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Flood Risk Modelling and Mapping

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Use of Mapping within this Document

Glossary of Terms

- CC – Return period inclusive for the predicted effects of Climate Change
- 1D – One-Dimensional
- 2D – Two-Dimensional
- AOD – Above Ordnance Datum (0m sea level, Newlyn, UK)
- Channel Cross Section – A one-dimensional view of a river channel
- Critical Storm – A storm that produces peak run off in the watershed
- Culvert – A device used to channel water
- Defended– A scenario in which river defences are used
- ESTRY Software – One-Dimensional hydraulic model – Representation of watercourses
- FRA – Flood Risk Assessment
- FEH – Flood Estimation Handbook
- Fluvial – Referring to the processes associated with rivers and streams
- GIS – Geographic Information System
- Hydraulic Model – The process of analysing the interaction of water and the connected environment
- Hydrology – The calculation of catchment based flow rates over time
- Inflow – Source of water within a modelled domain
- FMP Software – One-Dimensional hydraulic model – Representation of watercourses
- FMP-TUFLOW – Hydraulic linking program between FMP and TUFLOW (1D-2D)
- LiDAR – Light Detection and Ranging, remote sensing technology to measure distance
- Outflow – The process of water conveyance from a modelled domain
- Overtopping – The process of water being conveyed over a specific location
- Q100 – 1% probability fluvial event
- Q1000 – 0.1% probability fluvial event
- Q100CC – 1% fluvial event with the predicted effects of climate change
- QMED - The median of the set of annual maximum flood data (AMAX)
- SuDS – Sustainable Drainage Systems
- TUFLOW Software – Two-Dimensional hydraulic model – Representation of floodplain
- TUFLOW FV Software – Finite Volume hydraulic model
- Undefended – A scenario in which river defences are ignored

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1 Introduction

1.1 Overview

Edenvale Young Associated Ltd (EVY) has been appointed by Chalgrove Parish CouncIL to undertake a hydrological analysis and hydraulic modelling study of Chalgrove village in Oxfordshire. This encompasses the production of flood risk mapping. EVY were previously responsible for the development of a hydraulic model of the Chalgrove Brook for the Chalgrove Flood Action Group and the work described in this document builds upon the knowledge base acquired during the previous project. In particular, the modelling has been updated to include additional topographic survey which was collected by EVY during the summer of 2016 and further consideration has been given to the impact of housing development on Chalgrove Airfield.

The report covers the the following topics.'-

- Historical Flooding
- Planning policy
- Hydrological analysis
- Hydraulic modelling
- Flood Risk Mapping
- Conclusions.

The objective of the report is to improve the understanding of flood risk within the community. A range of return periods has been modelled commensurate with planning policy for Annual Exceedance Probabilities (AEP) of 1% (1-in-100 year event) and 0.1% (1-in-1000 year event) that define Environment Agency (EA) Flood Zones 3 and 2 respectively. In addition, 1% AEP plus 20%, 35% and 70% climate change have also been modelled. The results include: flood extent, flood depth and flood hazard.

It should also be noted that a number of housing projects have been brought forward for development. These are on the periphery of the village and include the airfield to the north of the village. It is anticipated that a better understanding of flood risk will improve decision making in relation to these, and other, developments.

1.2 Environment Agency Flood Mapping

Current EA flood mapping[1] identifies that a proportion of the village and areas downstream of the village are classified as being within Flood Zone 3. This is due to a risk of flooding from the Chalgrove Brook (see Figure 1).

This classification has a significant impact on development within the Parish. Flood Zone 3 is generally designated to the south of the High Street, affecting properties at the downstream end of the village between the Chalgrove Brook (known locally as the Back Brook) and the High Street. This includes properties on the High Street itself, Mill Lane, Flemming Avenue (and side roads), Quartermain Road, Adeane Road, Langley Road and Hardings. In addition, Church, Lane at the south-eastern end of the village is shown as being affected by flooding.

Figure 2 shows an extract from the EA *risk of flooding from surface water* dataset and includes the area to the north of Chalgrove at Chalgrove Airfield. Surface Water flooding relates to non-fluvial sources which includes run off from the rural and urban environment. The mapping shows flooding along the High Street and in other areas where heavy rainfall is likely to cause surface water flooding. Notably, Chalgrove Airfield is shown as relatively free from surface water flooding. The implication from this mapping is that the existing risk of surface water flooding to Chalgrove Airfield is generally Very Low.

[1] Flood Mapping for Rivers and Sea

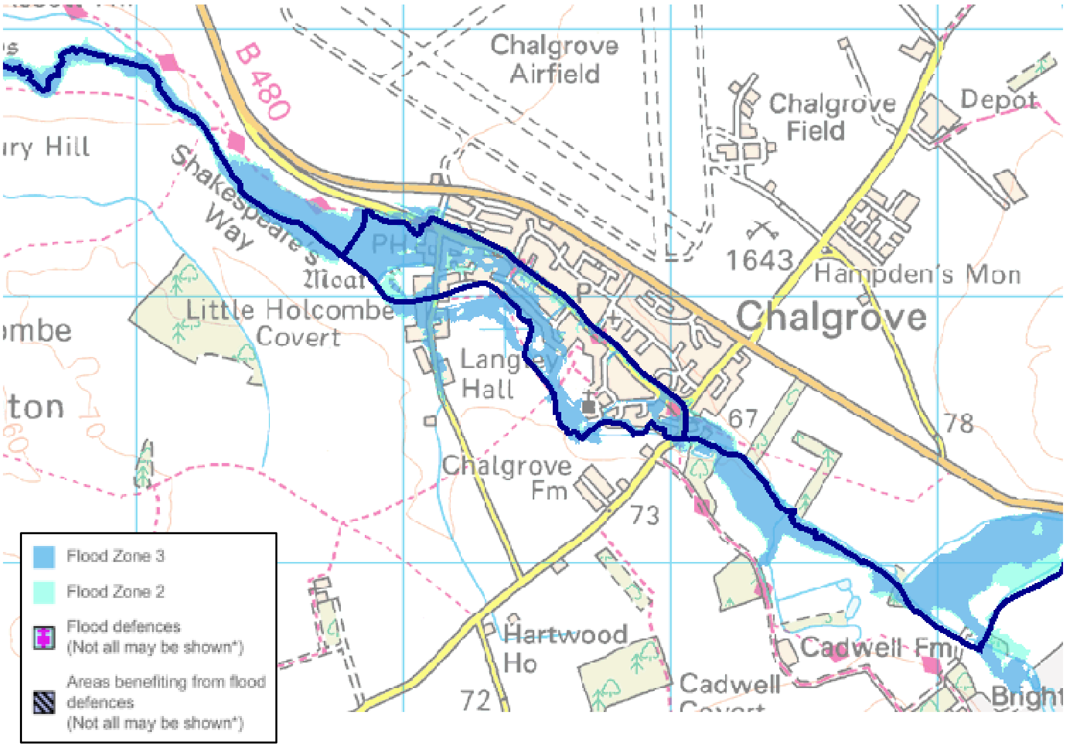


Figure 1 – EA Flood Map at location of site (click [here](#) for source data)

Figure 2| Surface Water Flood Risk (click [here](#) for source data)

1.3 Planning Context

The National Planning Policy Framework (NPFF), includes guidance associated with development in Flood Zones 1, 2 and 3. In general, development within Flood Zone 1 (Low Risk of Flooding) is acceptable for the majority of development types. This includes Highly Vulnerable developments such as hospitals, police and ambulance stations. Flood Zone 1 encompasses the area to the north of the High Street, Chalgrove airfield and upstream pockets between the High Street and Chalgrove Brook.

For developments in Flood Zone 2 or 3, NPPF gives details associated with the suitability of different forms of development. This is summarised in Table 1. Table 2 gives the vulnerability classifications for various types of development. By cross-referencing this information it is possible to determine which classifications of development are appropriate and where an Exception Test may be required. Development within Flood Zone 3b (the functional floodplain) is unacceptable except for essential infrastructure.

It should be noted that the Sequential Test is used in the first instance to direct developments away from locations at risk of flooding. The application of the Sequential Test is not considered in this report.

Table 1| Flood Risk Vulnerability Classification

	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	E	✓	✓	✓
Zone 3a	E	✗	E	✓	✓
Zone 3b	E	✗	✗	✗	✓*

E Exception Test Req'd ✓ Development is appropriate ✗ Development should not be permitted.

NPPF states that a Flood Risk Assessment (FRA) will be required to support a planning application where a development is:-

- in flood zone 2 or 3 including minor development and change of use
- more than 1 hectare (ha) in flood zone 1

- less than 1 ha in flood zone 1, including a change of use in development type to a more vulnerable class (e.g. from commercial to residential), where they could be affected by sources of flooding other than rivers and the sea (e.g. surface water drains, reservoirs)
- in an area within flood zone 1 which has critical drainage problems as notified by the EA

For example, a housing development which is classified as More Vulnerable within Flood Zone 1 would be suitable whereas a housing development within Flood Zone 3a would require an exception test and FRA to justify the development. However, More Vulnerable development in the Flood Zone 3b – which is generally considered to be the Functional Floodplain – would not be permitted.

It should be noted that the latest guidance contained in the NPPF should always be consulted in relation to the preparation of a planning application.

Table 2| Vulnerability Classification

Vulnerability Classification
Essential Infrastructure • Essential transport infrastructure (including mass evacuation routes) which has to cross the area at risk. • Essential utility infrastructure which has to be located in a flood risk area for operational reasons, including electricity generating power stations and grid and primary substations; and water treatment works that need to remain operational in times of flood. • Wind turbines.
Highly Vulnerable • Police and ambulance stations; fire stations and command centres; telecommunications installations required to be operational during flooding. • Emergency dispersal points. • Basement dwellings. • Caravans, mobile homes and park homes intended for permanent residential use. • Installations requiring hazardous substances consent. (Where there is a demonstrable need to locate such installations for bulk storage of materials with port or other similar facilities, or such installations with energy infrastructure or carbon capture and storage installations, that require coastal or water-side locations, or need to be located in other high flood risk areas, in these instances the facilities should be classified as 'Essential Infrastructure').

Vulnerability Classification
More Vulnerable • Hospitals • Residential institutions such as residential care homes, children's homes, social services homes, prisons and hostels. • Buildings used for dwelling houses, student halls of residence, drinking establishments, nightclubs and hotels. • Non–residential uses for health services, nurseries and educational establishments. • Landfill* and sites used for waste management facilities for hazardous waste. • Sites used for holiday or short-let caravans and camping, subject to a specific warning and evacuation plan.
Less Vulnerable • Police, ambulance and fire stations which are not required to be operational during flooding. • Buildings used for shops; financial, professional and other services; restaurants, cafes and hot food takeaways; offices; general industry, storage and distribution; non-residential institutions not included in the 'More Vulnerable' class; and assembly and leisure. • Land and buildings used for agriculture and forestry. • Waste treatment (except landfill* and hazardous waste facilities). • Minerals working and processing (except for sand and gravel working). • Water treatment works which do not need to remain operational during times of flood. • Sewage treatment works, if adequate measures to control pollution and manage sewage during flooding events are in place.
Water-Compatible Development • Flood control infrastructure. • Water transmission infrastructure and pumping stations. • Sewage transmission infrastructure and pumping stations. • Sand and gravel working. • Docks, marinas and wharves. • Navigation facilities. • Ministry of Defence defence installations. • Ship building, repairing and dismantling, dockside fish processing and refrigeration and compatible activities requiring a waterside location. • Water-based recreation (excluding sleeping accommodation). • Lifeguard and coastguard stations. • Amenity open space, nature conservation and biodiversity, outdoor sports and recreation and essential facilities such as changing rooms. • Essential ancillary sleeping or residential accommodation for staff required by uses in this category, subject to a specific warning and evacuation plan.

1.4 History of Flooding

A significant area of Chalgrove was flooded during February 2014. Chalgrove Flood Action Group gathered evidence of flooding during this event, a selection of which is reproduced in Figure 3 to Figure 8.

Figure 3| Flooding February 2014

Figure 4| Flooding February 2014

Figure 5| Flooding February 2014

Figure 6| Flooding February 2014

Figure 7| Flooding February 2014

Figure 8| Flooding February 2014

2 Hydrology

2.1 Introduction

In order to undertake detailed hydraulic modelling of the village, design hydrograph inflows were required for:

- Chalgrove Brook u/s bifurcation along Chalgrove High Street (circa. 1 km u/s confluence with Mill Lane Stream).
- Mill Lane Stream u/s confluence with Chalgrove Brook,

Chalgrove Brook drains east-south-east to west-north-west and is bounded in its headwaters by the chalk escarpment of the Chiltern Hills. Passing through the village, it is an essentially permeable (BFIHOST = 0.85; SPRHOST = 14.34) rural catchment of approx. 50 km^2, but does contain the market town of Watlington in its upper catchment.

In contrast Mill Lane Stream is a small and essentially rural catchment (< 2 km^2) draining from south to north joining the Chalgrove Brook just downstream of Chalgrove village, 16km southeast of Oxford. It is a relatively impermeable (BFIHOST = 0.355; SPRHOST = 46.01) catchment within a wider area of chalk downlands.

