Ouse. The highest velocities occur over the B2114 road and on the field on the eastern side of the road, where overtopping is occurring, resulting in higher velocities.

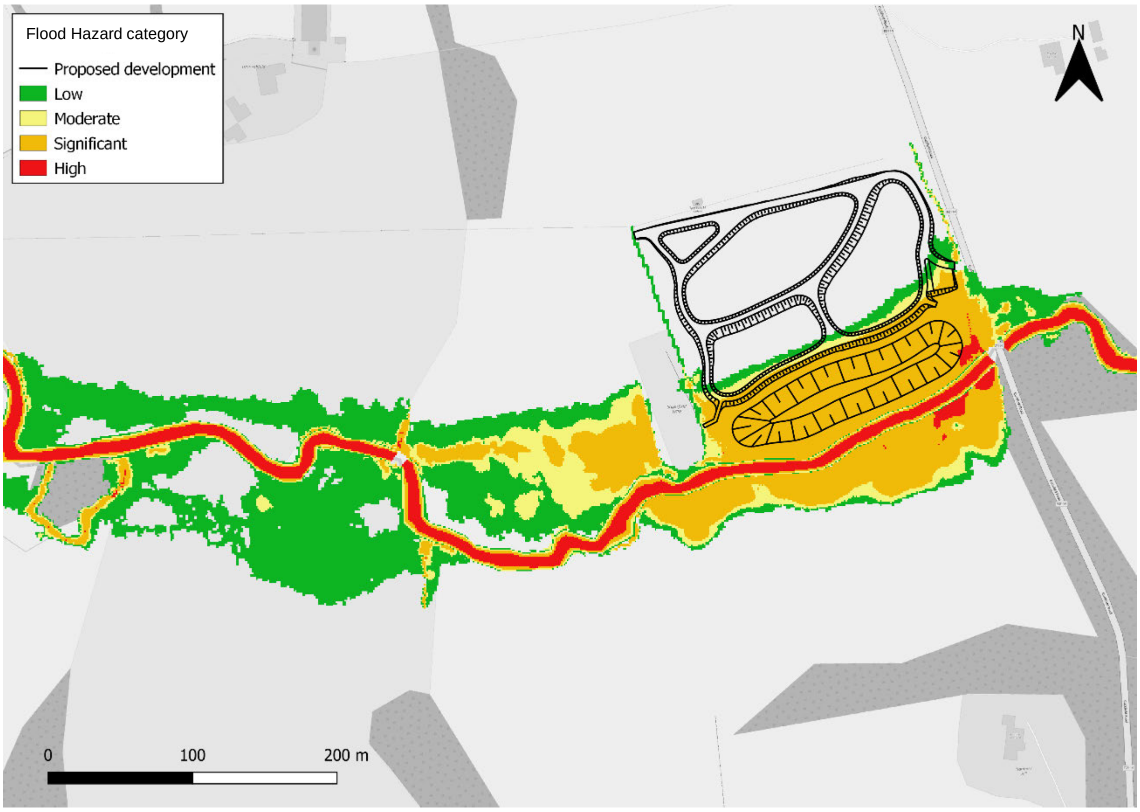
Figure 3.5: Modelled water velocities in the 1% AEP plus climate change event baseline scenario

3.2.4 Flood hazard

The flood hazard mapping[5] for the 1% AEP plus climate change event is shown in Figure 3.6. Most of the site has a "Significant" hazard, meaning "Danger to Most" people due to high depths that build up in the field. However, there is one smaller area on the access track on the left bank of the Ouse that is ranked as "Extreme", meaning "Danger to All". This is caused by higher velocities and flood depths at this location.

[5] Microsoft Word - Sub-Guidance of Safe Access and Exit (publishing.service.gov.uk)

Figure 3.6: Flood hazard mapping for the 1% AEP plus climate change baseline scenario



3.3 Surface Water

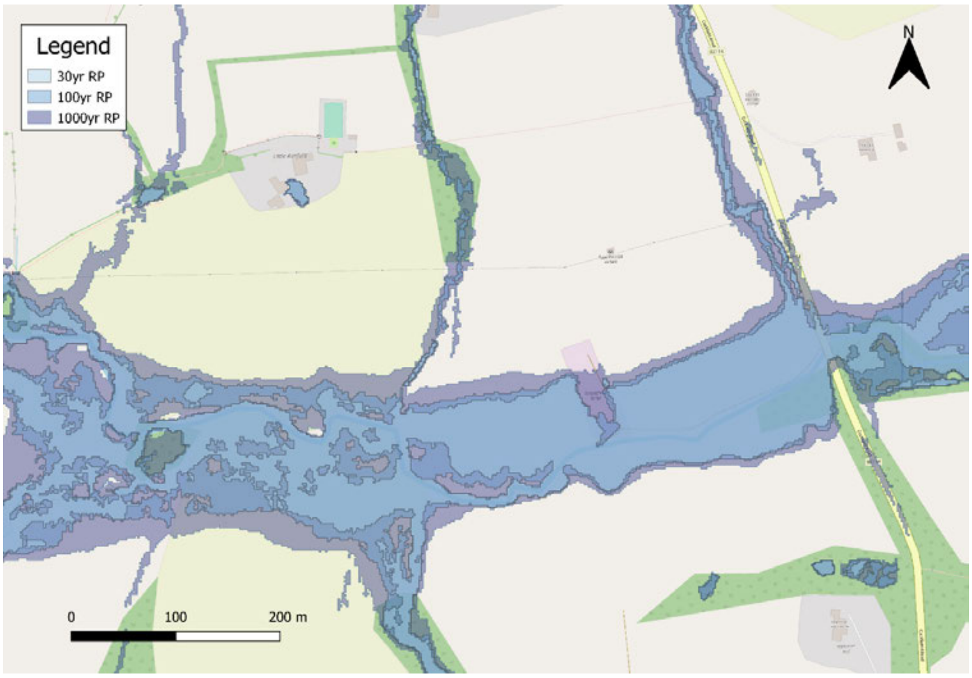
Surface water runoff from the field is currently managed by a series of agricultural field drainage pipes. Some of which were reported by the farmer to be in poor condition leading to waterlogging of soils and collection of surface water in the areas by the access track during a site visit on 1 February 2023 (Figure 3.7).

The National Risk for Flooding from Surface Water shows that the site could be at risk from surface water in the south and east of the site (Figure 3.8). It should be noted this national flood mapping assesses the surface water runoff flow paths before the water reaches the Main River. Much of this flooded area matches with the fluvial flooding from the B2114 drainage channel and local field drain to the west in the updated detailed hydraulic model in Section 3.2 Henceforth, flooding from the eastern and western perimeter drains will be considered together with Main River flooding in the detailed hydraulic model and the report.

Section 4.3 and Appendix D detail the management of surface water runoff from the remaining surface water runoff from the remaining areas on the site before it reaches the field drainage and B2114 drainage channels.

Figure 3.7: View from Staplefield WTW looking east across the site 1 February 2023

Figure 3.8: Risk of flooding from surface water



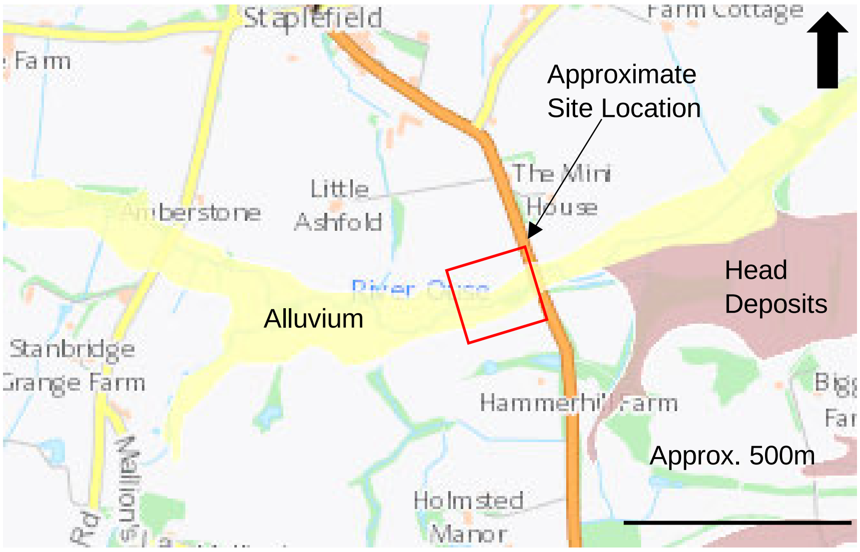
Where (RP=Return Period)

3.4 Groundwater

The hydrogeology of the site is comprised of:

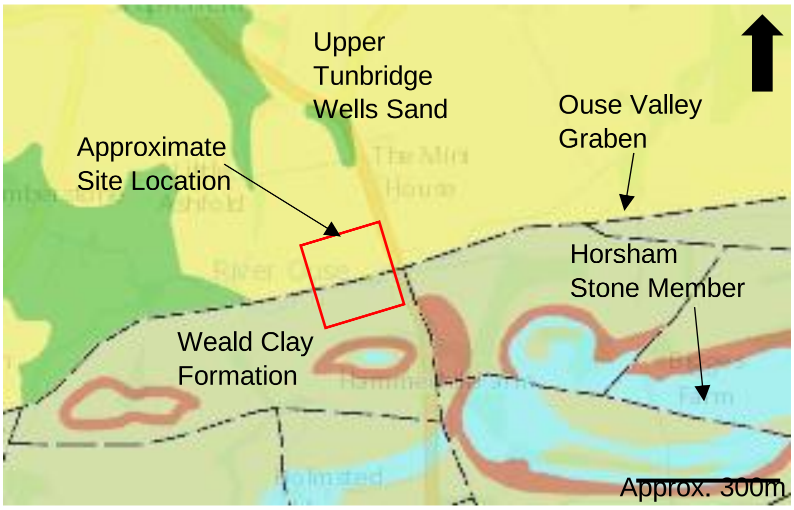
- Superficial deposits of Alluvium are classified as a Secondary Aquifer, a material capable of supporting water supplies at a local rather than strategic scale (Figure 3.9)
- Bedrock of the Upper Tunbridge Wells Sand is also classified as a Secondary A aquifer. The Weald Clay Formation is classified as Unproductive Strata (Figure 3.10).

Figure 3.9: Superficial Deposit distribution



Source: BGS GeoIndex Onshore (March 2023)

Figure 3.10: Bedrock Geology



Source: BGS GeoIndex Onshore (June 2023)

Table 3.1 summarises the groundwater levels obtained during the ground investigation monitoring visits completed on site. The levels suggest that the groundwater level across the site varies, with shallower levels generally encountered closer to the River Ouse. A minimum depth of 0m was observed at WS03 to the east of the field. However, this is where the farmer reported the field drainage pipes had collapsed, leading water in the observation area and was deemed to be poor surface water drainage rather than emergent groundwater flooding.

Table 3.1: Summary of Groundwater Monitoring Levels

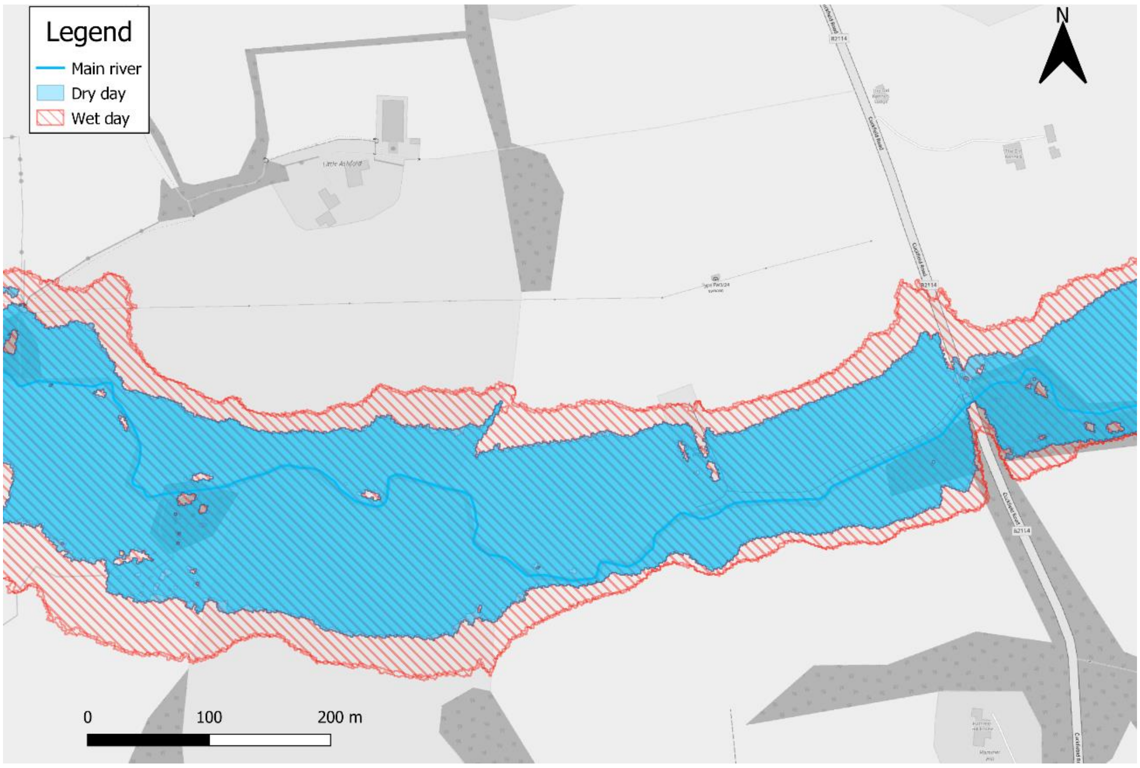
Borehole Location	Strata Installed	Maximum Water Level (mbgl) [mAOD]	Minimum Water Level (mbgl) [mAOD]
WS01	Weald Clay Formation	0.68 [54.72]	0.73 [54.67]
WS02	Alluvium/Weald Clay Formation	0.37 [51.36]	1.44 [50.29]
WS03	Alluvium/Weald Clay Formation	0.00 [51.98]	0.72 [51.26]
WS04	Weald Clay Formation	0.28 [53.71]	0.88 [53.11]

3.5 Artificial

A foul water sewage pipe passes under the north-western corner of the site to connect with the WTW to the west of the site. There are no manholes or records of surcharging of this pipe to cause flooding on the site.

Third party reservoirs are located upstream, presenting a risk of flooding from dam breach in the dry day and wet day scenarios. The flood extent is very similar to the 0.1%AEP fluvial flood. The ICW is only designed operate in and help to manage the water up to a 1%AEP +climate change event but this would also help manage water during a reservoir breach event and have a similar impact to the fluvial flooding which are considered in Section 4 of this report.

Figure 3.11: Risk of flooding from third party reservoirs



4 Post-development flood risk

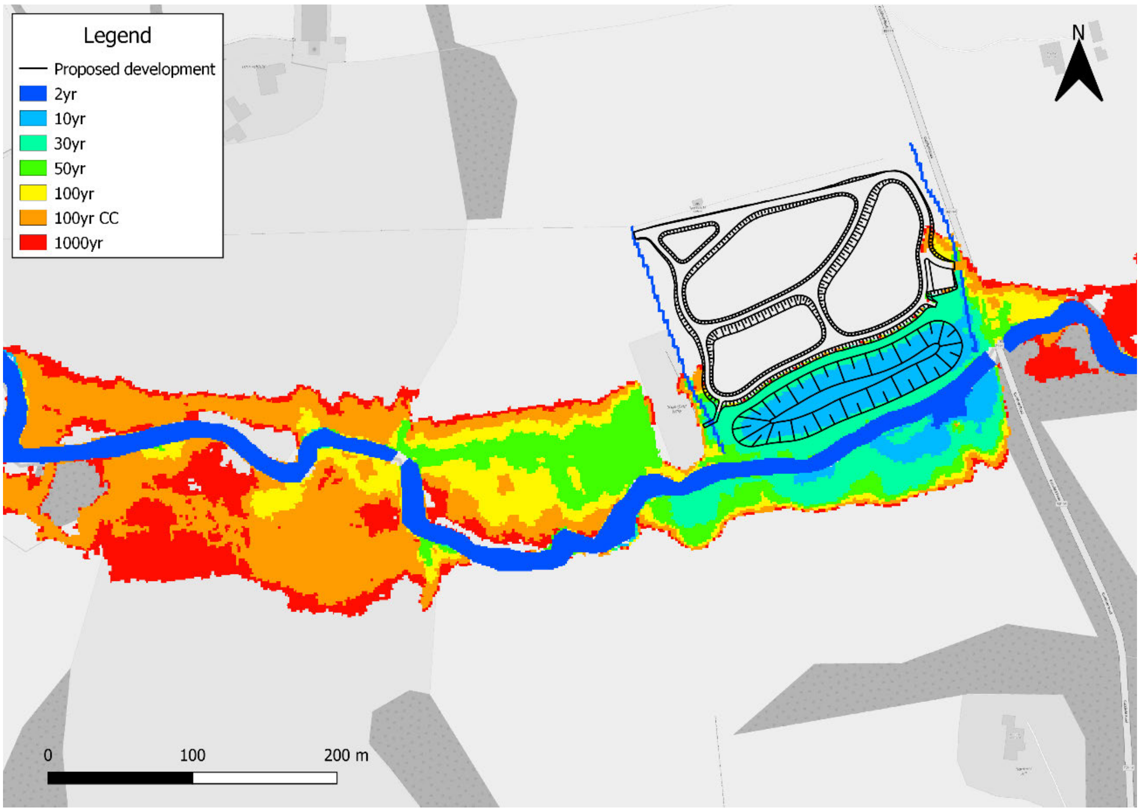
4.1 Fluvial flood risk to the development

The detailed hydraulic model indicated that flood levels varied from 52.4m to 52.6mAOD across the site and depths reached 2m in the flood mitigation area (Figure 4.1). The embankment is raised to a minimum of 53.5mAOD so the wetland and car park are kept safe and operational at all points during the 1%AEP plus climate change allowance flood event. Furthermore, the wetland and access/car park are also raised above flood levels.

