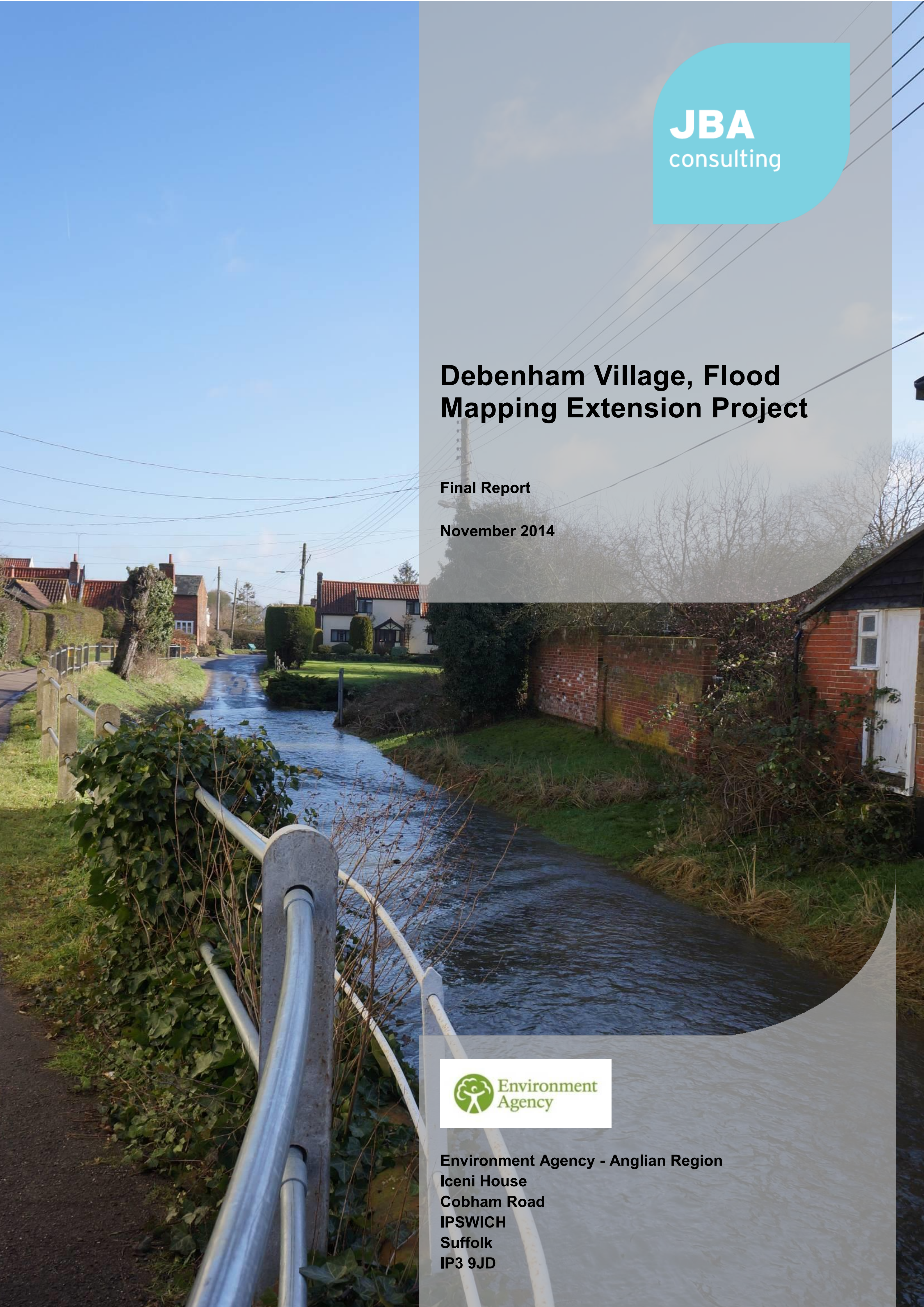

Debenham Village, Flood Mapping Extension Project

Final Report

November 2014

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Revision History

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Draft Report / October 2014		Rebecca Brown
Final Report / November 2014	Minor amendments	Rebecca Brown

Contract

This report describes work commissioned by the Environment Agency by reference AN169. The Environment Agency's representative for the contract was Rebecca Brown. Kevin Haseldine, Colin Riggs and Alex Siddaway of JBA Consulting carried out this work.

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Purpose

This document was prepared as a report for the Environment Agency. JBA Consulting accepts no responsibility or liability for any use that is made of this document other than by the Client for the purposes for which it was originally commissioned and prepared.

Acknowledgements

We are indebted to many people associated with Debenham for the help provided in completion of this project.

Firstly we would like to acknowledge the guidance provided by Rebecca Brown and Will Todd at the Environment Agency in Ipswich. Further thanks go to Suffolk County Council and Paul Bradford of the River Deben Holistic Water Management Plan who provided feedback on draft results at the interim meeting in Ipswich during September 2013.

Evidence of past flooding in Debenham was vital in the calibration of the hydrological estimates and the hydraulic model, much of which was provided by residents of Debenham. Peter Carter provided a detailed photographic record of major flood events dating back to the 1930s, without which it would have been challenging to ensure results matched the historical record.

Discussions with local residents at the community engagement meeting in November 2013 also aided modelling and provided the concept for one alleviation option (removal of the Cherry Tree drain bank).

Further thanks go to Debenham Community Centre for hosting the community engagement meeting.

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Executive summary

This study was commissioned by the Environment Agency in order to improve the understanding of flood risk at Debenham, a large village in central Suffolk. A key component of the project was to update the existing Flood Zones for the area, and to produce a range of potential flood alleviation options. This report builds on the final May 2014 report submitted by JBA, including extension of the hydraulic model and inclusion of local spot level topographic survey.

As part of the commission, hydrological modelling of the relevant catchments and the development of a hydraulic model of the Debenham area was undertaken; in places this involved updating an existing hydraulic model. The study reaches include the River Deben as it flows through the village, The Gulls and Cherry Tree Brook tributaries. In addition to fluvial modelling, the outputs also include a surface water model of the village with associated flood mapping.

Initial modelling significantly increased flood outlines from the original Flood Zones due to a combination of improved structures representation and updated hydrological analysis. The addition of a TUFLOW floodplain component allowed for detailed representation of the flow paths through the village. It was found that flooding occurs when the channel capacity was exceeded at Market Square, alongside surcharging of the Aspall Road culvert. Flooding also occurred from Cherry Tree Brook, in part driven by the backwater effect of high flows on the River Deben and the topographic constriction upstream of Fen Street.

Draft results were presented to both the Environment Agency and Debenham residents at a community engagement meeting in November 2013. Evidence of past flood events gathered from this meeting was used to update the hydrological estimates and the hydraulic model to produce more robust flood mapping outputs, resulting in increased flood extents.

Following the 2013-14 model update, results now indicate the Market Square in the centre of Debenham is flooded on average once every 20 years which ties in well with the photographic record and anecdotal evidence. Flood extents increased elsewhere in the village, such as the area near the fire station, providing a good match to photographs of flooding in 1993. Further work was undertaken in summer 2014 to refine the model, including spot level survey in Debenham to improve representation of flow paths south of Water Lane.

Part of this commission involved investigation of various flood alleviation options for Debenham following completion of the hydraulic modelling. These were discussed at a meeting with the Environment Agency and SCC in September 2013 and subsequently at the community engagement meeting in Debenham. Eight schemes were taken forward for further investigation during the final stages of this project, represented in hydraulic models. The modelling demonstrated that the option which provides the greatest flood risk benefit in terms of properties protected is the construction of an impounding reservoir on The Gulls watercourse near Aspall. Development of a two stage channel downstream of the village also has significant flood risk benefits.

In addition to hydraulic modelling, work was undertaken to assess the predicted economic damages associated with a given design flood and also the average damages per year. These results, alongside the model results, were used to calculate a preliminary GiA PF score for each scheme.

It is concluded that in addition to providing the greatest flood risk benefit, The Gulls reservoir also offers the highest PF score, of all the considered options, at 28%. It is recommended this option is considered by the Environment Agency; if taken forward a full feasibility and detailed design project is required.

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Abbreviations

1D .... One Dimensional (modelling)
2D .... Two Dimensional (modelling)
AAD .... Annual Average Damage
ASCII .... American Standard Code for Information Interchange
CESMM .... Civil Engineering Standard Method of Measurement 4
DTM .... Digital Terrain Model
FEH .... Flood Estimation Handbook
FCRM .... Flood and Coastal Risk Management
FDGiA .... Flood Defence Grant in Aid
FRISM .... Flood Risk Metrics software
GiA .... Grant in Aid
GIS .... Geographical Information System
HR .... Hydraulic Research, Wallingford
IDB .... Internal Drainage Board
ISIS .... Hydrology and hydraulic modelling software
LIDAR .... Light Detection and Ranging
mAOD .... metres Above Ordnance Datum
M-CM .... Multi-Coloured Manual
NaFRA .... National Flood Risk Assessment
NFCDD .... National Flood and Coastal Defence Database
NFU .... National Farmer's Union
NRD .... National Receptor Dataset
OM .... Outcome Measures
OS .... Ordnance Survey
PF .... Partnership Funding
Q100 .... Flow at the 100-year return period
QMED .... Median Annual Flood (with return period 2-years)
ReFH .... Revitalised Flood Hydrograph method
TUFLOW .... Two-dimensional Unsteady FLOW (a hydraulic model)
SCC .... Suffolk County Council
SFRM2 .... Strategic Flood Risk Management
SI .... Site Investigation
WFD .... Water Framework Directive

1 Introduction

1.1 Project overview

The Environment Agency commissioned JBA in 2013 to undertake hydrological analysis and hydraulic modelling with a view to improve the understanding of flood risk in Debenham, a large village in central Suffolk with a long history of fluvial flooding. A key component of the project was to update the existing Flood Zones for the area, and to produce a range of potential flood alleviation options.

New hydrological modelling of the relevant catchments and the development of a hydraulic model of the Debenham area was undertaken; involving the update of an existing hydraulic model originally constructed by JBA Consulting in 2007 and updated in 2010. The study reaches include the River Deben as it flows through the village, The Gulls and Cherry Tree Brook tributaries (see Figure 1-1). The 2, 5, 10, 20, 75, 100 and 1,000-year return periods were modelled, including a climate change allowance for the 20, 100 and 1,000-year events. In addition to fluvial modelling, outputs for the project included a surface water model of the village with associated flood mapping.

Draft results were presented to both the Environment Agency and Debenham residents at a community engagement meeting in November 2013. Evidence of past flood events gathered from this meeting was used to validate and improve the hydrological estimates and the hydraulic model to produce more robust flood mapping outputs, resulting in increased flood extents. This report and the associated results were delivered to the Environment Agency in May 2014.

Following delivery of this project, JBA was asked to extend the hydraulic model further upstream to assess the wider impacts of a potential flood storage reservoir, identified during options testing in the May 2014 report. The current report therefore supersedes the May report, including all the information regarding initial model development in addition to the changes made subsequently.

This report, consisting of 10 chapters, was written to Strategic Flood Risk Management (SFRM2) guidance[1] and provides an overview of the project:

1. Introduction
2. Community engagement meeting
3. Technical approach
4. Description and discussion of baseline model results
5. Economic damage assessment
6. Discussion of potential flood alleviation options
7. Cost benefit Partnership Funding analysis of the proposed options
8. Discussion of limitations and assumptions
9. Study deliverables
10. Conclusions and recommendations.

In addition to the main body of the report, various appendices are also provided. These support this document and provide a detailed technical explanation and audit trail of the approaches applied. These include hydrological analysis (Appendix A), hydraulic modelling (Appendix B), economic damage assessment (Appendix C) and cost estimates for the alleviation options (Appendix D).

1.2 Study area

Debenham is a large village located in central Suffolk, around 20km due north of Ipswich. In old English the word Debenham is derived from "village in a deep valley"; this accurately portrays the location of the old portion of the village, on the banks of the River Deben, which flows north-south through the settlement. Newer developments have been located on a spur of high ground running between the River Deben and the Cherry Tree Brook tributary. Development of the village is ongoing.

[1] Environment Agency, 2010. *Hydraulic Modelling and Risk Mapping Model Report Performance Scope.* (Created by the SFRM user group).

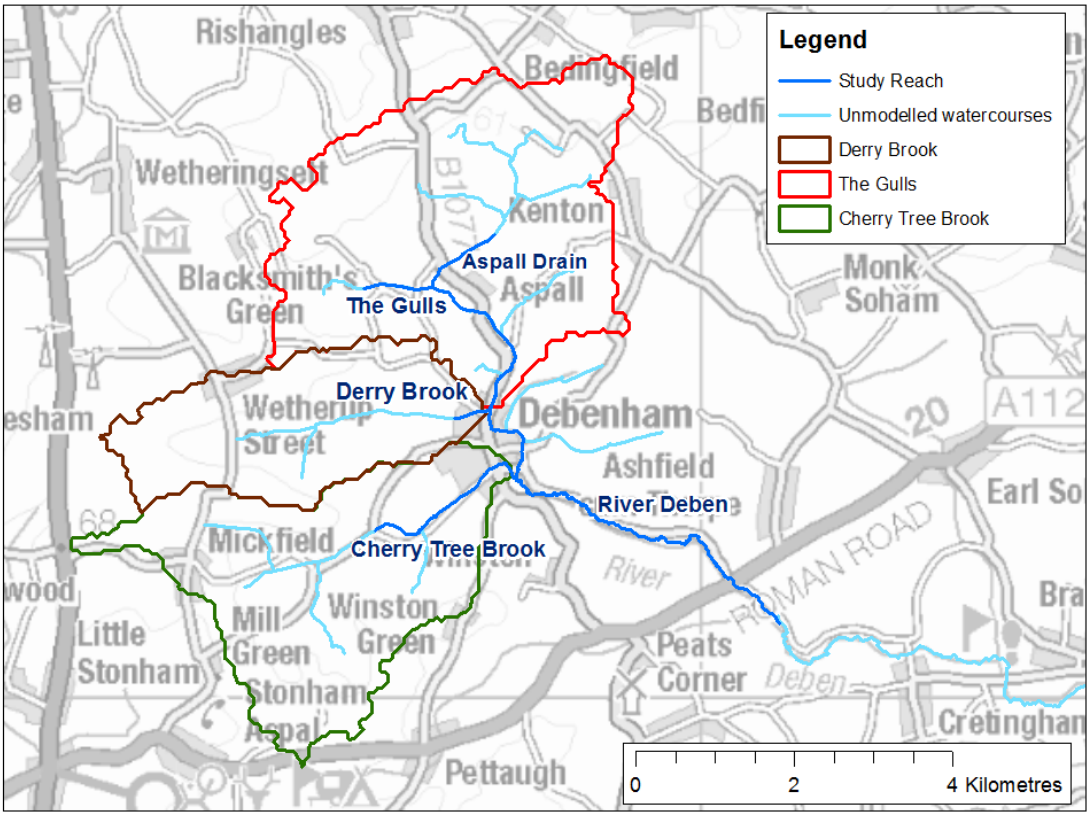
One of the defining features of the village is associated with the River Deben, coming in the form of Water Lane, where a 100m long ford conveys the watercourse over the road. Flooding of this area and the upstream Market Square is recorded in the provided flood history.

A significant confluence is present at the junction of Aspall Road and The Butts; downstream of this point the watercourse is referred to as the River Deben. The naming convention of the upstream watercourses differs between various sources, but residents of the village refer to the larger watercourse flowing from the north as The Gulls and the smaller watercourse from the west as Derry Brook. The Gulls drains a rural catchment including the land around Aspall, including the apple orchards associated with the cider of the same name.

The southern portion of the village is bounded by the rural Cherry Tree Brook watercourse, flowing east and joining the River Deben at Kenton Road. A small surface water drain also flows into Cherry Tree Brook, located at the southern end of the High Street.

The study reaches used in the May 2014 report included The Gulls from Aspall to Debenham, Cherry Tree Brook from Bush Corner to Debenham and Derry Brook/River Deben throughout the village. Downstream, the model extended to downstream of the A112 road bridge. For the current project The Gulls has been extended upstream to Redhouse Farm, and an unnamed tributary to The Gulls added to the east (hereafter referred to as Aspall Drain). Study reaches are shown in Figure 1-1 below.

Figure 1-1: Study location, subject watercourses



1.3 Available data

Various datasets were made available at the outset of the 2013-14 project including Geographical Information System (GIS) layers such as Ordnance Survey (OS) background mapping[2] and MasterMap[3], supplied by the Environment Agency, along with filtered and

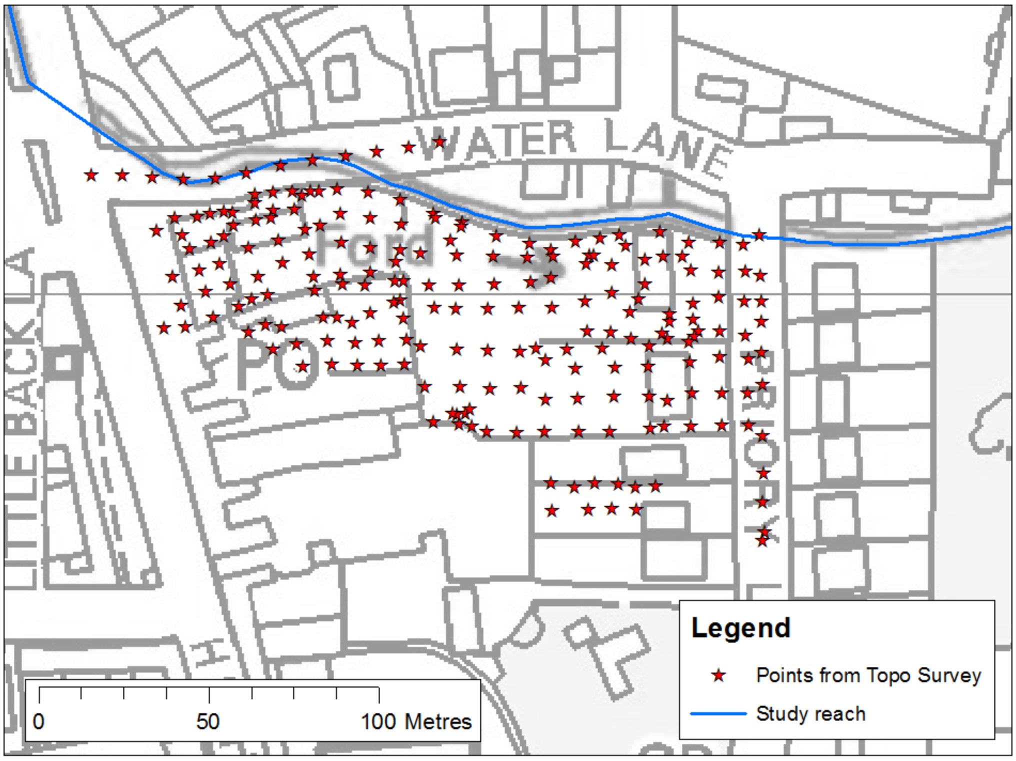
[2] OS 1:10,000, 1:25,000 and 1:50,000 mapping **(Licence number: Z17791)**

[3] OS Master Map data **(Licence number: Z17791)**

unfiltered Light Detection and Ranging (LIDAR) data[4]. Anglian Water sewer maps[5] were also provided. A photographic flood history was also supplied.

In addition, three topographic surveys were available for the channels in Debenham. This first of these was completed in 2007 and provided the topographic information required for the 2010 project. The second dataset was commissioned for the 2013-14 project, undertaken by Maltby Land Surveys Ltd in early 2013, providing updated survey at specific channel cross sections along with new survey of selected hydraulic structures. Finally, Maltby Land Surveys Ltd provided additional channel survey of The Gulls and Aspall Drain in summer 2014 to inform the extension modelling. This survey also included a spot level survey of land to the south of Water Lane, shown below in Figure 1-2. This area was surveyed to improve flow path representation and is discussed further in 3.2.

Figure 1-2: Location of Water Lane spot level topographic survey



There are two Environment Agency managed flow gauges within the River Deben catchment (Figure 1.3 in Appendix A); a crump weir (with associated rating curve) located at Naunton Hall and the other an ultrasonic gauge at Brandeston. Neither of these is included within the HiFlows-UK dataset and both are downstream of the current study domain. Three water level gauges are (or have been) located in the upper catchment near Debenham, one on each of the subject watercourses. Data from all gauges were supplied for the current project.

Rainfall records were also supplied from gauges at Needham Market, Stradbroke and Great Finborough.

Following the community engagement meeting held on 20 November 2013 (see chapter 2 for details) further datasets were made available for analysis. These included a more extensive photographic record of events (supplied by a local resident), additional hydrometric data until December 2013 and maps produced by residents at the community event.

[4] 1m and 2m LIDAR data **(Licence number: GMG-230-050213-12894)**

[5] Anglian Water Sewers Maps **(Data sharing agreement dated 21/03/2013)**

1.3.1 Previous studies

An existing ISIS one dimensional (1D) model of the River Deben and selected tributaries was provided by the Environment Agency for use in the 2013-14 project[6]. The model was originally constructed in 2007 and updated in 2010 by JBA Consulting[7]; part of a catchment wide study from upstream of Debenham to the tidal outfall at Woodbridge.

1.4 Flood history

Despite the low average annual average rainfall at Debenham, typical of the east of England, many extreme events have occurred in the upper Deben catchment. These heavy rainfall events, combined with the relatively low lying nature of some streets in Debenham, have resulted in the village experiencing frequent flooding in the recent past.

Photographs taken in 1936, 1937, 1944, 1947, 1956, 1968 and 1993 illustrate flooding at various locations in Debenham. Flooding appears to be driven by high levels on both the River Deben and Cherry Tree Brook. Significant flood depths were recorded at the Market Square, Water Lane, Priory Lane and the south of the High Street near the fire station and at Cross Green. A selection of the supplied photographs is provided in section 1.2.2 of Appendix A.

1.5 River Deben Holistic Water Management Project

In addition to the current project - focusing on flood risk management in Debenham - a study is currently underway regarding holistic water management in the River Deben catchment, authored by Bradford and Brighton Ltd.

The aim of this project is to link flood risk management with water resources management and Water Framework Directive (WFD) objectives throughout the River Deben catchment. Its concept came about following the Suffolk Flood Risk Management Strategy which advocated a more integrated approach to management, along with highlighting the water scarcity in East Anglia. Various stakeholders are involved, including the Environment Agency, Natural England, the National Farmers Union (NFU), Internal Drainage Boards (IDBs), Suffolk County Council (SCC) and water companies.

Phase 1 of the project identified the River Deben as a suitable catchment for investigation, given the high demand on water resources associated with agriculture, particularly around the Brandeston area (downstream of Debenham).