An FEH analysis was undertaken to establish design flows which included deriving QMED from catchment characteristics and growth curves using standard FEH statistical methods using WINFAP III. Full details of the analysis is provided in the accompanying flood estimation calculation record (see Appendix A).

Unfortunately, the inflows derived from the above analysis, gave rise to flood extents in Chalgrove that were significantly less extensive than the historical information suggested. This is almost certainly because recognised problems with using FEH characteristics in permeable catchments. Accordingly, the use of methods based on the FEH characteristics to establish QMED were set aside (including ReFH 2).

Unfortunately, there is limited local data in the form of gauge records on the Chalgrove Brook to improve confidence in the estimation of QMED and the growth curve. Accordingly, an analysis of Peak Over Threshold (POT) data was used to give a more realistic estimate QMED and this is described in more detail in Section 2.2.1.

In consultation with the EA it was also agreed that the growth curve used by JBA at Watlington would be used in lieu of the FEH growth curve. Design hydrograph shape has been calculated using the ReFH rainfall-runoff model. The hybrid method (scaling of design hydrographs) has been applied to derive design hydrographs for return period flows between 2 years and 1,000 years inclusive.

Table 3| JBA Growth Curve at Watlington (Flood Peaks m3/s

Site Code	1 in 2	1 in 5	1 in 20	1 in 30	1 in 50	1 in 75	1 in 100	1 in 100cc	1 in 1000
CB02	0.45	0.65	0.97	10.7	1.22	1.36	1.46	1.75	2.55

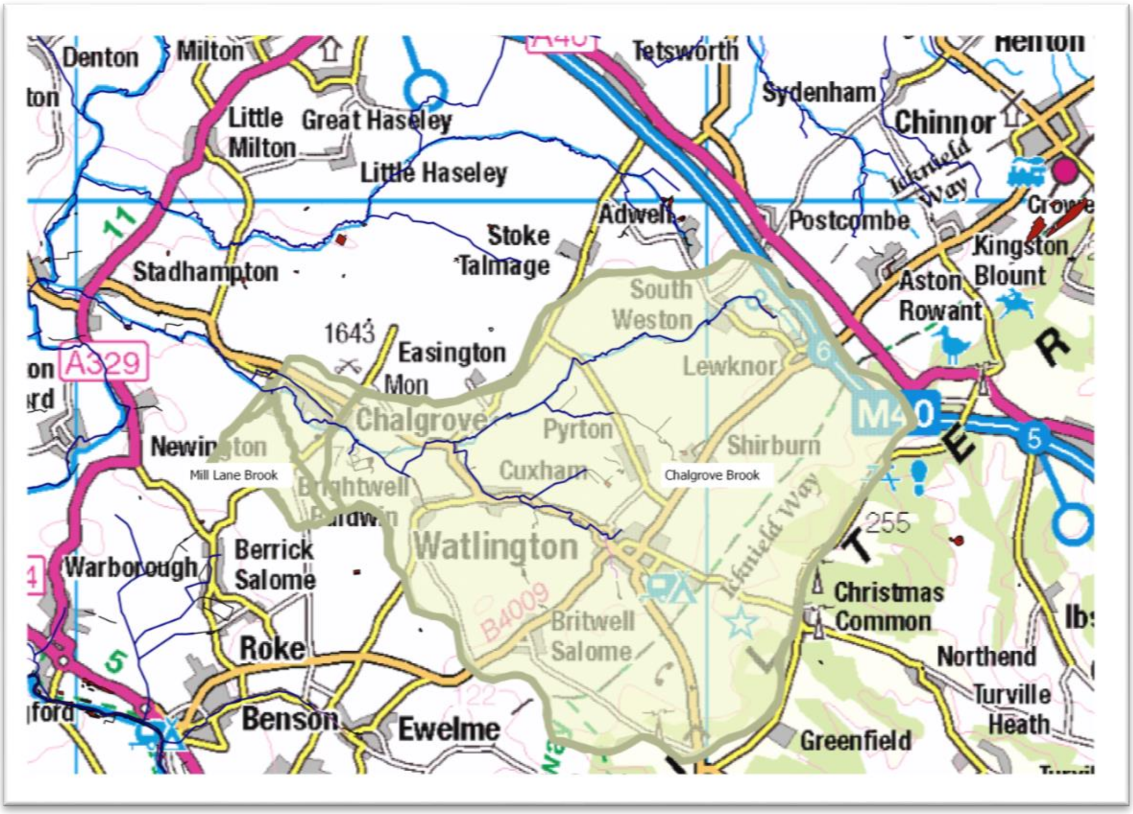


Figure 9| Chalgrove Brook Catchment

2.2 Design estimates

2.2.1 Chalgrove Brook hydrograph design

As noted in the previous section QMED was estimated using the current catchment descriptor equation and peaks over threshold data. Three years of observed levels were available from a gauging station located upstream of the Berrick Road bridge by the Grange in Chalgrove. Levels have been recorded just before the flow splits between the main channel and the High Street channel.

A rating curve was generated from the ISIS (now Flood Modeller Pro) model of the watercourse, from which a rating equation has been calculated, which may be expressed as:

- $Q = a(h - b)^c$

With:

- $a = 0.0179$
- $b = 65.5908$
- $c = 6.081$

This rating equation has been used to convert the level-time series data into a flow-time series, which is required for the generation of a three year peaks-over-threshold (POT) set. It is of note that the flows calculated in this way would have been higher if a lower value of roughness than 0.06 had been used.

POT data comprise a series of flood peaks which are bigger than a selected threshold. They provide a more complete description of flood behaviour than annual maximum data (from which QMED is normally derived). They can be useful in estimating QMED (even though this is defined as the median of the annual maxima), particularly where there are only short periods of record, such as in this case.

Events selected for inclusion in a POT series must be independent of one another; that is, two peaks must be separated by at least three times the average time to rise and the minimum discharge in the trough between two peaks must be less than two thirds of the discharge of the first of the two peaks. The events selected here meet these criteria.

QMED is estimated by noting the number of years of complete data available (in this case, 3) and looking up values of *i* (a ranked position in the POT series) and *w* (a weighting factor between two POT values) in table 2.1 of the Flood Estimation Handbook Volume 3. QMED is then estimated as a weighted average of the i^{th} and $(i+1)^{th}$ highest floods from the POT series:

- QMED = $w\,Q_i + (1 - w)Q_{i+1}$

Where

- Q_i is the peak flow of the i^{th} ranked event in the POT series,
- Q_{i+1} is the flow for the $(i+1)^{th}$ highest event from the series. For a three year record, $i = 2$
- $w = 0.1$ and therefore the peak flows from the second and third largest events in the series are required to calculate QMED.

The following POT events have been used in the QMED estimation:

Table 4| Highest Three Peak Flow Estimates in Three Year Rated Flow Record at Chalgrove Flood Warning Station

POT number	Flow (m^3/s)	Date
1	3.17	07/02/2014
2	2.3	15/02/2014
3	2.16	19/03/2013

Thus

- QMED = 0.1 x 2.3 + (1 – 0.1) x 2.16 = 2.174m^3/s.

This can be compared against an ReFH estimate of QMED = 1.4m^3/s (from catchment descriptors) and 0.312m^3/s from ReFH

2.2.2 Mill Lane Stream hydrograph design

No donor transfer was applied to Mill Lane Stream as neighbouring gauged catchments were considered hydrologically dissimilar (based on consideration of AREA and BFIHOST). The Kjeldsen urban adjustment method was applied to derive final estimates of QMED for this catchment.

The GL distribution was selected to estimate the growth curve for Mill Lane Stream based on its preferential goodness-of-fit score (z=-0.0044). After review for discordancy and heterogeneity, the pooling group remained heterogeneous (H2=2.1963).

2.2.3 Design Estimates

Final statistical method estimates of peak flow are given as follows:

Table 5| Statistical Method Peak Flow Estimates by Return Period (years)

	1 in 2	1 in 5	1 in 30	1 in 50	1 in 100	1 in 1000
Mill Lane Stream	0.400	0.570	0.949	1.09	1.30	2.37
Chalgrove Brook	2.174	2.904	4.313	4.776	5.470	8.463

For both Mill Lane Stream and Chalgrove Brook, the ReFH rainfall-runoff model was applied based on catchment descriptors to estimate hydrograph shape. The resultant design hydrographs were scaled such that their peak flows matched those as defined by the statistical method analysis. It should be noted that the estimates have been agreed with the EA.

2.3 Climate Change

The NPPF contains guidance on the application of climate change allowances. For river systems, this involves applying a percentage increase to the model inflows in order to account for the influence of climate change in the future. The allowances required will vary depending on the type of the development under consideration and their geographic location. In this context, Chalgrove is located within the Thames basin area.

Table 1 of the NPPF, which is reproduced below as Table 6, gives the climate change allowances appropriate to the Thames basin. NPPF requires that housing development (More Vulnerable) with an expected lifespan of 100 years located in Flood Zone 3a would require the application of the higher central and upper end estimates of 35% and 70%.; a housing development located in Flood Zone 2 would require the application of central and higher central of 20% and 35%.

Accordingly, the model results given later in this report have been presented for 1 in 100 year return period with climate change allowances of 20%, 35% and 70%. More information on the application of climate change to other development types is given in the NPPF.

Table 6| Climate Change Allowances

Allowance category	Total potential change anticipated for the '2020s' (2015 to 2039)	Total potential change anticipated for the '2050s' (2040 to 2069)	Total potential change anticipated for the '2080s' (2070 to 2115)
Upper end	25%	35%	70%
Higher central	15%	25%	35%
Central	10%	15%	25%

3 Hydraulic Modelling

3.1 Overview

The modelling undertaken as part of this project is based upon a 1D Flood Modeller Pro (FMP) model developed in 1998 and obtained from the EA. As part of the flood map challenge undertaken for Chalgrove Flood Action Group in 2015, EVY converted the 1D only model into a linked 1D-2D FMP-TUFLOW model using survey data and LiDAR supplied by the EA. However, this model did not include survey information for the High Street Brook; the watercourse was instead modelled in 2D as a series of Z-Lines, with layered flow constrictions representing driveways to property on the High Street. This representation is not considered to be ideal and potentially misrepresented flooding to the High Street.

In summer 2016 EVY surveyed the High Street Brook, including the crossings over the channel. These have now been explicitly represented in 1D within the FMP. In addition, the survey included ditches and other water conveyance routes upstream and downstream of the village. The updated extent of the model is shown in Figure 10.

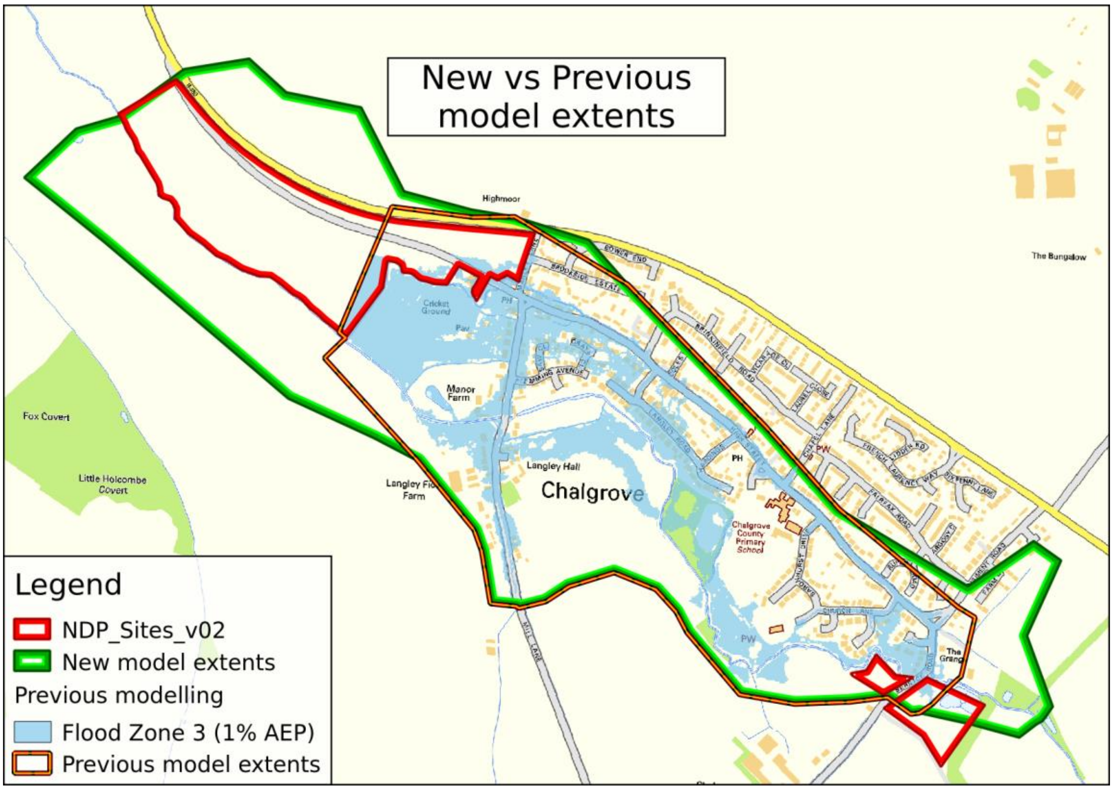


Figure 10| Changes in Model Extent

3.2 Software

The flood risk mapping was produced using Flood Modeller Pro (FMP) (version 6.7.0.110) in conjunction with TUFLOW (version 2016-03-AA-iDP-w64). The generally accepted accuracy for models of this nature is in the order of ±0.150m.It is worth noting that whilst it is convenient to make comparisons between return periods it should be recognised that small differences in depth or water levels given in by the hydraulic modelling results may not be physically measurable.