Figure 4.1: Post-development flood depths for the 1% AEP plus climate change scenario

A summary of the post-scheme flood extents is shown in Figure 4.2. The wetland embankment is raised high enough so that it blocks out flooding during all events simulated and therefore, is unaffected by river flooding. The lowest elevations of the flood mitigation area first floods from the 10% AEP event via overland flow from the drainage channels. The mitigation area collects and stores flood water and results in the 10% AEP event being spread out over a greater distance in the field. The access track first overtops during the 3.33% AEP event, over the lowered area of the track. With the mitigation in place, there are no significant changes in the flood extent affecting third parties, with the B2114 road controlling flood levels in a similar way to the baseline.

Figure 4.2: Summary of the post-scheme flood extents for the modelled scenarios

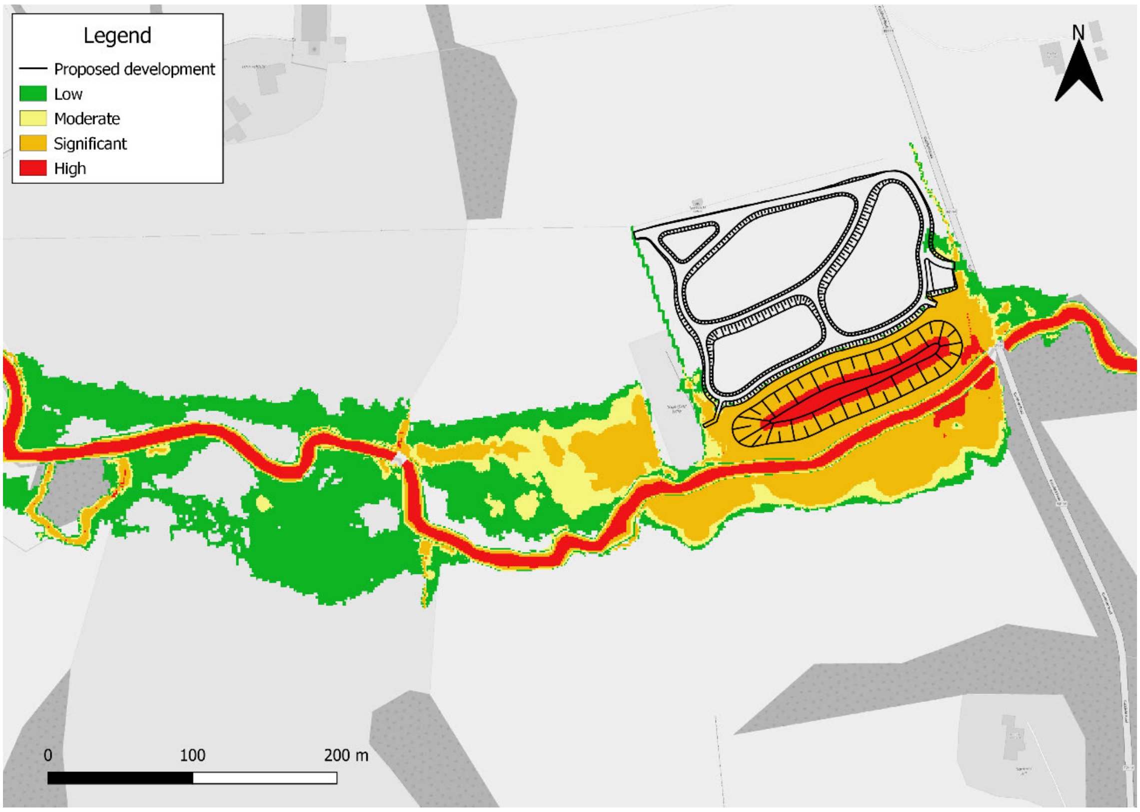


Post scheme-flood velocities are shown in Figure 4.3. Velocities are largely unchanged compared to the baseline as the B road still controls the pass forward flow and hence velocities across the site.

Figure 4.3: Post-development flood velocities for the 1% AEP plus climate change scenario

Flood hazard mapping for the post-scheme scenario is shown in Figure 4.4. An area of "High" hazard is shown at the flood mitigation area due to the higher depths. Compared to the baseline (Figure 3.6), the post-scheme additions are shown not to significantly increase flood hazard to third parties. The very high flood depths produced by the mitigation make it "Dangerous to All". Access should be restricted within the Southern Water site as set out in the operation and maintenance plan so that people are not entering the flood mitigation area during flood and appropriate safety access measures and warnings are applied to the site.

Figure 4.4: Post-development Flood hazard mapping for the 1% AEP plus climate change scenario



4.2 Fluvial flood risk from the development

The Embankment of the ICW was found to obstruct 1240m^3 of the 1%AEP + Climate change allowance flood event after shifting the footprint as far north as possible whilst still allowing for depth limits of the sewage main and access track to the north. This minimised the amount the ICW embankments obstructed the flood flows.

Floodplain mitigation was still provided on a volume for volume basis to compensate for the 1240m^3 lost and located by the River Ouse to fill once the access track overtops as it does currently under the baseline scenario. Pipes then allow the storage area to drain away within 11 hours which is approximately the same as the critical storm duration and encourage the flood mitigation area to empty prior to the next storm.

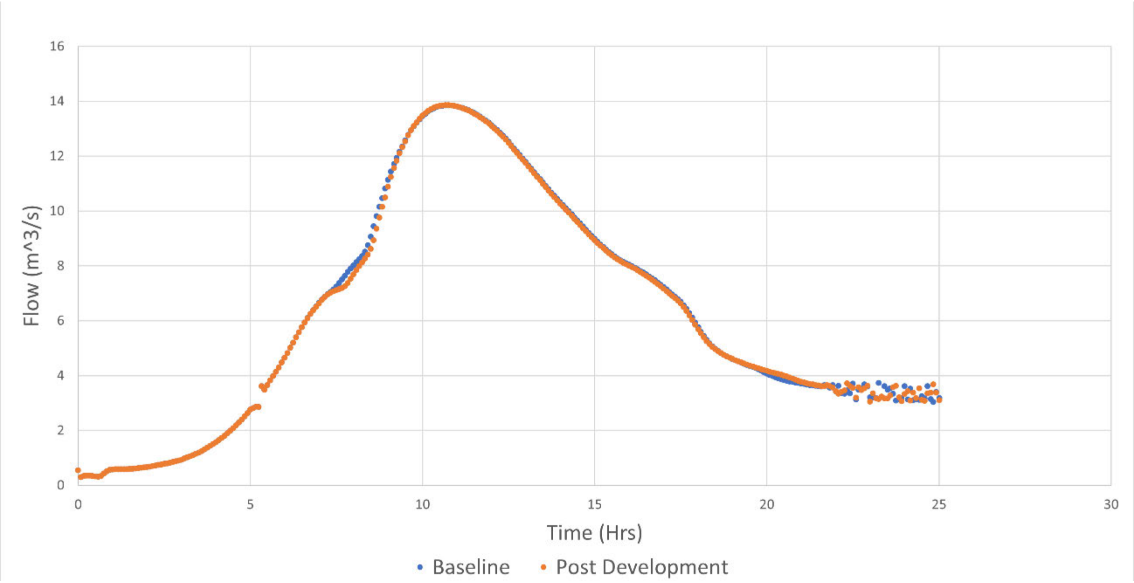
Figure 4.5 compares water levels for the 1% AEP plus climate change event, between baseline and post-scheme scenarios. It indicates no significant change in flood risk third party land upstream or downstream of the site where a 0.05m increase in flood risk on agricultural land (classed as moderately adverse) is classed as significant[6]. The B2114 bridge and road control flooding in this event, effectively turning the site field into a flood storage area. Despite the wetland scheme removing a large floodplain area, flooding will not increase elsewhere as the obstructed volume has been compensated by the flood mitigation area.

[6] 4.01.78-LA-113-revision-1-Road-drainage-and-the-water-environment-web.pdf (a55j16j16a-publicinquiry.co.uk)

Figure 4.5: Water level differences for the 1% AEP plus climate change scenario

Downstream, the flow hydrographs are similar throughout the event as the flood mitigation area design mimics the existing floodplain functionality (Figure 4.6). The flows passed forward are less than 1% different to the baseline and produce a similar flood duration at all levels so there is no change in flood risk downstream of the hydraulic model.

Figure 4.6: Comparison of the 1% AEP plus climate change flood flow hydrograph between baseline and post-development scenarios



4.3 Surface water management

Drawing 23539_4_13 - SURFACE WATER MANAGEMENT in Appendix D sets out the surface water management strategy for the 3.13ha developed within the re line boundary to meet the greenfield runoff rates stated in Table 4.1.

Table 4.1: Greenfield Runoff Rates

Return Period [years]	IoH124	FEH	ReFH2
	l/s	l/s	l/s
1	15.9	21.1	18.5
30	43.1	57.2	44.4
100	59.8	79.3	56.2

Approximately 60% of the runoff (1.8ha) is captured by the wetland cells and discharged at the registered discharge point at the existing permitted rate. A further 18% of the runoff area is captured by the flood mitigation area and grassed areas draining into the flood mitigation area (0.6ha) which can drain at the greenfield runoff rate using the outlet culverts Only 22% of the runoff area on the site will continue to drain to the River Ouse without attenuation based on the 0.5ha of grassed and 0.2ha of gravel to grass outside the wetland embankment. The reduced area and use of grassed or gravel material ensure the runoff rate is below the greenfield runoff rate. See Appendix D for further details.

4.4 Residual risk management

The 52214-UAX-ZZ-ZZ-OM-EN-00001 Operations & Maintenance Plan sets out how the operator of the site, Southern Water, will operate and maintain the flood mitigation area to mitigate residual risk that the storage is unavailable.

Visual checks will be carried out on the flood mitigation area and any maintenance works carried out as required. These checks will include:

- Visual sediment/silt level assessment. Levels are to be recorded and logged for review. Any maintenance works required for sediment/silt removal will be undertaken following review and method statement.
- Log and record water levels in FMA (water level gauge).
- Inspection of the water level gauge to ensure that it is clearly visible for water level monitoring.
- Visual check on vegetation within the FMA for differences in the composition or cover of the plants should be noted and recorded. Any significant/rapid changes in the colour of the vegetation or die off should be monitored. Any increased establishment of weeds/grass should be noted.
- Check of the outlet pipes to ensure there are no blockages or obstruction hindering flows from the FMA to the river.
- Check of the concrete headwall at the outfalls to the river shall be checked for any damage and debris.
- Inspection to note any further observations in the area including presence of wildlife.

5 Conclusions and recommendations

5.1 Summary of flood risk changes and management

The proposed development comprises of the construction of an integrated constructed wetland adjacent by to the Staplefield Wastewater Treatment Works. A new volume for volume compensation area has been integrated into the design to offset any floodplain volume obstructed by the new wetland in the up to the 1% AEP with climate change flood and a volume for volume flood mitigation area.

The key tasks completed as part of this FRA are as follows:

- A 1D-2D FMP-TUFLOW hydraulic model representing the River Ouse and two drainage channels has been created to assess flood risk to a site in Staplefield and provide an update to the baseline flood mapping from the national JFLOW model and Risk of Surface Water mapping products.
- Peak inflows for the model were generated using the lates FEH Statistical Method at the time of assessment in 2023. Tests identified the critical storm duration as 10.5 hours, used for all simulated flood events. The wetland development is deemed to be “water compatible” based on government guidance. Therefore, the central value has been used for the allowance for climate change runs.
- Seven events were simulated for the baseline modelling (AEP): 50%, 10%, 5%, 3.33%, 2%, 1%, 0.1% and 1% plus climate change.
- Sensitivity testing has shown the model to be moderately sensitive in terms of Manning’s “n” roughness and the representation of the B2114 bridge structure, and insensitive to changes in the downstream boundary conditions.
- Baseline modelling indicates the site begins to flood during the 10% AEP event when drainage channel culverts reach capacity. In larger events, the River Ouse overtops its banks and the B2114 bridge becomes the hydraulic control.
- The post-scheme scenario was modelled to represent the wetland, and flood mitigation areas. The same seven events modelled in the baseline were also modelled for the post-scheme scenario.

Table 5.1 summarises the key flood risk impacts up to the 1% AEP with climate change flood.

These flood risk impacts have been classified considering the main receptors of agricultural land and the Wastewater Treatment Works as Less Vulnerable under Table 2: Flood risk vulnerability classification within Flood Risk Assessment guidance (Flood risk and coastal change - GOV.UK (www.gov.uk)).

The classification for change in flood risk is as follows:

- High: >200mm change in water level or a change in flood hazard category
- Moderate: 100-200mm change in flood depth
- Minor: 50-100mm change in flood depth
- Very low or negligible: < 50mm change in flood depth.

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Table 5.1: Summary of Flood Risk Change

Source of Flood Risk	Flood risk to the scheme without mitigation	Action required/Mitigation applied	*Change in flood risk to the ICW within the red line boundary with flood risk mitigation in place*	*Change in flood risk to STW and third parties with mitigation*
Fluvial	High within the 1%AEP + Climate change flood extent	Location of raised embankment around the wetland area to be above the 1%AEP +Climate change flood event and 0.1%AEP flood extent. The floodplain mitigation area provides at least 1240 m^3 to offset the obstruction of floodwaters in the active floodplain.	*Similar flood risk to baseline as B2114 controls flood levels*	*No significant change*
Surface water	Low risk to the ICW Scheme itself.	The drainage strategy for the management of local surface water runoff from the impermeable surfaces ensure the post-development surface water runoff rate is less than greenfield runoff rate as most of the surface water for the site is captured in either the wetland or flood mitigation area.	*No significant change*	*No significant change*
Groundwater	*Low.* Near surface levels attributed to collapsed surface water drain.	Managed through geotechnical design of the flood mitigation area	*No significant change*	*No significant change*
Coastal	Not applicable	Not at risk from coastal flooding.	*Not applicable*	*Not applicable*
Reservoirs and canals	Reservoir breach risk present.	The flood mitigation would operate and help to manage the water during a reservoir breach event, but only to a 1% AEP+CC standard.	*See fluvial assessment*	*See fluvial assessment*

In conclusion, the Staplefield Integrated Wetland development is resilient to flooding and does not change flood risk to third parties for the following reasons:

- The embankment of the wetland area is built high enough so that the wetland is unaffected by flooding during all modelled events.
- With the flood mitigation area in place, the flood model predicts no significant change in flood risk to third-party greater than 0.05m.
- There are no significant third-party increases to flood depths, velocities, or flood hazard rating. Therefore, the IC does not increase flood risk to third parties.
- The deep water and zones of fast flowing water could be a health and safety hazard on site when the mitigation area is filling and requires management through design.
- The flood mitigation area has been designed to drain within 11 hours to drain the majority of the area before the next storm.

A. Site Drawings and Surface Water Management

The relevant scheme drawings for the flood risk assessment have been provided in the accompanying digital data:

23539_4_01 Site Location Map

23539_4_02 Existing Site Layout

23539_4_03 Proposed Site Layout

23539_4_04 Cross-Sections

23539_4_05 Typical Details

23539_4_06 Hydraulic Profile

23539_4_07 Landscape Plan

B. Flood estimation report

Introduction

This report template is a supporting document to the Environment Agency's Flood Estimation Guidelines. It provides a record of the hydrological context, the method statement, the calculations and decisions made during flood estimation and the results.