Phase 2 is underway and involves investigating management options including public engagement. Options under consideration include, but are not limited to:

- aquifer water storage
- river support systems
- winter stored surface water and
- flood attenuation.

A number of these options may serve dual benefits for water resources and flood mitigation. These concepts were considered during the development of flood alleviation schemes for the current project.

[6] River Deben Flood Risk Study and 2007 Topographic Survey **(Licence number: Z22442)**

[7] Environment Agency, 2010. *River Deben Model Review–Phase 2: Improvement of model.* Prepared by JBA Consulting.

2 Community engagement meeting

Following production of the hydraulic model, technical details of which can be found in Appendices A and B, draft results were presented to the Environment Agency and various stakeholders at a meeting in Ipswich in September 2013. In addition to flood mapping results, an array of potential flood alleviation options was presented, details of which are included in section 6.2 of this document. Following the meeting JBA was invited to present the findings to residents at a community engagement meeting held at Debenham Community Centre on 20 November 2013.

The purpose of the meeting was to keep local stakeholders informed and engaged of the ongoing work in Debenham, whilst seeking public opinion on the draft results and potential flood alleviation options. During the meeting many residents provided anecdotal evidence of flooding in the village, highlighting areas where it was felt draft outlines could be improved.

The general consensus was that the flood outlines presented were underestimating flood risk to Debenham. A number of locations were highlighted which have flooded in the recent past but that were not shown to flood in the draft outlines. Details are provided below in Table 2-1. A detailed map of Debenham, with the locations below highlighted, can be found in Appendix E.

Table 2-1: Summary of findings from the community engagement meeting

Location	Details	ID for Appendix E map
Market Square	Historic photographs helped to identify Market Square as a flood prone area at the project outset. However, Debenham residents suggested flood waters frequently extended further south than draft outlines. In the 1956 event flood water was present to the south of the Angel public house.	A
Chancery Lane	Draft flood outlines also appeared to underestimate extents at this location. Various residents remember flood waters being present along the length of the Lane, almost joining with flooding at the Market Square.	B
Little London Hill	One resident noted recent flooding of the garden of The Red House as recently as 2012. This site was visited during the meeting and it was observed that the garden sloped significantly downwards towards the channel. These considerations are taken into account in subsequent modelling and mapping of flood extents.	C
Priory Lane	Draft outlines along Priory Lane were thought to slightly underestimate the flooding experienced here. Photographs of the 1993 event confirm that significant flooding does occur at this location.	D
High Street - Cross Green	Flooding was recorded on the non-main river drain flowing behind the houses to the south of the High Street. Draft results indicate flood waters from Cherry Tree Brook flow up this drain and flood gardens; this matches well with the observed flooding. Householders in the area have previously excavated the small left bank of the drain to alleviate local flooding; formalising this method of mitigation is now considered in the options modelling.	E
Fen Street	A resident of Fen Street drew an outline on the mapping provided which was representative of an event in May 2013. This corresponded well with the draft 20-year outline, although the constrained nature of the topography at this location ensures outlines from various return periods are similar in extent.	F

General comments
Many residents observed water levels rising particularly rapidly in the 1993 event. This was widely attributed to the closure of tidal lock gates at Woodbridge, some 40km downstream of Debenham. Sensitivity analysis to the modelled downstream boundary, where water levels were increased by 1m, showed no changes to water levels at Debenham. This therefore demonstrates the River Deben is sufficiently steep to ensure closure of such gates does not influence water levels at Debenham. It should also be considered that no residents suggested blockage of structures were the cause of the 1993 event's atypical rate of rise. This indicates the rapid increase in water levels was most likely caused by a hydrological factor, this is considered in our subsequent analysis.
Perception among some residents was that Cherry Tree Brook contributed the majority of flood water to the village. Our hydrological analysis does not support this, with The Gulls being the largest contributing catchment both in terms of area and peak flows (Figure 1-1). However, it is true that flow in Cherry Tree Brook is the dominant influence on flood risk in the parts of the village that directly adjoin it.
One resident provided anecdotal evidence that no changes have been made to structures in Debenham since 1930. This is useful information as it confirms that any change in frequency of historic flood events was not driven by structure alterations.

3 Technical approach

This section of the report outlines the methodology employed to produce flood mapping outputs for Debenham. The techniques were initially used to produce draft outputs and subsequently refined, to better reflect the occurrences of flooding as highlighted at the community engagement meeting. These results were recorded in the May 2014 report. Subsequently, the model was extended upstream in The Gulls catchment, whilst further alterations have been made in Debenham to improve representation. This section records all technical work from 2013 to present.

3.1 Hydrology

A basic overview of the hydrological techniques employed to estimate design flow hydrographs at Debenham is provided here. In addition, some information is provided regarding the surface water component of the model. Full technical details are available in Appendix A.

The hydrological method statement highlights the lack of suitable flow data at Debenham; instead our analysis utilised data recorded at three level gauges. The two closest flow gauges to Debenham (Naunton Hall and Brandeston) are deemed to be unrepresentative of the catchment at Debenham; both are located much further downstream and are influenced by an area of permeable geology which does not affect Debenham. Moreover, the flow series at Brandeston is considered unreliable. Therefore these data sources were not used to refine hydrological estimates.

It should be noted that given the lack of flow data the hydrological estimates used are subject to a degree of uncertainty. We have, however, made full use of all available data (flow gaugings, level data etc.) in order to improve confidence as far as practical; this is explained in further detail in Appendix A.

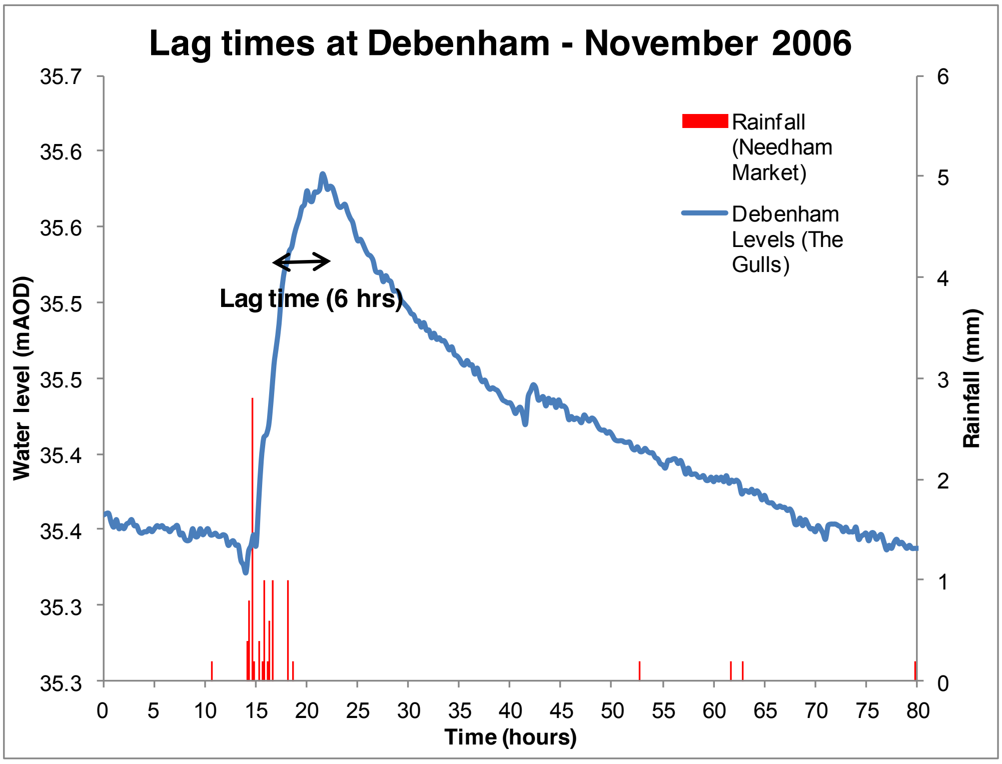
3.1.1 Fluvial inflows

The first step in most hydrological studies is to evaluate the available data against the project requirements and to select the most appropriate technical approach. For the Debenham study a range of options were considered and the Revitalised Flood Hydrograph (ReFH) methodology adopted for the final design flows for the following reasons:

- Makes the best use of available water level and rainfall data, allowing calibration of time-to-peak.
- Produces greater flow estimates that the Flood Estimation Handbook (FEH) Statistical Method. Although this should not be the sole justification it is noted that the existing Flood Zones underestimate flooding at Debenham; these were produced using FEH Statistical.
- When run through the hydraulic model the draft flow estimates produced flood outlines which were much more extensive than those previously available and appeared to correlate much better with the local flood history. However, findings from the community engagement meeting suggested draft outlines were still underestimating flood risk in Debenham; as a result of this improved local knowledge estimates were redefined.
- Allows for robust integration with the surface water model (which requires rainfall inputs).

ReFH is a rainfall-runoff model, which effectively routes a design rainfall event through a catchment, producing a design hydrograph for input into the hydraulic model. The availability of level and rainfall data allows the lag time associated with each watercourse to be estimated (the difference in time from the rainfall centroid to the peak level); this is then converted to time-to-peak and added as a parameter to the ReFH model. Figure 3-1 provides an example of lag time on The Gulls.

Figure 3-1: Catchment lag time, The Gulls



By analysing the entire record length of all three level gauges, it was found that each exhibits similar behaviour following an extreme rainfall event. Average lag time for each watercourse was converted to time-to-peak. The shortest time-to-peak value of all the watercourses was calculated at 4.0 hours on Cherry Tree Brook compared to the longest of 4.7 hours on The Gulls. It is therefore assumed that the same design event will occur on each catchment for a given rainfall event.

Additional data was obtained following the community engagement meeting for the 2013-14 period of record and added to the analysis dataset. It was also identified that the three contributing catchments exhibited a relationship between rainfall intensity and lag time; increased rainfall intensity resulted in decreased lag time (see Figure 3-2 for an example on Cherry Tree Brook). Given this finding, the upper quartile (top 25%) of maximum rainfall intensity events were extracted and used to calculate new time-to-peak values. This may go some way to explaining the rapid time to rise associated with water levels in the 1993 flood.

Shorter time-to-peak values result in a greater flood peak at Debenham, and also result in similarly timed peaks from each watercourse. Final parameters at each gauge are included in the Table 3-1 below.

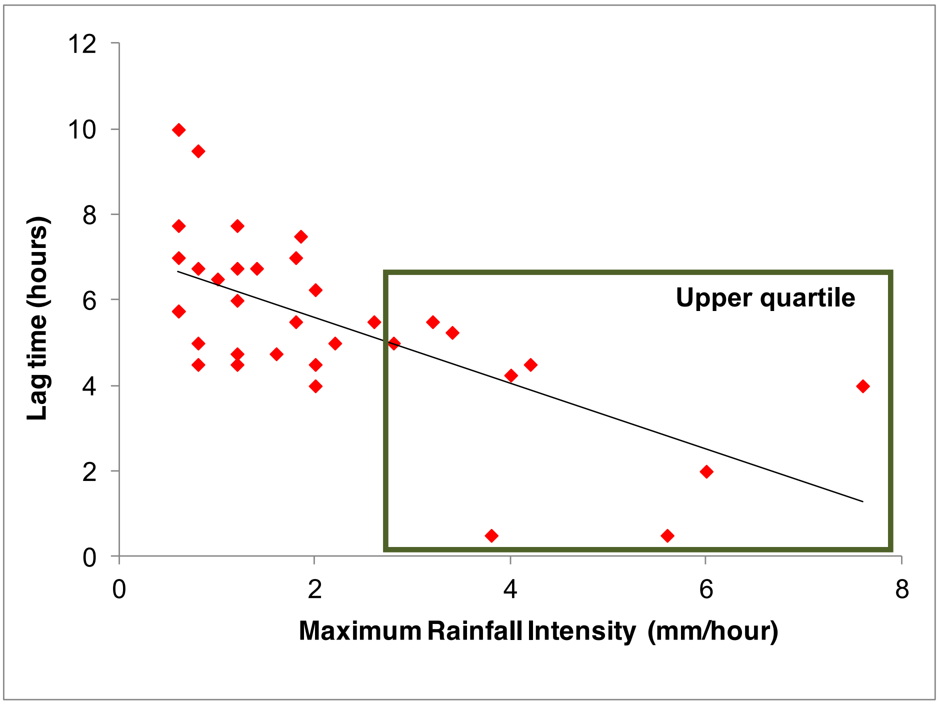
Table 3-1: Final time-to-peak values

Watercourse	Final time-to-peak (hours)
Derry Brook	3.26
The Gulls	3.53
Cherry Tree Brook	2.18

The fluvial inflows are modelled with a critical storm duration of 7.5 hours. This has been determined by running various durations through the hydraulic model and analysing results to establish which produced the greatest water levels.

Storm profiles are set to winter to reflect the rural nature of the contributing catchments, where long duration, low intensity storms are most likely to result in extreme floods.

Figure 3-2: Rainfall intensity and lag time relationship - Cherry Tree Brook



3.1.2 Direct rainfall inflows

The surface water component of the model requires direct rainfall to be added to the model over the village itself. In this version of the model, the area onto which rainfall is modelled was removed from the fluvial inflows.

The rainfall hyetograph was taken as the net rainfall produced from the ReFH model. The storm duration was set to one hour and the profile set to summer; surface water floods in urban areas are most likely to occur as a result of short, intense convective storms. In order to maintain consistency the fluvial critical storm duration was also set to summer. Further information regarding estimation of direct rainfall inflows is available in section 5.4, Appendix A.

3.2 Hydraulic modelling

New modelling was undertaken for Debenham using the existing 2010 ISIS model of the River Deben as a basis; updating and improving upon this broad scale model where necessary for the May 2014 model. Subsequent changes have extended the model upstream in The Gulls catchment.

3.2.1 One dimensional ISIS model

The existing 2010 1D model was updated to allow linking to TUFLOW. This involved trimming cross sections to the channel banks within Debenham. A number of additional open channel cross sections were added where new topographic survey was undertaken in 2013, and additional sections added on The Gulls and Aspall Drain to extended the model further upstream; these were surveyed in summer 2014. The purpose of the model extension was to assess the upstream impacts of a proposed flood storage reservoir on The Gulls, discussed further in 6.2.4.

Floodplain geometry was updated using new LIDAR data where these were retained in the 1D domain.

One of the main tasks was the update of all structures within Debenham. New topographic survey allowed for these to be represented in greater detail. One of the major structures which impacts water levels is the Aspall Road culvert near Market Square. This feature was updated in the model as a rectangular culvert, originally with a skew angle of 45º to account for the sharp change of channel direction immediately upstream, but subsequently changed to 60º to better reflect the losses associated with the structure (illustrated in Figure 3-3).

The downstream boundary was converted to a normal depth boundary; full details of this and the hydraulic structures are available in Appendix B.

Figure 3-3: Aspall Road culvert skew at Market Square

3.2.2 Two dimensional TUFLOW model

The original 2010 model used extended ISIS cross sections to represent the floodplain; this is a perfectly valid approach in some instances although an upgrade to a two dimensional (2D) TUFLOW model was considered more appropriate to represent detailed flow paths throughout the settlement and to facilitate surface water modelling.

TUFLOW models are based on a digital terrain model (DTM), in this case developed predominately using filtered LIDAR data. The ground model was read directly into TUFLOW, the modelling software converting the data into "z-points" (a GIS point file of elevations at grid cells centre, sides and corners). The z-points are used to produce an elevation grid, in this instance at a 2m resolution. The grid can be subsequently modified to represent other topographic features (such as building footprints or embankments) without alteration of the raw LIDAR data. It is also straightforward for future users to establish the changes made to the original topography.

Additional spot topographic survey was commissioned in 2014 immediately south of Water Lane in Debenham; this was identified as a flow path during the 5 and 10-year events from the 2013-14 model. However, anecdotal evidence gathered since implies this flow route is unlikely to be active during the 10-year event. This area includes dense vegetation, filtering of which can create inaccuracies in LIDAR elevations, a possible explanation for the larger than expected flood extents. Therefore it was considered appropriate to collect spot level survey in order to increase confidence in the modelled topography at this location. The supplied spot levels were converted into an ASCII grid and read directly into the TUFLOW model.

The TUFLOW domain is dynamically linked to ISIS cross sections using HX lines. These allow the boundary cells in TUFLOW to calculate the flow passing between the model domains based on water level in the ISIS model.

Direct rainfall modelling was also undertaken for Debenham to assess surface water flood risk. Rainfall was applied directly to the TUFLOW domain. For this model two test scenarios were run; the first of these assumes no infiltration and the second has a continuing loss of 10mm/hr throughout the model run. This infiltration was tested to establish the likely impact of both soil infiltration and also loss to the surface water sewer system. The results illustrated that outlines and depths were very similar between the no infiltration and infiltration model version. Therefore the no infiltration scenario was adopted for design model runs.

3.2.3 Defences

No formal defence schemes are present in Debenham, removing the need for a defended and undefended model scenario. However, the area is currently subject to a maintenance scheme undertaken by the Environment Agency, which removes excess vegetation from the channel and banks bi-annually.

3.2.4 Calibration following community engagement meeting

Given the draft outlines were deemed to underestimate flood extents, further work was undertaken to re-parameterise certain parts of the hydraulic model following the meeting. Without full calibration data (i.e. flows and levels from a given event) the alterations were made in an attempt to better match the historical flooding recorded and residents' anecdotal evidence. Whilst this is an appropriate method, model parameters should not be pushed outside realistic ranges; for example local factors not represented in the model, such as blockages, may have contributed to particular past events.

One dimensional hydraulic roughness

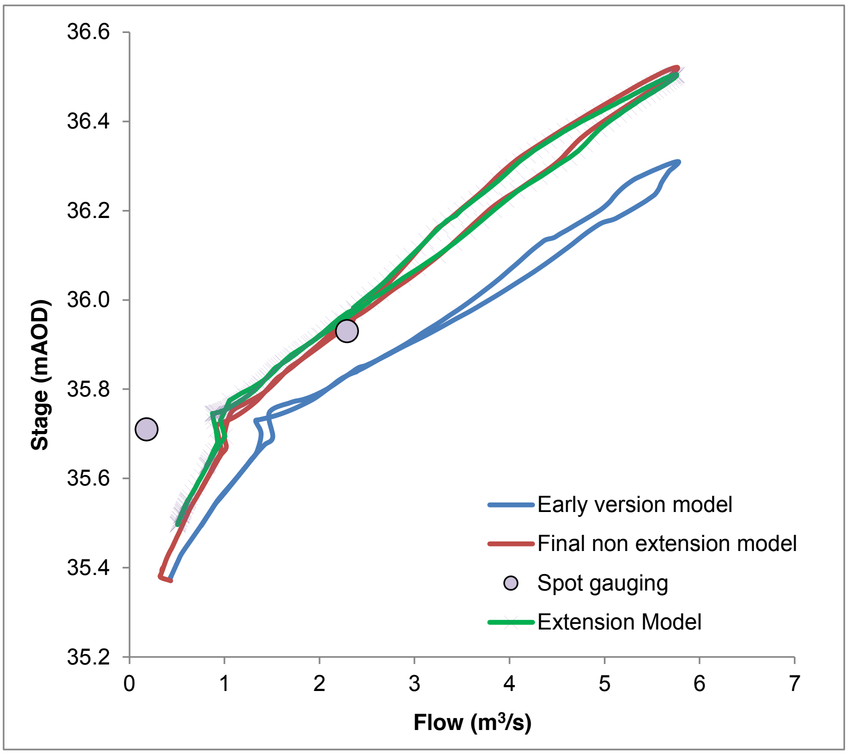
The most significant change to the draft model is associated with hydraulic roughness. Draft Manning's *n* values were estimated using Cowan's equation which accounts for various channel characteristics such as bed material, obstructions, vegetation, cross section variability and meandering. This approach is highly subjective.

Following the meeting, the Environment Agency provided two spot gaugings, one at the level gauge on The Gulls, and another at Low Road bridge on Cherry Tree Brook. These were undertaken in the winter when there was little vegetation growth in the channel. No level was supplied for Cherry Tree Brook and therefore was inferred from the supplied photograph; there is also significant hysteresis noted in the model results suggesting the gauging site is located within the backwater length of the River Deben (making it difficult to relate flow and level in Cherry Tree Brook). For this reason it is recommended that any further flow gaugings are undertaken further upstream on Cherry Tree Brook.

It is evident from these gaugings that the draft model under-predicted levels for a given flow. The low water levels at both sites and the aforementioned small outlines suggested an increase in hydraulic roughness may better represent in-channel resistance. Roughness was increased on an iterative basis until the results matched the spot gauging; the example below is from The Gulls.

Following supply of the May 2014 report, some further work was undertaken to improve hydraulic roughness representation in Debenham. This included reduction in roughness downstream of Priory Lane bridge, to account for the wood encased channel, and immediately downstream of the Cherry Tree Brook - River Deben confluence. Results in this locality were felt to overestimate flood risk to the nearby Fen Street previously.

Figure 3-4: Modelled rating curves on The Gulls



It is clear from the graph above that the final stage predicted by the new hydraulic model fits the observed higher flow gauging significantly better than the early development model version. However, there is a poor fit to the low flow gauging. This is likely to be due to the greater influence of small scale features and vegetation during low flows; such characteristics will not be well accounted for in the model given the relatively coarse cross section spacing. Whilst this implies the model does not perform as well at very low flows, the purpose of this study is to assess the flood risk at Debenham and therefore it is encouraging the results fit the higher spot gauging. If the model were to be used in future for low flow analysis is it recommended a finer resolution topographic survey is undertaken.

A similar trend is seen on Cherry Tree Brook where a steady flow of 1.77m^3/s results in a stage of 33.04mAOD, only around 0.03m lower than that recorded. It is not possible to illustrate this in the manner above due to the hysteresis present in the hydrodynamic model caused by the backwater effect of the River Deben.

This approach makes best use of the available data although should be re-visited if additional check gaugings become available. It is our belief that the resulting outputs offer the best representation the flood risk to the village using the information presently available. Roughness values remain within bands regularly used in hydraulic models.

It is strongly recommended that the model is re-visited if local flow data become available. The presence of a high quality flow gauging station on any of the main channels would significantly improve confidence in flow estimates, and also allow further calibration of the modelled levels against know flows.

Two dimensional hydraulic roughness

Roughness values were also increased for the 2D TUFLOW domain following the meeting to better represent the ground features such as trees and vegetation. Aerial photography was also re-queried to ensure all roughness zones were adequately accounted for.