3.3 Model Scenarios

The following modelling scenarios have been considered for the purposes of this flood risk mapping.

Table 7|Scenario and Model List

Source of Flood Risk	1 in 100 1% AEP	1 in 100 +20% cc 1% AEP	1 in 100 +35% cc 1% AEP	1 in 100 +70% cc 1% AEP	1 in 1000 0.1% AEP
Fluvial	x	x	x	x	x
Surface Water	x	-	-	-	x

3.4 1D modelling

The High Street Brook and crossings have been explicitly schematised in 1D. The majority of driveways across this channel were modelled as bridge units with spills in 1D. Some bridges at the downstream end of the High Street Brook were not included in the model as they were deemed hydraulically insignificant. This mainly relates to bridges which have flat bridge decks without solid parapets. Interpolates have also been added to increase the stability of the model.

A cross section and structure survey was carried out for Mill Lane Stream by EVY . The Mill Lane Stream has been added to the FMP model. The culvert on the Mill Leat has been represented in ESTRY.

The channel profile downstream of the High Street Brook (past the High Street bridge) has been based on the surveyed cross-section of the Chalgrove Brook and configured to match bed elevation of the watercourse at corresponding locations along the river. All such notes are clearly marked as comments in the FMP model. A sweetening flow boundary for the High Street Brook was added to prevent the channel from drying.

3.5 2D Modelling

As is standard practice, bank lines were placed at the top of the channel with elevations set to match the bank elevations in the FMP model. The placement of the banklines was selected to coincide with the high points visible in the LiDAR. For areas where nodes were duplicated but additional survey data was available, bed elevations were matched in FMP and bank lines were taken from topographic survey instead of the FMP nodes.

3.6 Roughness

The 1D Manning roughness from the original Chalgrove model (for Chalgrove Brook only) has been changed to match observations made during the field survey. The 1D roughness in the incoming model was 0.03, which was viewed as low according. The roughness was therefore adjusted to match a realistic vegetated channel roughness of 0.06 based upon the observed extent of vegetation within the various channels.

The 1D roughness of the Mill Lane Stream has been set upstream to match vegetated channel roughness at 0.06. A value of 0.04 was used where the channel is a concrete, U shaped and maintained, according to photographs supplied by CFAG.

The 1D roughness of the High Street Brook has been set at 0.02 to match the observations made during the field survey. It is understood that the watercourse is regularly maintained and kept clear of vegetation.

3.7 Model Run Parameters

The following modifications have been made to the default run parameters: -

- Dflood = 5

3.8 Time-step

The model runs with a 2D time-step of 1 second and a 1D time-step (ESTRY and FMP) of 0.5.

This conforms to standard practice.

3.9 Simulation Time

The model run time was 25 hours

3.10 Grid Size

The grid size was 2m.

3.11 Initialisation Assumptions

It has been assumed that: -

- un-surveyed small channels/ditches are dry in the initial conditions.

4 Results

4.1 Fluvial Flooding

Figure 15 to Figure 32 show the results of the hydraulic modelling for the following exceedance probabilities for water level, depth and hazard:

- 1 in 25 years (4 % AEP) (The functional Flood Plain)
- 1 in 100 years (1% AEP)
- 1 in 100 years (1% AEP) + 20% climate change
- 1 in 100 years (1% AEP) + 35% climate change
- 1 in 100 years (1% AEP) + 70% climate change
- 1 in 1000 year (0.1% AEP)

Figure 15 and Figure 16 delineates the extent of Flood Zones 2 and 3, based on these model results. Figure 17 shows the extent of Flood Zone 3b based on a return period of 1 in 25 years. The remaining figures record water level, depth and hazard for each of the return periods considered. It should be noted that the Flood Map does not include the influence of climate change.

Hazard is a measure of the impact on pedestrians of flooding and is a combination of velocity and depth, plus a debris factor. It is considered that shallow fast flowing water can be equally as dangerous as deep slow flowing water. The classification for hazard is divided into the following categories:-

		Colour coding
Low Risk		
Dangerous to some	includes children, the elderly and the infirm	
Dangerous to most	includes the general public	
Dangerous to all	includes emergency services	

Flood hazard is particularly important in relation to the provision of safe access and egress during flooding. Local Authorities are responsible for assessing whether there is safe access and egress to a property during the planning application process. In this context it should be noted that:

- In a 1 in 100 year event the flood hazard on the High Street is classified as dangerous to most people
- In a 1 in 1000 year event the flood hazard on the High Street is classified as dangerous to most people and dangerous to all.

Accordingly, safe access and egress from property between the High Street and the Chalgrove Brook would be compromised during a flood event. The flood mechanisms and chronology for a 1 in 100 year return period flood event are:

1. Surcharging of the culvert downstream of the Mill leat leads to out of bank flow on Mill Lane.
2. Flooding of the land downstream of the Cricket Ground
3. Overtopping of the Chalgrove Brook at Berrick Road leading to rapid flooding of Church Lane and the High Street
4. Flooding to Mill Lane via the High Street
5. Flooding to Langley Road
6. More extensive flooding through Chalgrove

During consultation with Chalgrove Parish Council it was noted that four properties on the High Street were shown as being inundated despite being elevate by approximately 1m above the High Street and the validity of the mechanism was questioned. Figure 11 to Figure 14 shows the progression of flooding in the vicinity of these four properties during the simulation at 11.00, 12.00, 12:45 and 13.45 hours into the simulation. Flooding does not come from the High Street. The properties are inundated from the rear via Langley Lane as a result of out of bank flow from the Chalgrove Brook.

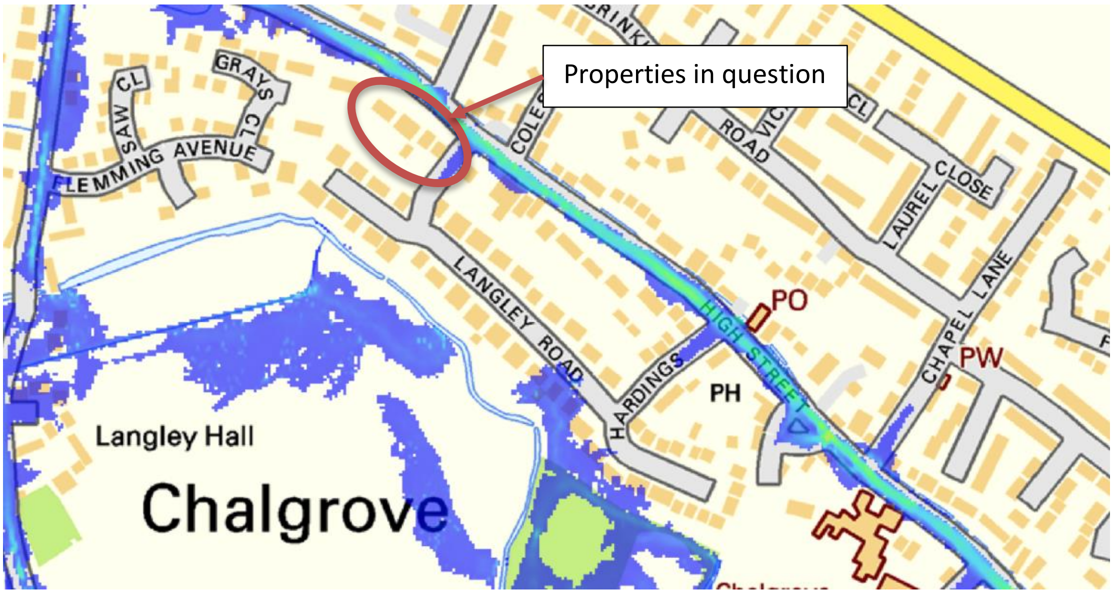


Figure 11| 1 in 100 years (Time 11:30 hours from commencement of model)

Figure 12| 1 in 100 years (Time 12:00 hours from commencement of model)

Figure 13| 1 in 100 years (Time 12:45 hours from commencement of model)

Figure 14| 1 in 100 years (Time 13:45 hours from commencement of model)

4.2 Surface Water Flood Mapping

Direct rainfall modelling using the FMP – TUFLOW software has been used to assess the risk of surface water flooding from the Chalgrove Airfield. The model domain has been extended to incorporate the airfield and a 1 in 100 and 1 in 1000 year rainfall boundary has been applied to the model.

This is a similar approach to that adopted by the EA and shown in Figure 2. However it should be noted that the resolution of the model is significantly higher and the surface water flood maps (see Figure 33 and Figure 34) show depths greater than 0.05m whereas Figure 2 shows flood depths greater than 0.3m.

The results indicate that the southern portion of the airfield drains towards the B480 in a westerly direction with some surface water entering the village along Marley Lane. Provision for conveying flow across Site 11 should be incorporated in the development plan.

Figure 15 | Flood Zones, based on the 1 in 100 year (Flood Zone 3, dark blue) and 1 in 1000 year (Flood Zone 2, pale blue) model results produced as part of this study

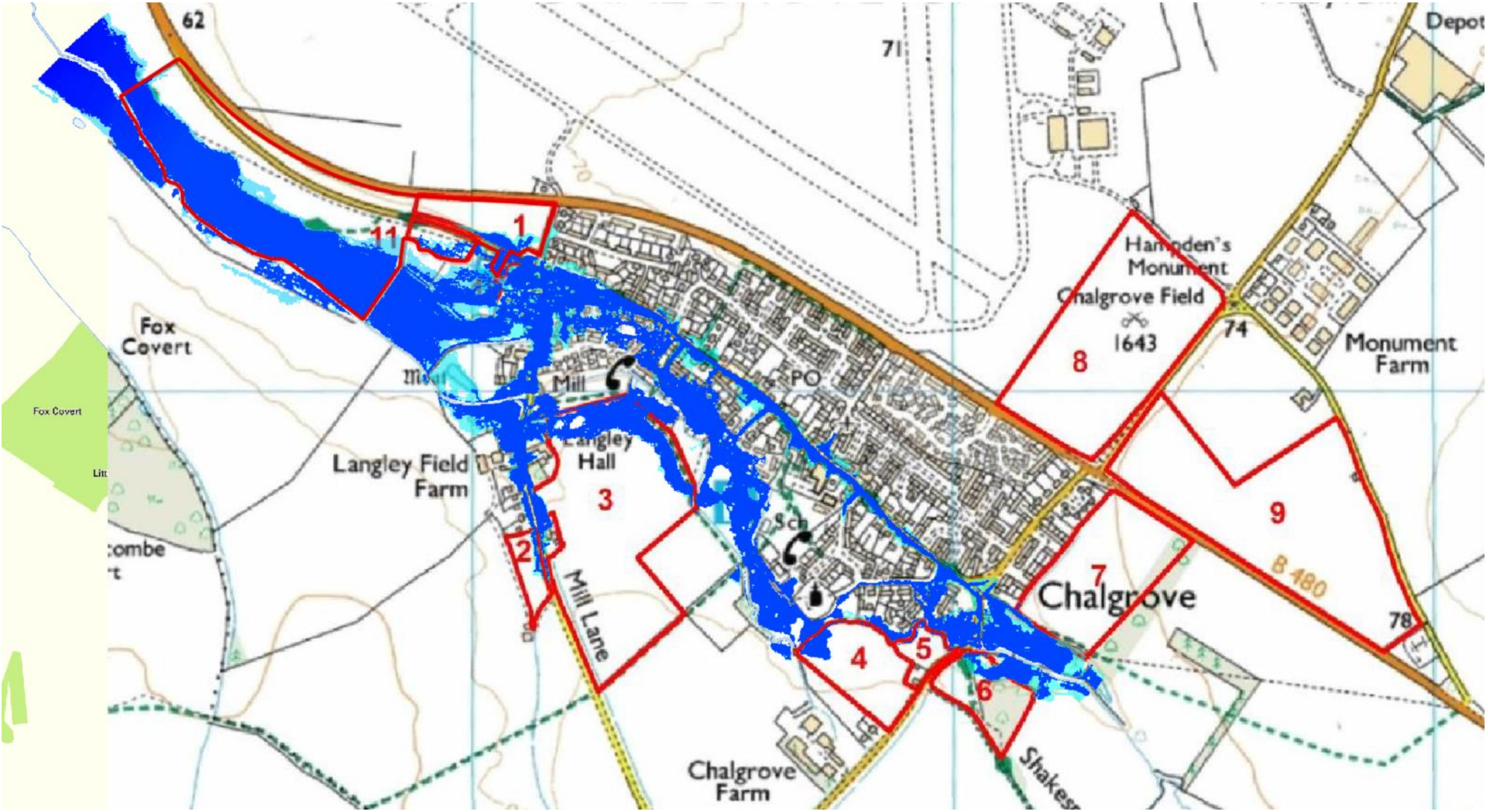


Figure 16| Flood Zones, based on the 1 in 100 year (Flood Zone 3, dark blue) and 1 in 1000 year (Flood Zone 2, pale blue) model results produced as part of this study (with development sites)