Contents

Approval

Revision stage	Analyst / Reviewer name & qualifications	Amendments	Date
Method statement preparation	Kieran Murnane BSc MSc M.CIWEM Level 1		31/01/23
Method statement sign-off	Christopher Rhodes BSc. MSc. C.WEM Level 2		31/01/23

Initial calculations preparation	Kieran Murnane BSc MSc M.CIWEM Level 1		31/01/23
Initial calculations sign-off	Christopher Rhodes BSc. MSc. C.WEM Level 2		31/01/23
Calculations - Revision 1 preparation	Kieran Murnane BSc MSc M.CIWEM Level 1	Updated for 10.5 storm and full range of return periods	04/04/23
Calculations - Revision 1 sign-off	Marianne Piggott BSc. C.WEM Level 2		04/05/23

Abbreviations

AEP	annual exceedance probability
AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CPRE	Council for the Protection of Rural England
FARL	FEH index of flood attenuation due to reservoirs and lakes
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
HOST	Hydrology of Soil Types
NRFA	National River Flow Archive
OS	Ordnance Survey
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph method
ReFH2	Revitalised Flood Hydrograph 2 method
SAAR	Standard Average Annual Rainfall (mm)
SPR	Standard percentage runoff
SPRHOST	Standard percentage runoff derived using the HOST soil classification
Tp(0)	Time to peak of the instantaneous unit hydrograph
URBAN	Flood Studies Report index of fractional urban extent
URBEXT1990	FEH index of fractional urban extent
URBEXT2000	Revised index of urban extent, measured differently from URBEXT1990
WINFAP-FEH	Windows Frequency Analysis Package – used for FEH statistical method

1. Summary of assessment

B.1 Summary

This table provides a summary of the key information contained within the detailed assessment in the following sections. The aim of the table is to enable quick and easy identification of the type of assessment undertaken. This should assist in identifying an appropriate reviewer and the ability to compare different studies more easily.

Catchment location	Staplefield, West Sussex, England. The site lies approximately 20km north of Brighton. The catchment area to the furthest downstream point of interest is 14.66 km^2.
Purpose of study and scope	To generate design flows for 3 inflow locations and a range return periods for use in a hydraulic model to assess risk of flooding to proposed constructed wetland at Staplefield WTW. Flood estimation for this study is considered "Routine" difficulty.
Key catchment features	Sandstone and siltstone geology – low to moderate permeability. Agricultural catchment with some forestry and a small area of discontinuous urban cover. Catchment contains several mill ponds which are expected to have minimal attenuation capability in a high magnitude storm event.
Flooding mechanisms	Fluvial flooding is likely to be dominant due to proximity to the river and lack of urban cover or significant aquifers for surface water and groundwater flooding mechanisms respectively.
Gauged / ungauged	The catchment is generally ungauged, with the next downstream flow gauge gauging a catchment almost 2.5 times the size of our downstream point catchment. There are numerous potentially suitable flow gauges nearby which can be used as donor sites.
Final choice of method	Hybrid method. FEH for events up to 0.1% AEP, ReFH2 for 0.1% AEP.
Key limitations / uncertainties in results	No calibration to observed results on watercourse of interest

Note on flood frequencies

The frequency of a flood can be quoted in terms of a return period, which is defined as the average time between years with at least one larger flood, or as an annual exceedance probability (AEP), which is the inverse of the return period.

Return periods are output by the Flood Estimation Handbook (FEH) software and can be expressed more succinctly than AEP. However, AEP can be helpful when presenting results to members of the public who may associate the concept of return period with a regular occurrence rather than an average recurrence interval. Results tables in this document contain both return period and AEP titles; both rows can be retained or the relevant row can be retained and the other removed, depending on the requirement of the study.

The table below is provided to enable quick conversion between return periods and annual exceedance probabilities.

Annual exceedance probability (AEP) and related return period reference table

AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1
AEP	0.5	0.2	0.1	0.05	0.033	0.02	0.0133	0.01	0.005	0.001
Return period (yrs)	2	5	10	20	30	50	75	100	200	1,000

B.2 Method statement

B.2.1 Requirements for flood estimates

Overview	The purpose of the study is to produce design flood flows for input into a 1D-2D linked FMP-TUFLOW model to assess the flood risk to a proposed constructed wetland at Staplefield Wastewater Treatment Works (WTW). Design hydrographs are required for eight AEP (%) events: 50%, 10%, 5%, 3.33%, 2% 1%, 0.1% and 1%+Climate Change (CC) AEP. The climate change allowance applied is based using the latest river]fluvial uplifts[7]. The constructed wetland is considered a "water compatible" development and is partially located in Flood Zone 2 and hence the central estimate for the Adur and Ouse Management Catchment to the 2080s epoch is applied. An uplift of 37% is applied directly to the 1% AEP event. Flow is estimated at three locations (River Ouse at Cuckfield Road bridge, Cuckfield Road drainage ditch, River Ouse at downstream lane intersection). A hydrograph is produced for each of these three locations for use in the hydraulic model.
Project scope	The study is a routine study for a relatively small, rural, ungauged catchment. The scope of the study is to derive inflows for a 1D-2D linked FMP-TUFLOW model using the ReFH2.3 and FEH statistical methods. Note that the latest FEH22 updates to guidance and methods were not available at the time of this study and thus have not been implemented within this work.

[7] Climate change allowances for peak river flow in England (data.gov.uk) (Accessed: 18/01/2023)

	There are no known existing detailed flood studies in the catchment of interest.

B.2.2 The catchment

Description	The catchment, with a total area of 14.7km^2, is shown in Figure 2.1 and is located approximately 20km north of Brighton. It has relatively low relief with a maximum elevation of 164mAOD and a minimum elevation of 50mAOD[8]. The area has a temperate climate, typical of the south of England (SAAR 1961-1990 of ~831mm). The soils are defined as being loamy and clayey with some impeded drainage[9]. The catchment is mostly underlain by moderately permeable bedrock, with some impermeable bedrock[10]. There are no major superficial geological deposits. The land use is mostly agricultural/grassland with some woodland and a small area of discontinuous urban cover[11]. The catchment contains several mill ponds, the operational rules of which, and thus operation and the attenuation impact during flood events cannot be quantified. It is assumed the average attenuation impact is represented by the FARL value assuming no active operation during flood.

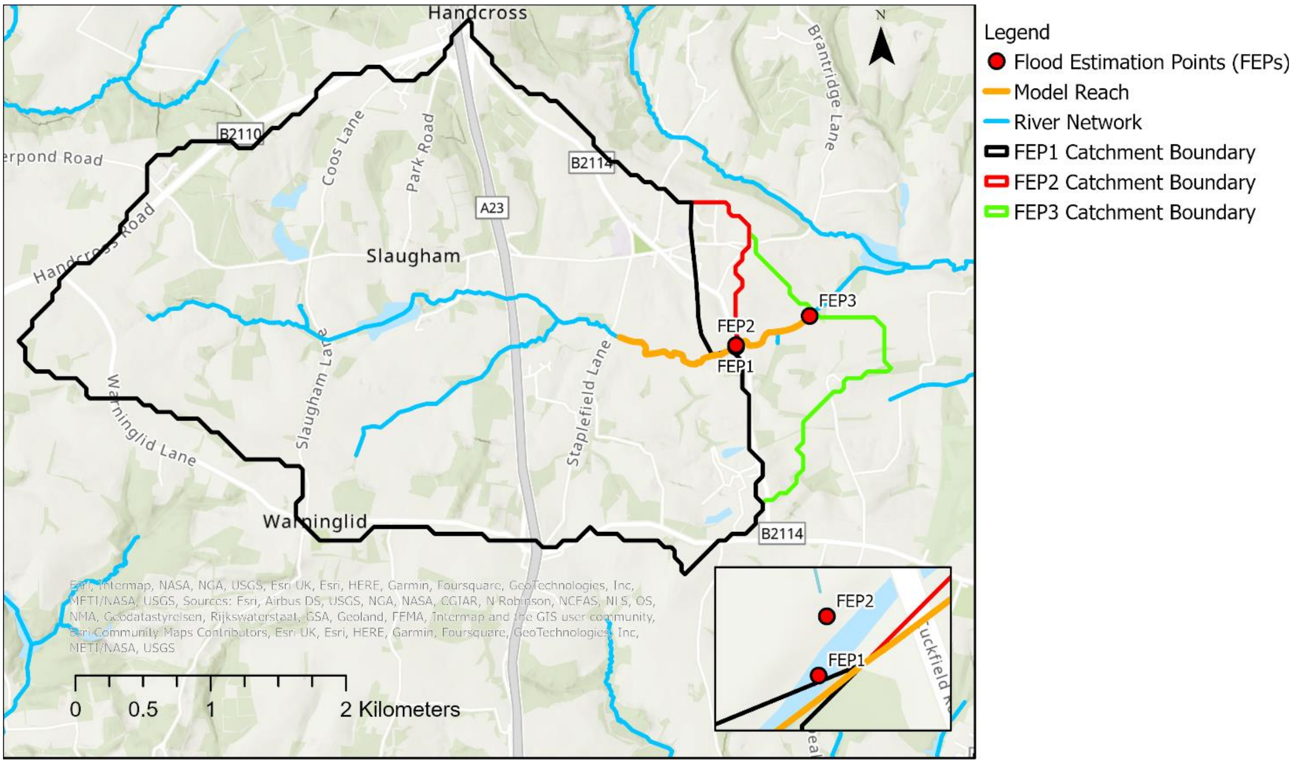
[8] LIDAR Composite DTM 2020 - 2m - data.gov.uk (Accessed: 18/01/2023)

[9] https://www.landis.org.uk/soilscapes/ (Accessed: 18/01/2023)

[10] BGS 625k Geology (Accessed: 18/01/2023)

[11] NRFA Station Mean Flow Data for 41030 - Ouse at Ardingly (ceh.ac.uk) (Accessed: 18/01/2023)

Overview of the study area



B.2.3 Source of flood peak data

Source	NRFA peak flows dataset, Version 11, released 07/09/2022. This contains data up to water year 2020-21.

B.2.4 Gauging stations (flow or level)

There are no flow gauges in the study area,

B.2.5 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available?	Source of data	Details
Check flow gaugings	No	No	N/A	There are no known flow gauges in the study catchment.
Historical flood data	Yes	Yes	Sussex Express (2013)[12]	Anecdotal evidence of historic surface water flooding issues in Staplefield village. No data available for fluvial flooding within the study reach.

[12] Sussex Express (2013) 'Staplefield given county help to beat flooding'. Available at: Staplefield given county help to beat flooding | SussexWorld (sussexexpress.co.uk) (Accessed: 13/02/2023).

Flow or river level data for events	No	No	N/A	There are no known flow gauges in the study catchment.
Rainfall data for events	No	No	N/A	As there was no flow gauge data available, event rainfall data was not required.
Potential evaporation data	No	No	N/A	As there was no flow gauge data available, potential evaporation data was not required.
Results from previous studies	No	No	N/A	No known data available.
Other data or information	Yes	Yes	Site visit	A site walkover was undertaken on the 27th January 2023 to aid catchment conceptualisation for the hydrological and hydraulic modelling. Evidence of recent flooding was observed through the presence of floodplain debris (likely deposited during a flood event) which was corroborated through anecdotal evidence from the site landowner. The site visit also allowed us to accurately derive the catchment for FEP 2, using surface drainage features (culverts, ditches) to accurately identify the catchment area.

B.2.6 Initial choice of approach

Is FEH appropriate?	Yes – FEH appropriate. Catchment is not heavily urbanised or sufficiently complex.
Initial choice of method(s) and reasons How will hydrograph shapes be derived if needed? Will the catchment be split into sub-catchments? If so, how?	As the catchment is ungauged and is relatively uncomplex it was determined that a combination of the FEH statistical method and ReFH2 rainfall-runoff method was appropriate. A semi-distributed (lumped) approach has been taken with the catchment being split into three sub-catchments with corresponding flow estimation points (FEPs). The selection of these FEPs was done to ensure all flow to the site would be accounted for. FEP1 was selected along the River Ouse downstream of the site location, whilst FEP2 was selected at the downstream extent of the B211 channel, to ensure that flooding to the site from this channel would be accurately represented. FEP3 was

	placed on the downstream extent of the model and will be used as a check, to ensure modelled flow matches what has been calculated. QMED at the three FEPs was estimated using catchment descriptors and was adjusted using an appropriate donor site. A pooling group of appropriate gauging stations was used to construct a growth curve to enable return period flows to be estimated. The list of pooled sites was initially generated in WINFAP, with sites being added/rejected based on the homogeneity of the pooling group and the similarity of catchment descriptors with the study site. The ReFH2 method was then used to generate an alternate set of peak flow values which were compared with those generated by the FEH statistical method. Hydrograph shape and volumes were derived using ReFH2.3.
Software to be used (with version numbers)	FEH Web Service / WINFAP 5 / ReFH2.3

B.3 Locations where flood estimates required

B.3.1 Summary of subject sites

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km2)	Revised AREA if altered (km2)
FEP1	Lumped catchment	River Ouse	River Ouse immediately upstream of Cuckfield Road	528177	127408	13.64	13.28

Site code	Type of estimate L: lumped catchment S: Sub-catchment	Watercourse	Name or description of site	Easting	Northing	AREA on FEH CD-ROM (km2)	Revised AREA if altered (km2)
FEP2	Subcatchment	Cuckfield Road drainage ditch	Drainage ditch immediately upstream of confluence with River Ouse	528179	127418	-	0.35
FEP3	Lumped catchment	River Ouse	River Ouse at lane downstream of Cuckfield Road	528721	127634	14.66	-

B.3.2 Important catchment descriptors at each subject site (incorporating any changes made)

Green text refers to catchment descriptors transferred from FEP1. Red text refers to catchment descriptors that have been manually derived for FEP2. More detail can be found in Section B.3.3.

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 1990 Delete if not required	URBEXT 2000	FPEXT
FEP1	0.93	0.36	0.55	3.3	54.6	831	0.012	0.013	0.039
FEP2	1	0.36	0.55	0.9	20.0	831	0.012	0.013	0.039
FEP3	0.93	0.36	0.54	3.8	53.6	831	0.011	0.012	0.045

B.3.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes	Catchment boundaries derived from the FEH web service were checked using available 2m LiDAR data[13]. Where LiDAR data was not available, coarser resolution elevation data from an online topographic viewer was used[14]. Catchment areas were checked using a geometric calculation in ArcGIS Pro which determined that no change to FEH AREA catchment descriptor was required for FEP 3. The FEP 1 catchment boundary was changed slightly following the outcomes of a site visit where identified surface drainage features (ditches, culverts etc) indicated a catchment area that was slightly smaller than that derived from the FEH web service.

[13] LIDAR Composite DTM 2020 - 2m - data.gov.uk (Accessed: 18/01/2023)

[14] England topographic map, elevation, terrain (topographic-map.com) (Accessed: 18/01/2023)

Record how other catchment descriptors were checked and describe any changes.	Catchment descriptors such as PROPWET, DPSBAR, BFIHOST, FARL and SAAR were sense-checked against data readily available for a nearby NRFA station downstream of our study area[15]. The catchment descriptors were suitably similar and thus were carried forward. URBEXT was checked against the readily available land cover datasets LCM2000 (on NRFA) and CORINE[16] which indicated that the URBEXT values were appropriate. The catchment outline of FEP2 was estimated using topographic data and as well as information from a site visit, with the catchment descriptors being derived using a combination of data transfer from FEP1 and manual derivation for certain parameters (AREA, DPLBAR, DPSBAR, FARL) using GIS. Urban extent was calculated in GIS using buildings data in the latest Ordnance Survey OpenMap – Local GIS layer[17] and, as the percentage urban cover was comparable with that of the catchment for FEP1, URBEXT for FEP2 was also derived with a data transfer from FEP1. The URBEXT for all FEP catchments were updated to 2023 values using the CPRE formulae in accordance with the EA Flood Estimation Guidelines.
Source of URBEXT	URBEXT2000.
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000. These were updated to 2023.

B.4 Application of Statistical method

What is the purpose of applying this method?	Design peak flows have been derived using the FEH statistical method to provide a comparison for the peak flows from the ReFH2.3 method.