Form loss coefficients

Form loss coefficients can be added to the 1D-2D boundaries in order to represent hydraulic losses associated with shear resulting from a change in flow direction and velocity over the banks. In the draft Debenham model this parameter was set to 0.25. Following the community meeting values were increased to 0.50 where fences were present on boundaries to make the transfer of water less efficient.

3.3 Outline production

Outlines for the 1D only portion of the model were produced using an automated routine where modelled water levels are mapped onto filtered LIDAR. This was completed for all areas of the model with a 1D floodplain component (i.e. upstream and downstream extents). The same technique enables mapping of water depths in the ISIS channel where TUFLOW represents the floodplain. 2D outlines were produced by contouring the TUFLOW grid outputs at 0m depth. By combining the two types, the flood extents are produced for the entire model.

For the direct rainfall modelling we employed the same technique, although because rainfall is applied to the entire TUFLOW domain, it is necessary to remove shallow areas of flooding. Following discussion with the Environment Agency and SCC any flooding below 0.10m was removed from the outlines.

Modelled outlines were converted in the Environment Agency's National Flood and Coastal Defence Database (NFCDD) format, alongside modelled water level and model extents.

4 Baseline results

The new baseline results generated for the current report (October 2014) are discussed here. These have built upon the draft outlines presented at the community engagement meeting and the final results associated with the May 2014 report. These results have been generated from the model including The Gulls and Aspall Drain extensions and the inclusion of the Water Lane topographic spot level survey.

4.1 Fluvial flood risk

4.1.1 Comparison with pre-community meeting results

Prior to description and analysis of baseline model results, it is helpful to understand the scale of alteration from the pre-community engagement meeting results to the current results. Table 4-1 highlights modelled peak water levels for the 20 -year events at key locations in Debenham.

Table 4-1: Change in water levels between model versions at key locations

Modelled cross section	Location	Pre-Community Event modelled water levels(mAOD)	May 2014 modelled water levels (mAOD)	Final Oct 2014 modelled water levels (mAOD)
		Q20	Q20	Q20
DEBN_5404u	Upstream of Aspall Road culvert at Market Square.	35.4	35.7	35.6
DEBN_5354	Water Lane	35.0	35.2	35.2
DEBN_4934u	Downstream of Priory Lane bridge, Kenton Road tributary.	34.3	34.6	34.6
CHRY_0253u	Upstream of Cherrytree Bridge.	33.6	34.0	33.9

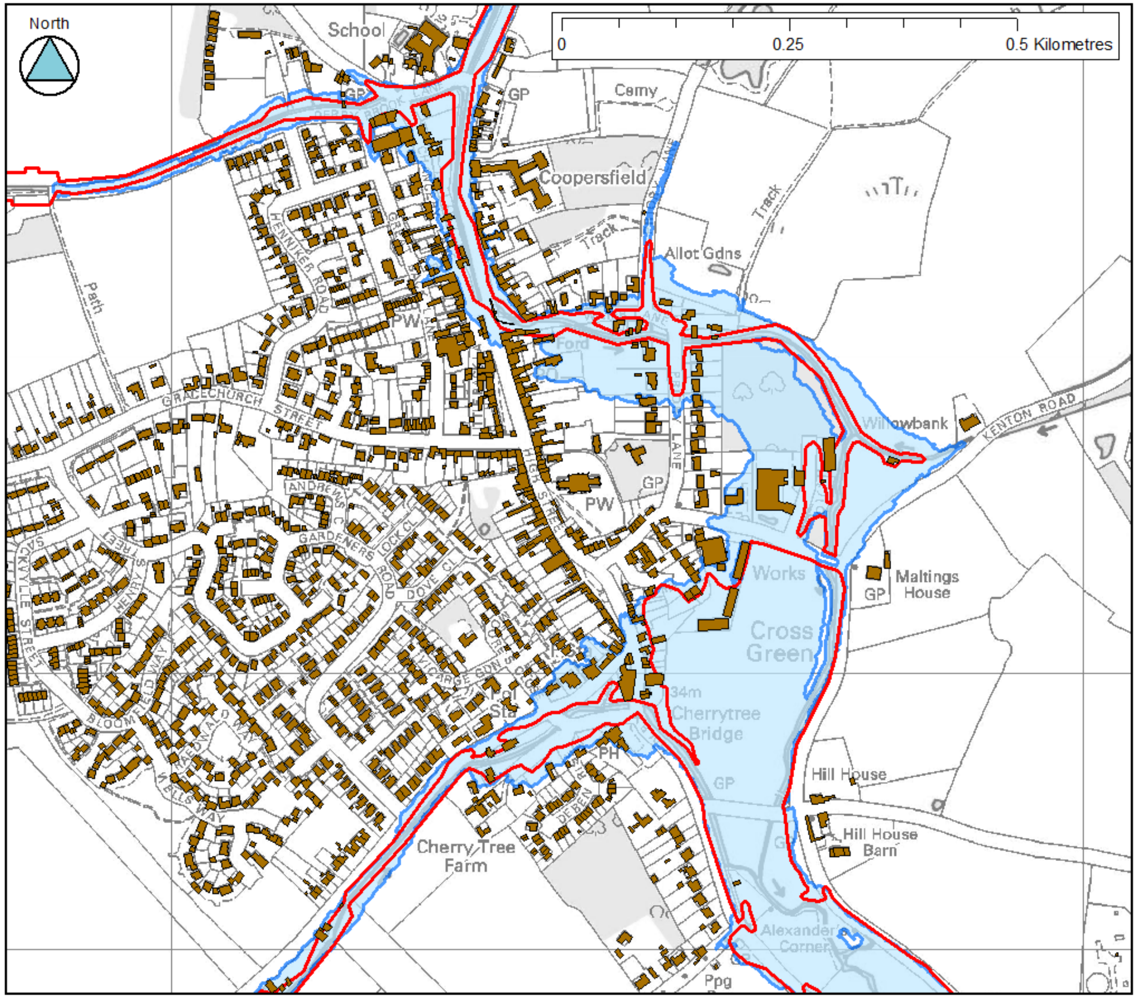
It is evident that water levels increased following the changes made to the draft model as discussed in section 3.2.4. It is encouraging that the largest increases in water level occur around Market Square and Cherrytree Bridge as these areas were highlighted as originally underestimating flood extents. Water levels at Water Lane have only increased slightly for the May 2014 model; changes here are reduced given the smaller adjustment made to 1D hydraulic roughness.

Adjustments made to the October 2014 model had a minor impact on the water levels recorded above from those associated with the May 2014 model. Slight reductions are apparent at Market Square and Cherry Tree Brook reflecting the minor alterations made in these localities.

4.1.2 Final results

Flood risk at Debenham can be ascribed to all three watercourses. Hydrological analysis demonstrated that each catchment is likely to respond in a similar manner to a given rainfall event in terms of time-to-peak. The highest peak flows for a given return period are associated with The Gulls (the largest catchment).

Figure 4-1: 100-year baseline results against existing Flood Zone 3



KEYPLAN Scale **1:600,000**

Thorndon
Bacton
Framlingham
Stowmarket
Woodbridge
Ipswich Urban Area
Capel St. Mary

Legend

- Buildings
- Flood Zone 3
- Q100 flood extent

Fluvial Flood Extent
(1 in 100-year)

Debenham Village Flood Modelling

Produced by: KRH 08/10/2014

Checked by: CER 08/10/2014

Figure 4-2: 20-year baseline results against historic flood photography

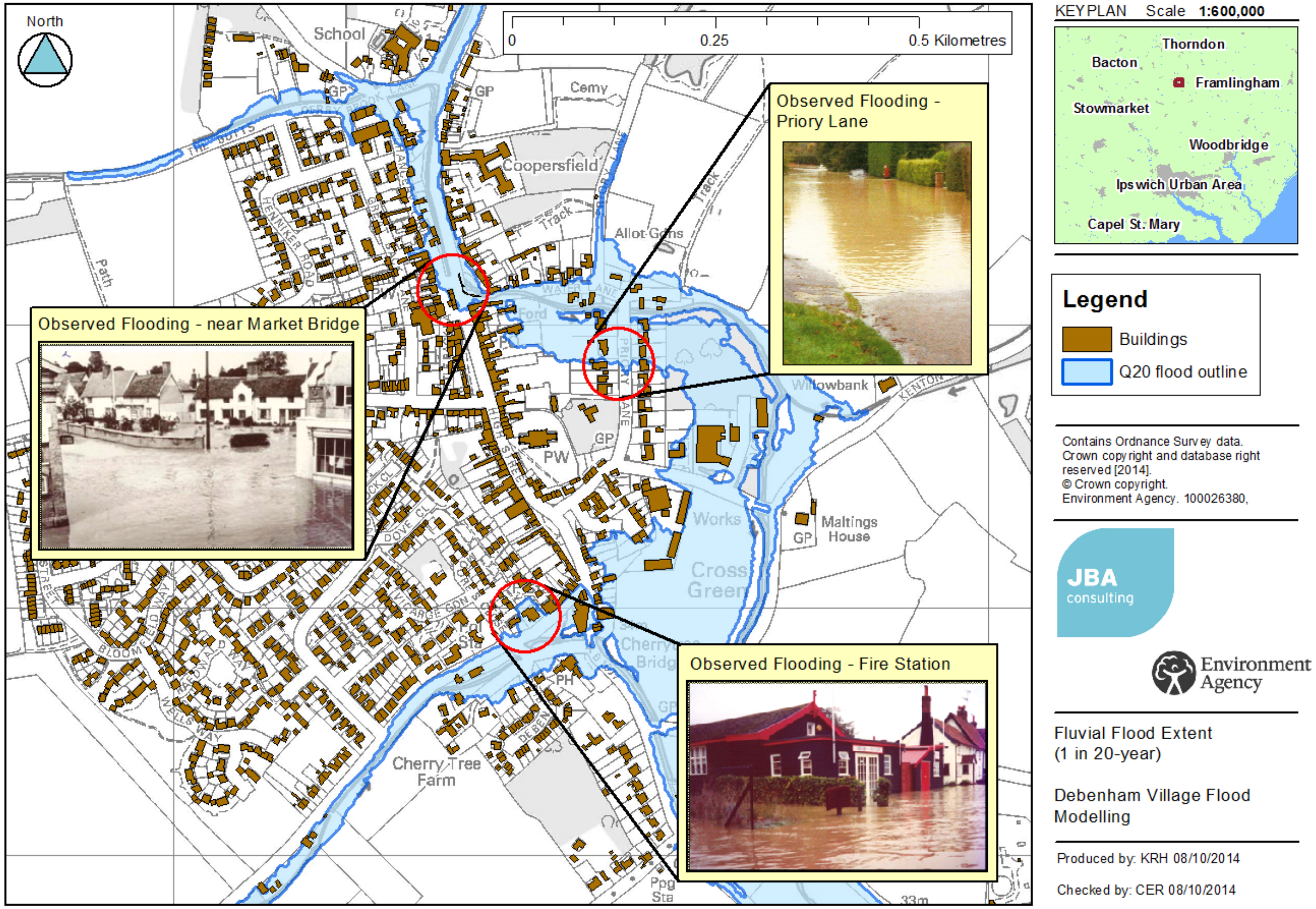


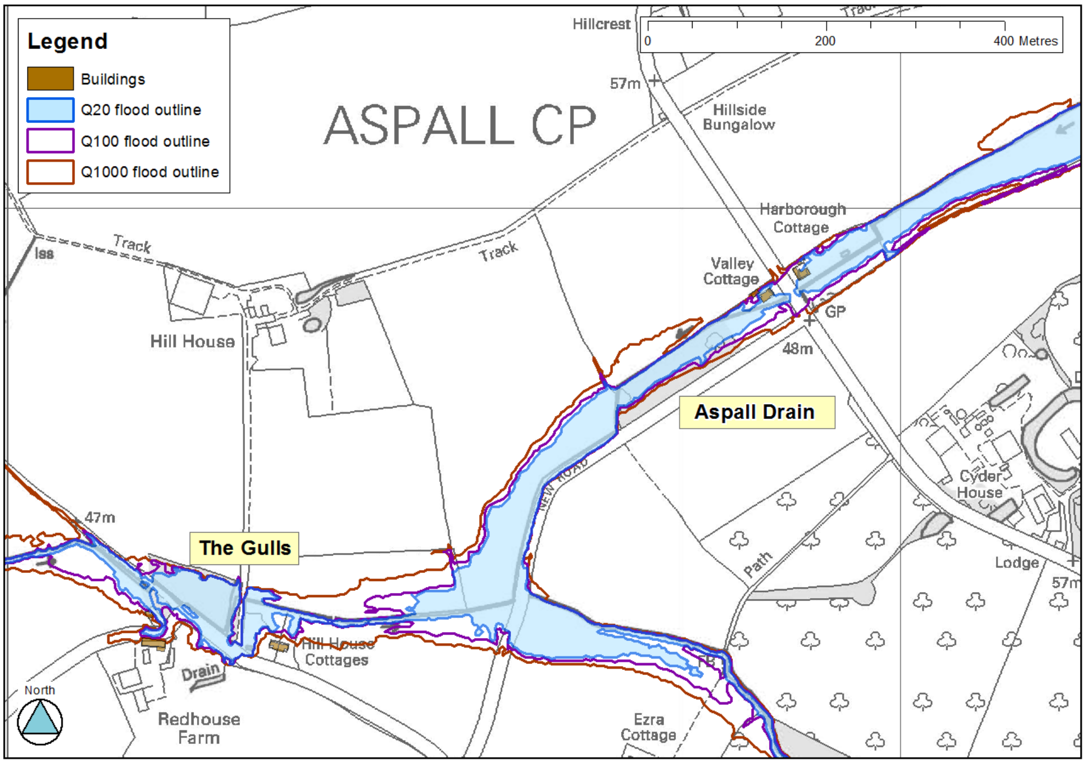
Figure 4-1 illustrates the new 100-year flood outline at Debenham in comparison to the existing Flood Zone 3. It is apparent that the new modelling has increased flood extents throughout the village, notably resulting in more flooded properties around Market Square, Priory Lane and the south of the High Street.

New flood outlines also provide a good match to historic flooding. Analysis of historic photographs indicated flooding occurs at Market Square roughly once every 20 years and therefore it is encouraging that the 20-year outlines (shown in Figure 4-2) corroborate this. Whilst it is not possible to assign a return period individually to the supplied photographs it appears the general trend is now well represented. One resident highlighted flooding at Market Square extended as far as The Angel public house in 1956, generally regarded as the most extreme event of the last 80 years. This was not the case in the draft hydraulic modelling, but following the updates discussed in section 3.2.4, this property is now becomes inundated between the 100 and 1,000-year events.

4.1.3 Changes as a result of additional October 2014 modelling

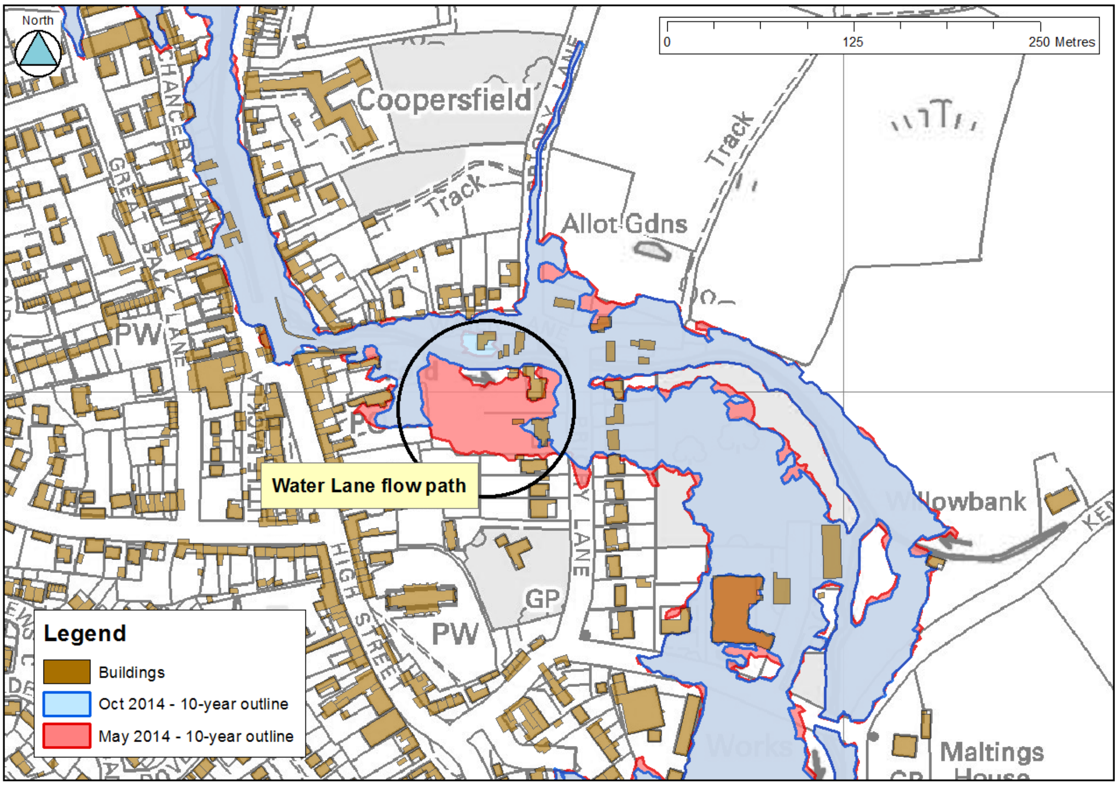
Flood outlines are now available for the extended reach on The Gulls and Aspall Drain (Figure 4-3). The only properties at risk of flooding in this area are the buildings associated with Redhouse Farm on The Gulls, and the cottages seen below on Aspall Drain.

Figure 4-3: Modelled flood outlines at Aspall



Following supply of the May 2014 report, anecdotal evidence has been provided suggesting the right bank flow path from Water Lane to Priory Lane is not active at the 10-year return period. The model has been updated in this area to include spot level survey (see Figure 1-2) and this has resulted in the removal of this flow path from the 10-year flood extent. A comparison of the May 2014 and current flood outlines is provided below (Figure 4-4).

Figure 4-4: Water Lane flow path



4.1.4 Concluding remarks

Flood risk at Debenham is caused by a combination of the following:

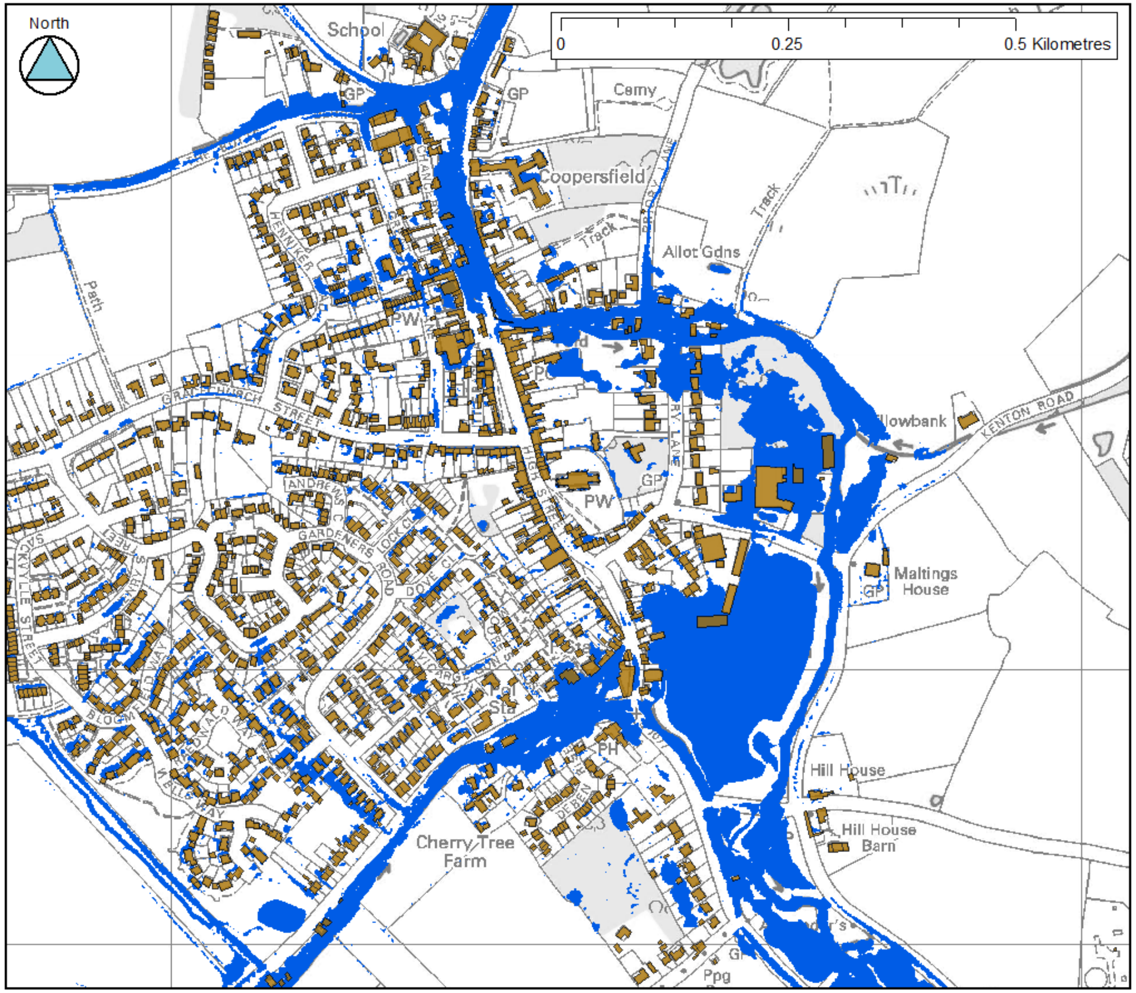
- Flooding at Market Square is driven by discharge rate exceeding conveyance capacity of the Aspall Road culvert and the upstream channel. Historic photographs of flooding in this location demonstrate that modelled outlines are indicative of a long standing issue.
- At the Derry Brook - Gulls confluence and the Cherry Tree - Deben confluence flooding is exacerbated by coincident flood peaks. This is particularly notable on Cherry Tree Brook where the backwater impact of the River Deben results in flooding in the non main river drain running adjacent to the High Street.
- Flooding at Cross Green and around the fire station is impacted by the narrow valley topography between Hill House Bridge and Fen Street on the River Deben. The constriction conveys insufficient water to prevent flooding at Cross Green. This mechanism exacerbates flood risk to the southern end of Debenham High Street.
- The hydraulic model demonstrates that additional frictional losses in the channel at Debenham result in greater water levels. Therefore it is suggested that the current maintenance regime in place at Debenham is continued.

Chapter 6 of this report examines potential flood alleviation options at Debenham. The options under investigation were developed based on the known flood mechanisms discussed above.