Figure 17 | Flood Zone 3b, based on the 1 in 25 year fluvial event

Figure 18 | Model Results showing flood water level for 1 in 100 Year Return Period

Figure 19 | Model Results showing flood depth for 1 in 100 Year Return Period

Figure 20|Model Results showing flood hazard for 1 in 100 Year Return Period

Figure 21 | Model Results showing flood water level for 1 in 100 Year Return Period plus a climate change allowance of 20%

Figure 22 | Model Results showing flood depth for 1 in 100 Year Return Period plus a climate change allowance of 20%

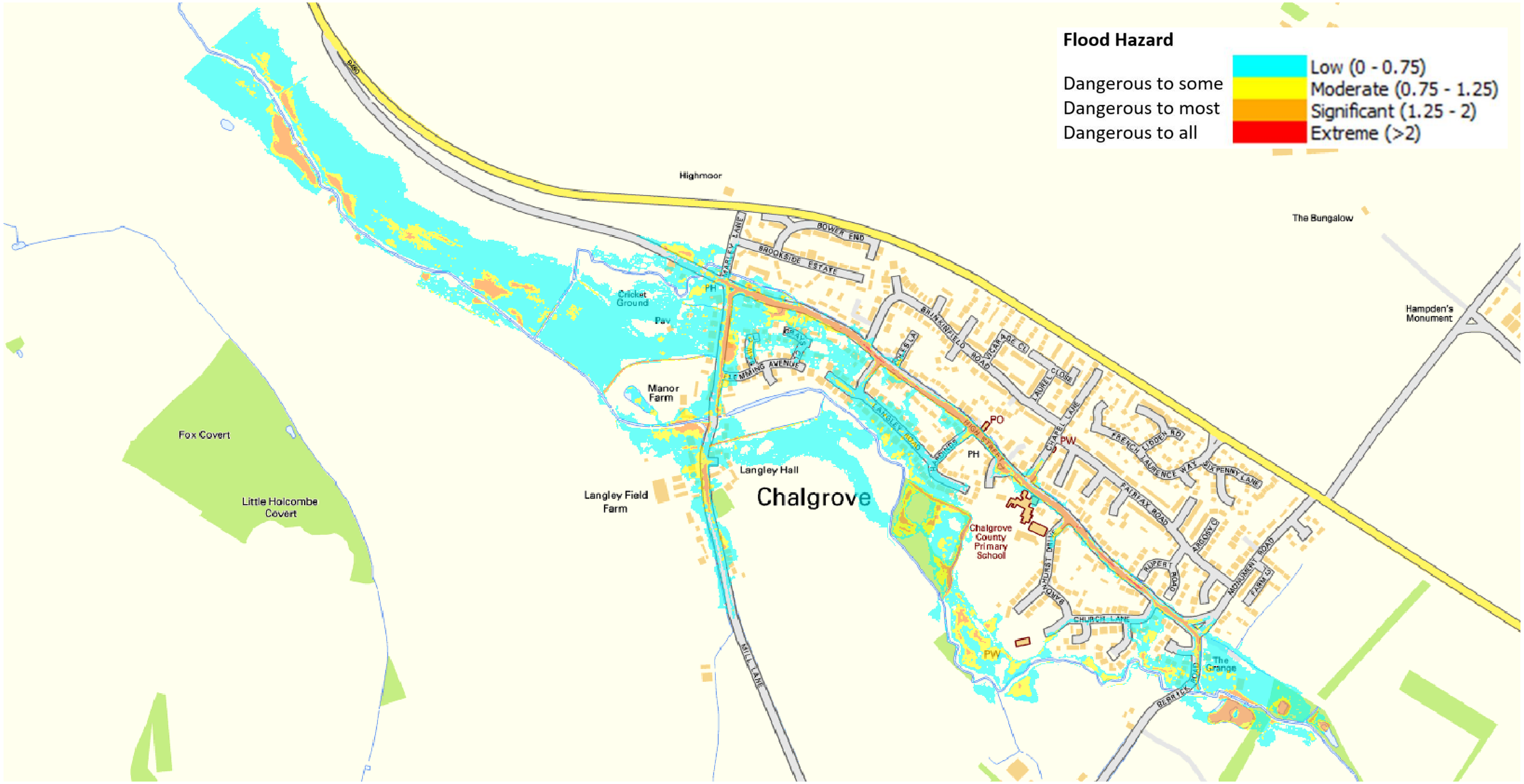


Figure 23 | Model Results showing flood hazard for 1 in 100 Year Return Period plus a climate change allowance of 20%

EdenvaleYoung

Figure 24 | Model Results showing flood water level for 1 in 100 Year Return Period plus a climate change allowance of 35%

Figure 25 | Model Results showing flood depth for 1 in 100 Year Return Period plus a climate change allowance of 35%

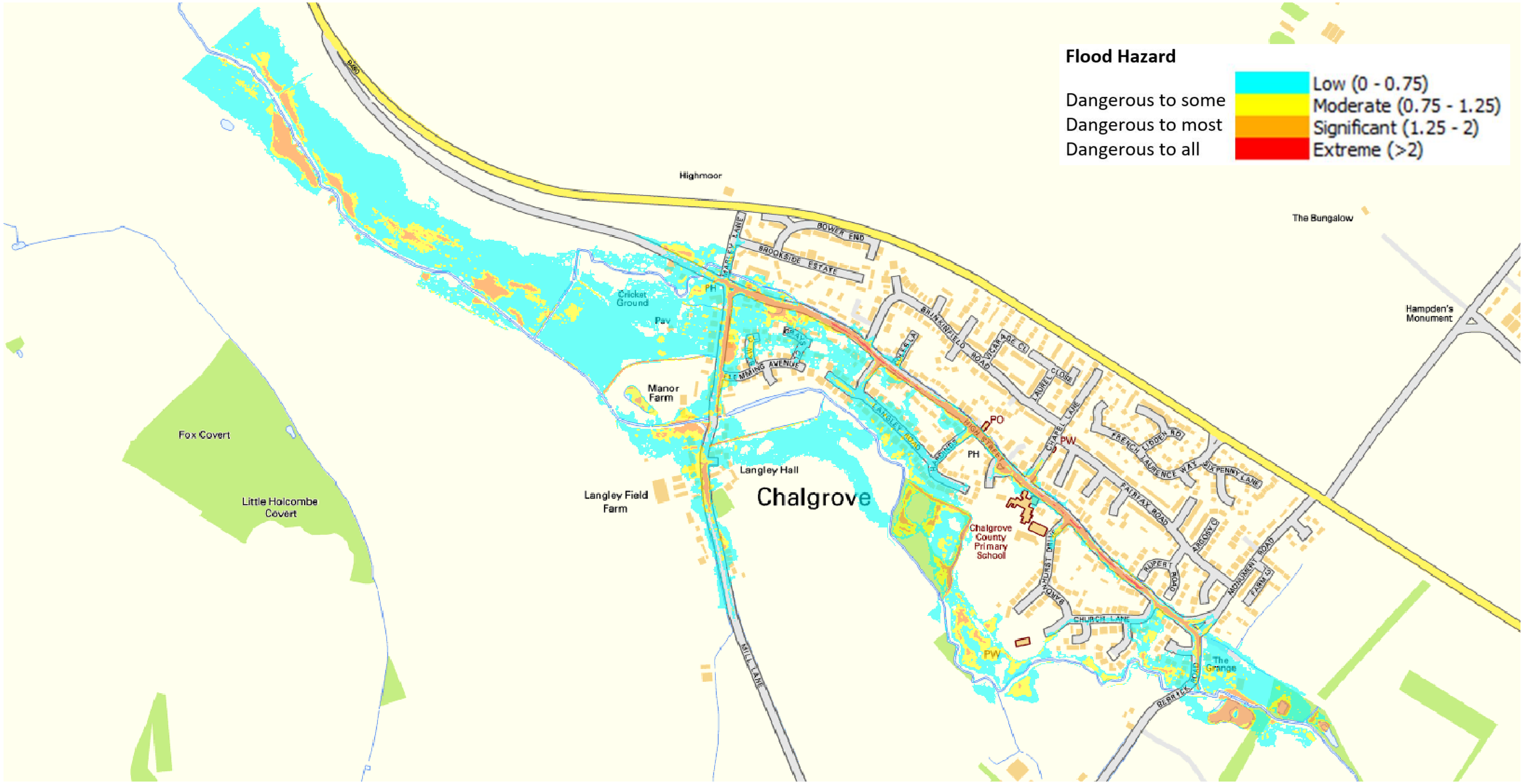


Figure 26 | Model Results showing flood hazard for 1 in 100 Year Return Period plus a climate change allowance of 35%

Figure 27 | Model Results showing flood water level for 1 in 100 Year Return Period plus a climate change allowance of 70%

Figure 28 | Model Results showing flood depth for 1 in 100 Year Return Period plus a climate change allowance of 70%

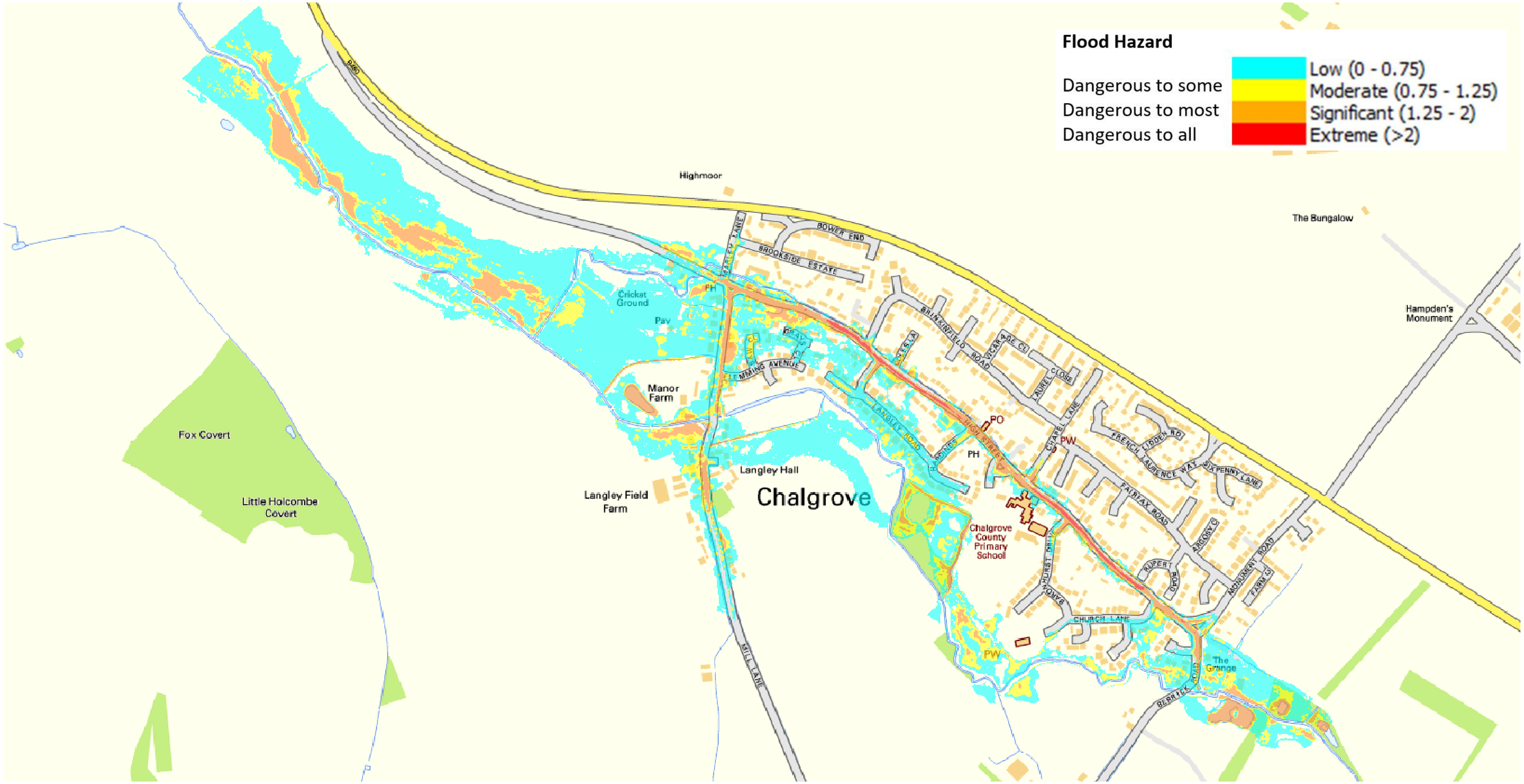


Figure 29 | Model Results showing flood hazard for 1 in 100 Year Return Period plus a climate change allowance of 70%