[15] NRFA Station Mean Flow Data for 41030 - Ouse at Ardingly (ceh.ac.uk) (Accessed: 18/01/2023)

[16] Corine land cover 2018 for the UK, Isle of Man, Jersey and Guernsey - EIDC (ceh.ac.uk) (Accessed: 20/01/2023)

[17] OS OpenMap - Local | OS Data downloads | OS Data Hub

B.4.1 Overview of estimation of QMED at each subject site

Site code	QMED (rural) from CDs (m3/s)	Final method	Data transfer					Urban adjust-ment factor UAF	Final estimate of QMED (m3/s)
			NRFA numbers for donor sites used (see 4.3)	Distance between centroids dij (km)	Moderated QMED adjustment factor, (A/B)a	If more than one donor			
						Weight	Weighted ave. adjustment		
FEP1	3.18	DT	41020	14.55	1.004			1.014	3.23
FEP2	0.19	DT	41020	14.40	1.004			1.014	0.19
FEP3	3.61	DT	41020	15.09	1.004			1.013	3.67

Are the values of QMED spatially consistent?	Yes
Method used for urban adjustment for subject and donor sites	Kjeldsen (2010)[18] / WINFAP v4[19]

Parameters used for WINFAP v4 urban adjustment if applicable

Impervious fraction for built-up areas, IF	Percentage runoff for impervious surfaces, PR_{imp}	Method for calculating fractional urban cover, URBAN
0.3	70%	From updated URBEXT2000

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer (with urban adjustment); CD – Catchment descriptors alone (with urban adjustment); BCW – Catchment descriptors and bankfull channel width (add details); LF – Low flow statistics (add details).

The QMED adjustment factor A/B for each donor site is moderated using the power term, a, which is a function of the distance between the centroids of the subject catchment and the donor catchment. The final estimate of QMED is $(A/B)^a$ times the initial (rural) estimate from catchment descriptors.

Important note on urban adjustment

The method used to adjust QMED for urbanisation published in Kjeldsen (2010)[18] in which PRUAF is calculated from BFIHOST is not correctly applied in WINFAP-FEH v3.0.003. Significant differences occur only on urban catchments that are highly permeable. This is discussed in Wallingford HydroSolutions (2016)[19].

[18] Kjeldsen, T. R. (2010). Modelling the impact of urbanization on flood frequency relationships in the UK. Hydrol. Res. **41**. 391-405.

[19] Wallingford HydroSolutions (2016). WINFAP 4 Urban adjustment procedures.

B.4.2 Search for donor sites for QMED (if applicable)

Comment on potential donor sites	As the study catchment is ungauged, it was determined that a data transfer was required to increase the confidence in QMED that would otherwise be estimated using catchment descriptors alone. The NRFA website was checked for donors nearest the study site including **41030 (Ouse @ Ardingly)**, **41024 (Shell Brook @ Shell Brook)** and **41026 (Cockhaise Brook @ Holywell)** although none of these had a peak flow record and thus were not suitable for use in QMED estimation. WINFAP 5 was used to identify other close sites. Despite being geographically closest (out of the stations with peak flow data) to the study site (9.3km distance), **41005 (Ouse @ Gold Bridge)** was rejected due to the impact of Ardingly Reservoir on flood flows, as well as the catchment being more than 12 times the size of our study catchment. **41020 (Bevern Stream @ Clappers Bridge)** is the next closest to the study site (16.0km distance). Although this is relatively far from our catchment, the catchment has adequately similar hydrological properties (SAAR, BFIHOST, FARL) and is only two times the size of the catchment. Therefore, it is considered that using this site to adjust the QMED estimation is better than estimating QMED using catchment descriptors alone. The third closest station with peak flow data to our study site is **41014 (Arun @ Pallingham)**. This station is 19.4km distance from our station which was deemed too far and, in combination with the significantly higher catchment area (26 times the size of our study catchment), led to the site being rejected.

B.4.3 Donor sites chosen and QMED adjustment factors

NRFA no.	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjustment ratio (A/B)
41020	AM	No	13.80	13.62	1.013

B.4.4 Derivation of pooling groups

Try to use as few groups as possible, this avoids step changes in flow estimates between flow estimation points for catchment-wide studies. If all catchments being assessed have AREA <25km^2 and similar SAAR, FARL and FPEXT values, normally use one group.

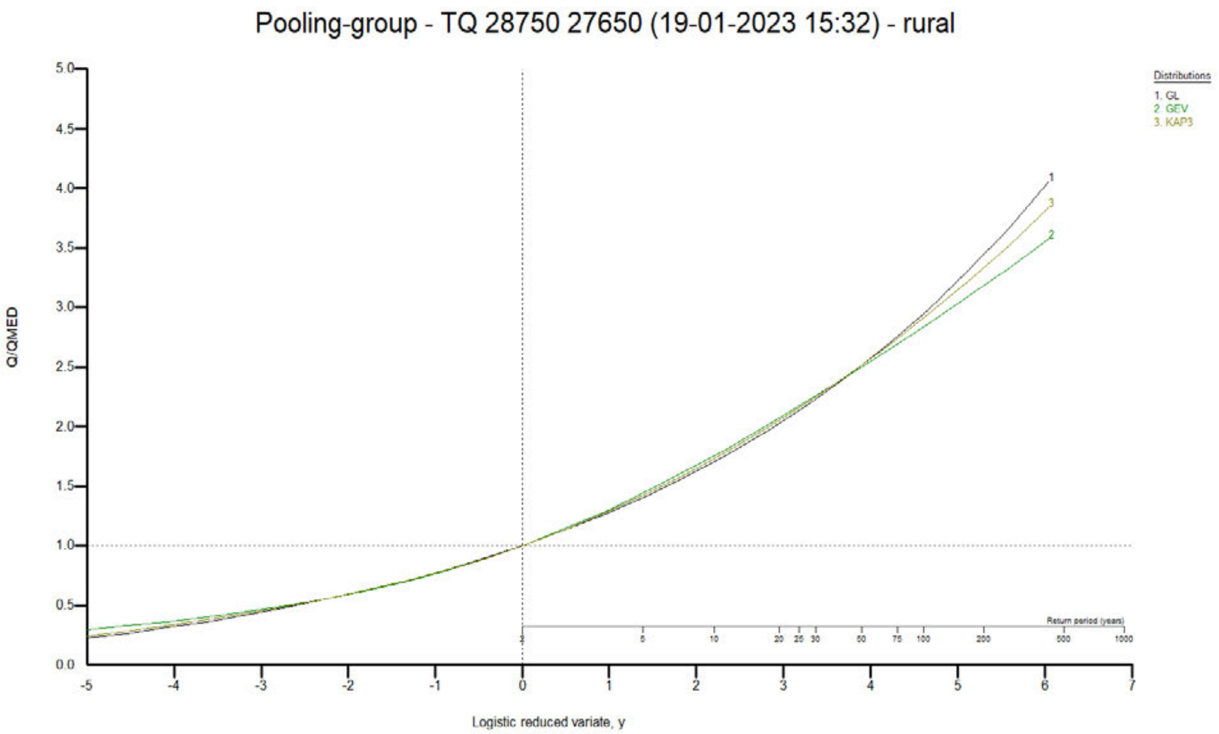
Section 4.3 of the Flood Estimation Guidelines provides further details on reviewing pooling groups.

As the three catchments being assessed had AREA <25km^2 and displayed similar SAAR, FARL and FPEXT values, a single pooling group was used.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged?	Changes made to default pooling group, with reasons	Weighted average L-moments
FEP1, 2, 3	FEP3	No	Numerous stations from the default pooling group were removed due to having BFIHOST19 values that deviated from the study catchment's value by >0.2, including the list provided in B.8.1	L-CV: 0.248 L-SKEW: 0.210
Note: Pooling groups were derived using the procedures from Science Report SC050050 (2008).				

B.4.5 Derivation of flood growth curves at subject sites

Figure 5.1: Pooling group growth curves



Source: WINFAP 5

<table>
<tr><th>Site code</th><th>Method</th><th>If P, ESS or J, name of pooling group</th><th>Distribution used and reason for choice</th><th>Note any urban adjustment or permeable adjustment</th><th>Parameters of distribution</th><th>Growth factor for 100-year return period / 1% AEP</th></tr>
<tr><td>FEP1</td><td rowspan="3">P</td><td rowspan="3">FEP1, 2, 3</td><td rowspan="3">General Extreme Value shown as best fitting distribution.</td><td>Urban adjustment applied using WINFAP V5, UAF = 1.014</td><td rowspan="3">Location: 0.863
Scale: 0.37
Shape: -0.062</td><td rowspan="3">2.831</td></tr>
<tr><td>FEP2</td><td>Urban adjustment applied using WINFAP V5, UAF = 1.014</td></tr>
<tr><td>FEP3</td><td>Urban adjustment applied using WINFAP V5, UAF = 1.013</td></tr>
<tr><td colspan="7">Notes
Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis
Urban adjustments are all carried out using the method of Kjeldsen (2010).
Growth curves were derived using the procedures from Science Report SC050050 (2008).</td></tr>
</table>

B.4.6 Flood estimates from the statistical method

Site code	Flood peak (m3/s) for the following return periods (in years)							
	1 in 2	1 in 10	1 in 20	1 in 30	1 in 50	1 in 100	1 in 1000	
	Flood peak (m3/s) for the following AEP (%) events							
	50%	10%	5%	3.33%	2%	1%	0.1%	
FEP1	3.23	5.68	6.69	7.29	8.07	9.16	15.74	
FEP2	0.19	0.34	0.40	0.43	0.48	0.54	0.93	
FEP3	3.67	6.45	7.59	8.28	9.16	10.39	17.80	

2. Revitalised flood hydrograph 2 (ReFH2) method

B.5 Application of ReFH2 method

What is the purpose of applying this method?	ReFH2.3 has been used to produce a series of peak flow estimates to be compared to those generated using the FEH statistical method, as well as to produce hydrograph shapes. The ReFH2 method was used as part of a hybrid approach to extend the growth curve produced using the FEH statistical method to the 0.1% AEP event.

B.5.1 Parameters for ReFH2 model

Site code	Method	Tp_{rural} (hours)	Tp_{urban} (hours)	Cmax (mm)	PR_{imp} % runoff for impermeable surfaces	BL (hours)	BR (for 1% AEP event)
FEP1	CD	3.4	2.6	360.9	70%	40.0	1.786
FEP2	CD	2.2	1.6	360.9	70%	29.7	1.769
FEP3	CD	3.7	2.8	352.6	70%	40.7	1.692
Brief description of any flood event analysis carried out			No flood event analysis was carried out due to a lack of available local data for the study catchment.				
Methods: OPT: Optimisation, BR: Baseflow recession fitting, CD: Catchment descriptors, DT: Data transfer (give details)							

B.5.2 Design events for ReFH2 method: Sub-catchments and intervening areas

Site code	Season of design event	Storm duration (hours)	Storm area for ARF (if not catchment area)	Reason for selecting storm
FEP1	Winter	10.5	0.96	Outcome of storm duration testing
FEP2	Winter	10.5	0.99	Outcome of storm duration testing
FEP3	Winter	10.5	0.96	Outcome of storm duration testing
Results of storm duration testing.		Storm duration testing was undertaken in the hydraulic model for a range of duration events. This found 10.5 hours to be the critical duration storm event for the entire study catchment.		

B.5.3 Flood estimates from the ReFH2 method

Site code	Flood peak (m3/s) for the following return periods (in years)							
	1 in 2	1 in 10	1 in 20	1 in 30	1 in 50	1 in 100	1 in 1000	1 in 100 + CC
	Flood peak (m3/s) for the following AEP (%) events							
	50%	10%	5%	3.33%	2%	1%	0.1%	1% + CC
FEP1	4.39	6.85	7.93	8.62	9.54	11.01	18.93	15.08
FEP2	0.15	0.24	0.27	0.30	0.33	0.38	0.66	0.52
FEP3	4.77	7.43	8.60	9.34	10.34	11.92	20.42	16.33

B.6 Discussion and summary of results

B.6.1 Comparison of results from different methods

Site code	Return period 2 years / 50% AEP			Return period 100 years / 1% AEP			Return period 1000 years / 0.1% AEP		
	FEH	ReFH2	Ratio	FEH	ReFH2	Ratio	FEH	ReFH2	Ratio
FEP1	3.23	4.39	1.36	9.16	11.01	1.20	13.10	18.93	1.45
FEP2	0.19	0.15	0.79	0.54	0.38	0.70	0.77	0.66	0.86
FEP3	3.67	4.77	1.30	10.39	11.92	1.15	14.87	20.42	1.37

B.6.2 Final choice of method

Choice of method and reasons	There is more confidence with the peak flow estimates derived by the FEH statistical method due to the use of a donor site and pooling group to produce these estimates. However, as the ReFH2.3 method produces significantly higher peak flows for the 0.1% AEP event, a hybrid approach which uses the ratio of the FEH and ReFH2 0.1% AEP peak flows to scale the FEH 0.1% AEP estimate to a more conservative estimate has been applied in accordance with best practice in the flood estimation guidance.
How will the flows be applied to a hydraulic model?	FEP1 and FEP2 will be used to provide inflow hydrographs will be input into the hydraulic model at the locations shown in Figure 2.1to meet the target flows at FEP3.

B.6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	The study assumes that applying a hybrid method, whereby the 0.1% AEP peak flows are taken from the ReFH2 approach, is a suitable approach. The peak flows up to the 0.1% AEP event are derived using the FEH statistical method and are then used to scale the ReFH2 hydrographs for these events.
Discuss any particular limitations	The main limitation of the study is the lack of local hydrometric data to calibrate or validate the flow estimates. An additional limitation of the study is the proximity of the QMED donor site which is further than would typically be desired.
Provide information on the uncertainty in the design peak flow estimates and the methodology used	Uncertainty has been assessed using the methodology from the Flood Estimation Guidelines document. Please see table in Section B7.1 for the 95% and 68% uncertainty bounds.
Comment on the suitability of the results for future studies	The results are suitable for meeting the scope of the current study.

Give any other comments on the study	N/A

B.6.4 Checks

Are the results consistent, for example at confluences?	Yes, the flows are consistent with the respective catchment areas.
What do the results imply regarding the return periods / frequency of floods during the period of record?	N/A – catchment is ungauged.
What is the range of 100-year / 1% AEP growth factors? Is this realistic?	The adopted (FEH statistical) 1% AEP growth factor is 2.83 which is considered realistic and to lie within the range of the FSR regional 1% AEP growth factors.
If 1000-year / 0.1% AEP flows have been derived, what is the range of ratios for 1000-year / 0.1% AEP flow over 100-year / 1% AEP flow?	The ratio of the adopted (hybrid) 0.1% AEP and 1% AEP for the three FEPs ranges between 1.71-1.73 which is considered typical.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	N/A – no previous studies reporting flow estimates were identified.
Are the results compatible with the longer-term flood history?	N/A – catchment is ungauged.
Describe any other checks on the results	The generated flow estimates were sense checked in the hydraulic model to ensure that flooding at least occurred in the 0.1% AEP event (reasonable assumption), as well as that flooding was simulated in higher frequency return periods for areas in the study reach anecdotally known to flood more regularly.

B.7 Final results

Site code	Flood peak (m3/s) for the following return periods (in years)							
	1 in 2	1 in 10	1 in 20	1 in 30	1 in 50	1 in 100	1 in 1000	
	50%	10%	5%	3.33%	2%	1%	0.1%	
FEP1	3.23	5.68	6.69	7.29	8.07	9.16	15.74	
FEP2	0.19	0.34	0.40	0.43	0.48	0.54	0.93	
FEP3	3.67	6.45	7.59	8.28	9.16	10.39	17.80	

B.7.1 Uncertainty bounds

This table reports the flows derived from the uncertainty analysis detailed in Section B.6.3. The 'true' value is more likely to be near the estimate reported in Section B.7 than the bounds. However, it is possible that the 'true' value could still lie outside these bounds.