4.2 Surface water flood risk

Surface water flood risk from the 100-year event is shown in Figure 4-5; it is apparent that Debenham is not particularly susceptible to this form of flooding predominantly given the steep valley sides in the study domain. The majority of flooding seen is from the channels, only small isolated areas are flooded from surface water. Where surface water flooding does occur, this is a result of water ponding behind obstructions such as buildings.

Figure 4-5: 100-year surface water baseline results



KEYPLAN Scale **1:600,000**

Legend

- Buildings
- Q100 DR Outline

JBA consulting

Environment Agency

Direct Rainfalll Flood Extent
(1 in 100-year)

Debenham Village Flood Modelling

Produced by: KRH 09/10/2014

Checked by: CER 09/10/2014

4.3 Sensitivity testing (May 2014 report)

Sensitivity testing is an important step in hydraulic model development; by testing the model it is possible to highlight areas where uncertainties exist and also to further understand the hydraulic processes occurring. Sensitivity for the Debenham modelling was undertaken in early 2014 and recorded in the May 2014 report. No additional sensitivity tests have been completed for the October 2014 model update, given the relatively minor changes to the hydraulic model.

For reference, the results of the testing undertaken for the May 2014 report are discussed below.

4.3.1 Hydraulic roughness

Adjusting hydraulic roughness by ±10% changes average water levels in the 1D model by ±0.04m. As such it can be construed that the model is not overly sensitive to changes in hydraulic roughness.

However, it should be considered that in-channel roughness values were increased on the basis of two check gaugings and anecdotal evidence. Greater certainty could be placed on the roughness values if further calibration data were made available, such as further check gaugings.

4.3.2 Downstream boundary

Sensitivity tests demonstrated that increasing or decreasing the downstream boundary water level by ±1m has no impact on water levels at Debenham. This is encouraging as it ensures any assumptions made at the boundary water level will not impact the findings of this study.

4.3.3 Sensitivity to flow

Changing the modelled peak flow by ±15% results in changes in water level across the model domain; the average increased flow is 0.08m and a maximum of 0.15m. There is a reduction of 0.09m on average when flows are decreased. This demonstrates how the uncertainty associated with hydrological estimates can impact model results. Whilst the results at Debenham match well with flood history, it should be noted that confidence in these would be increased with a high quality flow record upstream of the village.

4.3.4 Blockage scenarios

Two blockage scenarios were undertaken in Debenham. One of these reduces the width of the Aspall Road culvert near Market Square by 33% and the other reduces the bore area of the Cherry Tree Brook downstream culverts by 33%. No evidence of past blockage has been made available for this project, but it is anticipated large flood events would be likely to result in blockage of the Market Square culvert.

Both scenarios result in an increase in water levels of 0.05m immediately upstream of each structure. Blockage of the Aspall Road culvert increases flood extents by $600m^2$, blockage on Cherry Tree Brook results in an increase of $500m^2$. This suggests that partial blockage of each structure will not result in significant increases in water level.

5 Economic damage assessment

In addition to the production of flood mapping outputs, the hydraulic model results are also used to estimate economic damages within the study area. To do so, JBA's Flood Risk Metrics (FRISM) software was utilised. This is a GIS based impact analysis software that computes a range of flood risk metrics, including property damages, based on the techniques outlined in the Multi-Coloured Manual (M-CM - see section 5.3 for further details).

Annual average damages (AADs) are also calculated by the software, i.e. the average economic damage which can be expected as a result of flooding in a given year. These figures provide a useful dataset when assessing the cost-benefit of potential flood alleviation schemes, as discussed in 7.4.

Further details are available in the damage estimate check file (Appendix C). Results of this analysis have changed from those reported in May 2014, full details are provided in this section.

5.1 Available data

In order to calculate damage estimates, the following datasets were used:

- **Hydraulic modelling results**. Water levels were extracted from the October 2014 ISIS-TUFLOW model and trimmed to match the flood outlines. FRISM requires these datasets in an ESRI ArcMap raster grid format.
- **MasterMap data**. Essential in defining when a property is flooded.
- **National Receptor Dataset (NRD).** This national dataset of properties was trimmed to the study area. It includes information on building type, footprint area, floor level and M-CM code.
- **Threshold survey.** Maltby Land Surveys Ltd was commissioned to undertake a threshold survey for buildings estimated to be at risk of frequent flooding. In doing so, accurate elevation data were obtained for properties most at risk.

5.2 Data preparation

Prior to using FRISM, the NRD data was spliced together with the threshold survey (or LIDAR levels where no survey was available) to ensure all properties within the study area were attributed a threshold value. This was completed using ArcMap.

Secondly, the NRD dataset was trimmed to remove data points not required for damage calculations. We have used the Environment Agency's Flood Map for Surface Water Property Count[8] guidance document to define which features this applies to. A record of the property types excluded from the analysis is provided in Appendix C.

Properties were also removed if it was noted that the building no longer exists. A number of further assumptions were made, including:

- All properties designated "potential upper floor" were not included in the damage calculations, but maintained for the purposes of flooded property counts.
- Where the building use is not clear from available photography an assumption is made (i.e. if at the bottom of a garden this is assumed to be a domestic shed).
- The floor area stated in the NRD was used, unless this is missing in which case area was calculated from building footprints defined by MasterMap.

5.3 Multi-Coloured Manual (M-CM)[9]

The M-CM was produced by the Flood Hazard Research Centre at Middlesex University to outline the technique for undertaking evaluations of the benefits for risk management projects. This includes a methodology for calculating expected damages at an individual property for a

[8] Environment Agency, 2010. Flood Map for Surface Water - Property Count Method. Published by Environment Agency.

[9] Penning-Rowsell, E. *et al*., 2010. *The Benefits of Flood and Coastal Risk Management: A Handbook of Techniques - 2010 (The Multi-Coloured Manual).* Flood Hazard Research Centre.

given flood event. The techniques outlined form the basis of our in-house FRISM tool, used in the current study.

Included in the manual are depth-damage curves for various property types, catering for both residential and non-residential premises. These provide the damage (in pounds sterling (£)) per m^2 of flooded property for a given flood depth. These were most recently updated in 2013; the latest version of our FRISM software includes new short duration residential and non-residential (no basement) curve, which represent a direct update from those available in the 2010 M-CM. Examples of the curves used in the current study are available in Appendix C.

5.4 Flood RISk Metrics

FRISM uses the M-CM techniques to compute a variety of metrics by combining flood modelling results together with the receptor data discussed above. The metrics that can be calculated depend on both the geometry type of the receptor data and the type of modelling results used. As water level grids were produced for this project, detailed property level analysis was computed and included depths and damages at each property (based on either the survey property threshold level or that extracted from LIDAR). As multiple events were modelled, the long term AADs were calculated alongside damages for a given return period. Results from these calculations are summarised in sections 5.5.1 and 5.5.2.

Firstly, property counts were undertaken. NRD points were linked to the building footprint data based on their spatial relationship. Commercial properties were counted as flooded if any part of their associated building footprint intersects with the flood outline and the threshold exceeded. The same technique is used for residential properties, although these are classed as flooded, and a damage calculated, if the water depth in the property is -0.3m or greater. This value is representative of national average curves and accounts for the fact some properties have basements and cellars whilst others do not. The NRD dataset does not record any basement flats in Debenham (these would be given a Floor Level parameter of dB; "definite basement"). Upper floor properties can be included in the flooded count, but have no associated damage.

During the community engagement meeting residents were asked to mark properties where basements and cellars were known to be present in order to refine the economic damage calculations. However, only five properties were highlighted and therefore it is considered more appropriate to use the standardised national approach; applying a different depth-damage curve for these five properties would impart bias into the results. If more detailed results are required collection of comprehensive data regarding basement/cellar locations and characteristics is recommended, involving commissioning of survey. This would involve establishing the floor level of each feature, ground level of potential water ingress points and the purpose (i.e. storage).

5.4.1 Flood RISk Metrics depth damages

FRISM calculates the depth of flooding within each building footprint by subtracting the threshold level from the water level grid (produced by the hydraulic model) and attributes these depth values to the property. As an example, if the mean water level at a property was 20.60m and a property threshold of 20.25m was set, the resultant flood depth would be 0.35m.

Damages were calculated using the well-established methods set out in the M-CM, assigning depth-damage curves for each property type. These methods were implemented in the software as per the Hydraulic Research (HR) Wallingford (2008) Technical Note on National Flood Risk Assessment (NaFRA) Economic Calculations[10]. The depth was used in conjunction with property type and the relevant M-CM depth-damage curves to obtain damage per metre squared (£/m^2). This was multiplied by the floor area of the property to obtain a property damage value.

Please note:

1. Damage calculations for upper floors were not undertaken, although these are included in flooded property counts.
2. Damages were not capped to maximum property price or valuation.

[10] Panzeri, M. and Mauz, J., 2008. *NaFRA 2007 Technical Note; Economic Calculations*. HR Wallingford

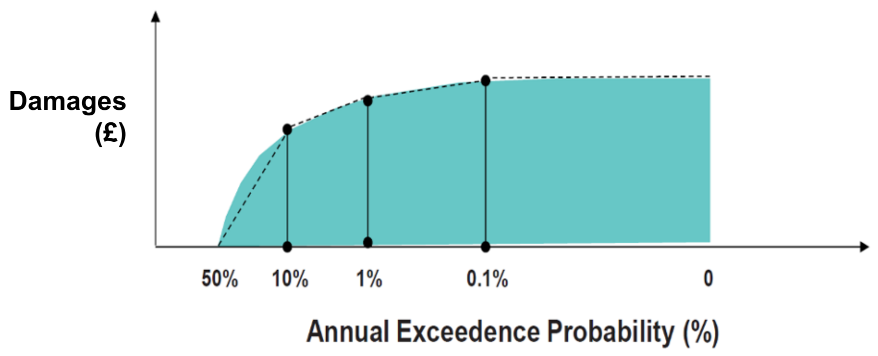
5.4.2 Flood RISk Metrics annual average damages

As a range of modelled return periods were available, FRISM was used to apply a probability of occurrence to those results to calculate AAD estimates. The model was run for seven different return periods (2, 5, 10, 20, 75, 100 and 1,000-year); no climate change allowance is accounted for in the damage estimates. AADs represent the notional long term average or expectation of consequence in any given year and are a useful way of comparing flood risk between different areas.

The principle of the equations used is illustrated in the Figure 5-1 example below, with the area under the curve integrated to give the AAD metric value. This assumes that the onset of flooding (or zero damages) is the 1 in 1-year event and that the damages for rarer events do not increase beyond those incurred at the 1 in 1,000-year (0.001% AEP) event.

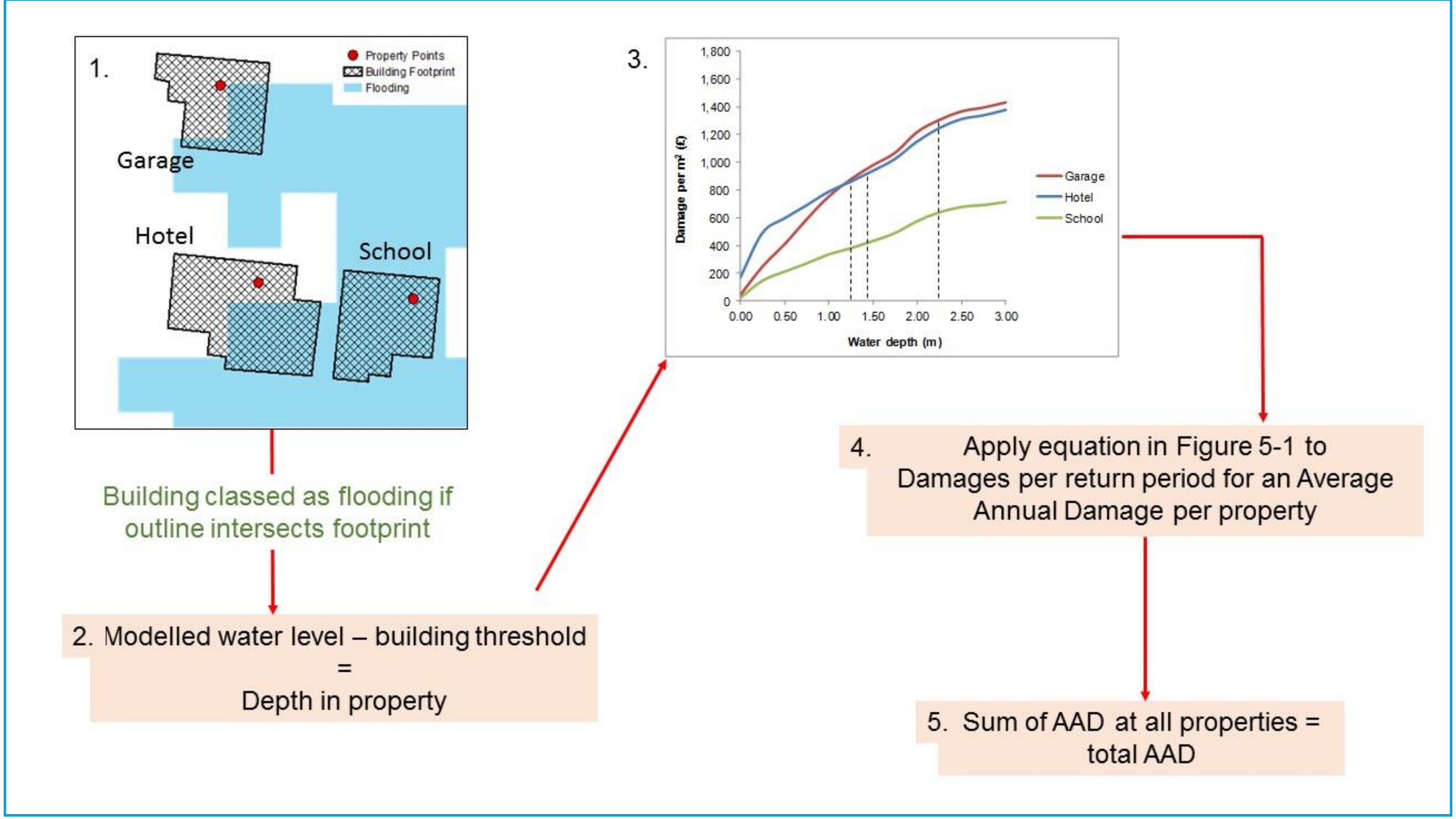
Figure 5-1: Approximation of annual average risk based on a limited number of events

RP (Years)	Ratio	Damage (£)	Contribution (£)
1	1.000 (a)	0 (b)	$=(a - a^1) * ((b + b^1)/2)$ $= (1.000 - 0.500) * ((0 + 500)/2)$ = 125.00
2	0.500 (a^0)	500 (b^0)	$=(a^0 - a^1) * ((b^0 + b^1)/2)$ $= (0.500 - 0.200) * ((500 + 1,000)/2)$ = 225.00
5	0.200 (a^1)	1,000 (b^1)	= 150.00
10	0.100 (a^2)	2,000 (b^2)	= 315.00
100	0.010 (a^3)	5,000 (b^3)	= 67.50
1,000	0.001 (a^4)	10,000 (b^4)	= 10.00
AAD = Sum of contributions			**= £892.50**



The calculation of economic damages is summarised in Figure 5-2 below.

Figure 5-2: Flow chart of annual average damage estimation



5.5 Economic damage results

JBA's FRISM software produces detailed results for a number of damage measures. These measures include the total damages (for a given flood event) and annualised results (AADs). These detailed results are provided alongside this report and the following section is intended as a summary of these results.

FRISM was employed to estimate baseline AADs and also to assess the potential economic benefit each alleviation option could provide, discussed further in section 6.3.

5.5.1 Baseline Flood RISk Metrics results

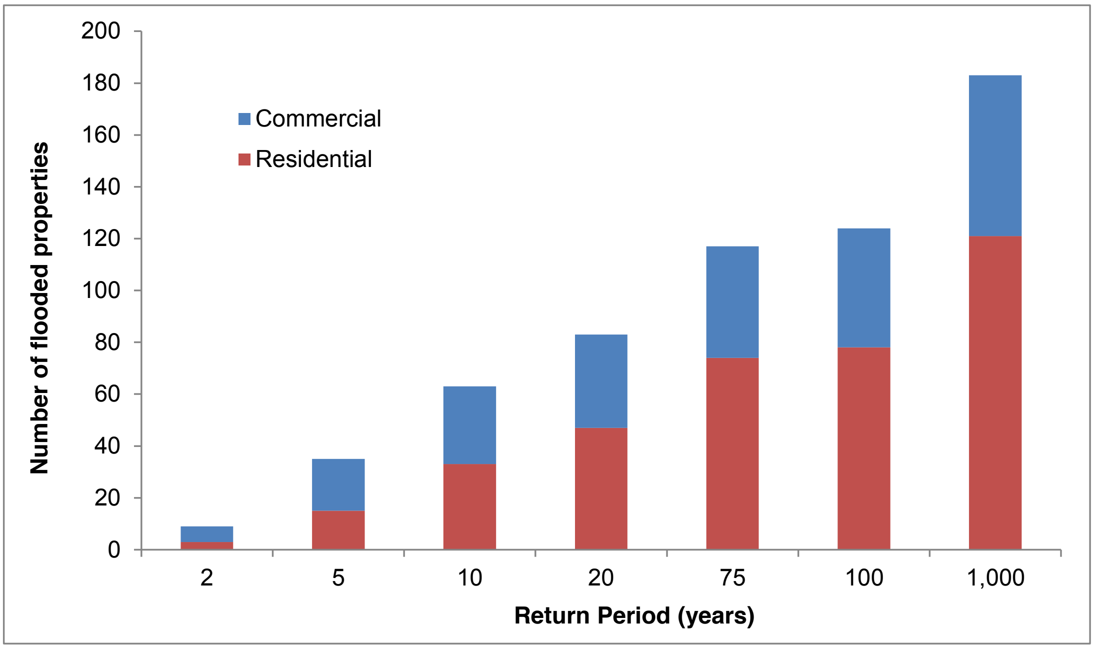
Table 5-1 below provides the total estimated damages for a given return period event, including the May 2014 results and the current October 2014 results. The figures presented here are the sum of economic damage associated with every property in Debenham; residential properties are assumed to have basements which impart damages once flood water exceeds a depth of -0.3m, as per national guidance when more detailed data is unavailable. The residential damage column below includes any NRD points with the OS class "dwelling". Utility properties (electricity sub-stations, telecommunications, pumping stations and sewage treatment works) are not included in the table below as no damage is sustained on these features even at the 1,000-year event, each is located on high ground. Infrastructure such as roads are not included in the current economic damage assessment.

Please note that not all properties classed as flooded have an associated damage; upper floor premises are classed as flooded (due to lack of access during a flood event), but have no resultant economic damage.

Table 5-1: Damages estimates per return period

Return Period (years)	Total damages (£000s) - May 2014	Total damages (£000s) - Oct 2014	Residential damages (£000s) - Oct 2014	Commercial damages (£000s) - Oct 2014
2	57	46	30	16
5	267	197	73	124
10	529	393	126	267
20	850	693	270	423
75	1,762	1,609	764	845
100	1,958	1,828	890	938
1,000	4,794	4,509	2,709	1,800

Figure 5-3: Total flooded properties per return period (years) - October 2014 results



It is useful to begin by analysing the damages estimated per return period rather than referring to the AAD estimates. At Debenham the majority of damages are incurred in a relatively narrow river corridor. The estimates per return period have decreased compared to the May 2014 results for the following reasons:

- Inclusion of topographic survey at Water Lane has removed many properties from the flood outline, particularly notable at the 5 and 10-year return periods.
- Reduced hydraulic roughness downstream of Priory Lane bridge and the area around Fen Street has decreased water levels for a given return period. This decreases the damage associated with each property affected. The impact of this is particularly notable given one property at Fen Street floods at the 2-year event.

The following observations are made with regard to economic damage estimates:

- A significant number of the properties at risk are located either side of the channel upstream of Market Square, areas between Water Lane and Priory Lane and also the residential areas at the south of the High Street. This matches well with historic evidence.
- The greatest individual damages are estimated at Fen Street, where two residential properties are flooded even at the 2-year return period. Residents noted that flooding occurred regularly at this location although incurred damages every two years may overestimate this risk. Hydraulic roughness in this reach has been reduced to counter this and has resulted in lower damage estimates in this area. A greater check gauging record at Debenham would improve flow estimates and therefore increase certainty in flood outlines here.
- Many properties flood around the Cross Green floodplain between the River Deben and Cherry Tree Brook. This is partially as a result of water backing up the non-main river drain discharging to Cherry Tree Brook, a mechanism highlighted by residents at the community engagement meeting.

No damages are predicted on the left bank of Derry Brook, despite the suggestion of flooded gardens at the Red House on Little London Hill (see Table 2-1). This aligns with the observations made on site that the house is located on higher ground than the garden.

5.5.2 Flood RISk Metrics annual average damage results

Cumulative AADs at Debenham are estimated at £185,600. As discussed in section 5.4.2 this is a function of the damages incurred for each return period flooded. The value is significantly reduced from the May 2014 report (total AAD = £226,300) because of the changes discussed in section 4.1.3; the impacts of these alterations are particularly notable at shorter return periods, hence has a large impact on the AAD value.

The greatest single damage is incurred at Fen Street as discussed above, although most properties incurring damages are located in the centre of Debenham. The following maps (Figure 5-4, Figure 5-5 and Figure 5-6) demonstrate these results, the first shows all properties. The residential map (Figure 5-5) only includes properties marked as "dwelling" in the OS feature class. Garages are all deemed commercial, as the same definition is given both to household garages and commercial garages.