Figure 30 | Model Results showing flood water level for 1 in 1000 Year Return Period

Figure 31 | Model Results showing flood depth for 1 in 1000 Year Return Period

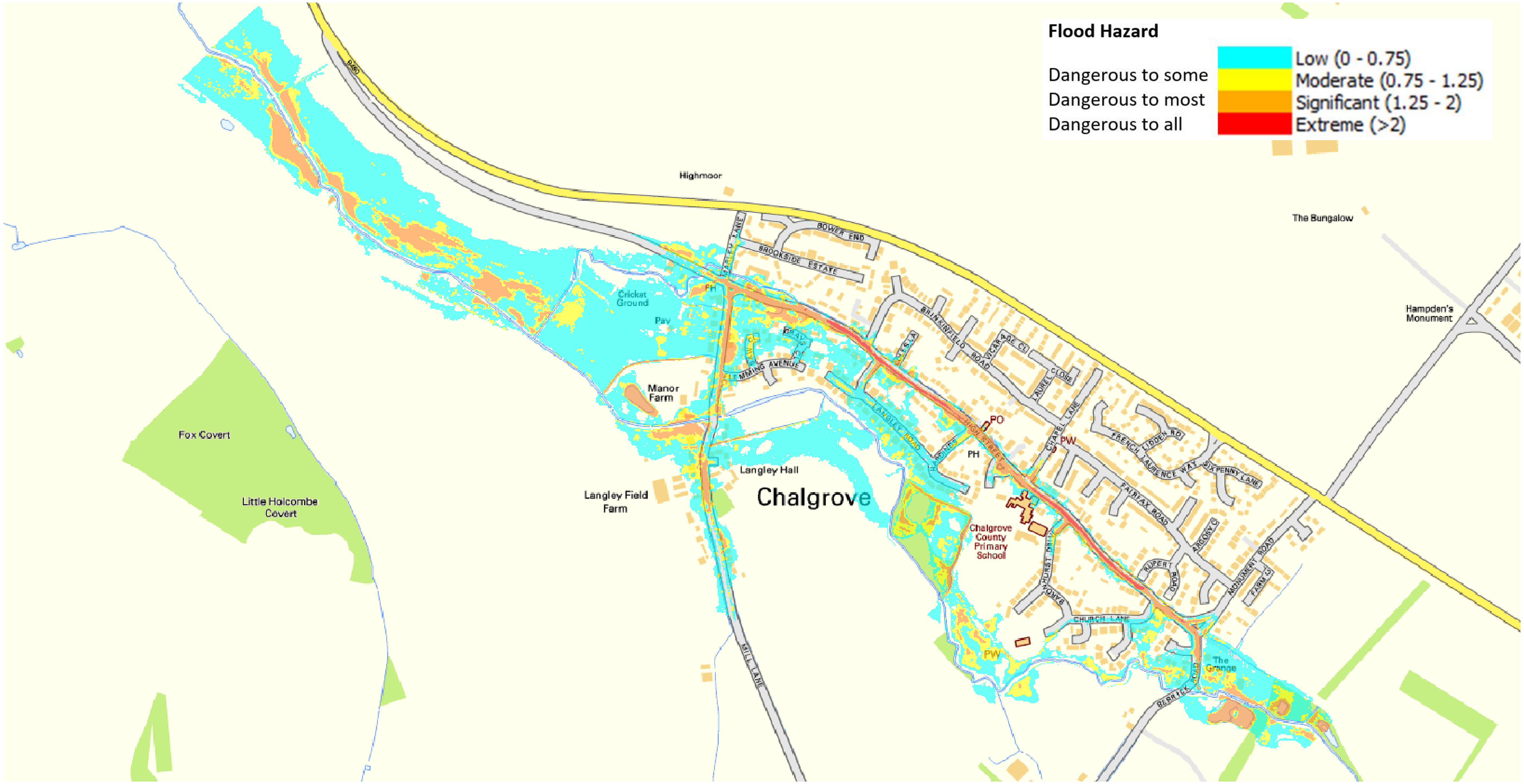


Figure 32 | Model Results showing flood hazard for 1 in 1000 Year Return Period

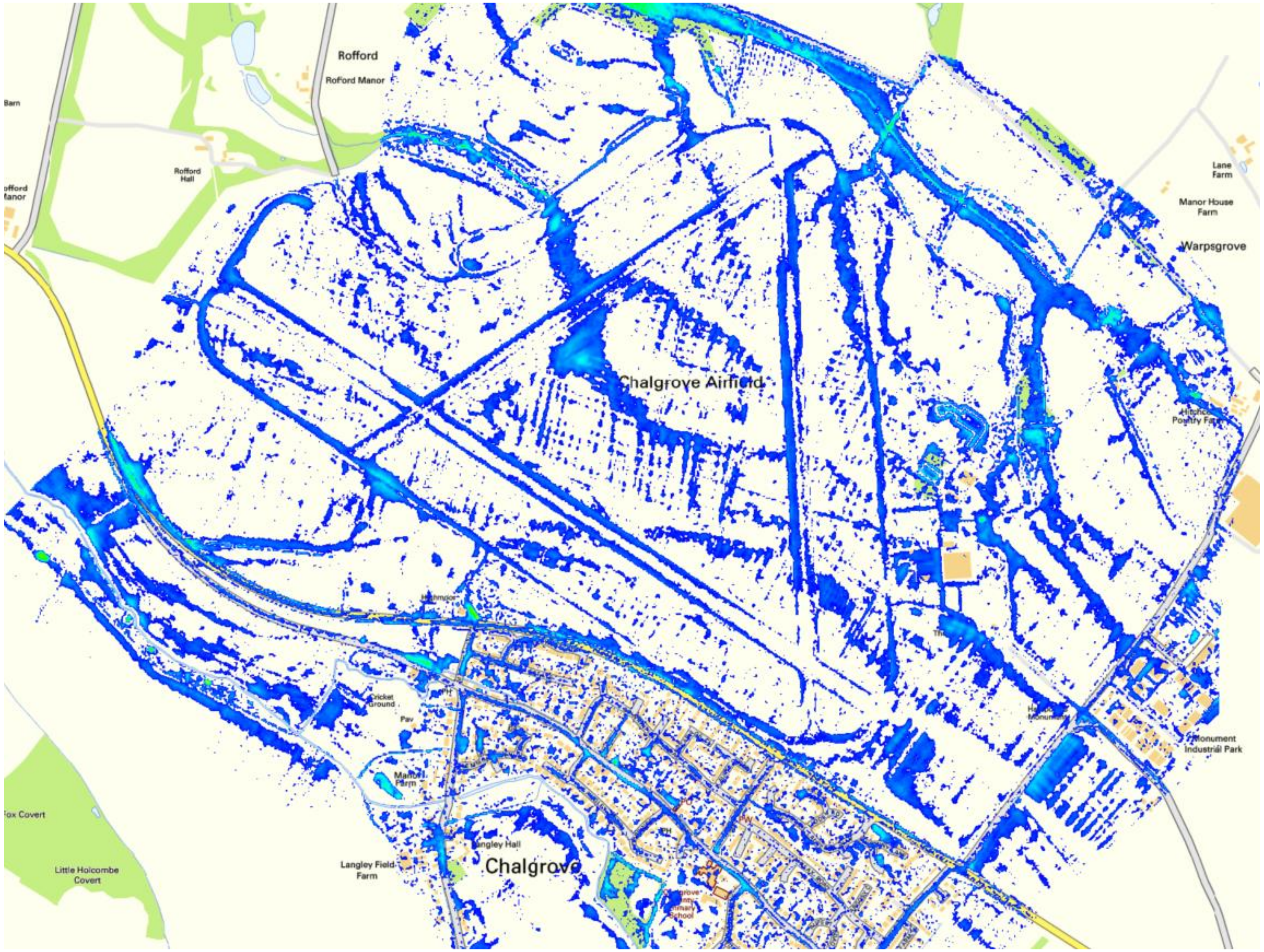


Figure 33 | Direct rainfall model: Surface water results showing flood hazard for 1 in 100 Year Return Period (Depths Greater than 0.05m)

Figure 34 | Direct rainfall model : Surface water results showing flood hazard for 1 in 1000 Year Return Period (Depths Greater than 0.05m)

5 Conclusions and Recommendations

5.1 Conclusions

EVY have undertaken a flood risk mapping study for the village of Chalgrove in Oxfordshire. This study has built upon previous modelling carried out for Chalgrove Flood Action Group and has included hydrological analysis and hydraulic modelling with the aim of improving flood risk mapping of the village. The modelling includes recent topographic survey of the High Street, including the High Street Brook, and associated structures. In addition, the model domain has been increased upstream and downstream of the village compared to the earlier modelling.

The modelling has confirmed that:-

- A significant proportion of property within Chalgrove is at risk of flooding and within Flood Zones 2 and 3.
- There is significant flooding to the High Street. The hazard along the High Street associated with this flooding mechanism is categorised as “dangerous to most” in the 1 in 100 year flood event and rises to “dangerous to all” in the 1 in 1000 year event. This is particularly pertinent in relation to Chalgrove Primary School which relies on access from the High Street and would be inundated in a 1 in 100 year event.
- In relation to housing and other forms of development, flood hazard would be a significant consideration for development to the south of the High Street and north of Chalgrove Brook.
- Properties on Berrick Road, Church Road, Langley Road, Fleming Avenue, Saw Close, Grays Close and Mill Lane are vulnerable to flooding. In general, properties on the High Street are largely unaffected apart from areas downstream of Langley Drive.
- The risk of surface water flooding to Chalgrove Airfield is generally classified as Low by the Environment Agency mapping. The direct rainfall modelling indicates that the southern half of the airfield drains towards the B480 with a route for surface water entering the village via Marley Lane.

- The introduction of development on Chalgrove Airfield could potentially have an impact to surface water flooding within Chalgrove as a result of an increase in impermeable surfaces. Accordingly, it is considered that Chalgrove Parish Council should be aware of this risk and ensure that planning conditions associated with the discharge of surface water from the site are restricted to greenfield flow rates / QBAR or less.
- Development site 11 could be vulnerable to surface water flooding from Chalgrove Airfield and provision must be made within the masterplan to accommodate surface water flowing across the B480.

5.2 Recommendations

It is recommended that:-

- Development within Chalgrove in areas classified as Flood Zone 1 and which have safe access and egress in times of flooding should be considered in preference to developments in Flood Zones 2 and 3. This is commensurate with sequential testing contained in the NPPF. Areas to the north of the High Street would be suitable for housing development.
- Development within Flood Zone 3b (the functional flood plain) is deemed to be unacceptable.
- The flood hazard to the High Street in a 1 in 100 and 1 in 1000 year event is rated as dangerous to most or dangerous to all. The presence of a Primary School on the High street means that there is a serious risk to children and the general public. It is therefore recommended that that this is addressed in order that the risk can be mitigated.
- A network of flood wardens is established within the community to manage flood risk and provide advice and assistance during a flood event. All flood wardens should be aware of the restrictions and dangers of flooding particularly in relation to pedestrians and vehicles.
- Flooding to the High Street appears to be related to the configuration of the bifircation during high flows. It is recommended that this flooding mechanism is investigated in more detail with a view to mitigating the risk of flooding to the High Street.

Appendix A – Flood Estimation Record

Flood estimation calculation record

Introduction

This document is a supporting document to the Environment Agency's flood estimation guidelines. It provides a record of the calculations and decisions made during flood estimation. It will often be complemented by more general hydrological information given in a project report. The information given here should enable the work to be reproduced in the future. This version of the record is for studies where flood estimates are needed at multiple locations.

Contents

Approval

	Signature	Name and qualifications	For Environment Agency staff: Competence level (see below)
Calculations prepared by:	Matthew Scott	Matthew Scott (BSc, PGDip, PGDip, DEA, C.WEM)	
Calculations checked by:			
Calculations approved by:			

Environment Agency competence levels are covered in Section 2.1 of the flood estimation guidelines:

- Level 1 – Hydrologist with minimum approved experience in flood estimation
- Level 2 – Senior Hydrologist
- Level 3 – Senior Hydrologist with extensive experience of flood estimation

ABBREVIATIONS

AM	Annual Maximum
AREA	Catchment area (km^2)
BFI	Base Flow Index
BFIHOST	Base Flow Index derived using the HOST soil classification
CFMP	Catchment Flood Management Plan
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
POT	Peaks Over a Threshold
QMED	Median Annual Flood (with return period 2 years)
ReFH	Revitalised Flood Hydrograph 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 Method statement

1.1 Overview of requirements for flood estimates

Item	Comments
Give an overview which includes: • Purpose of study • Approx. no. of flood estimates required • Peak flows or hydrographs? • Range of return periods and locations • Approx. time available	Flood risk assessment on behalf of Chalgrove Flood Action Group related to proposed developments on Mill Lane Brook. Flood estimates required for Mill Lane Brook u/s confluence with Chalgrove Brook, and for Chalgrove Brook u/s bifurcation along Chalgrove High Street (circa. 1 km u/s confluence with Mill Lane Brook). Design hydrographs required for return periods between QMED and 0.1% AEP, approximately 1.5 days available for analysis.