Site code	Flood peak (m3/s) for the following return periods (in years)													
	2		10		20		30		50		100		1,000	
	Flood peak (m3/s) for the following AEP (%) events													
	50		10		5		3.33		2		1		0.1	
	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper	Lower	Upper
FEP 1 (95%)	2.04	5.11	3.46	9.32	4.02	11.11	4.37	12.17	4.76	13.64	5.31	15.75	8.50	29.28
FEP 1 (68%)	1.29	8.12	2.10	15.34	2.41	18.47	2.62	20.38	2.82	23.00	3.11	26.92	4.56	54.30
FEP 2 (95%)	0.12	0.30	0.21	0.56	0.24	0.66	0.26	0.72	0.28	0.81	0.31	0.93	0.50	1.74
FEP 2 (68%)	0.08	0.48	0.13	0.92	0.14	1.09	0.15	1.20	0.17	1.37	0.18	1.59	0.27	3.22
FEP 3 (95%)	2.31	5.80	3.93	10.58	4.56	12.61	4.97	10.58	5.40	15.48	6.03	17.87	9.61	33.11
FEP 3 (68%)	1.47	9.21	2.39	17.42	2.73	20.96	2.98	23.14	3.21	26.11	3.53	30.55	5.16	61.41

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, hydraulic model, or reference to table below)	Inflows for hydraulic model_v2.xlsx

B.8 Annex

B.8.1 List of gauging stations used in final QMED pooling group

Station	Distance (SDM)	Years of data	QMED AM	L-CV		L-SKEW		Discordancy
				Observed	Deurbanised	Observed	Deurbanised	
25019 (Leven @ Easby)	0.023	43	5.677	0.334	0.335	0.373	0.372	2.062
27051 (Crimple @ Burn Bridge)	0.469	49	4.564	0.217	0.218	0.143	0.142	0.09
27010 (Hodge Beck @ Bransdale Weir)	0.531	41	9.42	0.224	0.224	0.293	0.293	1.011
49005 (Bolingey Stream @ Bolingey Cocks Bridge)	0.658	11	5.777	0.262	0.263	0.207	0.206	2.272
41020 (Bevern Stream @ Clappers Bridge)	0.723	52	13.78	0.201	0.203	0.166	0.164	0.484

Station	Distance (SDM)	Years of data	QMED AM	L-CV		L-SKEW		Discordancy
				Observed	Deurbanised	Observed	Deurbanised	
28058 (Henmore Brook @ Ashbourne)	0.793	13	10.6	0.145	0.147	-0.046	-0.049	2.28
24007 (Browney @ Lanchester)	0.89	15	10.981	0.222	0.222	0.212	0.211	0.932
9006 (Deskford Burn @ Cullen)	0.924	11	21.783	0.29	0.29	0.139	0.139	1.091
53017 (Boyd @ Bitton)	0.935	48	13.908	0.24	0.242	0.081	0.079	0.823
44011 (Asker @ Bridport East Bridge)	0.995	26	15.958	0.225	0.227	0.172	0.169	0.146
44003 (Asker @ Bridport)	0.995	14	12.354	0.224	0.226	0.17	0.168	1.096
41022 (Lod @ Halfway Bridge)	1.012	51	15.9	0.295	0.297	0.183	0.181	0.811
49004 (Gannel @ Gwills)	1.045	52	14.51	0.257	0.259	0.114	0.113	0.498
28041 (Hamps @ Waterhouses)	1.059	36	26.582	0.218	0.218	0.272	0.272	1.081

Station	Distance (SDM)	Years of data	QMED AM	L-CV		L-SKEW		Discordancy
				Observed	Deurbanised	Observed	Deurbanised	
73015 (Keer @ High Keer Weir)	1.107	31	12.421	0.208	0.209	0.195	0.195	0.323

Numerous stations from the default pooling group were removed due to having BFIHOST19 values that deviated from the study catchment's value by >0.2, including the list below:

26016 (Gypsey Race @ Kirby Grindalythe)

26017 (Ings Beck @ South Newbald)

27073 (Brompton Beck @ Snainton Ings)

44008 (South Winterbourne @ Winterbourne Steepleton)

7011 (Black Burn @ Pluscarden Abbey)

26014 (Water Forlornes @ Driffield)

44013 (Piddle @ Little Puddle)

39033 (Winterbourne Stream @ Bagnor)

Several stations were added based on their proximity to the study catchment. These included:

44011 (Asker @ Bridport East Bridge)

44003 (Asker @ Bridport)

41022 (Lod @ Halfway Bridge)

49004 (Gannel @ Gwills)

28041 (Hamps @ Waterhouses)

73015 (Keer @ High Keer Weir)

This process left the following stations in the finalised pooling group:

25019 (Leven @ Easby)

27051 (Crimple @ Burn Bridge)

27010 (Hodge Beck @ Bransdale Weir)

49005 (Bolingey Stream @ Bolingey Cocks Bridge)

41020 (Bevern Stream @ Clappers Bridge)

28058 (Henmore Brook @ Ashbourne)

24007 (Browney @ Lanchester)

9006 (Deskford Burn @ Cullen)

53017 (Boyd @ Bitton)

44011 (Asker @ Bridport East Bridge)

44003 (Asker @ Bridport)

41022 (Lod @ Halfway Bridge)

49004 (Gannel @ Gwills)

28041 (Hamps @ Waterhouses)

73015 (Keer @ High Keer Weir)

C. Hydraulic model development

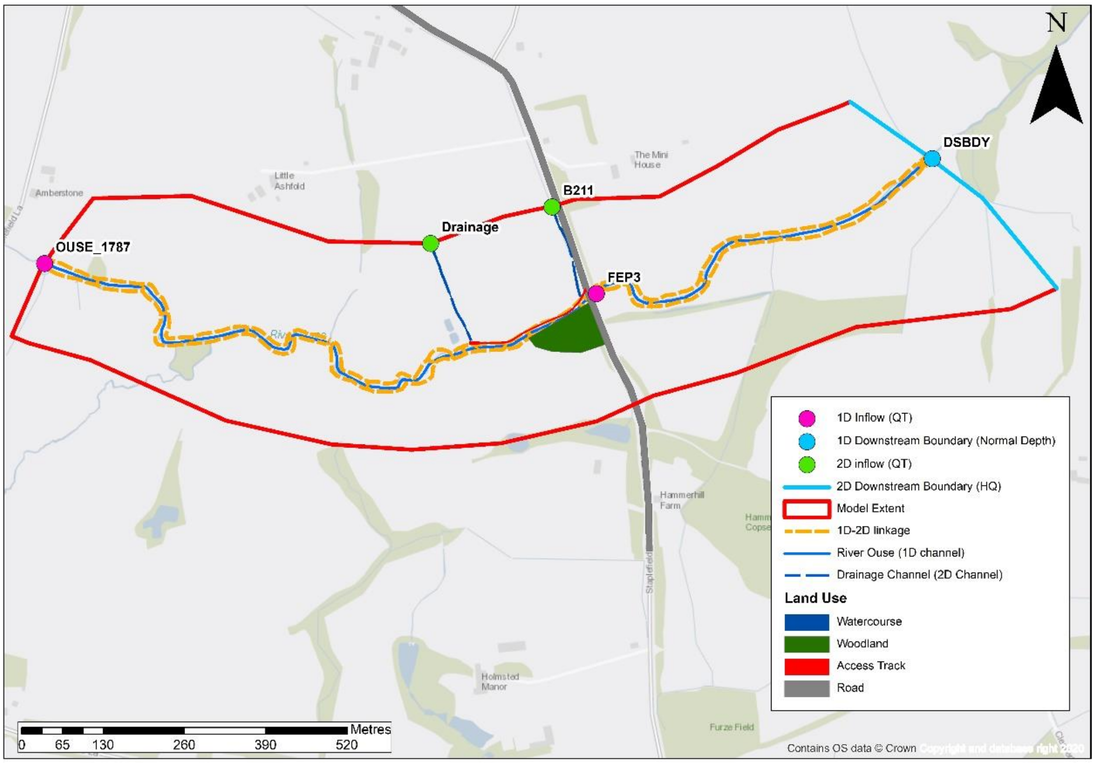
C.1 Baseline model build

The watercourses and floodplain were developed as a 1D-2D linked Flood Modeller Pro-TUFLOW (FMP-TUFLOW) model. The 1D-2D approach allows the watercourse and floodplain to be represented in sufficient detail to accurately model flood risk, using industry recognised software. The River Ouse has been modelled in the 1D domain using FMP. Modelling the River Ouse in 1D allows for a more accurate representation of the channel, using cross-sections that pick up detailed elevation changes. The two drainage channels to the east and west of the site have been modelled in the 2D domain with bed level and banks enforced into the floodplain in the 2D grid to replicate the channel capacity and threshold of flooding. These channels were modelled in the 2D domain to improve model stability over the steep gradient flowing into a small culvert adjoining the River Ouse 1D model. The three culvert structures on the drainage channels have been modelled as 1D ESTRY culverts embedded into the 2D domain. The culverts are the key constraint to flow on these drainage channels and modelling them as embedded 1D structures allow them to be modelled in sufficient detail. The remaining model area is simulated in the 2D TUFLOW to model the spreading of water across the field once it spills of the River Ouse of local drainage channels.

C.1.1 Model extent

The model extent (Figure C.1Figure C.1:) was determined to include all features that could influence flood risk to the site. The upstream extent of the model is located downstream of Staplefield Lane and extends to a bridge located approximately 600m downstream of the site. The bridge was chosen as the downstream extent as a backwater calculation determined that this bridge is far enough downstream not to have an impact on the site. The upstream point was chosen as this should cover all possible flow paths that could contribute to flooding on site. The total modelled area is 49.6ha. The length of watercourse modelled in the 1D domain is 1.8km. The total length of tributaries watercourses modelled in the 2D domain is 0.35km.

Figure C.1: Model Extent and Schematic



C.1.2 Boundaries

Two 1D Flow-Time (QT) inflow boundaries have been applied to the model: one at the upstream extent of the modelled Ouse and a lateral inflow applied upstream of the B2114 bridge. These locations represent the total flow arriving to the site at FEP3 on Figure C.1: C-1 split between the River Ouse upstream catchment and local sub-catchments for the drainage channels as set out in Section 3.2.

A normal depth boundary has been applied at the downstream extent of the River Ouse in the 1D domain far enough downstream such that the assumed floodplain slope does not influence flood levels at the site.

C.1.3 Application of design flows

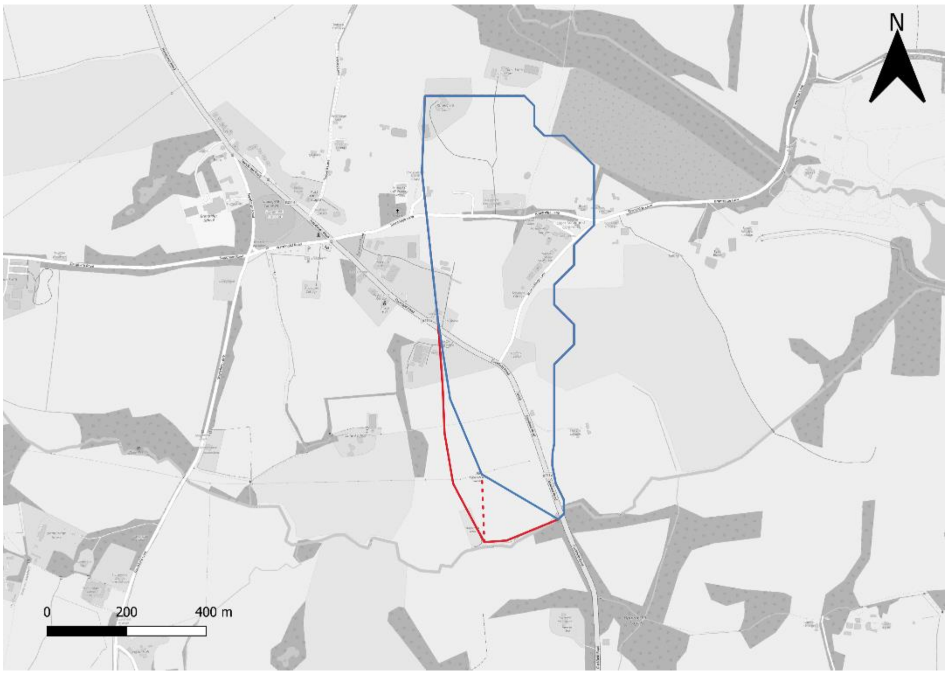
The FEH statistical method was used to derive peak flows. There was more confidence with this method due to the use of a donor site and pooling groups to produce these estimates. For the 0.1% AEP event, a hybrid method was used to produce peak flows by using the ratio of the ReFH2 flows to scale the 0.1% AEP estimate in accordance with the flood estimation guidance[20].

Peak flows were estimated for three locations (Flow Estimation Points, FEPs). One on the Ouse at the downstream extent of the site (upstream of the B2114 bridge), one on the Ouse at the downstream extent of the model, and one at the downstream extent of the B2114 eastern drainage channel, just before it joins the Ouse (Figure C.2). The 10.5 hour storm duration was identified as the critical duration following the sensitivity testing (see Section C.2.3). The peak flows for the 10.5 hour storm duration are shown Table C.1: . The peak flows for FEP3 are larger than the combined FEP1 and FEP2 flows as it includes an area of the catchment

[20] Estimate flood flow from rainfall and river flow data (source) - GOV.UK (www.gov.uk)

downstream of the B2114 bridge not covered by FEP1 or FEP2. Full details of the hydrological assessment can be found in Appendix B.

Figure C.2: Flow Estimation Point locations



The red catchment shown in Figure C.2 has been applied to the western ditch to represent the area draining form upstream and the local field area. In reality some of the local field drains via field drainage pipes to the River Ouse. However, this has been lumped together with the western ditch inflow as a conservative assumption of flood risk along the western boundary of the site.

The remainder of FEP2 catchment is shown in blue on Figure C.2. This has been applied at the B211 inflow and represents the eastern drainage ditch by the highways which was confirmed on site to drain Staplefield village.

Table C.1: Peak flow estimates at FEPs for the 10.5 hour storm duration

Site Code	Catchment Area (km^2)	Flood peak (m^3/s) for the following AEP (%) events						
		50%	10%	5%	3.33%	2%	1%	0.1%
FEP1	13.28	3.23	5.68	6.69	7.29	8.07	9.16	15.74
FEP2	0.35	0.19	0.34	0.40	0.43	0.48	0.54	0.93
FEP3	14.66	3.67	6.45	7.59	8.28	9.16	10.39	17.80

The model inflow locations are shown in Figure C.2 . FEP 1 was applied to the upstream extent of the model, at the inflow named “OUSE_1787”. FEP2 was split between the two drainage channels, with 98.9% applied at the upstream extent of the eastern B2114 channel (inflow named “B211”), and 1.2% applied at the western channel (inflow named “Drainage”). The remainder of the flow (found by subtracting combined peaks from FEP1 and FEP2 from FEP3) were applied at “FEP3” downstream of the B2114 bridge as a lateral inflow. The FEP target flows are shown in TableC.1 and the peak inflows as they were applied to the model, are shown in Table C.2.

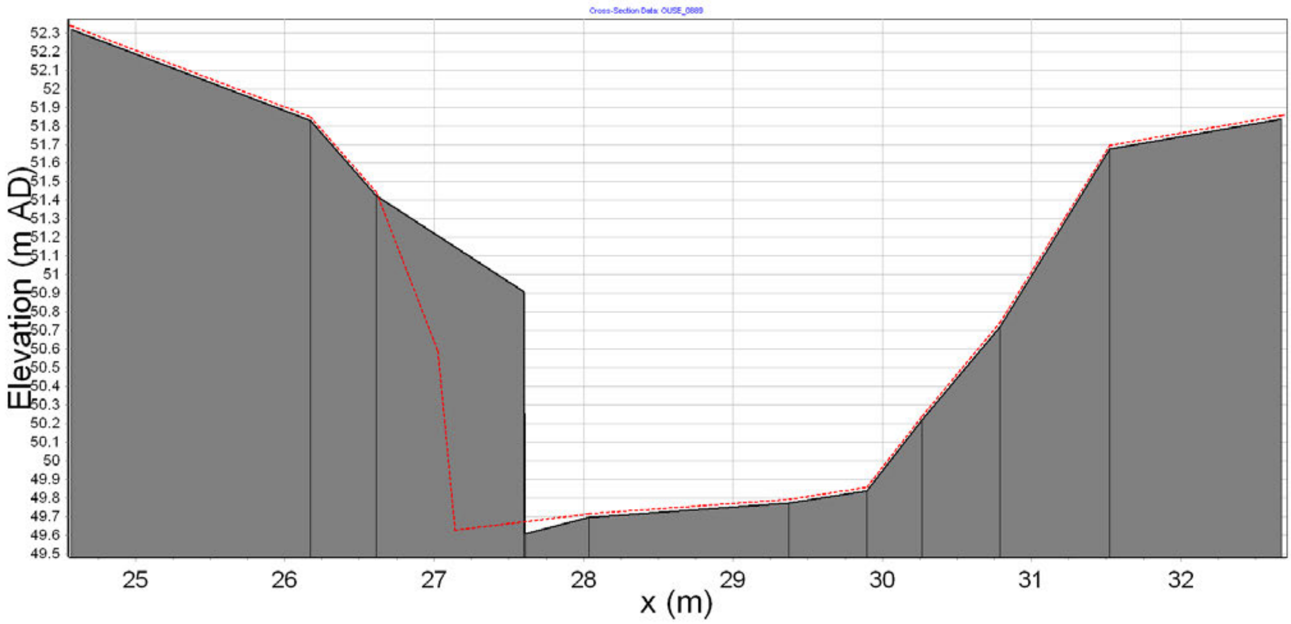
Table C.2: Applied Modelled Inflow Peaks