Figure 5-4: Property AAD results at Debenham based on the depth and frequency of flooding

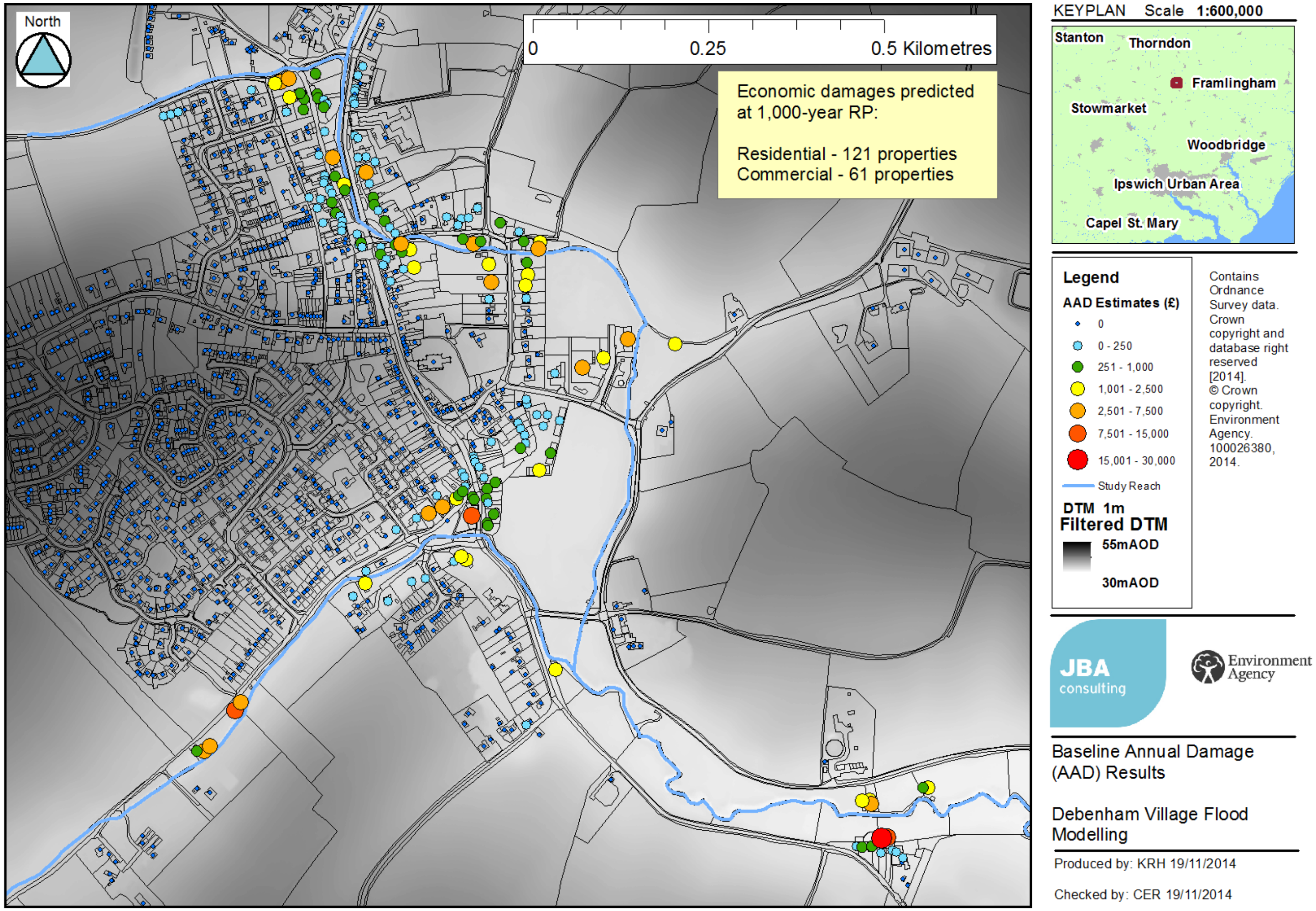


Figure 5-5: Residential property AAD results at Debenham based on the depth and frequency of flooding

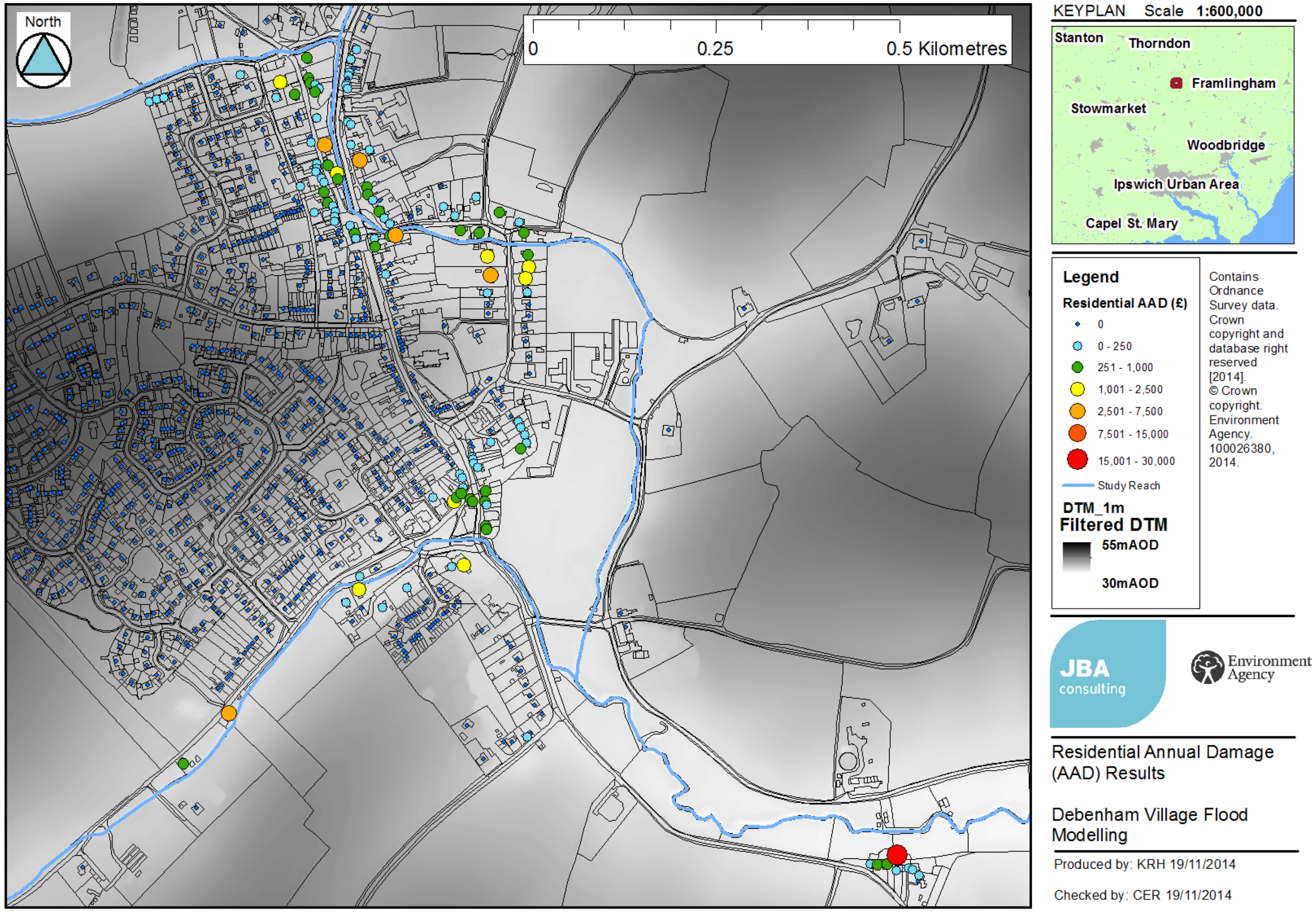
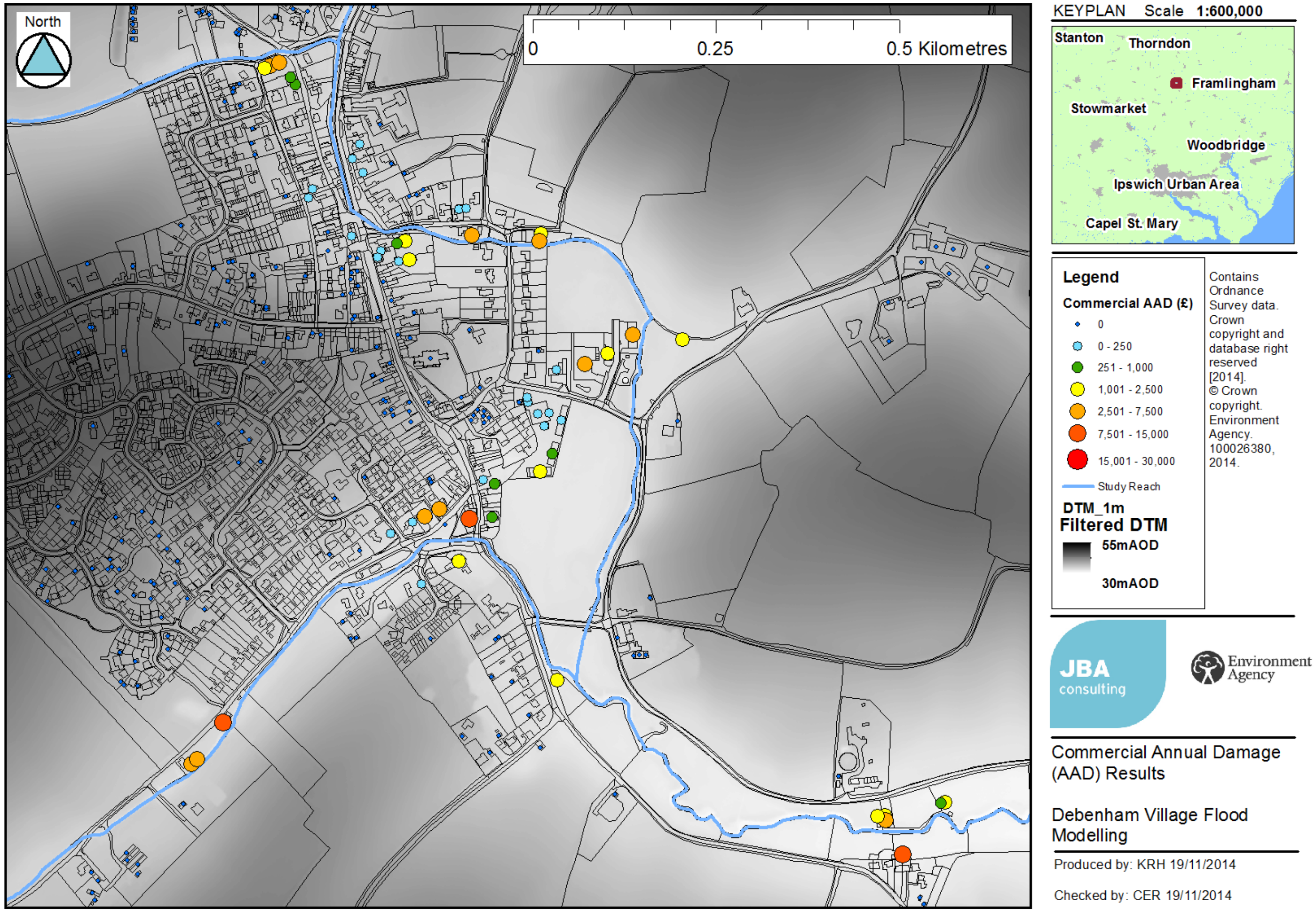


Figure 5-6: Commercial property AAD results at Debenham based on the depth and frequency of flooding



6 Flood alleviation options appraisal

6.1 Introduction

Having completed draft modelling of Debenham, the Environment Agency requested investigation of various flood alleviation options in and around the village. Many of these were suggested by the Environment Agency in the project brief, whilst a number were proposed by JBA following initial modelling. Removal of the non-main river drain bank of Cherry Tree Brook was also suggested by Debenham residents. The draft results from these investigations were presented to the Environment Agency in September 2013, many of which were carried forward to be presented at the community engagement meeting. No changes were made to the suggested options from May 2014 to October 2014, although the hydraulic model was re-run to reflect the baseline model changes discussed in 4.1.3.

Section 6.2 only focuses of those options which were shown to offer flood risk benefits to Debenham. Various options proposed were shown to have little or no impact on water levels, whilst others actually increased flood risk. After discussion with the Environment Agency, the following measures were discounted from further consideration:

- Decreased roughness attained by increased channel maintenance. This was found to have a large impact downstream of the River Deben - Cherry Tree Brook confluence but it was considered that a two stage channel or alleviation channel would offer greater environmental benefits.
- Addition of an alleviation channel across Cross Green floodplain from Cherry Tree Bridge to upstream of Hill House Bridge on the River Deben. Such a feature increased flooding in the Cherry Tree Brook channel as water from the River Deben propagated further upstream on this watercourse.
- Removal of sediment blockage downstream of Cherry Tree Bridge was shown to have little impact on levels.
- Widening of the culvert beneath Derry Brook Lane on the River Deben was shown to have little impact on levels.
- Addition of 0.5m embankments upstream of Market Square increased flood risk. In-channel levels were increased to an extent where more water overtopped the left bank of The Gulls at Aspall Road upstream of the walls. Such a scheme would also be aesthetically unappealing.
- Removal of natural floodplain embankments encircling the Cross Green floodplain, lowering of Kenton Road and Cross Green Road. This allowed flood water from Cherry Tree Brook to enter the floodplain earlier, but resulted in lost floodplain storage as the River Deben flood wave propagated downstream and increased flooding as a result.
- Lowering of Kenton Road and Cross Green Road was shown to have little impact on levels.
- Widening of the channel immediately downstream of Water Lane was shown to have little impact on levels.
- Addition of the flood storage area on Derry Brook was discounted as it was deemed more beneficial to include such a feature on either The Gulls or Cherry Tree Brook.

Other options presented at the meeting were considered more effective at reducing flood risk to Debenham. These are discussed in greater detailed below.

6.2 Options

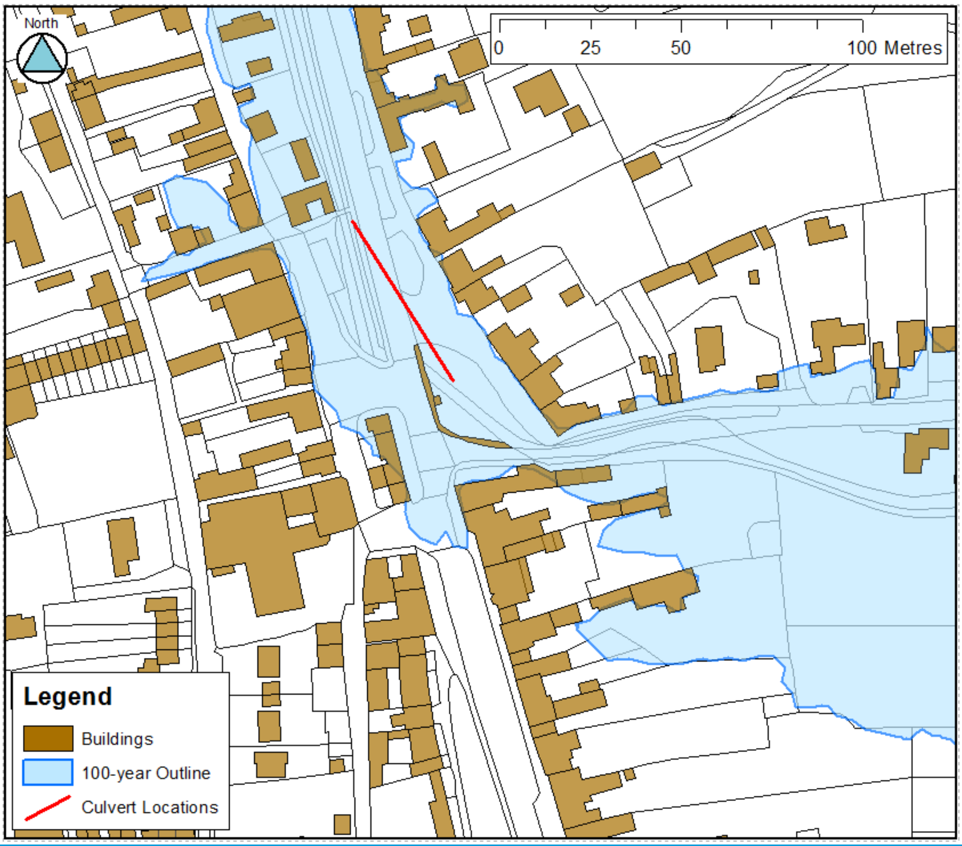
Please note that the alleviation options below simply provide an indication of the possible benefits; no attempt was made to optimise design to maximise the cost-benefit ratio. If any of the schemes below are taken forward this analysis would form part of an outline design phase along with detailed analysis of other factors such as environmental and geomorphological impacts.

6.2.1 Option A: Aspall Road culverts

The culvert beneath Aspall Road and the associated tight bends in the channel at both the entry and exit reduce discharge capacity through the village. This perception is reinforced by the extensive historical flooding in the adjacent Market Square. We therefore investigated whether flood risk in this area could be mitigated by increasing conveyance under Aspall Road. This was included in the model as three $1m^2$ rectangular culverts. The flood relief culvert is connected to the channel upstream of the Market Street culvert, bypassing Aspall Road culvert and discharging into the channel immediately upstream of Water Lane.

During the course of testing this option water levels downstream of the new culverts were checked against baseline results to ensure flood risk downstream did not increase. As demonstrated in Table 6-1 and Table 6-2 any differences are negligible, although this should be assessed further during a detailed design phase.

Figure 6-1: Option A - culvert location

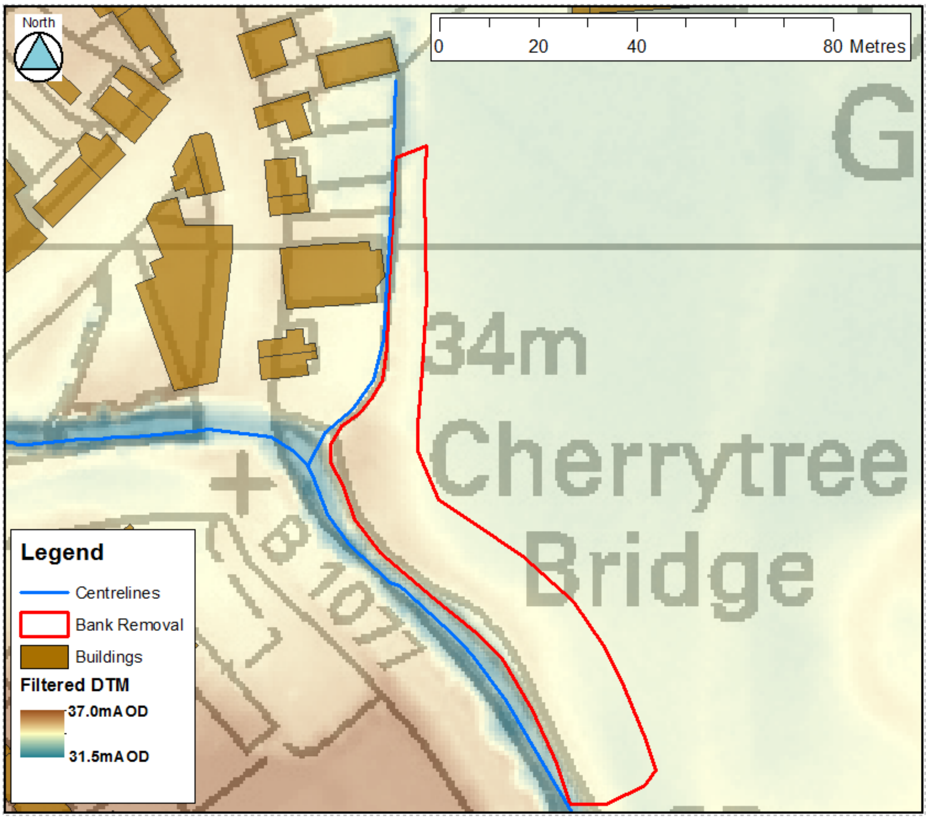


6.2.2 Option B: Cherry Tree Brook drain bank

Whilst at the community engagement meeting householders from the southern end of the High Street described how floodwaters regularly encroached into back gardens from the right bank of the non-main river drain which discharges into Cherry Tree Brook. As a result of this flooding residents have manually excavated portions of the left bank to allow water to drain into the Cross Bank floodplain.

Therefore, Option B involves the removal of the left embankment from this drain, and also on a small reach of Cherry Tree Brook. This option aims to allow floodwaters into the Cross Green floodplain sooner.

Figure 6-2: Option B - Cherry Tree drain bank



6.2.3 Options C-F: Conveyance around Fen Street

As highlighted in the flood modelling and discussed in section 4.1.2, a constriction in the valley topography upstream of Fen Street results in flood waters backing up during extreme events, rapidly inundating the Cross Green floodplain and properties on the bank of Cherry Tree Brook. Options C, D, E and F all examine the potential of increasing conveyance through this reach and therefore decreasing the volume of water to be stored on the floodplain upstream. If less stored volume is required the resulting flood extents are likely to be reduced.

Any option that increases conveyance through this section will result in greater pass on flows and the impact of this on downstream flood risk must be considered. However, as there is very little flood risk immediately downstream of our model domain, and by the time there is, the contributing catchment is much larger, the impact is likely to be minimal. If the increase in pass on flows was deemed unacceptable additional compensatory floodplain storage could be facilitated in the rural reach downstream. Again this would need to be explored in more detail as part of a full feasibility study.

6.2.3.1 Option C: Two stage channel

Option C involves the development of a two-stage channel downstream of the Cherry Tree Brook and River Deben confluence. By lowering the floodplain levels on either bank, additional conveyance capacity is created through this section. Low flows in the main channel will remain unaffected. This approach emulates the operation of a natural floodplain but in a more controlled manner. In the model the floodplain was lowered 12.5m either side of the channel banks. If this design is to be taken forward an allowance should be made for consultation with a geomorphologist.