1.2 Overview of catchment

Item	Comments
Brief description of catchment, or reference to section in accompanying report	Mill Lane Brook is a small essentially rural catchment (< 2km^2) draining from south to north joining the Chalgrove Brook just downstream of Chalgrove village, 16km southeast of Oxford. Chalgrove Brook itself drains east-south-east to west-north-west and is bounded in its headwaters by the chalk escarpment of the Chiltern Hills. Passing through the village of Chalgrove, it is an essentially rural catchment of approx. 50 km^2, but does contain the market town of Watlington in its upper catchment.

1.3 Source of flood peak data

Was the HiFlows UK dataset used? If so, which version? If not, why not? Record any changes made	Yes – Version 3.3.2**,** April 2014

1.4 Gauging stations (flow or level)

(at the sites of flood estimates or nearby at potential donor sites)

Water-course	Station name	Gauging authority number	NRFA number (used in FEH)	Grid reference	Catch-ment area (km²)	Type (rated / ultrasonic / level…)	Start and end of flow record
Pang	Pangbourne		39027	463450 176600	175.68	Crump weir	1968 - 2014
Winterbourne Stream	Bagnor		39033	445250 169350	45.34	Crump weir	1962 – 2014
Ock	Abingdon (Old)		39018	448600 196950	248.21	Compound crump weir	1962 – 1979
Ock	Abingdon (New)		39081	448150 196650	233.60	Crump weir	1979 – 2014
Lambourn	Shaw		39019	446950 168250	235.21	Crump weir	1962 – 2014

1.5 Data available at each flow gauging station

Station name	Start and end of data in HiFlows-UK	Update for this study?	Suitable for QMED?	Suitable for pooling?	Data quality check needed?	Other comments on station and flow data quality – e.g. information from HiFlows-UK, trends in flood peaks, outliers.
Pangbourne	1968 - 2014	No	✓		Not in scope	Gauged above QMED. which is below bank full and below the modular limit.
Bagnor	1962 – 2014	No	✓	✓	Not in scope	Slight overestimation of flows, but flow is within bank full.
Abingdon (Old)	1962 – 1979	No	✓		Not in scope	
Abingdon (New)	1979 – 2014	No	✓		Not in scope	
Shaw	1962 – 2014	No	✓	✓	Not in scope	Gauged above QMED / to within 16% of AMAX3, within bank full and flow estimates thought to be reliable.
Give link/reference to any further data quality checks carried out						

1.6 Rating equations

Station name	Type of rating e.g. theoretical, empirical; degree of extrapolation	Rating review needed?	Reasons – e.g. availability of recent flow gaugings, amount of scatter in the rating.
Pangbourne	Standard Crump weir calculation using tail levels. Rating estimates flow to a good level of accuracy	Not in scope	
Bagnor	Protected flows derived from standard Crump weir calculation.	Not in scope	
Abingdon (Old)	One rating applied across period of record. Rating by current meter observation. Bypassing means station underestimates high flows.	Not in scope	
Abingdon (New)	Standard Crump weir calculation using tail levels. Part of the weir is subject to non-modular flow conditions (0.5m). Inaccurate at low flows, but otherwise reasonably accurate. Slight overestimation could result in anomalies at higher flows.	Not in scope	
Shaw	Protected flows derived from standard Crump weir calculation. Rating is deemed accurate.	Not in scope	
Give link/reference to any rating reviews carried out			

1.7 Other data available and how it has been obtained

Type of data	Data relevant to this study?	Data available ?	Source of data and licence reference if from EA	Date obtained	Details
Check flow gaugings (if planned to review ratings)					
Historic flood data – give link to historic review if					

carried out.					
Flow data for events					
Rainfall data for events					
Potential evaporation data					
Results from previous studies					
Other data or information (e.g. groundwater, tides)					

1.8 Initial choice of approach

Is FEH appropriate? (it may not be for very small, heavily urbanised or complex catchments) If not, describe other methods to be used.	FEH statistical and ReFH appropriate for Mill Lane Brook 'small catchment' (ref. SC090031). FEH statistical appropriate for Chalgrove Brook – requires consideration of permeable adjustment; ReFH not suitable for estimating peak flows due to high permeability of the catchment but can be used as part of hybrid method to estimate design hydrograph shape.
Outline the conceptual model, addressing questions such as: • Where are the main sites of interest? • What is likely to cause flooding at those locations? (peak flows, flood volumes, combinations of peaks, groundwater, snowmelt, tides…) • Might those locations flood from runoff generated on part of the catchment only, e.g. downstream of a reservoir? • Is there a need to consider temporary debris dams that could collapse?	Main site of interest is towards the lower end of Mill Lane Brook, @ confluence with Chalgrove Brook & ~ 400 m u/s confluence; flooding may be influenced by downstream water levels in Chalgrove Brook. Sites of interest are sufficiently close such that a single estimate of flows for Mill Lane Brook (@ confluence) are considered appropriate for estimating design inflows.
Any unusual catchment features to take into account? e.g. • highly permeable – avoid ReFH if BFIHOST>0.65, consider permeable catchment adjustment for statistical method if SPRHOST<20% • highly urbanised – avoid standard ReFH if URBEXT1990>0.125; consider FEH Statistical or other alternatives; consider method that can account for differing sewer and topographic catchments • pumped watercourse – consider lowland catchment version of rainfall-runoff method • major reservoir influence (FARL<0.90) – consider flood routing • extensive floodplain storage – consider choice of method carefully	Mill Lane Brook is a relatively impermeable small catchment within a wider area of chalk downlands (FEH catchment BFIHOST = 0.355; SPRHOST = 46.01). Chalgrove Brook is highly permeable (FEH catchment BFIHOST = 0.85; SPRHOST = 14.34); requires application of permeable adjustment for statistical estimation of peak flows and careful consideration of hydrological similarity for both potential QMED donor transfer and composition of pooling group. ReFH suitable for estimation of hydrograph shape only. No local data available to improve confidence in statistical method / rainfall-runoff model parameters / design hydrograph shape.
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	FEH statistical to estimate peak flows for both Mill Lane Brook and Chalgrove Brook catchments; ReFH to estimate peak flows & hydrograph shape for Mill Lane Brook catchment; hydrograph shape only for Chalgrove Brook catchment (consider different initial conditions to model 'wet' and 'dry' antecedent conditions as variations to normal design method).
Software to be used (with version numbers)	FEH CD-ROM v3.0[1]

[1] FEH CD-ROM v3.0 © NERC (CEH). © Crown copyright. © AA. 2009. All rights reserved.

	WINFAP-FEH v3.0.002[2] / ~~ReFH spreadsheet~~ / ~~ReFH Design Flood Modelling Software~~ / ISIS

[2] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

2 Locations where flood estimates required

The table below lists the locations of subject sites. The site codes listed below are used in all subsequent tables to save space.

2.1 Summary of subject sites

Site code	Watercourse	Site	Easting	Northing	AREA on FEH CD-ROM (km^2)	Revised AREA if altered
ML1	Mill Lane Brook	u/s confluence with Chalgrove Brook	463100	196950	1.59	
CB1	Chalgrove Brook	u/s bifurcation to High Street (~ 1 km u/s confluence with Mill Lane Brook)	464050	196450	45.09	
Reasons for choosing above locations						

2.2 Important catchment descriptors at each subject site (incorporating any changes made)

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
ML1	1	0.29	0.355	1.52	12.5	615	46.01	1990=0; 2000=0.0063	0.2331
CB1	0.988	0.29	0.85	6.9	51.1	680	14.34	1990=0.0128; 2000=0.0165	0.0701

2.3 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Catchment boundaries checked against OS VectorMap District, OS Open Rivers and OS Terrain 50 raster and vector data and Environment Agency main rivers data; and MagicMap OS mapping (www.magic.gov.uk). Some uncertainty noted to upper Mill Lane Brook catchment boundary but evidence not conclusive to require modification of FEH catchment area (top part of catchment may drain away from Mill Lane Brook; additional catchment near top of catchment may drain towards Mill Lane Brook). Additional checks undertaken for the Mill Lane Brook catchment using 1m LiDAR data obtained from Environment Agency; contoured at 1m elevation intervals. FEH catchment boundary confirmed as appropriate.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	FARL checked visually against OS Open Map Local surface water layer and deemed appropriate. URBEXT checked visually against OS VectorMap District data and deemed appropriate. Soil properties not explicitly checked against soil mapping (CSAI Soilscapes viewer license precludes its use for commercial activities), but BFIHOST & SPRHOST values reported assessed as consistent with regional characteristics. Lower permeability of Mill Lane Brook catchment consistent with anecdotal evidence of flooding on brook.
Source of URBEXT	URBEXT1990 for ReFH / URBEXT2000 for statistical analysis
Method for updating of URBEXT	CPRE formula from FEH Volume 4 for URBEXT1990 updating / CPRE formula from 2006 CEH report on URBEXT2000

3 Statistical method

3.1 Search for donor sites for QMED (if applicable)

Comment on potential donor sites

Mention:

- Number of potential donor sites available
- Distances from subject site
- Similarity in terms of AREA, BFIHOST, FARL and other catchment descriptors
- Quality of flood peak data

Include a map if necessary. Note that donor catchments should usually be rural.

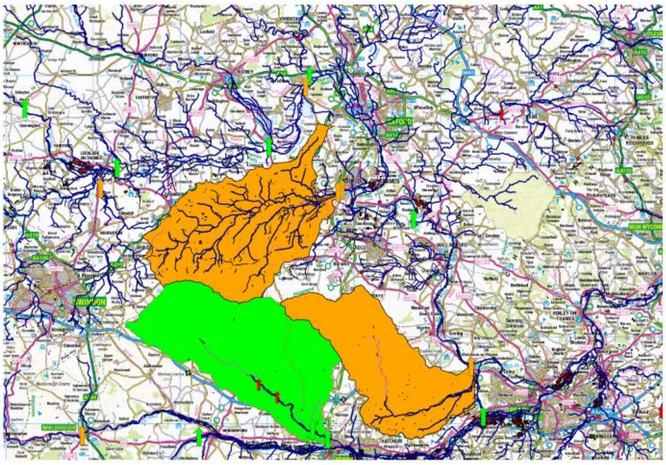
ML1:

Mill Lane Brook catchment is a small catchment (AREA = 1.59 km2) of moderate permeability (BFIHOST = 0.355) in an area of generally high permeability → geographically close catchments are typically hydrologically *dissimilar*. TOP 5 closest essentially rural catchments suitable for QMED or pooling selected as potential donor catchments.

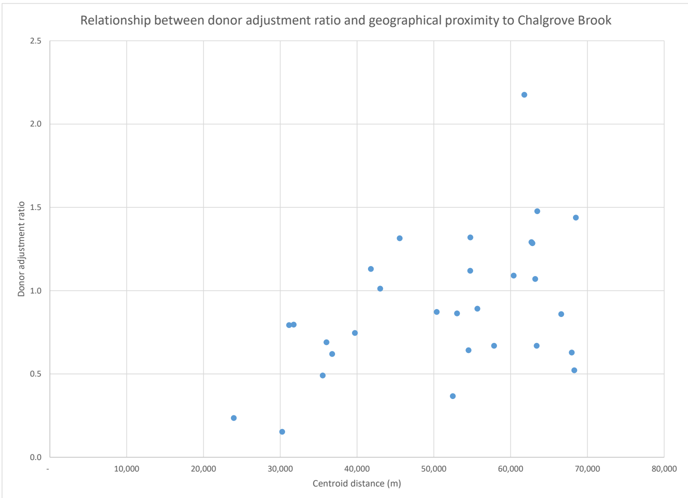
CB1:

Chalgrove Brook catchment is a moderate sized catchment (AREA = 45.09 km2) of high permeability (BFIHOST = 0.85) in an area of generally high permeability → geographically close catchments are typically hydrologically *similar*. TOP 5 closest essentially rural catchments suitable for QMED or pooling selected as potential donor catchments. Additionally, there is a local gauge on the Chalgrove Brook upstream of the Berrick Road bridge by The Grange in Chalgrove, which is in the immediate proximity of the target site. This gauge has a little over three years of data.

ML1 & CB1 potential donor catchments are identical (see figure below: suitable for QMED catchments shown in orange; suitable for pooling shown in green; subject catchments shown in shaded green).



Analysis of all essentially rural catchments suitable for QMED or pooling within a 70 km distance of CB1 shows a strong correlation between distance and donor adjustment ratios (see below).