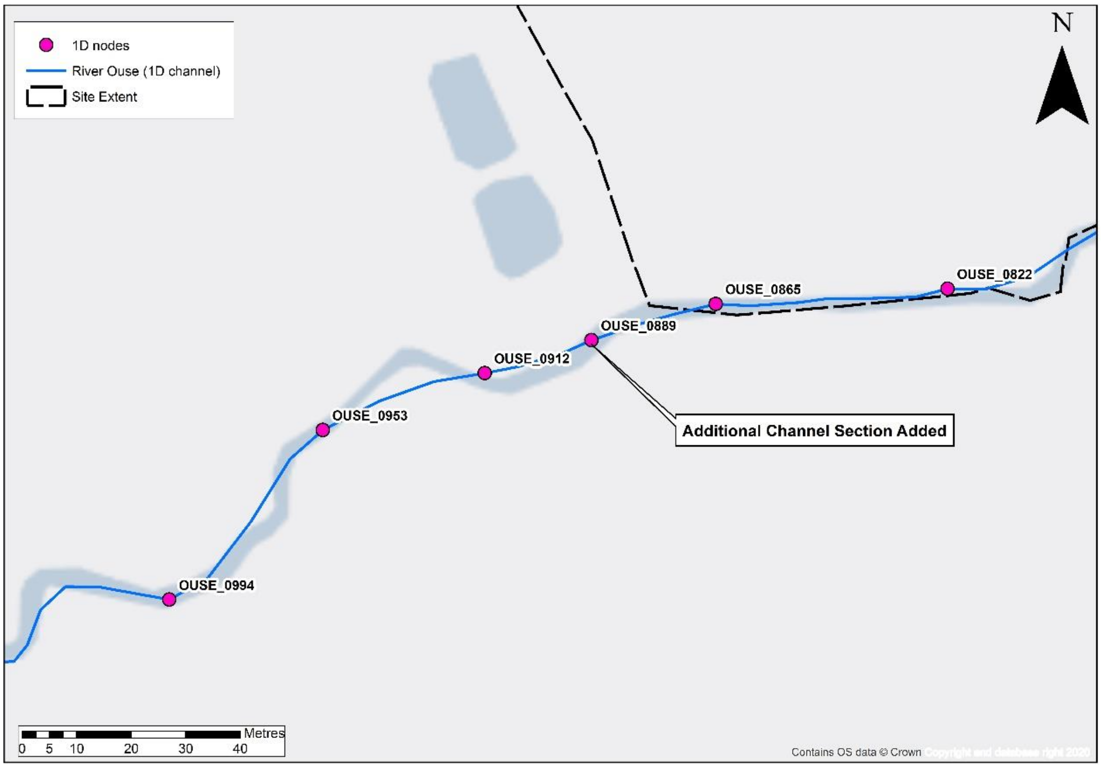
Site Code	Flood peak (m3/s) for the following AEP (%) events							
	50%	10%	5%	3.33%	2%	1%	0.1%	
OUSE_1787	3.23	5.68	6.69	7.29	8.07	9.16	15.74	
B211	0.188	0.336	0.395	0.425	0.474	0.534	0.919	
Drainage	0.002	0.004	0.005	0.005	0.006	0.006	0.011	

C.1.4 Open channel

The 1D open channel sections for the River Ouse were represented based on the channel survey, captured by Maltby Surveys Ltd. The cross-sections were trimmed to the top of bank to match the width of the channel shown in the 2D domain by LiDAR or topographic survey. During a site visit, a double shelf was observed on the left bank of the River Ouse just upstream of the site location, which was not picked up in the channel survey in 2023. This double shelf narrows the width of the channel in this location and causes a possible flow constriction that may affect flooding on site. To represent this double shelf, a copy of the upstream cross-section was made, and the double shelf was added (node label OUSE_0889). The width and height of the shelf was estimated based on visual observations and available data from the upstream and downstream surveyed cross-sections. All other 1D open channel sections were modelled as shown in the cross-section survey. A 2D Z-line was used to enforce the bank levels along a section of the River Ouse at the site to ensure the 1D bank level matched the 2D elevation. This was done to counter the discrepancies between the channel survey and topographic survey.

Figure C.3: Channel Section OUSE_0889,





Source: Cross section shows shelf/two stage channel on left bank with the upstream cross-section OUSE_0912, shown in red dashed line for reference and location of the additional node shown in the map.

The drainage channels on the perimeter of the site were modelled in the 2D domain, being incorporated into the DEM using 2D Z-shape layers (Points and Lines). For each drainage channel, a Z-shape was used to represent the channel bed levels and another Z-shape was used to represent the bank levels. For the Eastern channel (B2114 channel), the available cross-section survey was used to represent the bed and bank elevations. Topographic elevations of the bed and bank levels for the western channel were extracted from the site topographic survey .

C.1.5 Key structures

A total of six structures have been represented in the baseline model: three along the River Ouse modelled in FMP, and three along the drainage channels modelled in ESTRY. The representation of the structures in the model is summarised in Table C.2.

Table C.3: Key structures in the model

Name	Location (Easting and Northing)	Type	Dimensions	Manning's "n" roughness value
Dilapidated Access Track Bridge (OUSE_1198bu)	527770 127364	USBPR1978 (Non-Arched) Bridge, FMP	Invert level: 52.077mAOD Soffit level: 52.390mAOD Max Width: 3.458m Length: 4.64m	0.04 - 0.05
B2114 Road Bridge (OUSE_0660bu)	528180 127452	Arched Bridge, FMP	Invert level: 49.492mAOD Springing Level: 49.946mAOD Soffit level: 50.97mAOD Max Width: 1.723m Length: 7.58	0.035 – 0.07
Access Bridge (OUSE_0005bu)	528715 127645	USBPR1978 (Non-Arched) Bridge, FMP	Invert level: 47.321 Soffit level: 49.64 Max width: 6.098 Length: 4.86m	0.05
Farm access culvert (B211_0077C)	528170 127487	Circular culvert, ESTRY	Invert level: 51.38mAOD Soffit level: 52.18mAOD Diameter: 0.80m Length: 6.20m	0.024
Access track culvert (East) (B211_0012C)	528183 127426	Circular culvert, ESTRY	Invert level: 49.97mAOD Soffit level: 50.42mAOD Diameter: 0.45m Length: 8.95	0.013
Access track culvert (West) (Drainage culvert)	528006 127351	Circular culvert, ESTRY	Invert level: 50.87mAOD Soffit level: 51.25mAOD Diameter: 0.38m Length: 6.556m	0.02

C.1.6 Floodplain

The composite 2m DTM (flown in February 2021) was used to represent the base elevations across the model (Figure C.4) Site specific topographic survey (undertaken by Maltby Surveys Ltd in November 2022) was used to represent elevations across the site (Figure C.5). Site elevations range from 50.5mAOD in the south-east of the site (not including the watercourses) to 57.5mAOD at the north of the site. An anomalous low area of the topographic survey was "smoothed out" near the downstream extent of the western drainage channel. The low point was up to 1m lower than the surrounding ground and located off the left bank of the Ouse. It appears to be a processing error when creating the ascii, that incorrectly merged the elevations of the watercourse and the ground. The site topographic survey shows that the Water Treatment

Works is protected by raised embankments with elevations of 53.0mAOD, approximately 1m higher than the surrounding ground.

A polygon was added to smooth the transition between the boundary of the LiDAR and topographic survey, to prevent potential glass walling of flood water. The elevations match well with the bank levels in the channel survey for the rest of the model extent. Therefore, no further topographic changes were made to the LiDAR across the model extent.

The elevations of an access track on the left bank of the watercourse were enforced using a 2D Z-shape layer to ensure they were being accurately represented as the level of the access track is important in determining when out of bank flooding first occurs.

Figure C.4: LiDAR coverage across site

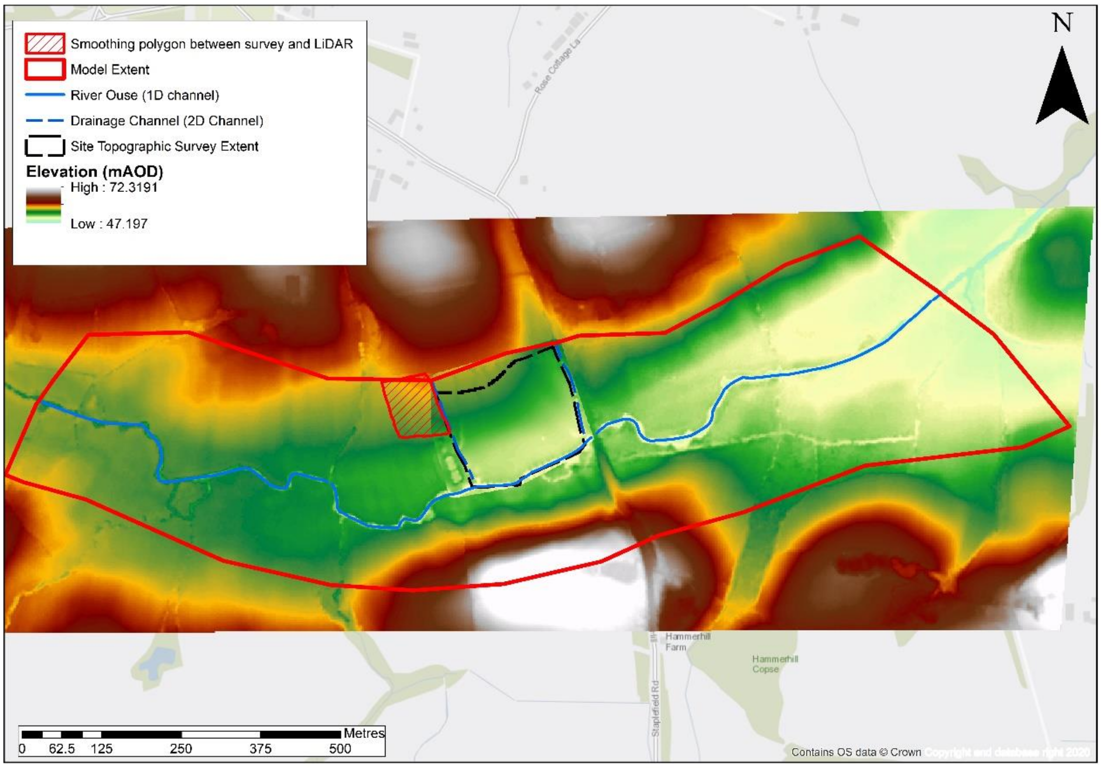
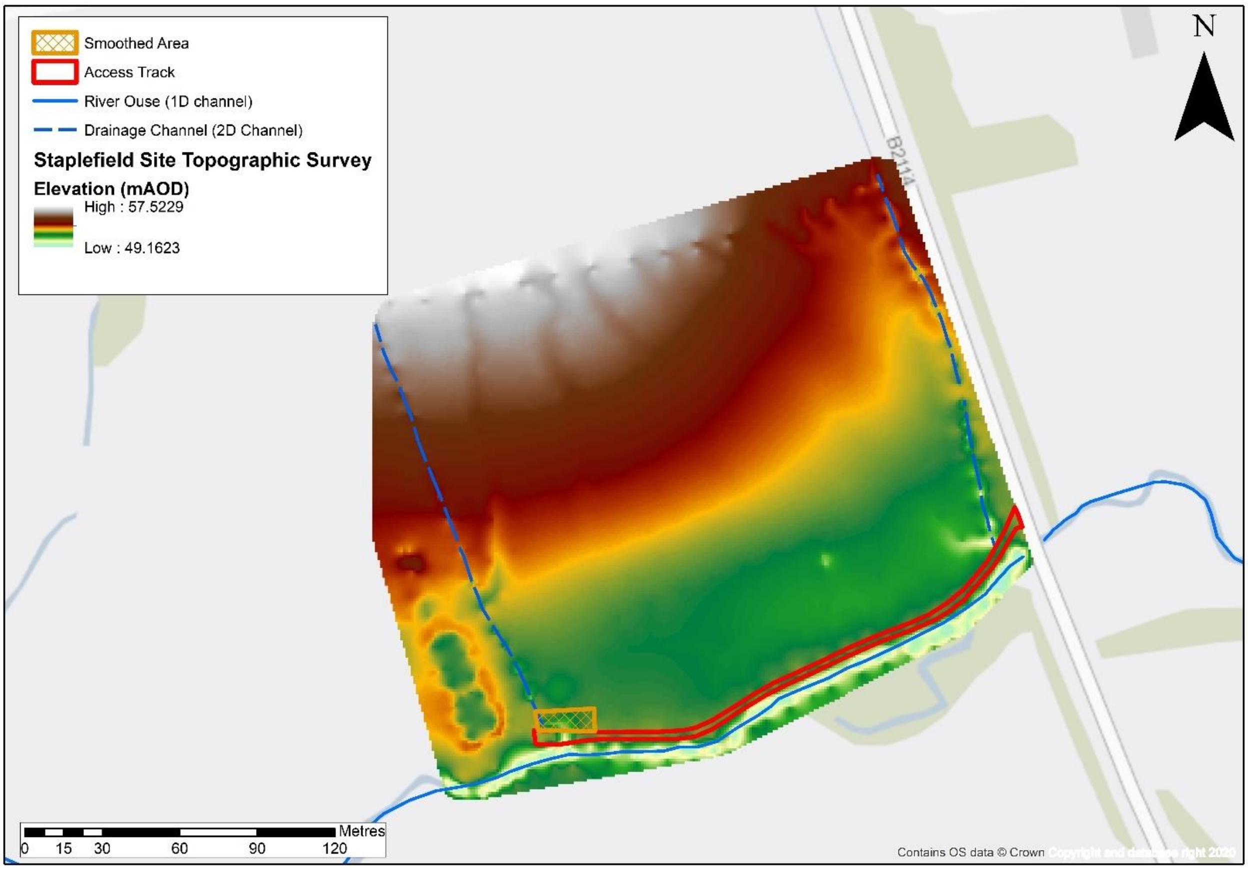


Figure C.5: Site topographic data



C.1.7 Model roughness

The ranges of Manning's n values used to represent roughness in the 1D domain are shown in Table C.3. The Manning's n values were based on Chow (1959). A conservative value of 0.05 was applied for the majority of the channel bed in the 1D domain. This is due to high vegetation and a high prevalence for blockages that was observed on a site visit. A higher value of 0.06 was applied at one node (OUSE_0953) to representing a sharp meander, to account for form loss values. Channel banks were also given a high value ranging from 0.07 (medium to dense brush), to 0.10 (dense brush/woodland). Photographs provided by the surveyor for each cross-section were used to determine channel bank roughness. Further details on the roughness values chosen for the structures can be found in Section C.1.5..

Table C.4: 1D Roughness Values

Description	Manning's "n" roughness value range
Open Channel Bed	0.04 – 0.06
Open Channel Banks	0.07 – 0.10
ESTRY Culverts	0.013 – 0.024

The 2D roughness values used in the model are shown in Table C.4. Due to lack of available data (e.g. MasterMap) the majority of the model in the 2D domain is represented with the "General Surface" value of 0.05. For the majority of the model, this value is considered appropriate as most of the surrounding area is farmland or grasslands. Important features with differing roughness values, such as the access track, road, and wooded areas have been added in manually.