6.2.3.2 Option D: Two stage channel with upstream weir removal

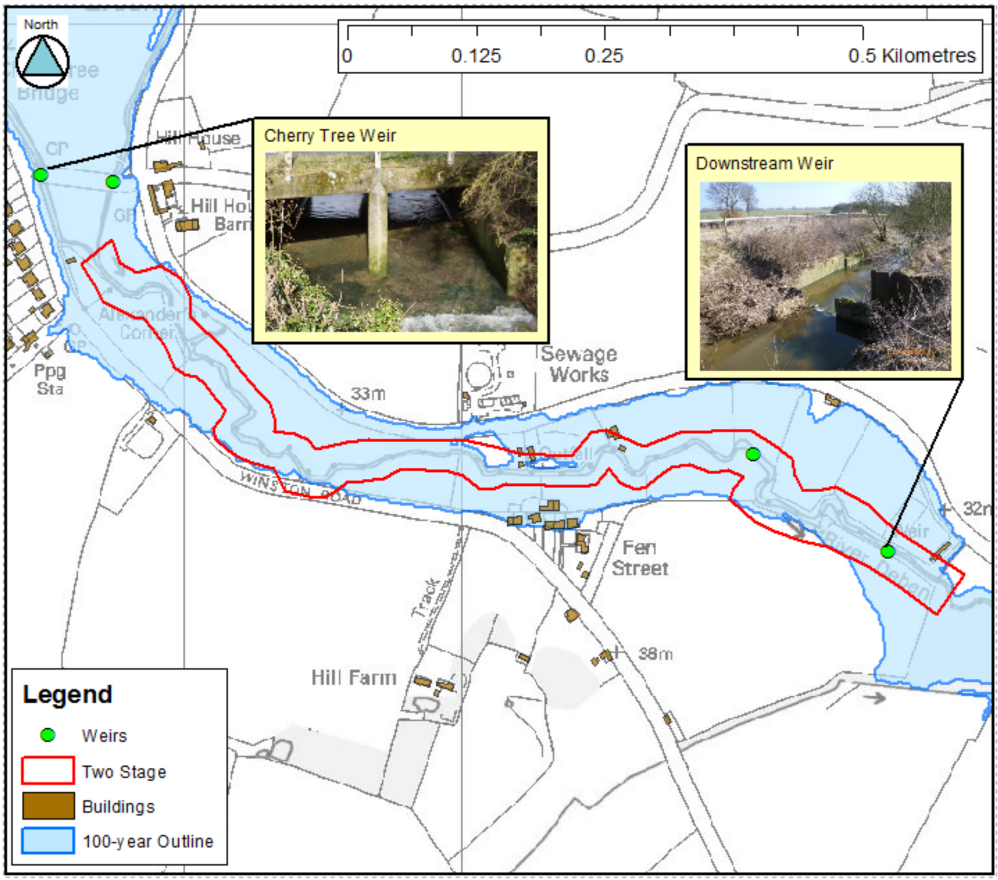
Option D includes the two stage channel as above with the removal of upstream weir features beneath the road bridges on Cherry Tree Brook and the River Deben. With these features in place the effectiveness of the two stage channel is limited; water levels upstream of the weirs are controlled by these features before they become drowned out. Removing the weirs would allow additional flood water into the two stage channel sooner.

6.2.3.3 Option E: Two stage channel with downstream weir removal

Two weirs are present on the River Deben downstream of Fen Street (Figure 6-3). The purpose of these is not known. Removal of these features would aid progression of the flood wave through the Fen Street area, although their impact is limited in extreme events given their tendency to drown out.

Figure 6-3 illustrates the proposed location of the two stage channel and the weirs.

Figure 6-3: Options C, D and E - two stage channel and weirs

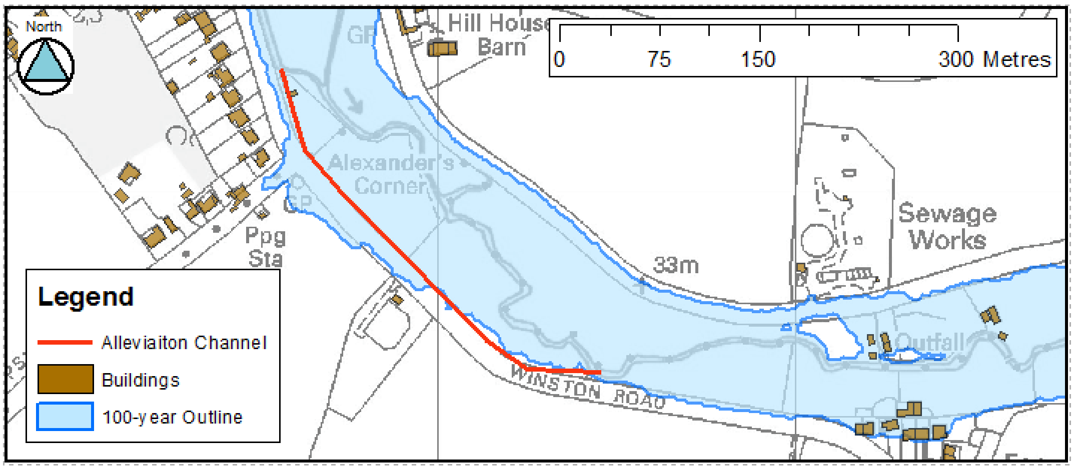


6.2.3.4 Option F: Alleviation channel

Option F is similar to the two stage channel - increasing conveyance downstream of the Cherry Tree Brook - River Deben confluence. A channel was added from Cherry Tree Brook (upstream of the confluence) to a meander adjacent to Winston Road.

The channel is included in the model as 1.5m deep and 10m wide. A higher elevation spill is included at the upstream extent to ensure the channel only becomes active at return periods above QMED ensuring low flows in Cherry Tree Brook and the River Deben remain unaffected.

Figure 6-4: Option F - Alleviation channel



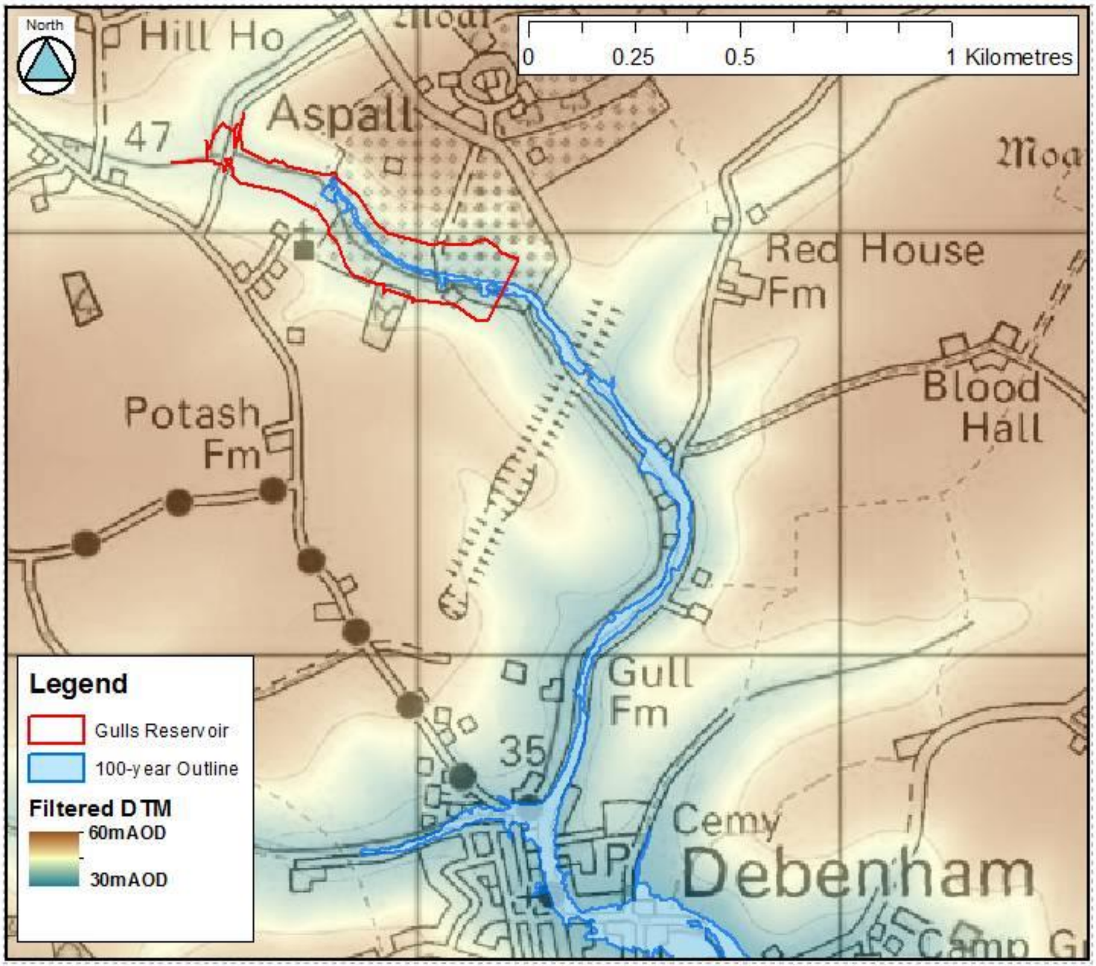
6.2.4 Option G: The Gulls flood storage reservoir

The possibility of additional upstream storage was also investigated. Two sites were identified as potential reservoir locations; one on The Gulls and one on Cherry Tree Brook. The aim of these features would be to attenuate extreme events to reduced flood risk downstream in Debenham. There are potential additional benefits such as storage of water offline to supply supplementary flows in the summer months; such benefits may fall in line with the aims of the River Deben Holistic Water Management Plan.

By including a reservoir around Aspall on The Gulls, the model demonstrated that the 100-year flow hydrograph can be attenuated to such a degree that only approximately a 2-5 year peak flow conveys down the watercourse. This is not translated into a 2-year event at Debenham given the un-attenuated inflows from Derry Brook, but it does mitigate the majority of the flood risk during an event of this magnitude. In order to store this volume of water an average crest height of around 3m would be required, with a maximum height of around 5m. A spillway was added above the 100-year water level to allow overtopping at events greater than the 100-year.

At this stage we have assumed the flow will be regulated only via culverts. The use of moveable sluice gates or hydro-brakes may reduce the volume of water to be stored but would add to the expense. It should also be considered that currently inflows to the reservoir remain unchanged from the baseline modelling scenarios. The construction of a reservoir on either The Gulls or Cherry Tree Brook would fundamentally change the critical storm duration resulting in the most extreme flood at Debenham, given the increased attenuation. This is not considered currently and would need to be modelled, alongside a range of potential reservoir outlet units during a feasibility study.

Figure 6-5: Option G - The Gulls reservoir

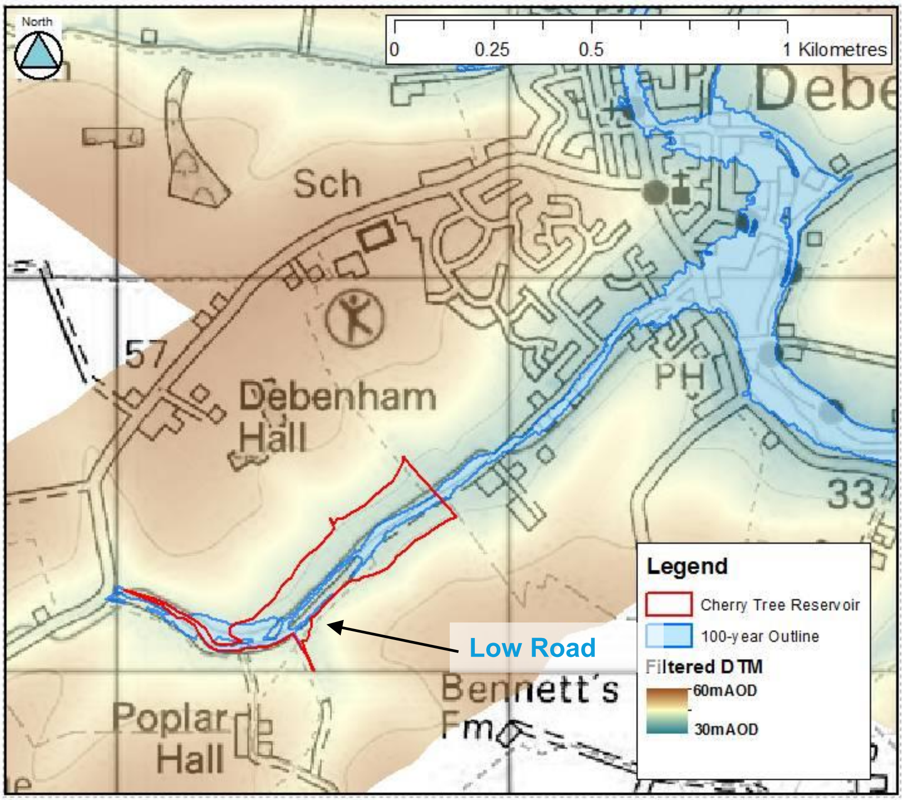


6.2.5 Option H: Cherry Tree Brook flood storage reservoir

There is also a potential reservoir site on Cherry Tree Brook, where the steep valley sides suggest a favourable location for an impounding reservoir. This was modelled in a similar manner to that on The Gulls, with outlet culverts designed to attenuate the peak 100-year flow to QMED.

A further consideration at this site is the requirement to alter Low Road. This carriageway currently runs adjacent to the watercourse and as such would need to be re-situated on higher ground if the proposed reservoir was constructed.

Figure 6-6: Option H - Cherry Tree Brook reservoir



6.2.6 Implications of the Reservoirs Act (1975)

The estimated storage capacity of The Gulls and Cherry Tree Brook reservoirs are likely to fall within the scope of the Reservoirs Act, 1975 (i.e. because they have will have a storage capacity greater than 25,000m^3) and thus the design, construction, inspection, maintenance, monitoring and decommissioning of any dam structure will need to comply with all the requirements of the act (and be overseen by a qualified reservoirs panel engineer). The required stored volumes will be finalised in a detailed design phase.

6.2.7 General comments on flood storage reservoirs

The following significant issues have emerged during our studies:

- The required dams are in the region of 3 to 5m high and would be substantial engineering structures, requiring major earthmoving operations, and potentially raising planning issues.
- The land comprising the dam embankment would probably have to be purchased.
- The impounded area would be prone to intermittent flooding that would tend to damage crops and impact upon agricultural activities, thus leading to compensation claims.
- There would need to be a willing candidate to own and maintain the dams.
- The dams would in themselves pose a flood risk to the downstream villages.

6.3 Options modelling results

The table below (Table 6-1) records the change in water level at given locations for the alleviation options, as well as the change in 20-year flooded property count and AADs. The observation points below are the same as those outlined in Table 4-1; please refer to this for details. Before referring to the table, a number of considerations are required:

- Baseline AAD results have reduced due to improvements in topographic representation and hydraulic roughness in Debenham. Therefore the resultant reduction in AAD per option is decreased from those reporting in May 2014.

- Since the May 2014 issue of our results, required dam elevations were raised to reflect improvements made to the elevation-area relationship used to define storage capacity in the proposed reservoir. This has resulted in less available storage and the required embankment height is greater than that recorded in May 2014 (see section 7.3.7.1).
- "Net properties with reduced AAD" below records whether AAD values reduced per property between alleviation and baseline scenarios. Properties where AAD reduction is less than £10 are removed from the analysis to avoid overstating benefits.

Table 6-1: Alleviation option impacts (20-year)

Option ID	Observation point	Option water level (mAOD)	Reduction from baseline (m)	Properties with damages reduced to £0	Properties with reduced AAD	Reduction in AAD (£)
A	DEBN_5404u	35.5	0.1	7	39 (31 res, 8 com)	12,350
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.9	-			
B	DEBN_5404u	35.6	-	-2	19 (11 res, 8 com) NB: 27 with increased AAD	-3,330
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.9	-			
C	DEBN_5404u	35.6	-	14	56 (26 res, 30 com)	59,640
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.6	0.3			
D	DEBN_5404u	35.6	-	15	68 (39 res, 29 com)	69,600
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.4	0.5			
E	DEBN_5404u	35.6	-	14	56 (26 res, 30 com)	59,760
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.6	0.3			
F	DEBN_5404u	35.6	-	0	12 (2 res, 10 com)	610
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.9	-			
G	DEBN_5404u	35.5	0.1	19	140 (93 res, 47 com)	54,380
	DEBN_5354	35.1	0.1			
	DEBN_4934u	34.5	0.1			
	CHRY_0253u	33.8	0.1			
H	DEBN_5404u	35.6	-	9	66 (38 res, 28 com)	37,690
	DEBN_5354	35.2	-			
	DEBN_4934u	34.6	-			
	CHRY_0253u	33.8	0.1			

Table 6-2: Alleviation option impacts (100-year)

Option ID	Observation point	Option water level (mAOD)	Reduction from baseline (m)	Properties with damages reduced to £0	Properties with reduced ADD and total reduction in ADD
A	DEBN_5404u	35.7	0.1	10	As above in Table 6-1
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.2	-		
B	DEBN_5404u	35.8	-	0	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.2	-		
C	DEBN_5404u	35.8	-	14	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.1	0.1		
D	DEBN_5404u	35.8	-	16	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.0	0.2		
E	DEBN_5404u	35.8	-	14	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.1	0.1		
F	DEBN_5404u	35.8	-	0	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.2	-		
G	DEBN_5404u	35.7	0.1	23	
	DEBN_5354	35.2	0.1		
	DEBN_4934u	34.6	0.1		
	CHRY_0253u	34.1	0.1		
H	DEBN_5404u	35.8	-	6	
	DEBN_5354	35.3	-		
	DEBN_4934u	34.7	-		
	CHRY_0253u	34.1	0.1		

Table 6-1 and Table 6-2 demonstrate the potential benefits associated with the range of alleviation options under considerations at Debenham. The cost implications of such schemes are discussed in chapter 7.

- The greatest annual damage reduction is associated with the construction of a two stage channel downstream of the Cherry Tree Brook - Deben confluence, with removal of the weir beneath Hill House Bridge on both watercourses (Option D), followed by the other two stage channel options and the impounding reservoir on The Gulls (Option G). The Gulls reservoir protects the most properties at the 20-year return period, given the majority of properties in Debenham flood are located downstream. Outlines associated with The Gulls reservoir do not notably decrease, although the associated depths do given the constrained nature of the floodplain.

- Modelling has suggested that construction of a reservoir on The Gulls does not impact flood outlines in the reach immediately upstream; i.e. around Hill House Farm.
- The two stage channel schemes (Options C, D and E) offer significant benefits, but these are confined to the area of the fire station and Cross Green. Removal of the upstream weirs at Hill House Bridge further enhances the benefit.
- A flood storage scheme on Cherry Tree Brook (Option H) offers significant benefits, but protects less properties than a comparable scheme on The Gulls.
- Increasing conveyance near Market Square (Option A) reduces AADs by approximately £12,350, although few properties are removed from the 20-year flood outline. This is because the outline does not reduce significantly as a result of the additional conveyance, although properties are flooded to lower depths.
- The alleviation channel (Option F) has little benefit, conveying significantly less flow than the similar two stage channel.
- Removal of the left bank of the non-main river drain (Option B) is shown to increase flood risk to the properties adjacent to this watercourse during a 20-year event on both Cherry Tree Brook and the River Deben. This is because flood water from Cherry Tree Brook enters the Cross Green floodplain sooner in this scenario, reducing storage capacity when the larger River Deben peak flow occurs. Removal of the bank would likely reduce flood risk to the south of the High Street if an event was to occur on Cherry Tree Brook only, but hydrological analysis suggests this is likely to be accompanied by an extreme event of the River Deben.

7 Partnership Funding analysis of proposed alleviation options

7.1 Flood Defence Grant-in-Aid funding

In England, Local Authorities, IDBs, the Environment Agency and other government organisations are entitled to bid for Flood Defence Grant-in-Aid (FDGiA) funding from Department for Environment Food and Rural Affairs (Defra) (administered by the Environment Agency) to assist with the delivery of Flood and Coastal Risk Management (FCRM) schemes.

The funding regime encourages stakeholders to contribute towards the cost of flood relief schemes, such as those proposed for Debenham. The greater the proportion of stakeholder contribution, the higher the Partnership Funding (PF) score, and therefore the greater chance of securing FDGiA funding for the project. The Environment Agency ranks all bids nationally in descending order of PF score and will allocate the available funding accordingly until all the available budget is allocated. It is uncertain from year to year how much capital funding is available and therefore what PF score is required to secure GiA.

For this project the feasibility of obtaining FDGiA was assessed for each alleviation option shown to provide a flood risk benefit. Option B (removal of the Cherry Tree drain bank) was removed from the options given the negative impact associated with this option. The results were originally reporting in May 2014; calculations have now been updated as a result of the additional modelling and also the increased embankment height required on The Gulls reservoir.

7.2 Partnership Funding score - cost benefit analysis

PF was calculated for each scheme using the PF Calculator, a spreadsheet tool which follows the FDGiA procedures outlined in the Environment Agency's guidance document[11].

From 2012/3, the funding available to any scheme is scored using Outcome Measures (OMs) which relate to the economic damages avoided (OM1), the number of households protected (OM2), coastal erosion benefits (OM3) and wider benefits of the scheme (OM4); these are assessed against the capital cost of the scheme, ongoing post-construction costs and also the funding from other sources to produce the final PF score.

7.2.1 Outcome Measure 1

The economic benefits of each scheme were calculated using AAD estimates from JBA's FRISM software (as discussed in chapter 5); the reduction in damages between the baseline and alleviation options provides the present day annual benefit. However, given the scheme in question is designed to last into the future, it is necessary to calculate the "Whole-Life Benefit". For the purposes of the Debenham project it is assumed that the expected design life of all schemes is 75-years. In order to project economic benefits an HM Treasury Green Book discount rate of 3.5% is applied each year; this effectively ensures £1 today is worth more than £1 in the future as a result of inflation. The sum of 75-years discounted benefits produces the Whole-Life Benefit.

The engineering costs for each scheme were calculated and are recorded in section 7.3. For the purpose of the cost calculations it is assumed that the construction of each scheme would undertaken within the current financial year and therefore discounting of costs is not required. The ongoing maintenance costs associated with each scheme, known as the "Post-Construction Costs" have been discounted assuming a 75-year life.

7.2.2 Outcome Measure 2

OM2 incorporates the properties at risk into the PF calculation, categorised into very significant risk (20-year return period), significant risk (75-year return period) and moderate risk (200-year

[11] Environment Agency., 2011. *Estimating Outcome Measure contributions and using the FSGiA Funding calculator for Flood and Coastal Erosion Risk Management project*. Environment Agency Operating Instruction.

return period). For this project, moderate risk is taken as the 1,000-year event given the lack of a 200-year model version.

It is assumed that all properties in Debenham are located in the 60% least economically deprived area.

7.2.3 Outcome Measures 3 and 4

Debenham is not located on the coast and therefore OM3 is not relevant. At this stage OM4 is not considered as it falls outside the scope of the current commission.

7.3 Development of alleviation scheme costs

Scheme costs were estimated using rates from the Civil Engineering Standard Method of Measurement (CESMM4)[12] Carbon and Price Book 2013 and a general specification in accordance with Sewers for Adoption, 7th Edition[13] and/or Civil Engineering Specification for the Water Industry (CESWI)[14]. The costs below are based on the design of alleviation options as included in the hydraulic model, unless stated otherwise. These do not represent optimised designs and therefore the costs below should be considered indicative.

No information was available for this study regarding the route, size, depth, level, condition or ownership of any utilities. The location of utilities could pose major constraints to the flood alleviation options and therefore could impact construction costs. Where utilities are deemed likely to be encountered (e.g. under roads, near sewage works) a crude initial estimate of the cost associated with service diversions has been included in the preliminary price.

Details of costs are available in Appendix D.