3.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjust-ment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
n/a	Rejected for ML1 due to hydrological dissimilarity Selected for CB1	POT	No	2.174	1.4	1.553
39027	Rejected for ML1 due to hydrological dissimilarity – AREA & BFIHOST Selected for CB1	AM	N/A	2.14	9.08	0.235
39018	Rejected for ML1 due to hydrological dissimilarity – AREA & BFIHOST Rejected for CB1 due to lower BFIHOST value & relatively higher adjustment ratio value	AM	N/A	10.45	13.18	0.793
39081	Rejected for ML1 due to hydrological dissimilarity – AREA & BFIHOST Rejected for CB1 due to lower BFIHOST value & relatively higher adjustment ratio value	AM	N/A	10.53	13.23	0.796
39033	Rejected for ML1 due to hydrological dissimilarity – AREA & BFIHOST Selected for CB1	AM	N/A	0.39	2.57	0.153
39019	Rejected for ML1 due to hydrological dissimilarity – AREA & BFIHOST Selected for CB1	AM	N/A	3.55	7.23	0.490
Which version of the urban adjustment was used for QMED at donor sites, and why? Note: The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8).				No urban adjustments applied to donor sites as all sites essentially rural.		

3.3 Overview of estimation of QMED at each subject site

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	
ML1	CD	0.397	N/A	N/A	N/A	N/A	N/A	N/A	Rural=0.397 Urban=0.400
CB1	POT	2.174	N/A	N/A	N/A	N/A	N/A	N/A	2.174
~~CB1(alt1)~~	~~DT~~	~~1.40~~	~~39027~~	~~23.950~~	~~0.285~~	~~0.662~~	~~0.334~~	~~0.2211~~	
			~~39033~~	~~30.254~~	~~0.251~~	~~0.624~~	~~0.333~~	~~0.2078~~	
			~~39019~~	~~35.524~~	~~0.226~~	~~0.851~~	~~0.333~~	~~0.2834~~	

Site code	Method	Initial estimate of QMED (m^3/s)	Data transfer						Final estimate of QMED (m^3/s)
			NRFA numbers for donor sites used (see 3.3)	Distance between centroids d_{ij} (km)	Power term, a	Moderated QMED adjustment factor, $(A/B)^a$	If more than one donor		
							Weight	Weighted average adjustment factor	
							→	~~0.7123~~	~~Rural=0.997~~ ~~Urban=1.10~~
~~CB1(alt2)~~	~~DT~~	~~1.40~~	~~39027~~	~~23.950~~	~~1~~	~~0.235~~	~~0.334~~	~~0.0785~~	
			~~39033~~	~~30.254~~	~~1~~	~~0.153~~	~~0.333~~	~~0.0509~~	
			~~39019~~	~~35.524~~	~~1~~	~~0.490~~	~~0.333~~	~~0.1632~~	
							→	~~0.2926~~	~~Rural=0.410~~ ~~Urban=0.454~~

Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?	N/A. QMED for CB1 sensitive to donor transfer method and adoption of adjustment factor. CB1(alt) calculated without use of adjustment factor for consideration of uncertainty (not adopted for best estimate)
Which version of the urban adjustment was used for QMED, and why?	Kjeldsen (2010), except for CB1, where local data was used with no urban adjustment due to the permeable nature of the catchment.

Notes

Methods: AM – Annual maxima; POT – Peaks over threshold; DT – Data transfer; CD – Catchment descriptors alone.

When QMED is estimated from POT data, it should also be adjusted for climatic variation. Details should be added.

When QMED is estimated from catchment descriptors, the revised 2008 equation from Science Report SC050050Error! Bookmark not defined. should be used. If the original FEH equation has been used, say so and give the reason why.

The guidelines recommend great caution in urban adjustment of QMED on catchments that are also highly permeable (BFIHOST>0.8). The adjustment method used in WINFAP-FEH v3.0.003 is likely to overestimate adjustment factors for such catchments. In this case the only reliable flood estimates are likely to be derived from local flow data.

The data transfer procedure is from Science Report SC050050. The QMED adjustment factor A/B for each donor site is given in Table 3.3. This 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 estimate from catchment descriptors.

If more than one donor has been used, use multiple rows for the site and give the weights used in the averaging. Record the weighted average adjustment factor in the penultimate column.

3.4 Derivation of pooling groups

The composition of the pooling groups is given in the Annex. Several subject sites may use the same pooling group.

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
~~**ML1.1**~~	~~ML1~~	~~No~~	~~Site 49006 removed as short record (6 yrs); no discordant sites; h2=3.1391 (heterogeneous)~~	~~L-CV=0.224~~ ~~L-Skew=0.215~~

Name of group	**Site code from whose descriptors group was derived**	**Subject site treated as gauged?** (enhanced single site analysis)	**Changes made to default pooling group, with reasons** Note also any sites that were investigated but retained in the group.	**Weighted average L-moments**, L-CV and L-skew, (before urban adjustment)
ML1.2	ML1	No	Flat GCs / low l-cv & l-skew sites reviewed (27073; 203046; 206006): - *27073 excluded as flow record is unrepresentative of topographical catchment (BFIHOST=0.887);* - *203046 excluded as no gaugings above QMED and single segment rating curve applied across stage range;* - *206006 excluded as no information available on gaugings or ratings* Steep GCs / high l-cv & l-skew sites reviewed (45816; 28033; 25019) - 45816 retained as no bypassing and thought to perform well to flows beyond QMED - 28033 retained as all recorded flows contained by structure & remain modular - 25019 retained as well contained channel with gaugings higher than QMED. AMAX1 significantly higher than other AMAX but with associated substantial flooding Sites 44008, 36010 and 49003 added to meet 500 year criterion; site 44008 shows steep GC / high l-cv & l-skew but retained as all recorded flows contained & rating confirmed by gaugings; no discordant sites; h2=2.1963 (heterogeneous)	L-CV=0.256 L=Skew=0.259
~~**CB1.1**~~	~~CB1~~	~~No~~	~~Default using ‘flood attenuation’ SDM.~~ ~~No discordant sites; h2=3.0618 (heterogeneous)~~ ~~Site 39033 shows steep GC / high l-cv & l-skew but no obvious record length / data quality issues so retained (AMAX1 dated July 2007).~~	~~L-CV=0.283~~ ~~L-Skew=0.123~~
~~**CB1.2**~~	~~CB1~~	~~No~~	~~Composition based on ‘permeability’ SDM.~~ ~~Site 39033 discordant (D=3.461) but retained as no obvious record length / data quality issues (AMAX1 dated July 2007); h2=4.2463 (strongly heterogeneous). No other sites with steep / flat GCs or high / low l-cv / l-skew values identified.~~	~~L-CV=0.235~~ ~~L-Skew=0.029~~

Name of group	Site code from whose descriptors group was derived	Subject site treated as gauged? (enhanced single site analysis)	Changes made to default pooling group, with reasons Note also any sites that were investigated but retained in the group.	Weighted average L-moments, L-CV and L-skew, (before urban adjustment)
CB1.3	CB1	No	Composition based on distance to subject site, permeability (SPRHOST < 30) & catchment size (AREA < 10x subject site AREA). No discordant sites; h2=2.5954 (heterogeneous). Site 39033 shows steep GC / high l-cv & l-skew but no obvious record length / data quality issues so retained (AMAX1 dated July 2007); site 42009 also shows steep GC / high l-cv & l-skew but no obvious record length / data quality issues so retained (AMAX1 dated December 2000)	L-CV=0.220 L-Skew=0.150

Notes

Pooling groups were derived using the revised procedures from Science Report SC050050 (2008). Amend if not applicable.

The weighted average L-moments, before urban adjustment, can be found at the bottom of the Pooling-group details window in WINFAP-FEH.

3.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group (3.4)	Distribution used and reason for choice	Note any urban adjustment or permeable adjustment	Parameters of distribution (location, scale and shape) after adjustments	Growth factor for 100-year return period
ML1	P	ML1.2	GL best fit (z=-0.0044); ~~GEV z=-1.3499, PIII z=-2.0204~~	Kjeldsen urban adjustment applied ($URBEXT_{2000}$ = 0.0065)	Location=1 Scale=0.255 Shape=-0.26	3.257
CB1	P	CB1.3	GL best fit (z=-1.7338); ~~GEV z=-3.9549, PIII z=-4.0726~~; compatible with FEH permeable adjustment	Permeable adjustment applied	Location=1 Scale=0.212 Shape=-0.177	2.516

Notes

Methods: SS – Single site; P – Pooled; ESS – Enhanced single site; J – Joint analysis

A pooling group (or ESS analysis) derived at one gauge can be applied to estimate growth curves at a number of ungauged sites. Each site may have a different urban adjustment, and therefore different growth curve parameters.

Urban adjustments to growth curves should use the version 3 option in WINFAP-FEH: Kjeldsen (2010).

Growth curves were derived using the revised procedures from Science Report SC050050 (2008)

The composition of the pooling group for site CB1 was based on distance and consideration of hydrological similarity (AREA and BFIHOST only) as regional geology is dominant / site is highly permeable (FEH catchment BFIHOST = 0.85; SPRHOST = 14.34). Other PG compositions were tested (see annex for further details) but discounted as being less suitable for representing local hydrological conditions.

Any relevant frequency plots from WINFAP-FEH, particularly showing any comparisons between single-site and pooled growth curves (including flood peak data on the plot), should be shown here or in a project report.

3.6 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)								
	2	5	10	30	50	75	100	200	1000
ML1	0.400	0.570	0.702	0.949	1.09	1.21	1.30	1.56	2.37
CB1	2.174	2.904	3.422	4.313	4.776	5.172	5.470	6.248	8.463

4 Revitalised flood hydrograph (ReFH) method

4.1 Parameters for ReFH model

Note: If parameters are estimated from catchment descriptors, they are easily reproducible so it is not essential to enter them in the table.

Site code	**Method:** OPT: Optimisation BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details)	**Tp (hours)** Time to peak	**C_{max} (mm)** Maximum storage capacity	**BL (hours)** Baseflow lag	**BR** Baseflow recharge
ML1	CD				
CB1	CD				

Brief description of any flood event analysis carried out (further details should be given below or in a project report)	None.

4.2 Design events for ReFH method

Site code	**Urban or rural**	**Season of design event (summer or winter)**	**Storm duration (hours)**	**Storm area for ARF** (if not catchment area)
ML1	Rural	Winter	6.25	
CB1	Rural	Winter	10.25	
dsConf			11.75	

Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?	

4.3 Flood estimates from the ReFH method

Site code	**Flood peak (m^3/s) or volumes (m^3) for the following return periods (in years)**								
	2	**5**	**10**	**30**	**50**	**75**	**100**	**200**	**1000**
ML1	0.498	0.660	0.791	1.00	1.12	1.23	1.31	1.55	2.35
CB1	0.312	0.544	0.787	1.37	1.77	2.17	2.50	3.52	7.76

5 FEH rainfall-runoff method

5.1 Parameters for FEH rainfall-runoff model

Methods:
FEA : Flood event analysis
LAG : Catchment lag
DT : Catchment descriptors with data transfer from donor catchment
CD : Catchment descriptors alone
BFI : SPR derived from baseflow index calculated from flow data

Site code	Rural (R) or urban (U)	Tp(0): method	Tp(0): value (hours)	SPR: method	SPR: value (%)	BF: method	BF: value (m^3/s)	If DT, numbers of donor sites used (see Section 5.2) and reasons

5.2 Donor sites for FEH rainfall-runoff parameters

No.	Watercourse	Station	Tp(0) from data (A)	Tp(0) from CDs (B)	Adjustment ratio for Tp(0) (A/B)	SPR from data (C)	SPR from CDs (D)	Adjust-ment ratio for SPR (C/D)
1								
2								

5.3 Inputs to and outputs from FEH rainfall-runoff model

Site code	Storm duration (hours)	Storm area for ARF (if not catchment area)	Flood peaks (m^3/s) or volumes (m^3) for the following return periods (in years)							
			2							
Are the storm durations likely to be changed in the next stage of the study, e.g. by optimisation within a hydraulic model?										

6 Discussion and summary of results

6.1 Comparison of results from different methods

This table compares peak flows from various methods with those from the FEH Statistical method at example sites for two key return periods. Blank cells indicate that results for a particular site were not calculated using that method.