Table C.5: 2D roughness values

Model Code	Description	Manning's "n" roughness value
2	General Surface	0.05
4	Inland Water	0.035
5	Woodland	0.10
6	Path (Used for access track)	0.04
8	Road or track (used for B2114 road)	0.02

C.2 Baseline model proving

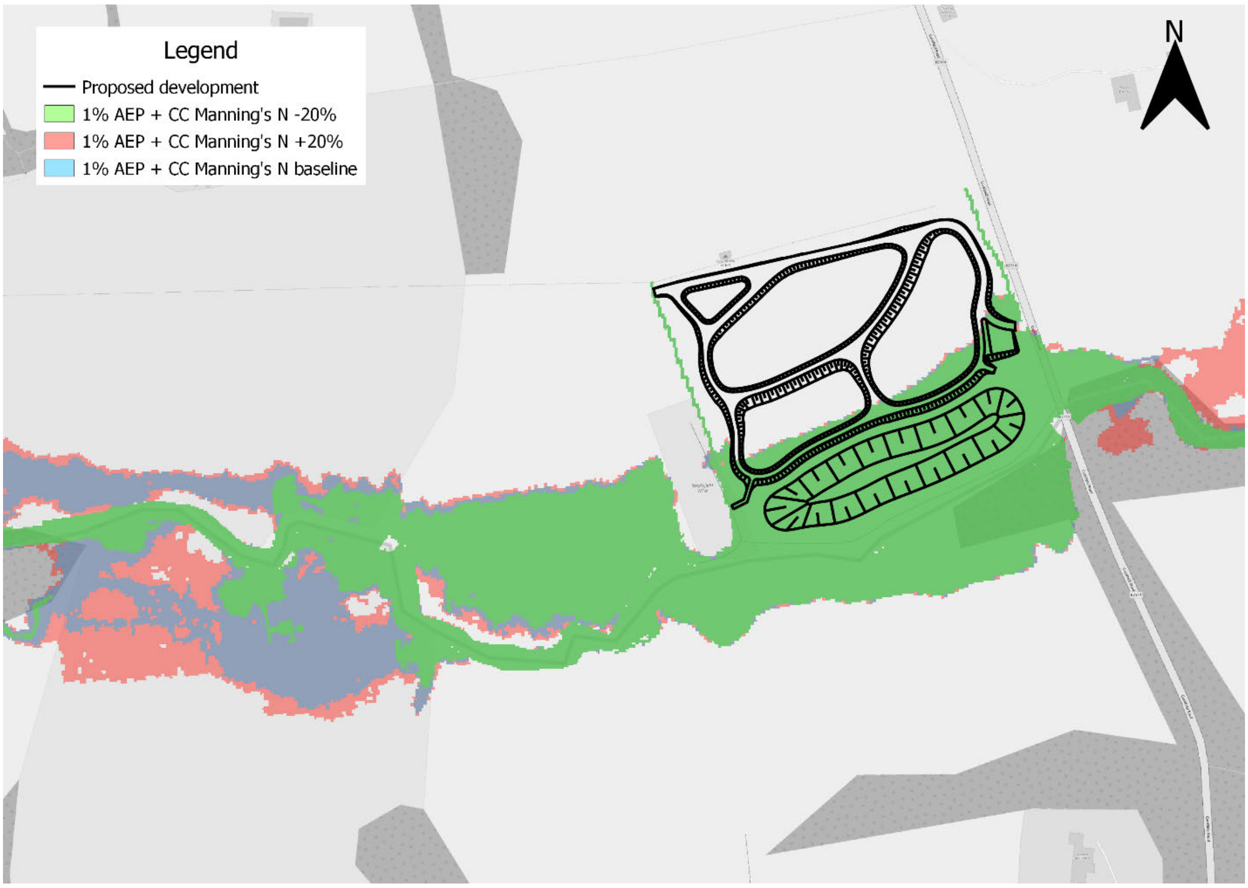
The following sensitivity tests were carried out for the model. All sensitivity tests were simulated for the 1% AEP plus climate change event

- Manning's "n" roughness increased by ±20%
- The slope value of the downstream boundary increased/decreased
- The B2114 bridge was modelled using an orifice unit for all flows rather than a bridge unit transitioning to orifice flow.
- Storm duration testing – used to determine the critical storm duration that would be used for all subsequent model simulations
- FARL Sensitivity test – Uses model inflows derived from hydrological calculations "FARL" value set to 1

C.2.1 Manning's "n" roughness sensitivity test

For the Manning's "n" sensitivity test, all 1D and 2D roughness values were increased and decreased by 20%. For most of the model, decreasing the Manning's "n" roughness decreases the flood extent compared to the baseline, whilst increasing the roughness increases the flood extent (Figure C.6). This is because with the larger roughness values, more resistance is applied to the water, making it flow less freely, resulting in more out of bank flooding. However, there is one area of the site field which has a larger flood extent occurring in the Manning's -20% scenario than in the baseline and +20% scenario. Flood depths across the field are generally higher too in -20% roughness than in the baseline (up to 0.08m). This increase is due to less out of bank flooding occurring upstream, resulting in more flow reaching the B2114 bridge constriction. The flow arriving then spills out of bank by the bridge as the bridge structure is not able to pass this additional water. Despite these changes, the extents at site are not significantly different through the site due to backwater from the bridge at the 1 in 100 year + climate change event. The modelled results on site are considered moderately sensitive to changes in Manning's "n" values. However, flood extents upstream and downstream of the site seem more sensitive to changes in Manning's "n" as these areas are located beyond the backwater effect of the road bridge.

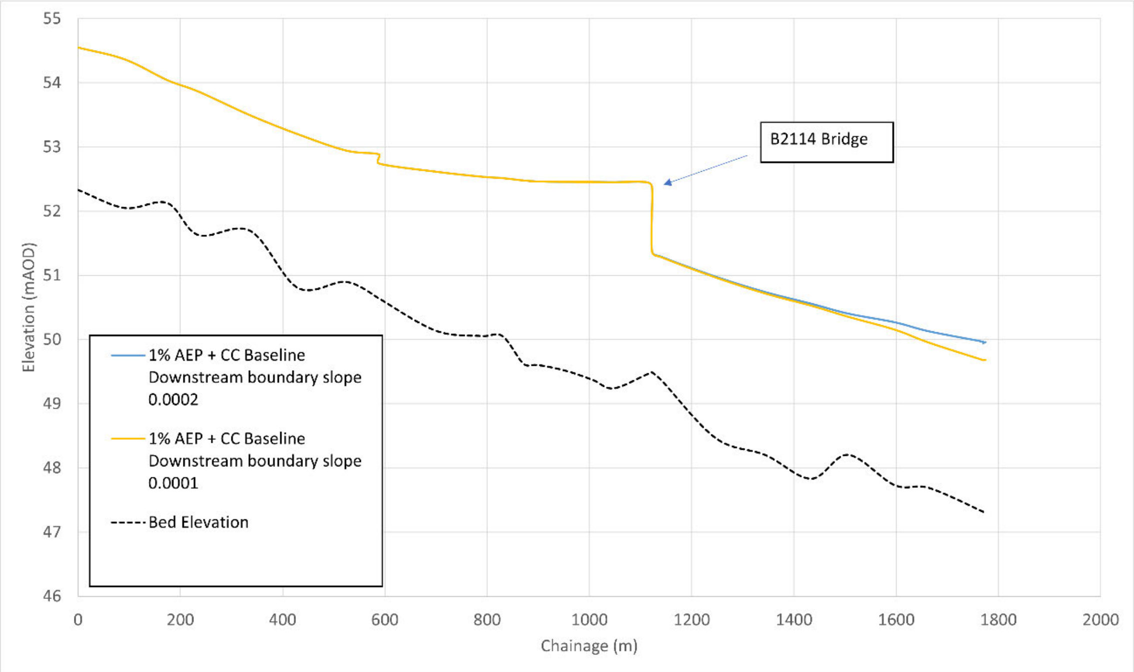
Figure C.6: 1% AEP plus Climate Change results for the Manning's "n" sensitivity test



C.2.2 Downstream boundary sensitivity test

Two downstream boundary sensitivity tests were simulated: one where the slope value of the downstream boundary slope value in the 1D and 2D domains was decreased (i.e., made less steep) and one where the slope value was increased (made steeper). During both scenarios, changes in flood depths occur near the downstream extent of the model, however, does not affect water levels on site or anywhere upstream of the B2114 bridge (Figure C.7). Therefore, the model is deemed to be insensitive at the proposed site to changes in the downstream boundary conditions.

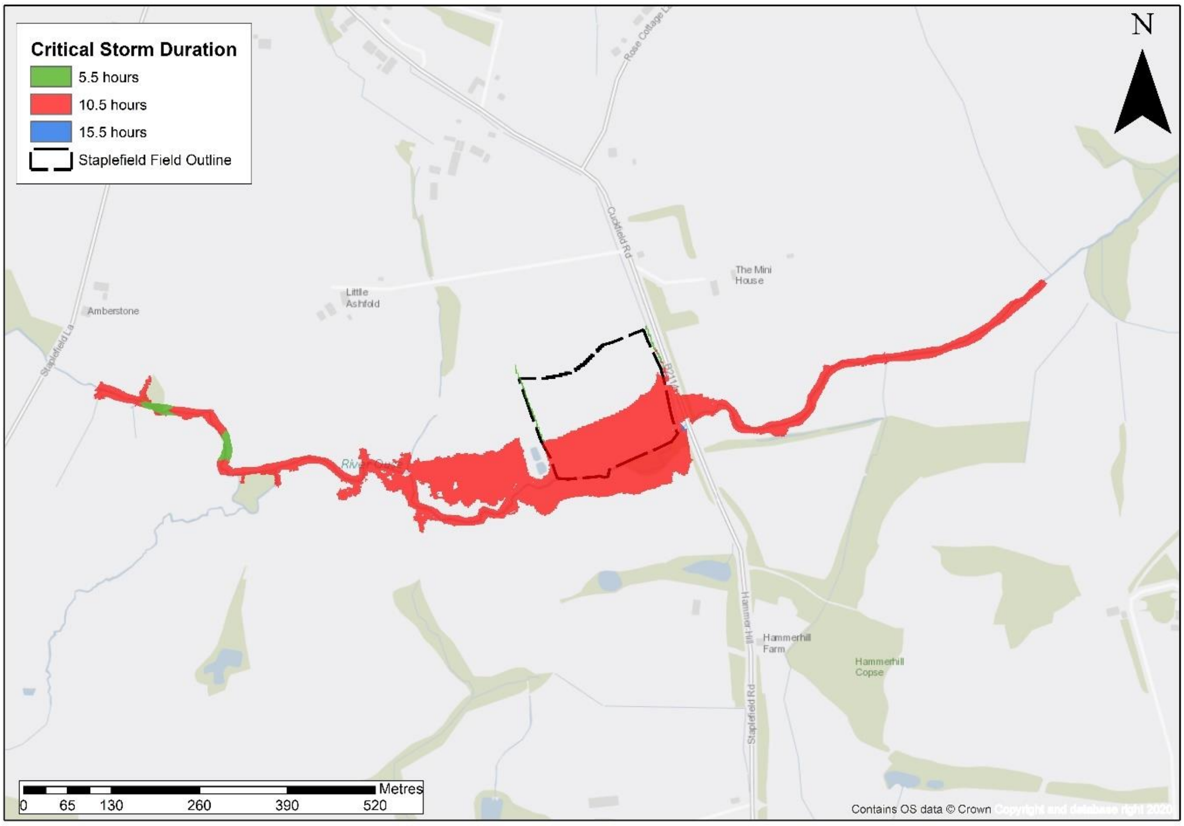
Figure C.7: 1% AEP plus climate change water level comparison: downstream boundary sensitivity test



C.2.3 Storm Duration Testing

Three different storm durations were tested using the 1% event to determine which one would be the critical duration and produce the highest flood depths. The durations tested were the 5.5-hour, 10.5-hour and 15.5-hour. The results of the test are shown in Figure C.8. The 10.5 hour duration is shown to be clearly the critical duration across almost the entire model extent, including at the site and at all locations where out of bank flooding occurs. Therefore, all simulations were simulated with the 10.5 hours storm duration.

Figure C.8: Storm Duration testing results



C.3 Assumptions and limitations

The following are a set of assumptions and limitations that have been used to build the hydraulic model that should be considered when assessing the results.

- The hydrological analysis is an area of uncertainty and is produced based on a number of assumptions. The catchment is ungauged, meaning there is no real-world data to compare the hydrological analysis to. Further details on assumptions and limitations from the hydrology can be found in the Flood Estimation Calculation Record in Appendix B.
- The data used to represent the topography of the model, including topographic survey and LiDAR DTM is assumed to be accurate. An analysis between the topographic survey and LiDAR showed discrepancy with the topographic survey generally being lower than the LiDAR (generally around 0.2m lower). The channel survey bank elevations on site generally match well with the elevations of the topographic survey, whilst also matching well with the LiDAR throughout the rest of the model. Therefore, no modifications to the LiDAR or survey have been made aside from the representation of features such as the access track. In one area, the transition between the survey area and LiDAR had to be smoothed out with a Z-shape polygon to prevent glass walling (though no flooding occurs in this area anyway). Based on the assessment, the topographic data used in the modelling is deemed to be appropriate.
- No detailed channel survey was available to represent the western drainage channel, and therefore bed and bank elevations have been determined from the topographic survey data. Therefore, both the western and eastern (due to stability issues when modelled in the 1D domain) drainage channels have been modelled in the 2D domain, with 1D embedded culverts. Whilst modelling the channels in the 2D domain is not as accurate as modelling

them 1D-2D, the representation of the flood risk of these smaller drainage channels in the 2D domain should be appropriate given the available data.

  – The culvert at the downstream extent of the western drainage channel has been modelled based on the topographic survey and is deemed to be a sufficient representation.
  – The roughness for the drainage channels was set to be 0.05 for vegetation blockage effects in the baseline. This assumption should be reviewed at detailed design.

- The model has been optimised for flow arriving to the site, and not for flood mapping upstream and downstream. Observations on site and anecdotal evidence from the landowner suggest that the field to the west of the sewage treatment works floods more frequently than is shown in the hydraulic modelling, which shows it to first flood during the 1% AEP event. Localised blockages were not considered explicitly in the model as there exact location and extent varies event to event such as a blockage from the dilapidated access bridge upstream. Instead this effect was incorporated in the selection of the Manning's n for the reach. Ultimately, flooding to the proposed development site is still shown to be frequent, and reduced flooding to the western field would only increase flooding to the development site, making results more conservative. Therefore, the results and modelling are deemed to be appropriate to use for this assessment.
- The sensitivity test with the FARL value shows a large increase in water levels when the FARL value was increased to 1 in the hydrological calculations. A check was performed of the catchment and the default value of 0.93 was considered appropriate due to the presence of bodies of water in the upstream catchment. Therefore, there was no reason to assume that the calculations using the FARL value of 1 would be the most appropriate to use for the modelling study. However, the test did show how sensitive the model can be to changes in the hydrological assumptions.

C.4 Baseline modelled scenarios and performance

Seven baseline scenarios were simulated to provide an understanding of flooding patterns.

Table C.6: Modelled Baseline Scenarios

Scenario	Return Period	AEP (%)	Model File
Baseline	2yr	50%	Staplefield_Baseline_10_5h_Q0002_026
Baseline	10yr	10%	Staplefield_Baseline_10_5h_Q0010_026
Baseline	30yr	3.33%	Staplefield_Baseline_10_5h_Q0030_026
Baseline	50yr	2%	Staplefield_Baseline_10_5h_Q0050_026
Baseline	100yr	1%	Staplefield_Baseline_10_5h_Q0100_026
Baseline	100yrCC	1% +CC	Staplefield_Baseline_10_5h_Q0100cc37_026
Baseline	1000yr	0.1%	Staplefield_Baseline_10_5h_Q1000_026

The key model run parameters are as follows:

- The cell grid size of the model is 2m. The 2D timestep has been set to 1 second (half the grid cell size) and the 1D timestep to 0.5 seconds (half the 2D timestep). This is considered appropriate for the area and scheme that are being modelled.
- The model run time is 25 hours to fully simulate beyond the peak of the flood for all events.
- The global matrix dummy coefficient has been set to 0.001. This has been set to prevent the model from crashing. Automatic Preissmann slots have been turned on to improve stability. No other parameters have been changed from default settings.
- The convergence plot from FMP for the 0.1% AEP event shows the model is within the recommended tolerance with no reported non-convergences.
- In TUFLOW, the peak cumulative Mass Error % during the 0.1% AEP event is -0.76%, which occurs at the start of the simulation, within the accepted range of ±1%. The final cumulative ME is -0.23%. No negative depths occur during the simulation for all events.
- Three check messages occur in TUFLOW during the baseline scenario. The check messages exist to ensure that the features of the model are being represented as intended. The check messages are not significant and the features mentioned are being modelled as intended. No further Warning or Error messages occur.
  – CHECK 1393 occurs twice: “Node XXXX linked to external 1D scheme Node OUSE_XXXX using a X1DH link”.
  – CHECK 2118 occurs once: - “Lowered SX ZC Zpt by 0.04m to 1D node bed level”. This message shows that the elevation of the 2D cell has been lowered to match the invert level set by the culvert as intended.

C.5 Post-Development model modifications

The post-development/scheme model consisted of two major components.

- The ICW wetland and associated embankment and pumping station
- The flood mitigation areas and access track

C.5.1 Wetland area

The wetland embankment has been represented by the 2d_ZSh layers to raise elevations up to a minimum of 53.5mAOD to the crest of the embankment and car park area. 2d_Zsh polygons have been used to transition from the toe to the crest of the embankment and to represent the access ramps).

C.5.2 Flood mitigation area

The flood mitigation area compensates for the floodplain lost due to the construction of the wetland, mitigating a volume of approximately 1,2400m^3. The mitigation area has a minimum elevation of 50.4mAOD at the centre and banks that vary from 51.3mAod in the southeast to the 51.72mAOD in the north-west to tie into exisitng ground elevation.

At the lowest points in the mitigation area are 300mm flapped culvert that drain the water from the mitigation area back into the Ouse. These have not been explicitly represented in the model but are assumed to be present to ensure the flood mitigation area if empty prior to the flood and can drain down within 11 hours(similar to the critical storm duration of 10.5 hours)

C.6 Post-Development modelled scenarios and performance

Seven post-development scenarios were simulated to provide an understanding of flooding patterns.