7.3.1 Option A: Aspall Road culverts

Although included in the hydraulic model as three orifice units, a box culvert with a similar cross sectional area was included in the cost estimate as this was deemed more cost effective. The CESMM rate for a box culvert assumes a standard pre-cast culvert with no modifications or additional requirements such as skewed ends or low flow channels. As this culvert installation involves modifying a highway, costs relating to road closures were accounted for. Two service diversions were also assumed. Potential high velocity flows at the culvert outlet could cause scour, therefore an estimate of scour protection has been included in the price.

A cost for further hydraulic modelling is included so more detailed design of the culvert inlet and outlet and scour protection can be carried out.

7.3.2 Option B: Cherry Tree Brook Drain Bank

This option is not included in the FDGiA calculations as it has been shown to have a negative impact on flood risk. Properties to the west of the small non-main river drain flood to a great depth, given the storage capacity of the Cross Green floodplain is utilised earlier by Cherry Tree Brook floodwaters as opposed to those from the River Deben. Therefore when the Deben flood peak arrives this storage is no longer available, resulting in greater flooding.

7.3.3 Option C: Two stage channel

This is primarily an earthworks scheme, which is where much of the cost arises for this option. Lowering the floodplain requires approximately 23,300m^3 of material to be excavated and removed from site. Transportation of excavated subsoil to landfill accounts a significant portion of the scheme cost. If the subsoil could be re-used elsewhere, or sold as fertile alluvial soils, the cost of the scheme would be considerably reduced.

A number of access tracks and loading/storage areas are required to transport the excavated subsoil to tip. These access tracks are on fields possibly containing livestock, therefore the whole length of the works will be require temporary fencing.

[12] Institution of Civil Engineers., 2012. CESMM4: Civil engineering standard method of measurement.

[13] Water UK and Water Research Centre PLC., 2012. Sewers for Adoptions, 7th Edition.

[14] Water Research Centre PLC., 2011. Civil Engineering Specification for the Water Industry, 7th Edition (CESWI).

7.3.4 Option D: Two stage channel with upstream weir removal

In addition to the two stage channel this option includes the removal of the upstream weirs beneath the road. The weirs are actually box culverts with raised inverts which perform the same function as a weir. Therefore this option has accounted for the cost of a culvert replacement which would be installed at a lower level.

7.3.5 Option E: Two stage channel with downstream weir removal

This option includes the removal of the downstream weirs as well as the two stage channel. Removing the downstream weirs is more straightforward than the upstream weirs so this is the cheaper option.

7.3.6 Option F: Alleviation channel

The alleviation channel option was priced using the two stage channel as a proxy by calculating a cost per metre length of channel for the two stage channel and multiplying this cost by the length of the alleviation channel. As they are both principally earthwork schemes they have similar components; earthworks, access tracks and site compounds.

7.3.7 Options G and H: Reservoirs

The development of costs for both reservoir options was completed at a high level, with no detailed or optimised design considered. Costs for the reservoirs are subject to the following assumptions and limitations:

- We have no Site Investigation (SI) data so have no information regarding soil types or ground conditions at the site. This could have a major bearing on the feasibility of providing a dam at this location and/or how the dam is designed and thus the cost of the dam.
- We have largely based our costs on rates within CESMM4 Carbon and Price Book. Rates can vary widely from region to region and with the state of the economy.
- The costs assume that all material to construct the dam (clay, sub-soil, topsoil) are imported to site, the conservative option. If SI work shows that material could be excavated from the site and reused in the dam, this will reduce the cost of the dam significantly.
- The costs assume an earth dam. It might be more cost-effective to provide a concrete dam (especially if all earth needs importing to site).
- It is assumed that no excavation will be required other than that associated with the construction of the embankment.
- The estimated quantities of material required for the dam have been based on simplified geometrical shapes. In particular The Gulls dam cross-section has been split into three simplified sections which assume a flat base. In reality the geometry of the dam will vary.
- Ground levels across the valley have been estimated from LIDAR data.
- No plans showing services or statutory utilities were available for the costing exercise. Service diversions could substantially increase the cost of the scheme.
- Environment impacts of the dam are not considered at present.

7.3.7.1 Option G: The Gulls flood storage reservoir

To contain the 100-year flood event the reservoir crest would, at its maximum, need to be approximately 5m high. However due to the shape of the valley, the embankment would taper off and reduce to zero at the side of the valley. For calculation purposes the embankment was split into three sections. Sections A and C assumed an average embankment height of 2m for a combined length of 115m, whilst section B assumed an average height of 5m for a 30m long section. This method is very crude but is suitable for a first pass for costing the scheme. The total embankment length is approximately 145m.

Onward flow at lower return periods is maintained in the model by two culverts (600mm and 700mm diameter). However, for simplicity, in the pricing one 900mm diameter pipe was used with a roughly equivalent flow area. If this option is taken forward this should be addressed, consideration should also be given to the cost effectiveness of other flow regulation structures such as moveable sluice gates or hydro brakes. This would require further, more detailed, modelling.

As part of the scheme, permanent access tracks will be installed for use during construction and for maintenance activities post construction. The landowner will need to be compensated for the area of land taken up by the access track and also for the land taken up by the embankment footprint; both these costs have been included in the pricing. However, at present no compensation for the landowner for inundation has been accounted for. This could be in the form of a 'Right to Flood' payment or the cost of buying the area of land that will be inundated. Any such payments/purchase could considerably increase the cost of the scheme.

It should also be noted that the current proposed embankment encroaches onto the B1077 due to the gradients required on side slopes and drainage/access to the embankment toe, although not to the extent of the Cherry Tree reservoir discussed below. The location chosen does not represent an optimised scheme but simply aims to illustrate the potential flood risk benefit which could be obtained. Given the short distance of road affected this is not been considered in the costing as for Cherry Tree reservoir. Any future work would require detailed reservoir design and additional modelling, at which point the location of the embankment is likely to be refined, based on the advice of engineers.

7.3.7.2 Option H: Cherry Tree Brook flood storage reservoir

The Cherry Tree Brook flood storage reservoir was designed to store flows up to and including the 100-year event. Allowing for 0.50m freeboard a maximum embankment height of approximately 5m would be required at the valley bottom but the height would reduce to zero where the embankment meets the valley sides. Therefore an average embankment height of 3m has been assumed for a total embankment length of 208m.

Given that an average embankment height of 3m was used in both cases, Cherry Tree Brook flood storage reservoir was priced using The Gulls flood storage reservoir as a proxy. A cost per metre length of embankment was calculated for The Gulls which was multiplied for the total length of Cherry Tree Brook reservoir embankment. Additional cost has been added to this option for the relocation of Low Road, which currently runs along the bottom of the valley, to a higher elevation, above the estimated 100-year water level.

Both reservoir options were priced for flood storage purposes; if longer term storage for water supply was sought additional design work would be required and would increase cost. It is also recommended that local soil cores are taken to evaluate local soil properties as part of any feasibility study.

7.4 Partnership Funding results

The results below record the scheme costs, benefits and final PF score. It should be considered that "houses protected" represents the changes from the baseline model results. Where a negative result is recorded this reflects the movement of properties originally in a higher risk category to a lower risk category.

7.4.1 Option A: Aspall Road culverts

Table 7-1: Option A Partnership Funding results

Number of houses protected	Very Significant Risk	7
	Significant Risk	2
	Moderate Risk	-7
Design and construction cost (£)		277,824
Future maintenance costs (£)		28,128
Estimated Whole Life Cost (£)		**305,952**
Estimated Whole Life Benefit (£)		**347,319**
PF Score (%)		**19**
Comments	Low PF score, but better than a number of options presented below. Relatively low cost scheme given the small extent, although this would involve significant	

	disruption with culvert installation beneath Aspall Road. Few properties removed from flood extents hence the low benefits, but many upstream properties flooded to a lower depth.

7.4.2 Option C: Two stage channel

Table 7-2: Option C Partnership Funding results

Number of houses protected	Very Significant Risk	14
	Significant Risk	-1
	Moderate Risk	-10
Design and construction cost (£)		1,520,904
Future maintenance costs (£)		As existing
Estimated Whole Life Cost (£)		**1,520,904**
Estimated Whole Life Benefit (£)		**1,677,610**
PF Score (%)		**11**
Comments	Low PF score; despite protection of a number of properties (particularly around the fire station area) the scheme is expensive given the extensive earth works required. Consideration should be given to local relocation of earth.	

7.4.3 Option D: Two stage channel with upstream weir removal

Table 7-3: Option D Partnership Funding results

Number of houses protected	Very Significant Risk	15
	Significant Risk	3
	Moderate Risk	-13
Design and construction cost (£)		1,720,904
Future maintenance costs (£)		14,064
Estimated Whole Life Cost (£)		**1,734,968**
Estimated Whole Life Benefit (£)		**1,957,648**
PF Score (%)		**11**
Comments	Low PF score as for Options C and E. Upstream weir removal protects two additional properties at the 20-year return period but incurs additional costs.	

7.4.4 Option E: Two stage channel with downstream weir removal

Table 7-4: Option E Partnership Funding results

Number of houses protected	Very Significant Risk	14
	Significant Risk	-1
	Moderate Risk	-10
Design and construction cost (£)		1,550,904
Future maintenance costs (£)		As existing
Estimated Whole Life Cost (£)		**1,550,904**
Estimated Whole Life Benefit (£)		**1,680,958**
PF Score (%)		**10**
Comments	Low PF score as for Options C and D. Downstream weir removal protects no additional properties.	

7.4.5 Option F: Alleviation channel

Table 7-5: Option F Partnership Funding results

Number of houses protected	Very Significant Risk	0
	Significant Risk	0
	Moderate Risk	1
Design and construction cost (£)		554,248
Future maintenance costs (£)		16,601
Estimated Whole Life Cost (£)		**570,849**
Estimated Whole Life Benefit (£)		**17,102**
PF Score (%)		**0**
Comments	Very low PF score given the few properties protected and the significant earthworks involved in construction. An optimised design to protect a greater number of properties may bring this option more in line with the two stage channel results, although PF score is still likely to remain low.	

7.4.6 Option G: The Gulls flood storage reservoir

Table 7-6: Option G Partnership Funding results

Number of houses protected	Very Significant Risk	19
	Significant Risk	3
	Moderate Risk	-9
Design and construction cost (£)		586,055
Future maintenance costs (£)		94,533
Estimated Whole Life Cost (£)		**680,588**
Estimated Whole Life Benefit (£)		**1,529,603**
PF Score (%)		**28**
Comments	This option results in the highest PF score of those considered. The construction of the impounding reservoir is relatively in-expensive given the small embankment required, although this is not an optimised design. Attenuation of flood flows protects a significant amount of properties. Consideration could be given to combining this option with the two stage channel. If the excavated earth from the new channel could be used in embankment construction this may reduce the costs of both schemes and also provide greater economic benefits. A significant third party contribution is required to achieve a PF score of 100%.	

7.4.7 Option H: Cherry Tree Brook flood storage reservoir

Table 7-7: Option H Partnership Funding results

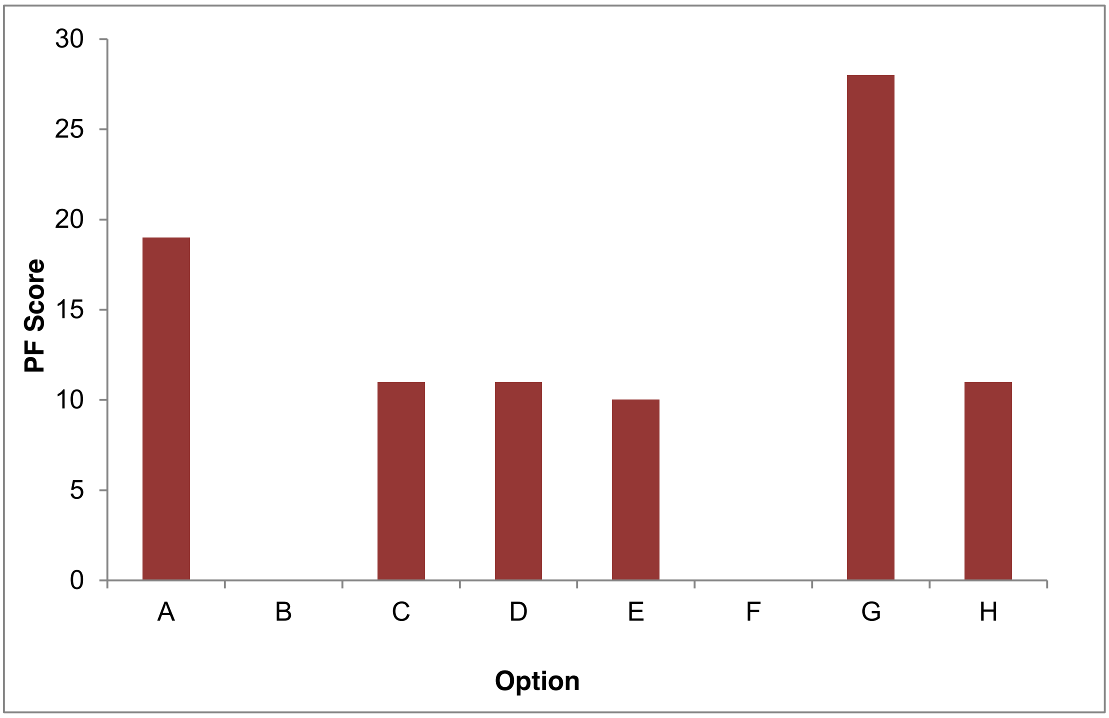
Number of houses protected	Very Significant Risk	9
	Significant Risk	-2
	Moderate Risk	-1
Design and construction cost (£)		829,168
Future maintenance costs (£)		94,533
Estimated Whole Life Cost (£)		**923,701**
Estimated Whole Life Benefit (£)		**1,060,014**
PF Score (%)		**11**
Comments	This option results in a similar PF score to the two stage channel. The	

	construction is more expensive than The Gulls reservoir given the relocation cost of the Low Road carriageway but fewer properties are protected. Consideration could be given to combining this option with the two stage channel. If excavated earth from the new channel could be used in embankment construction this may reduce the cost of both schemes and provide greater economic benefits.

7.4.8 Summary

Figure 7-1 below illustrates the calculated PF score for each of the options investigated. It is apparent that The Gulls impounding reservoir produces the highest PF score.

Figure 7-1: Partnership Funding scores



Further consideration should be given to The Gulls option. In addition to flood risk the reservoir could be designed to help maintain low flows in dry periods and also irrigation for the Aspall orchards. If this option were to be taken forward the design would be developed in conjunction with the River Deben Holistic Water Management Project. It is also recommended that any further study considers potential cost savings associated with implementing a combined solution. For example, whether soil extracted from the two stage channel can be used for construction of a reservoir embankment.

8 Assumptions and limitations

Hydraulic models and flow estimates are produced in an attempt to represent real world systems. It is inevitable that some stages of the process will be subject to assumptions made by the modeller and limitations imposed by data availability and software ability.

Uncertainty is reduced in Debenham by provision of level data by which to calibrate the time-to-peak parameter of the ReFH model. Anecdotal and photographic evidence of flooding since the 1930s have further helped to bench-mark model results. However, a number of uncertainties are still present.

8.1 Hydrology

As discussed above, the hydrological estimates have undergone some calibration, although some uncertainties and limitations still exist:

- Analysis has suggested time-to-peak on the contributing catchments reduces as rainfall intensity increases. Whilst this is a well-defined trend, uncertainty would be reduced with provision of a longer gauge record.
- It was assumed that each catchment is likely to produce the same design return period flow for a given rainfall event. Whilst each catchment is similar, it is likely that each responds slightly differently. This assumption is conservative.

8.2 Hydraulic modelling

A number of additional assumptions were made in the Debenham model, many of which are assumed in most hydraulic models:

- In-channel roughness values were derived based on modelling judgement alongside two supplied check gaugings and photographic evidence of flood history at Debenham. Whilst results match the observed events well, it should be considered that this is a source of uncertainty; roughness values were increased throughout the model (to varying degrees) as result. Confidence would be improved with further check gauging.
- Building footprints in the village are stamped up from ground level by 0.3m to represent floor level. Where threshold survey data are available these elevations were used.
- Spills coefficients at structures set to realistic levels based on photography.
- Loss coefficients are defined in 1D/2D model boundaries to account for lateral losses.

8.3 Economic damage estimation

In addition to the above, estimation of AAD values were also used some assumptions such as:

- All properties designated "potential upper floor" are not included in the damage calculations, but maintained for the purposes of flooded property counts.
- Where the building use is not clear from either NRD data or available online photography an assumption is made (i.e. if at the bottom of a garden this is assumed to be a domestic shed).
- The floor area stated in the NRD is used, unless this is missing in which case area is calculated from building footprints.

Limitations associated with the techniques used include:

- Lack of threshold survey across the entire 1,000-year flood extent results in some uncertainty regarding damage estimates.
- National average depth-damage curves are used, which include basements for residential properties. To further refine estimates, properties with basements could be recorded and a different curve applied to these.

8.4 Cost benefit analysis

The main limitation of the cost benefit analysis is that the scheme design is indicative at this stage of the project. These designs were not optimised to protect against a given return periods; this should be considered when analysing the results.

9 Deliverables

In addition to the final report and associated appendices, a number of digital deliverables are supplied as an output for this project. Deliverable outputs include:

- Topographic survey deliverables;
- Hydraulic modelling files;
- Results files;
- NFCDD output (for 2, 5, 10, 20, 75, 100 and 1,000-year);
- NFCDD output for climate change scenarios (for 20, 100, 1,000-year plus 15% allowance);
- Depth, velocity and hazard grids;
- PDF mapping of baseline results;
- PDF mapping of option results;
- PDF mapping of AAD results;
- FRISM results.

10 Conclusions and recommendations

10.1 Conclusions

Detailed flood mapping demonstrated that Debenham village is at significant risk of flooding from all main watercourses draining to the settlement. This project built upon previous studies and information provided by the Environment Agency, SCC and local residents to improve understanding of the flood mechanisms operating at Debenham. We have, in turn, ensured these extents correlate well with the observed flood history in the village; results now indicate the Market Square in the centre of Debenham is flooded on average once every 20-years. Flood extents were increased elsewhere in the village, such as the area to the south of the High Street and the fire station, providing a good match to photographs of flooding in 1993.

The modelling demonstrated that Debenham is at low risk for surface water flood risk; lack of historic surface water flooding supports this conclusion.

Part of this commission involved the investigation of various flood alleviation options for Debenham following completion of the hydraulic modelling. These were discussed at a meeting with the Environment Agency and SCC in September 2013 and subsequently at a community engagement meeting in Debenham. Eight schemes were discarded following this meeting and eight were taken forward (see sections 6.1 and 6.2) for further investigation, represented in hydraulic models. Modelling has demonstrated that construction of an impounding reservoir on The Gulls watercourse near Aspall, or development of a two stage channel, serves greatest flood risk benefit to Debenham in terms of number of properties protected.

In addition to hydraulic modelling, work was undertaken to assess the predicted economic damages associated with a given design flood and also the average damages per year. These results, alongside the model results, were used to calculate the GiA Partnership Funding score. It is concluded that as well as providing the greatest flood risk benefit, The Gulls reservoir also offers the highest PF score of all the considered options, at 28%. This is reduced from the previously estimated value (in the May 2014 report) given the lower economic benefit, in turn impacted by the lower baseline AAD results.

10.2 Recommendations

In light of the assumptions and limitations listed in chapter 8, the following are recommended:

- A greater availability of hydrometric data would increase confidence in design flow estimates and therefore hydraulic modelling results in Debenham. This project sought to use all available information although a thorough flow record on one of the major watercourses would aid future hydrological modelling.
- In the absence of a permanent flow gauge we recommend the Environment Agency continue a programme of check gauging in Debenham.
- It is recommended that the current maintenance regime in Debenham is continued. Increases in hydraulic roughness result in increased water levels; the narrow geometry of the channels in Debenham ensure this are sensitive to increased vegetation growth.
- Thorough consideration should be given to The Gulls reservoir flood storage option as this has the potential to significantly reduce flood risk at Debenham. If this option is preferred, further input from a range of specialities is required.
- The two stage channel offers significant flood risk benefits for the southern extent of Debenham, although the costs associated with re-shaping and removing material from the floodplain are high. Cost savings could be achieved if material was deposited locally; it is suggested some consideration is given to this possibility. Given the highly fertile natural of this alluvial soil it is possible that this could be sold to local farms.
- Combining both The Gulls reservoir and two stage alleviation options should be considered for cost saving and potentially increased benefits. Please note this combination has not been modelled.
- Removal of the left bank of the non main river drain near Cherry Tree Brook is not recommended. Whilst removal initially has localised benefits, this will result in reduced

floodplain storage when a large flood event occurs on both Cherry Tree Brook and the River Deben, potentially increasing flood risk to the area in question.

- It is recommended other sources of funding are considered to increase the PF score of the preferred option.

Appendices

A Appendix A - FEH Calculation Record

Flood estimation calculation record

Introduction

This calculation record is based on a supporting document to the Environment Agency's flood estimation guidelines (Version 4, 2012). 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

	Name and qualifications
Calculations prepared by:	Kevin Haseldine BSc MSc
Calculations checked by:	Duncan Faulkner MA MSc FCIWEM C.WEM CSci

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	The Environment Agency commissioned JBA in 2013 to update the existing modelling undertaken at Debenham, Suffolk, in order to assess flood risk. This included updates to the hydrological estimates and addition of a TUFLOW domain to the existing ISIS hydraulic model. Outputs helped to inform the Agency on flood mechanisms in the area and also included development of management options for the village. Following this study JBA were commissioned again to extend the hydraulic model upstream (alongside other update discussed in the main report) in 2014. This required estimation of the hydrology on one watercourse, The Gulls, to account for the extended model reach. Required modelled return periods include: 2-year 10-year 75-year 1,000-year 5-year 20-year 100-year Inclusion of climate change impact is required for the 20-year, 100-year and 1,000-year return periods. This document records the calculations involved in the hydrological update for Debenham. Previous flow estimates are considered and updated where appropriate, given the availability of new techniques and additional data.

1.2 Overview of catchment

1.2.1 Catchment characteristics

Debenham is situated on the upper reaches of the River Deben, with the watercourse flowing through the centre of the village. Throughout the settlement the River Deben is joined by various tributaries, the most significant of which are The Gulls, Derry Brook and Cherry Tree Brook, with confluences at the north and south extents of the village respectively (see Figure 1-1). The River Deben is only known as such downstream of The Gulls – Derry Brook confluence. Downstream of the Cherry Tree - Deben confluence the contributing catchment area is 34.07km^2. All these watercourses are considered in the current project alongside Aspall Drain, a small channel joining The Gulls at Aspall.

No major settlements are located in the catchment with the exception of Debenham itself. A number of small villages and hamlets are present, such as Aspall and Wetherup Street, but these consist of a scattering of buildings at most. As such the catchment can be considered predominantly rural, reflected in an URBEXT2000 value on the FEH CD-ROM of 0.009.