Site code	Ratio of peak flow to FEH Statistical peak					
	Return period 2 years			Return period 100 years		
	ReFH	Other method	Other method	ReFH	Other method	Other method
ML1	ReFH=0.498 Statistical=0.400 Ratio=1.25			ReFH=1.31 Statistical=1.30 Ratio=1.01		
CB1	ReFH=0.312 Statistical=2.174 Ratio=0.144			ReFH=2.50 Statistical=5.47 Ratio=0.457		

6.2 Final choice of method

Choice of method and reasons – include reference to type of study, nature of catchment and type of data available.	The hybrid method has been adopted to provide design hydrographs for the required return period range (2 to 1000 years). Local data used to improve confidence in statistical method at CB1 only. No local data available to improve confidence in rainfall-runoff model parameters / design hydrograph shape. **ML1:** SC090031 recommends that the FEH statistical approach and ReFH model are appropriate for estimating QMED on small catchments, but that methods tend to underestimate QMED on lower rainfall catchments (bias=0.844 and 0.747 respectively). SC090031 also shows that the FEH pooling group statistical approach is appropriate for estimating growth curves. Environment Agency guidelines (OI 197_08) note that QMED data transfer should consider hydrological similarity in addition to geographical proximity. No data transfer has been applied for ML1 as the catchment is considered hydrologically dissimilar to neighbouring catchments, i.e., all neighbouring catchments are highly permeable. - QMED based on catchment descriptors; no data transfer applied - Pooling group composition based on ‘flood attenuation’ similarity (current standard); GL distribution adopted with urban adjustment - Design standard ReFH model (to be scaled as part of hybrid method) **CB1:** Environment Agency guidelines (OI 197_08) note that the ReFH method should not be used for estimating peak flows on permeable catchments, and that QMED estimation from catchment descriptors on permeable catchments is extremely uncertain. The evaluation of the QMED data transfer process has shown that there appears to be a regional trend of QMED over-estimation when using catchment descriptors. Local data and POT have been used to inform QMED at this site. The Environment Agency guidelines (OI 197_08) also note that the hydrological response of permeable catchments may respond differently to non-permeable catchments and as such manual editing of the pooling group composition to consider permeability may provide an improved estimate of growth curves. Different pooling group compositions have been tested for CB1 and it has been concluded that composition based on local catchments of similar hydrological conditions (considering AREA and BFIHOST) provide the best growth curve estimates (e.g. due to lower h2 and z-score values). - QMED based on POT from local gauge data - Pooling group composition based on local catchments of similar hydrological

conditions; GL distribution adopted with permeable adjustment

- Design standard ReFH model (to be scaled as part of hybrid method)

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)

Primary assumption for Mill Lane Brook is that FEH methods are appropriate for estimating peak flows and hydrograph shape / volume on small catchments for wide range of event severity (2 year to 1000 year return period), as reported by SC090031 (May 2012); including non-application of QMED donor transfer due to dissimilarity of subject catchment and neighbouring catchments.

Primary assumptions for Chalgrove Brook are that:

- QMED assessment from POT is more accurate than from catchment descriptors with donor transfer;
- The associated growth curve is best represented by considering local catchments of similar hydrological characteristics (AREA and BFIHOST only); and
- Although the ReFH model is not considered appropriate for estimating peak flows (due to high permeability of the subject site catchment), it can be scaled when using the hybrid method to provide appropriate estimates of hydrograph shape and volume.

Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed

Limitations inherently linked to main assumptions on suitability of FEH methods in small / permeable catchments in which methods are being applied at / beyond the extreme range of catchment types for which they were developed, including, *inter alia*:

- Estimating QMED from catchment descriptors for ML1;
- Estimating QMED from POT for CB1;
- Constructing pooling groups based on hydrological similarity (in which AREA similarity is a dominant factor);
- Application of the design method in permeable catchments where differences between antecedent and event conditions are more pronounced.

Give what information you can on uncertainty in the results – e.g. confidence limits for the QMED estimates using FEH 3 12.5 or the factorial standard error from Science Report SC050050 (2008).

QMED uncertainty estimates cannot be readily quantified for small catchments as model error at extremes is not well represented by sample average.

For small catchments in general, SC090031 provides an indication of typical uncertainty (bias and RMSE) associated with FEH (& other) methods for estimating QMED peak flow:

	All catchments (73)		**Urban catchments only (9)**		**No urban or permeable catchments (54)**		**No high-rainfall catchments (44)**	
	RMSE	Bias	RMSE	Bias	RMSE	Bias	RMSE	Bias
ADAS 345	1.180	0.662	1.787	0.258	1.006	0.834	1.212	0.529
ReFH	0.759	0.925	**0.523**	**0.941**	**0.438**	1.185	0.912	0.747
IH 124	0.675	0.823	0.651	1.100	0.566	0.841	0.739	**0.862**
FEH statistical	**0.547**	**0.977**	0.588	0.729	0.464	**1.107**	**0.593**	0.844

QMED uncertainty estimates for permeable catchments are also difficult to quantify – OIU 197_08 provides example illustrations – but may reasonable be considered to be greater than typical catchment uncertainty. Application of the donor transfer process suggests uncertainty may be of the order of FSE=2.

QMED estimates from POT with three years of data have 68% confidence intervals at 80% and 125% of estimate (source: table 2.2, FEH vol 3).

Comment on the suitability of the results for future studies, e.g. at

Results provided herein are only suitable for the current study due to the particularities of the catchment.

nearby locations or for different purposes.	
Give any other comments on the study, for example suggestions for additional work.	Analysis of local gauge data could be revised once the record length has had time to develop to improve confidence in the estimation of QMED on Chalgrove Brook, and combined with catchment rainfall to improve estimation of ReFH model parameters.

6.4 Checks

Are the results consistent, for example at confluences?	
What do the results imply regarding the return periods of floods during the period of record?	
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	The 100 year growth factor is approximately 3.25 for Mill Lane Brook (central within the typical range) and 2.5 for Chalgrove Brook (towards the lower end of the typical range)
If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	The 1000 year to 100 year return period ratio is approximately 1.8 on Mill Lane Brook and 1.5 on Chalgrove Brook, the latter lower value consistent with the permeable nature of the catchment.
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	
Are the results compatible with the longer-term flood history?	
Describe any other checks on the results	Final results to be checked through hydrodynamic modelling to assess whether resultant modelled flood extents are consistent with local knowledge of catchment flooding processes, with particular reference to the 100 year return period flood.

6.5 Final results

Site code	Flood peak (m^3/s) or volume (m^3) for the following return periods (in years)								
	2	5	10	30	50	75	100	200	1000
ML1	0.400	0.570	0.702	0.949	1.09	1.21	1.30	1.56	2.37
CB1	2.174	2.904	3.422	4.313	4.776	5.172	5.470	6.248	8.463

If flood hydrographs are needed for the next stage of the study, where are they provided? (e.g. give filename of spreadsheet, name of ISIS model, or reference to table below)	 iSIS.zip

7 Annex - supporting information

7.1 Pooling group composition

ML1.1

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	2.098	35	1.84	0.169	0.333	1.091
27073 (Brompton Beck @ Snainton Ings)	2.289	32	0.813	0.197	-0.022	1.155
27051 (Crimple @ Burn Bridge)	3.414	40	4.539	0.222	0.149	0.717
45816 (Haddeo @ Upton)	3.457	19	3.456	0.324	0.434	1.244
28033 (Dove @ Hollinsclough)	3.694	33	4.666	0.266	0.415	0.661
26802 (Gypsey Race @ Kirby Grindalythe)	3.952	13	0.109	0.261	0.199	0.324
25019 (Leven @ Easby)	3.989	34	5.538	0.347	0.394	1.606
20002 (West Peffer Burn @ Luffness)	4.094	41	3.299	0.292	0.015	1.718
25003 (Trout Beck @ Moor House)	4.109	39	15.164	0.176	0.291	0.503
47022 (Tory Brook @ Newnham Park)	4.126	19	7.331	0.257	0.071	0.973
25011 (Langdon Beck @ Langdon)	4.161	26	15.878	0.241	0.326	1.51
91802 (Allt Leachdach @ Intake)	4.232	34	6.35	0.153	0.257	1.081
203046 (Rathmore Burn @ Rathmore Bridge)	4.238	30	10.934	0.136	0.091	0.792
206006 (Annalong @ Recorder)	4.29	48	15.33	0.189	0.052	1.215
54022 (Severn @ Plynlimon Flume)	4.357	37	15.031	0.155	0.168	1.299
27010 (Hodge Beck @ Bransdale Weir)	4.36	41	9.42	0.224	0.293	0.112
Total		521				
Weighted means		521		0.224	0.215	

ML1.2

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
76011 (Coal Burn @ Coalburn)	2.098	35	1.84	0.169	0.333	1.123
27051 (Crimple @ Burn Bridge)	3.414	40	4.539	0.222	0.149	0.821
45816 (Haddeo @ Upton)	3.457	19	3.456	0.324	0.434	0.909
28033 (Dove @ Hollinsclough)	3.694	33	4.666	0.266	0.415	0.625
26802 (Gypsey Race @ Kirby Grindalythe)	3.952	13	0.109	0.261	0.199	0.393
25019 (Leven @ Easby)	3.989	34	5.538	0.347	0.394	0.921
20002 (West Peffer Burn @ Luffness)	4.094	41	3.299	0.292	0.015	1.991
25003 (Trout Beck @ Moor House)	4.109	39	15.164	0.176	0.291	0.451
47022 (Tory Brook @ Newnham Park)	4.126	19	7.331	0.257	0.071	1.111
25011 (Langdon Beck @ Langdon)	4.161	26	15.878	0.241	0.326	1.481
91802 (Allt Leachdach @ Intake)	4.232	34	6.35	0.153	0.257	0.933
54022 (Severn @ Plynlimon Flume)	4.357	37	15.031	0.155	0.168	1.994
27010 (Hodge Beck @ Bransdale Weir)	4.36	41	9.42	0.224	0.293	0.114
44008 (South Winterbourne @ Winterbourne Steepleton)	4.412	33	0.42	0.395	0.332	1.115
36010 (Bumpstead Brook @ Broad Green)	4.511	45	6.759	0.418	0.228	1.605
49003 (de Lank @ de Lank)	4.511	46	13.559	0.232	0.241	0.413
Total		535				
Weighted means				0.256	0.259	

CB1.1

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
36004 (Chad Brook @ Long Melford)	0.329	45	4.938	0.306	0.199	0.404
53017 (Boyd @ Bitton)	0.406	39	13.073	0.243	0.112	0.297
30004 (Lymn @ Partney Mill)	0.423	50	6.778	0.236	0.059	0.297
39033 (Winterbourne Stream @ Bagnor)	0.438	50	0.393	0.336	0.369	2.441
36007 (Belchamp Brook @ Bardfield Bridge)	0.526	48	4.628	0.384	0.129	1.516
36003 (Box @ Polstead)	0.54	49	3.841	0.31	0.109	0.563
41022 (Lod @ Halfway Bridge)	0.552	39	16.044	0.287	0.214	2.102
26003 (Foston Beck @ Foston Mill)	0.556	52	1.739	0.243	-0.015	0.666
37016 (Pant @ Copford Hall)	0.562	47	8.502	0.285	0.049	0.423
42011 (Hamble @ Frogmill)	0.57	40	8.028	0.159	0.013	1.641
33032 (Heacham @ Heacham)	0.6	44	0.461	0.315	0.099	0.65
Total		503				
Weighted means				0.283	0.123	

CB1.2

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.438	50	0.393	0.336	0.369	3.461
26003 (Foston Beck @ Foston Mill)	0.556	52	1.739	0.243	-0.015	0.188
33032 (Heacham @ Heacham)	0.6	44	0.461	0.315	0.099	1.586
33054 (Babingley @ Castle Rising)	0.613	36	1.129	0.214	0.069	0.236
26803 (Water Forlornes @ Driffield)	0.773	13	0.684	0.215	0.069	1.735
39042 (Leach @ Priory Mill Lechlade)	0.792	40	3.147	0.199	0.05	0.364
34012 (Burn @ Burnham Overy)	0.925	46	1.024	0.226	-0.137	0.969
43014 (East Avon @ Upavon)	1.052	41	3.616	0.206	0.051	0.44
39028 (Dun @ Hungerford)	1.171	44	2.207	0.216	-0.04	0.192
39020 (Coln @ Bibury)	1.35	49	3.61	0.18	0.028	0.754
39035 (Churn @ Cerney Wick)	1.621	43	3.46	0.16	-0.115	1.55
33007 (Nar @ Marham)	1.844	29	3.527	0.226	0.021	0.463
33029 (Stringside @ Whitebridge)	2.043	47	2.673	0.245	-0.108	1.061
Total		534				
Weighted means				0.235	0.029	

CB1.3

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
39033 (Winterbourne Stream @ Bagnor)	0.438	50	0.393	0.336	0.369	2.112
39042 (Leach @ Priory Mill Lechlade)	0.792	40	3.147	0.199	0.05	0.293
42009 (Candover Stream @ Borough Bridge)	0.899	41	0.992	0.264	0.364	1.082
43014 (East Avon @ Upavon)	1.052	41	3.616	0.206	0.051	1.437
42007 (Alre @ Drove Lane Alresford)	1.07	43	2.205	0.158	0.128	1.399
39028 (Dun @ Hungerford)	1.171	44	2.207	0.216	-0.04	1.352
39020 (Coln @ Bibury)	1.35	49	3.61	0.18	0.028	0.901
39019 (Lambourn @ Shaw)	2.349	50	3.545	0.218	0.143	0.007
42010 (Itchen @ Highbridge & Allbrook Total)	2.818	54	9.363	0.146	0.138	1.278
39006 (Windrush @ Newbridge)	2.924	62	11.05	0.194	0.241	0.613
39034 (Evenlode @ Cassington Mill)	3.146	42	24.7	0.225	0.133	0.526
Total		516				
Weighted means				0.22	0.15	

7.2 Additional supporting information

chalgrove-PoolingGroupAnalysis.xlsx

FEH.statistical.method.permeable.adjus

FEH.statistical.method.permeable.adjus

FEH.statistical.method.permeable.adjus