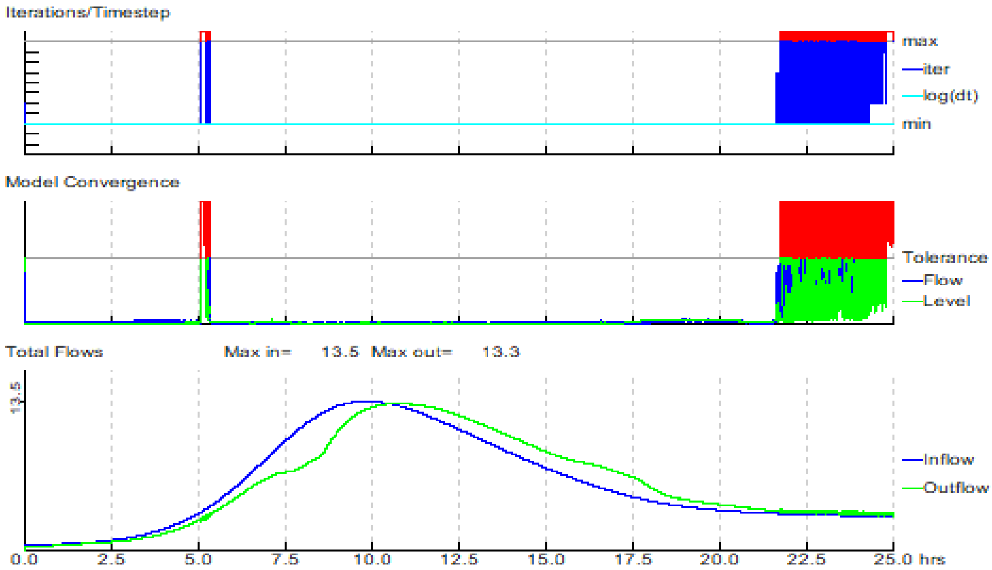
Table C.7: Modelled Post- Scheme Scenarios

Scenario	Return Period	AEP (%)	Model File
Baseline	2yr	50%	Staplefield_Postdev_10_5h_Q0002_026
Baseline	10yr	10%	Staplefield_Postdev_10_5h_Q0010_026
Baseline	30yr	3.33%	Staplefield_Postdev_10_5h_Q0030_026
Baseline	50yr	2%	Staplefield_Postdev_10_5h_Q0050_026
Baseline	100yr	1%	Staplefield_Postdev_10_5h_Q0100_026
Baseline	100yrCC	1% +CC	Staplefield_Postdev_10_5h_Q0100cc37_026
Baseline	1000yr	0.1%	Staplefield_Postdev_10_5h_Q1000_026

The key run parameters for the post-development simulations are as follows:

- No additional parameters have been changed for the post-scheme scenario, compared to the baseline
- The convergence plot from FMP for the 0.1% AEP event is shown in the figures below. It shows that the model is simulating stably, with no non-convergence occurring beyond the tolerance level.
- In TUFLOW, the peak cumulative Mass Error % during the 0.1% AEP event is -0.76%, which occurs at the start of the simulation, within the accepted range of ±1%. The final cumulative ME is -0.22%. No negative depths occur during the simulation for all events.
- An additional two check messages occur compared to the baseline (five total). Both of them relate to the addition of the pipe from the flood mitigation area. Neither are significant and the pipe is being represented as intended.
  - CHECK 1402 – “More than one culvert connected but could not create manhole at Node "B211_0012C.2". Check culvert inverts and directions”.
  - CHECK 2118 – “Lowered SX ZC Zpt by 0.05m to 1D node bed level”. This message shows that the 2D cell level has been lowered to match the invert level of the culvert of 50.40mAOD, as intended.

Figure C.9: Convergence plot for the 1% AEP post-scheme event



Datafile: ...\MODEL\FMP\DAT\STAPLEFIELD_026.DAT
Results: ...\STAPLEFIELD_POSTDEV_10_5H_Q0100CC37_026.zzl
Ran at 13:28:26 on 16/11/2023
Ended at 14:30:00 on 16/11/2023
Start Time: 0.000 hrs
End Time: 25.000 hrs
Timestep: 0.5 secs

Current Model Time: 25.00 hrs
Percent Complete: 100 %

Recommendations

Future flood modelling and hydrology in this area should consider the following:

- Details of any biodiversity net gain (BNG) should be considered in terms of flood risk before development. It is assumed that any BNG works will be located outside the flood extents or design to ensure no change to flow paths or floodplain capacity.
- Details of the upstream reservoir operation should be considered further to understand if they could alter and attenuate the flood flows more than suggested by the FARL estimates.
- A more detailed representation of the distribution or tributaries and inflows away from Staplefield ICW as flood mapping across the wider catchment was not focus of this study.

D. Surface Water Drainage

Drawing 23539_4_13 - SURFACE WATER MANAGEMENT provided in the accompanying digital handover and reproduced here.

NOTES

1. ALL DIMENSIONS AND ALL LEVELS IN METRES UNLESS NOTED OTHERWISE.
2. DRAWING TO BE READ IN CONJUNCTION WITH ASSOCIATED DESIGN DOCUMENTS.

LEGEND

- OWNERSHIP BOUNDARY
- SITE BOUNDARY (3.400ha)
- PROPOSED ELEVATION CONTOURS (0.5m INTERVALS)
- EXISTING DRAIN
- EXISTING CULVERT
- RISING MAIN (TBC BY GTB)
- PIPEWORK
- 1:2 CUT/FILL SLOPE TO EXISTING GROUND
- ACCESS ROAD
- 1:2 CUT SLOPE
- TREATMENT CELL (VEGETATED)
- 1m WIDE SLOPED ACCESS RAMP
- ROCK MATTRESS
- SLOPE TO EXISTING GROUND
- FLOOD MITIGATION BASE AREA
- 52.50 PROPOSED LEVELS
- 51.55 EXISTING LEVELS
- MH PROPOSED MANHOLE
- TREATMENT CELL AREA (12,889m²)
- FLOOD MITIGATION BASE AREA (930m²)
- GRASSED AREA (4,928m²)
- GRASSED AREA (5,477m²)
- GRASSED AREA (2,703m²)
- GRAVELLED AREA - FLOWS INTO ICW TREATMENT CELLS (2,555m²)
- GRAVELLED AREA - FLOWS TO SURROUND GRASSED AREAS (1,853m²)

TREATMENT CELL AREAS:
CELL 1 = 487m²
CELL 2 = 5,399m²
CELL 3 = 4,418m²
CELL 4 = 2,585m²
TOTAL CELL AREA = 12,889m² = 1.289 ha
FLOOD MITIGATION VOLUME = 3480m³

CURRENT VERSION INFORMATION

Initial Status or WIP

XXXXXX.

DATE	ORIG	CHKD	REVD	APPR	REV	STS	REASON FOR ISSUE
12/01/24	JF	AC	AC	AC	P02	S3	REVIEW AND COMMENT
24/11/23	JF	AC	AC	AC	P01	S3	REVIEW AND COMMENT

Southern Water
Southern House
Yeoman Road
Worthing
West Sussex
BN13 3NX
telephone (01903) 264444
fax (01903) 691435

GTb JOINT VENTURE
Galliford Try Infrastructure Limited
and Binnies UK Limited
Joint Venture,
Lewes Road, Falmer,
Brighton, East Sussex,
BN1 9PY

VESI ENVIRONMENTAL
Integrating our water back into nature

SBN SOLUTIONS BY NATURE
A Binnies-Salix Joint Venture

PROJECT TITLE
STAPLEFIELD WTW

DRAWING TITLE
SURFACE WATER MANAGEMENT

SITE UNIT MNEMONIC	SITE UNIT NO.	MASTER SIZE
STPFIELD	101464	A1

PRN	SCALE	STATUS
752214	1:500	S3

S.W. DRAWING NO.	REVISION
752214-UAX-ZZ-ZZ-DR-EN-00013	P02

Southern Water
Integrated Constructed Wetland

Technical Note: Greenfield Runoff Rates

January 2024

1. INTRODUCTION

Greenfield runoff-rates have been calculated for a site proposed for a wetland treatment facility adjacent to Staplefield WTW. An assessment of the impact on greenfield runoff rates post-development has also been made.

2. EXISTING GREENFIELD RUNOFF-RATES

Staplefield Greenfield runoff rates have been calculated using three methods:

1) IoH124 report
2) FEH method
3) ReFH2 rainfall runoff (plot scale equations using FEH22 rainfall)

Methods 1 and 2 have been carried out using the tool provided by the UK SUDS website: https://www.uksuds.com/. The *pro forma* are included in Appendix A. The greenfield runoff rates are calculated for the entire site area (3.13 ha) and can then be calculated for each component of the site (wetland treatment cell, flood mitigation pond, gravelled areas) on a *pro rata* by area basis if required (refer to Appendix B). The site layout is provided in Dwg. no. 23539_4_13 SURFACE WATER MANAGEMENT; refer to Appendix C. The ReFH2 method uses plot scale calculations using catchment descriptors from a small nearby catchment imported into the ReFH2.3 software and then similarly calculated on a *pro rata* by area basis. The greenfield runoff rates for the whole site are presented in Table 1 below and are attributable to the existing site, which is an agricultural field:

Table 1: Greenfield runoff rates

Return Period [years]	IoH124	FEH	ReFH2
	l/s	l/s	l/s
1	15.9	21.1	18.5
30	43.1	57.2	44.4
100	59.8	79.3	56.2

3. POST-DEVELOPMENT RUNOFF-RATES

The agricultural field drains southwards into the River Ouse. The creation of a treatment wetland will effectively create attenuation ponds into which the majority of the rainfall across the site will end up, either directly or through runoff. Around 20% of the site will continue to drain without attenuation into the River Ouse, the majority of which will be grassed, with a small proportion gravelled. Gravelled areas will be permeable, which will continue to allow infiltration, so its impact on runoff rates will be small. It can therefore be inferred that runoff rates to the River Ouse will be below the existing greenfield runoff rates once the treatment wetland is constructed.

APPENDIX A.1: UK SuDS IoH124

hrwallingford

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Timothy Paddison
Site name:	Stapleford
Site location:	Cuckfield Road

Site Details

Latitude:	51.03170° N
Longitude:	0.17567° W
Reference:	808248419
Date:	Dec 01 2023 15:29

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Runoff estimation approach

IH124

Site characteristics

Total site area (ha): 3.1345

Methodology

Q_{BAR} estimation method:	Calculate from SPR and SAAR
SPR estimation method:	Calculate from SOIL type

Soil characteristics

	Default	Edited
SOIL type:	4	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.47

Hydrological characteristics

	Default	Edited
SAAR (mm):	840	840
Hydrological region:	7	7
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

Default Edited

Q_{BAR} (l/s):	18.73	18.73
1 in 1 year (l/s):	15.92	15.92
1 in 30 years (l/s):	43.08	43.08
1 in 100 year (l/s):	59.75	59.75
1 in 200 years (l/s):	70.05	70.05

APPENDIX A.2: UK SuDS FEH

hr wallingford

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Timothy Paddison
Site name:	Stapleford
Site location:	Cuckfield Road

Site Details

Latitude:	51.03170° N
Longitude:	0.17567° W
Reference:	2485487649
Date:	Dec 01 2023 16:04

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha):	3.1345

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	28
BFI / BFIHOST:	0.337
Q_{MED} (l/s):	16.77
Q_{BAR} / Q_{MED} factor:	1.14

Hydrological characteristics

	Default	Edited
SAAR (mm):	840	840
Hydrological region:	7	7
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates

Default Edited

Q_{BAR} (l/s):	19.06	24.85
1 in 1 year (l/s):	16.2	21.12
1 in 30 years (l/s):	43.83	57.16
1 in 100 year (l/s):	60.79	79.28
1 in 200 years (l/s):	71.27	92.94

Appendix B: Staplefield WTW Components

SUDS Tool - IoH124:

Event (1 in x years)	Site Area (3.13 ha)		Cell 1	Cell 2	Cell 3	Cell 4	Grassed	Gravel to treatment	Gravel to grass	Flood Mitigation
Areas (ha):			*0.0482*	*0.5399*	*0.4418*	*0.2585*	*1.3108*	*0.2555*	*0.1852*	*0.093*
	l/s	l/s/ha	l/s	l/s	l/s	l/s	l/s	l/s	l/s	l/s
1	15.92	5.1	0.24	2.74	2.24	1.31	6.66	1.30	0.94	0.47
30	43.08	13.7	0.66	7.42	6.07	3.55	18.02	3.51	2.55	1.28
100	59.75	19.1	0.92	10.29	8.42	4.93	24.99	4.87	3.53	1.77
200	70.05	22.3	1.08	12.07	9.87	5.78	29.29	5.71	4.14	2.08

SUDS Tool - FEH method:

Event (1 in x years)	Site Area (3.13 ha)		Cell 1	Cell 2	Cell 3	Cell 4	Grassed	Gravel to treatment	Gravel to grass	Flood Mitigation
Areas (ha):			*0.0482*	*0.5399*	*0.4418*	*0.2585*	*1.3108*	*0.2555*	*0.1852*	*0.093*
	l/s	l/s/ha	l/s	l/s	l/s	l/s	l/s	l/s	l/s	l/s
1	21.12	6.7	0.32	3.64	2.98	1.74	8.83	1.72	1.25	0.63
30	57.16	18.2	0.88	9.85	8.06	4.71	23.90	4.66	3.38	1.70
100	79.28	25.3	1.22	13.66	11.17	6.54	33.15	6.46	4.68	2.35
200	92.94	29.7	1.43	16.01	13.10	7.66	38.87	7.58	5.49	2.76

ReFH2 - plot scale equations

Catchment Area* = *89.75* *ha* *Catchment:* *527800,127250*

Event (1 in x years)	Site Area		Cell 1	Cell 2	Cell 3	Cell 4	Grassed	Gravel to treatment	Gravel to grass	Flood Mitigation
Areas (ha):			*0.0482*	*0.5399*	*0.4418*	*0.2585*	*1.3108*	*0.2555*	*0.1852*	*0.093*
	l/s	l/s/ha	l/s	l/s	l/s	l/s	l/s	l/s	l/s	l/s
1	530	5.9	0.28	3.19	2.61	1.53	7.74	1.51	1.09	0.55
30	1270	14.2	0.68	7.64	6.25	3.66	18.55	3.62	2.62	1.32
100	1610	17.9	0.86	9.69	7.93	4.64	23.51	4.58	3.32	1.67
200	1840	20.5	0.99	11.07	9.06	5.30	26.87	5.24	3.80	1.91

Appendix C: Site Layout

E. Digital Handover

The following digital files have been provided together with this report:

The site plan and scheme drawings listed in Appendix A

22296 Topographic Site Survey

2296- Channel Survey

Staplefield Hydrology files of the WINFAP and ReFH2 projects to support Appendix B.

And

The packaged modelling files as set out in Table E.1

Table E.1 : Packaged Model Handover files

Version No.	Scenario	Modeller	Status/Scenario	Event (Return period)	Flood Modeller Control File (IEF)	BC Database (bc_dbase)	TUFLOW Control File (TCF)	TUFLOW Geometry File (TGC)	TUFLOW Boundary File (TBC)	TUFLOW Materials File (TMF)	FMP Channel File (DAT)
26	Baseline Manning N +20%	VV	10.5-hour	Q0100CC37	Staplefield_Base_10_5h_Q0100cc37_026_MN+20.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~s3~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026_+20MN.dat
26	Baseline Manning N -20%	VV		Q0100CC37	Staplefield_Base_10_5h_Q0100cc37_026_MN-20.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~s3~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026_-20MN.dat
26	Baseline +DSBDY	VV		Q0100CC37	Staplefield_Base_10_5h_Q0100cc37_026_DSBDY+.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~s3~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026_DSBDY+.dat
26	Increased inflows	VV		Q0100CC37	Staplefield_Base_10_5h_Q0100cc37_026_inflow.ief	bc_dbase_Staplefield_Sen001_10_5h_013.csv	Staplefield_~s1~_~s2~_~s3~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	Baseline	VV		Q0002	Staplefield_Base_10_5h_Q0002_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	Baseline	VV		Q0010	Staplefield_Base_10_5h_Q0010_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	Baseline	VV		Q0030	Staplefield_Base_10_5h_Q0030_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	Baseline	VV		Q0050	Staplefield_Base_10_5h_Q0050_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat

26	Baseline	VV	Q0100	Staplefield_Base_10_5h_Q0100_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	Baseline	VV	Q1000	Staplefield_Base_10_5h_Q1000_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q0002	Staplefield_PostDev_10_5h_Q0002_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q0010	Staplefield_PostDev_10_5h_Q0010_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q0030	Staplefield_PostDev_10_5h_Q0030_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q0050	Staplefield_PostDev_10_5h_Q0050_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q0100	Staplefield_PostDev_10_5h_Q0100_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat
26	PostDev	VV	Q1000	Staplefield_PostDev_10_5h_Q1000_026.ief	bc_dbase_Staplefield_10_5h_013.csv	Staplefield_~s1~_~s2~_~e1~_026.tcf	Staplefield_026.tgc	Staplefield_020.tbc	Staplefield_001.tmf	Staplefield_026.dat

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