The upper River Deben catchment is relatively steep considering it location in the east of England, dropping 15.9m/km (indicated by the DPSBAR catchment descriptor). Of the incoming tributaries, Cherry Tree Brook has the steepest catchment at 17.2m/km. The upper River Deben catchment is actually shallower than the overall Deben; where the watercourse becomes tidally influenced (at Woodbridge) the DPSBAR is 22.9m/km.

Typically for eastern England, the River Deben catchment receives a low average annual rainfall of around 590mm; despite this various flood events have been recorded at Debenham in the recent past (see section 1.2.2). The frequent flooding is likely due to the occasional extreme rainfall events in the area, as evidenced by the typically steep rainfall growth curves associated with the east of England.

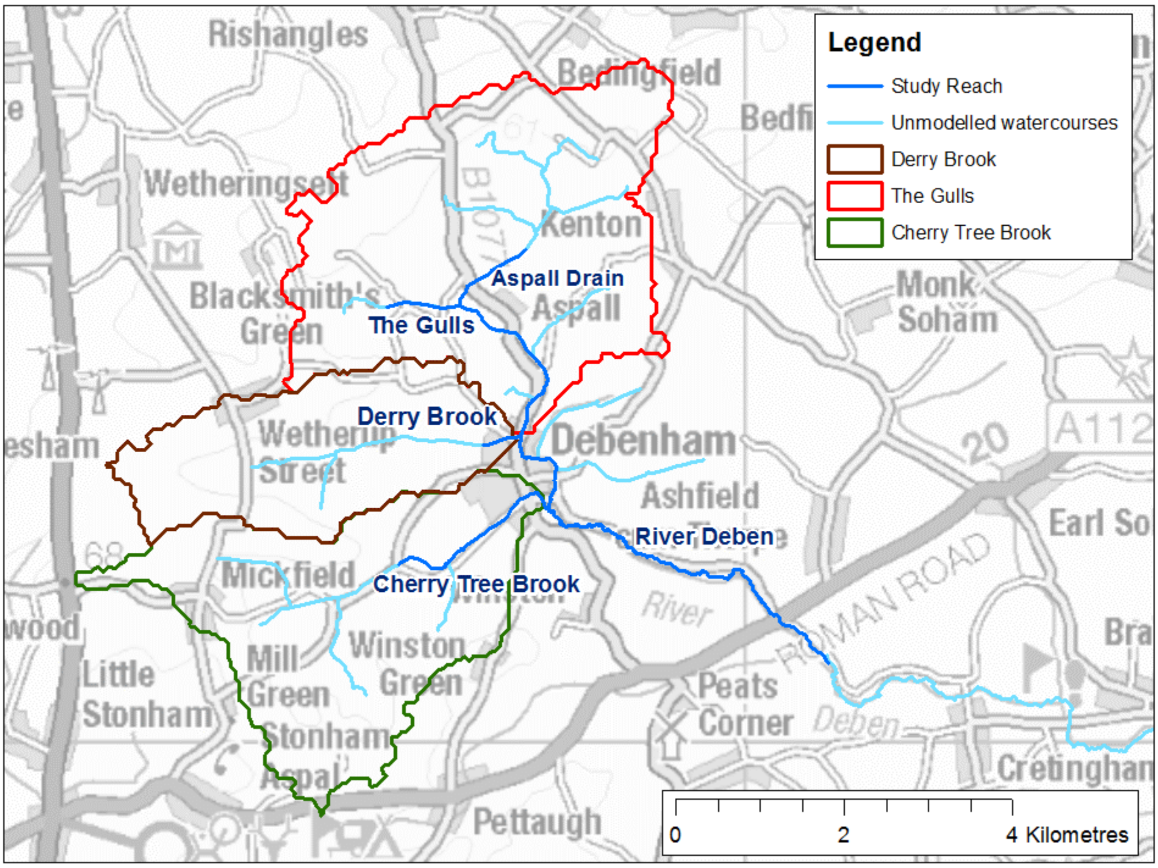


Figure 1-1: Upper Deben catchment and surrounds

Contains OS data © Crown copyright and database right 2014

The upper catchment is underlain by sedimentary bedrock of gravels, silts and clays. Some chalk deposits are located within the river valley, although these are not extensive. Soils in the catchment consist of two types, slowly permeable base rich loams and clays and lime-rich loams and clays with impeded drainage, leading to a BFIHOST value of 0.320, indicating moderately low permeability.

It should be considered that downstream of the study reach, the River Deben flows over a band of permeable chalk bedrock. This is highlighted in Figure 1-2, which is taken from the National River Flow Archive website for the flow gauge at Naunton Hall on the River Deben. This needs to be fully considered in the hydrological analysis. Naunton Hall was previously considered as a QMED donor site for Debenham, although this band of permeability was not discussed.

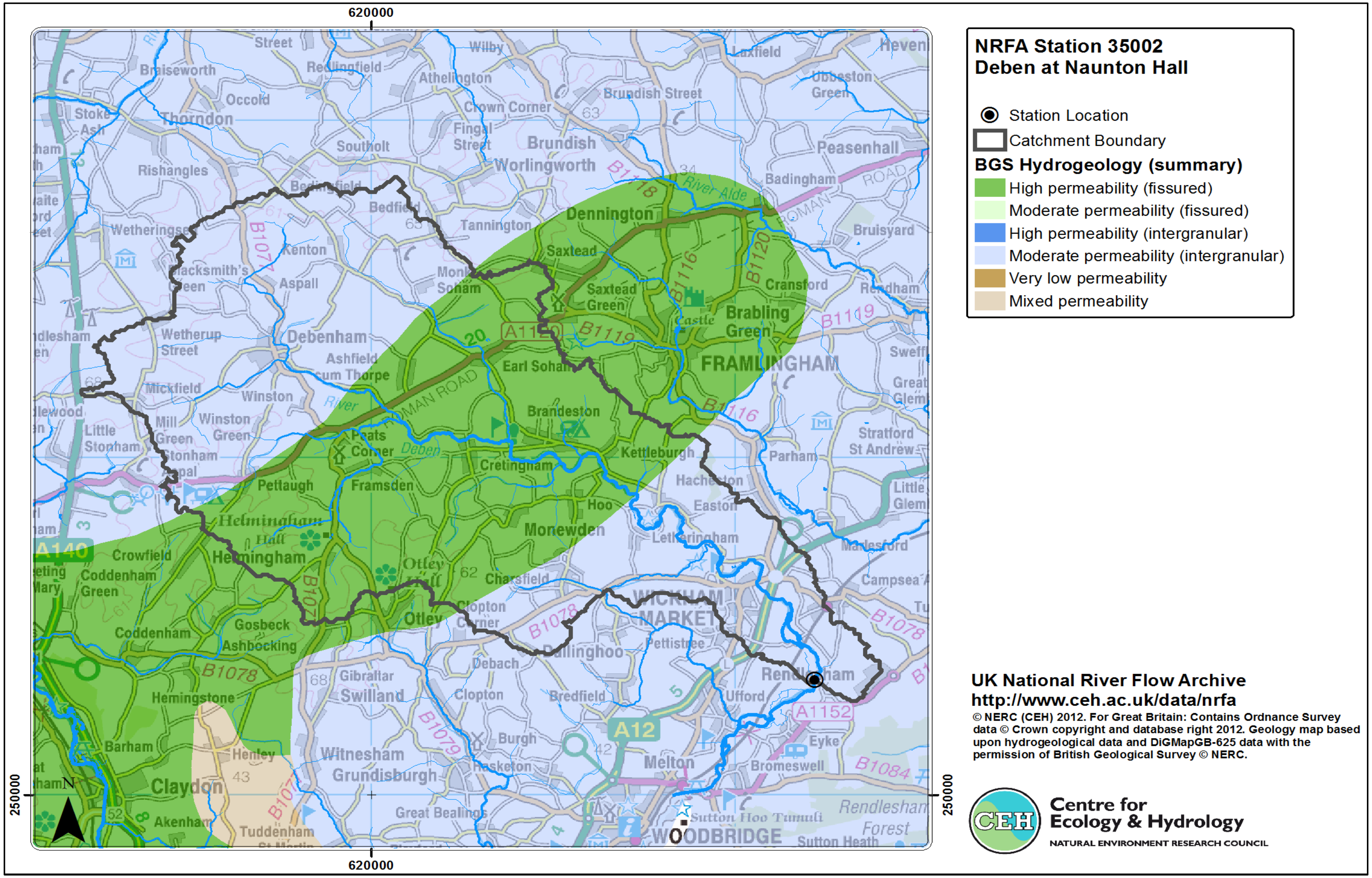
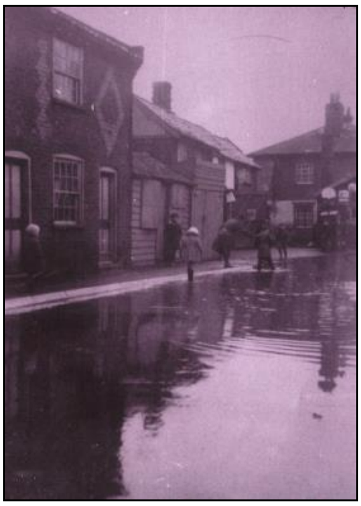


Figure 1-2: River Deben catchment geology

1.2.2 Flood history

Despite the low average annual average rainfall, the River Deben has been the cause of many flood events in the past. Information exists as far back as 1912, when villagers noted sheaves of corn being carried through the village by the flood flows. Photographs taken in 1936, 1937, 1944, 1947, 1956, 1968 and 1993 showing flooding in Debenham have been provided by the Environment Agency and local residents. Flooding appears to be driven by high levels on both the River Deben and Cherry Tree Brook. A selected number of these photographs are shown below.

Photographs of historical flood events, Debenham	
	**1936** Flooding at south extent of the High Street
	**1944** Flood water from River Deben at Market Square
	**1947** High Street, former public house near Market Square.

	**1956** View of the Market Square from Little Back Lane. The River Deben flows into a culvert beneath the High Street behind the wall shown to the left.
	**September 1968** Water Lane, looking east
	Looking north-west from near the Market Square
	**October 1993** Water Lane, looking east
	Flooded pottery on Low Road, adjacent to Fire Station.

	Northern extent of Chancery Lane, near Deben House.
	Aspall Road from near the Market Square.

These photographs provide a helpful record of flooding in the village, but it is not possible to accurately rank the events because it is not known whether they record the flood peak. It is obvious the Market Square has flooded on at least five occasions (1944, 1947, 1956, 1968 and 1993) in the past 70 years (1944-2014, no Market Square photos available for the 1936 event), with flood waters spilling from the River Deben. The provided photographs only show flooding at the southern end of the village in 1936 and 1993, although it is unlikely these were the only events.

The JBA report produced in 2010[1] noted that the October 1993 event was estimated (at the time) to have a return period of around 25-years and flooded 33 properties. Residents at the Debenham community engagement meeting held in November 2013 (following delivery of draft results of the 2013 project) suggested the 1956 and the 1993 events were the most severe.

Despite the lack of flow record or the ability to rank the flood events by severity, this evidence is very helpful in ensuring the modelling results are representative of the flooding processes occurring at Debenham. For further details of the model checking and calibration please refer to both the hydraulic model check file and the final report.

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	No HiFlows-UK data are used in the current study given the lack of suitable flow gauge at Debenham or in the nearby catchments. This is discussed further below.

[1] Environment Agency., 2010. *River Deben Model Review – Phase 2: Improvement of model*. Prepared by JBA Consulting.

1.4 Gauging stations

Watercourse	Station name	Station number	Grid reference	Catchment area (km²)	Type	Start and end of flow record
Deben	Naunton Hall	35002	632173, 253353	163.1	Rated	August 1964 - date
Deben	Brandeston	35035	623859, 260369	101.5	Originally rated for low flows, now ultrasonic	July 1999 – date (ultrasonic from 2002)
Deben	Debenham	L350605	617607, 264201	13.2	Level	2001 – present
Deben	Cherry Tree	L350601	617530, 262950	10.2	Level	1995 – 2001
Deben	Derry Brook	L350604	617250, 263620	6.6	Level	1996 – 2001

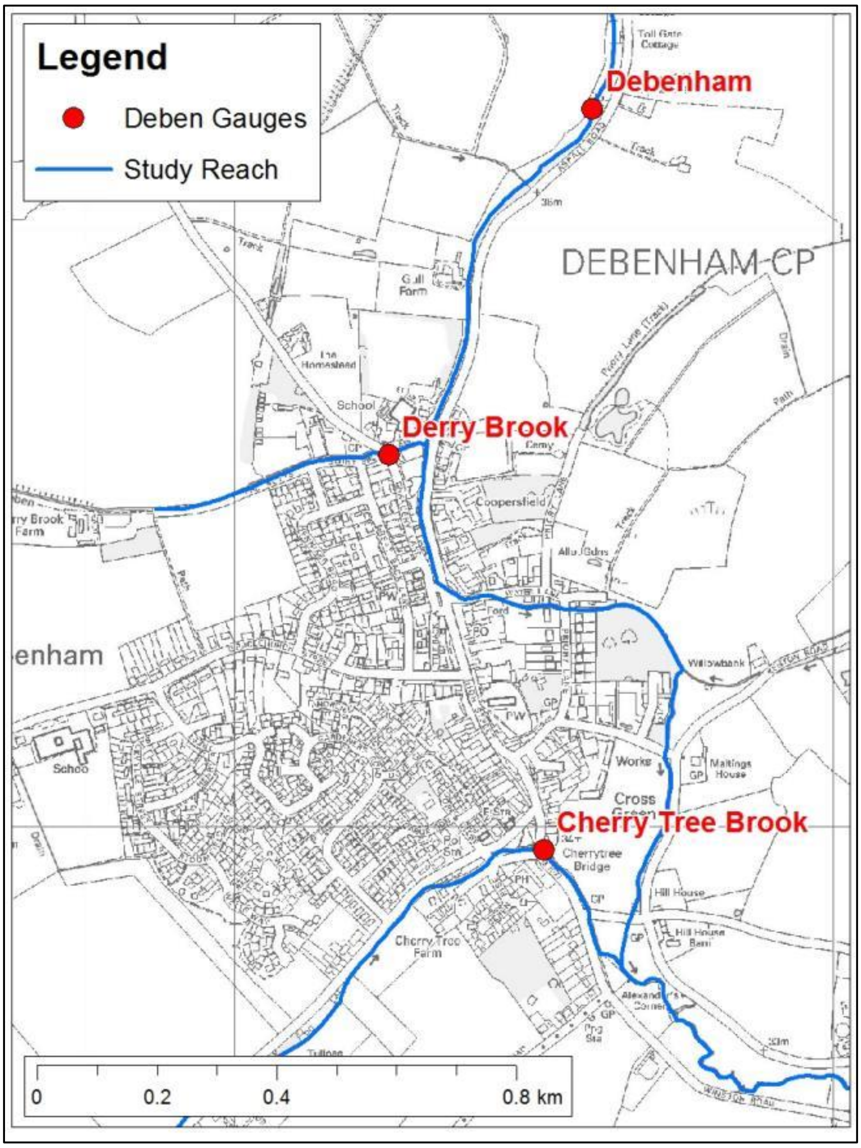


Figure 1-3: Debenham Level Gauges

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.
Naunton Hall	Not in HiFlows-UK	Yes	No	No	Yes	Theoretical rating applied to stages above modular limit – treat high flows with caution.
Brandeston	As above	Yes	No	No	Yes	Original rating was developed for low flow investigation when a weir was in place. This was removed around 2002 and the gauge is now fully ultrasonic. The EA have informed us that this does not account for any bypassing.

1.6 Rating equations and suitability

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.
Naunton Hall	Theoretical	No	Gauge not to be used in current study – see discussion below.
Brandeston	n/a	n/a	Ultrasonic

1.6.1 Naunton Hall

Flow data from Naunton Hall gauge shall not be used in the current project as it is believed the catchment draining to this gauge is not representative of that at Debenham, given the difference in catchment size and the band of permeable geology around Brandeston (see Figure 1-2). The 2010 JBA report considered the use of this gauge and rejected it given AMAX data resulted in QMED estimates lower than those produced using catchment descriptors alone. Given the current project was commissioned because the 2010 outlines underestimated flood extents compared to the recorded flood history, it is our view that hydrological estimates need to be increased, not decreased.

1.6.2 Brandeston

As discussed briefly above, the gauge at Brandeston is ultrasonic and does not account for any bypassing flows. Given the low channel banks at the gauge it is likely the site has bypassed on various occasions since the gauge's installation. This, together with the fact part of the contributing catchment includes the permeable bedrock discussed above, ensures the site is not suitable for use in flood peak estimation.

1.7 Flood mechanisms and characteristics

1.7.1 Initial time-to-peak analysis

As discussed above (section 1.2.2), floods in the village occur as a result of channel capacity exceedance. The historical photographs suggest flooding occurs most frequently at the Market Square (from the River Deben channel) and at the southern end of the High Street (from Cherry Tree Brook).

No flow data are available at Debenham, although data from three level gauges have been provided for use with the current project. Both Derry Brook and Cherry Tree Brook gauges were operational at the same time, although no overlapping record exists for the gauge on the Gulls.

It seems likely that all three watercourses will peak at a similar time given the similar catchment sizes and characteristics, although it is difficult to conclude this precisely given the lack of overlapping data for The Gulls. However, the provision of a rainfall time-series at Needham Market (the closest gauge to Debenham) allows time-to-peak (Tp) estimation to be undertaken on each watercourse.

Tp is closely related to catchment lag times, which can be derived from rainfall and flow/level data. For this project the relative timings of rainfall at Needham Market and the associated peak level at each gauge were compared. JBA's in-house Hydrometric Database allows extraction of each event's lag time (the difference between the rainfall centroid and peak water level recorded); an example is shown below, recorded on The Gulls.

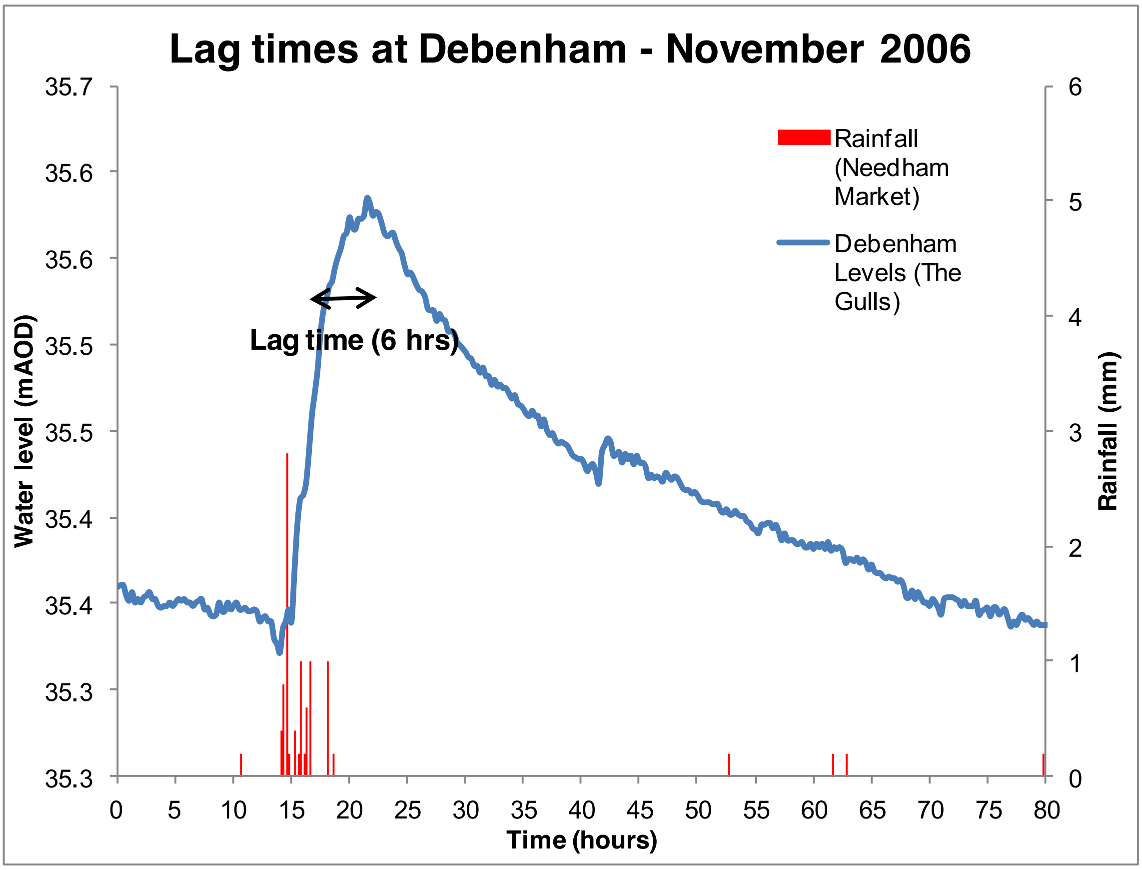


Figure 1-4: The Gulls lag analysis

For Tp estimation, the FEH recommends that lag for rural catchments should be estimated using at least 18 months of data, which was available for each watercourse in the study. A number of considerations were made regarding which flood events to use:

- Only events over a certain level threshold were included for each watercourse
- No double peaked flood events were considered
- Only those events preceded by significant rainfall at Needham Market, with an obvious centroid, were included.

Lag is calculated for at least 10 events on each watercourse. The geometric mean (as recommended in the FEH) of these is then estimated and the resulting lag converted to Tp using the following equation:

$$Tp = 0.879\left(LAG^{0.951}\right)$$

The events used in this analysis are recorded in section 7.2 of this document.

By comparing the average Tp on each watercourse, we can build up a better understanding of how these respond to rainfall events. The Tp values also help refine the hydrological model (see section 5 for details). Please note the values below are estimated at the gauge site on each watercourse. In order to apply these to the upstream flow estimation points an adjustment factor is used (discussed in section 5).

Watercourse	Derry Brook	The Gulls	Cherry Tree Brook
Tp (hours)	4.12	4.65	3.95

It is apparent that peak levels on each watercourse occur at roughly the same time following a rainfall event. This implies that each catchment behaves in a similar manner and that an extreme rainfall event is likely to result in a similar return period event on each watercourse, assuming similar antecedent conditions.

1.7.2 Further time-to-peak analysis

1.7.2.1 Updated record

At the community engagement meeting in November 2013, residents considered that draft flood outlines better represented flood risk than the existing Flood Zones, but continued to underestimate flood risk at Debeham. As a result the time-to-peak analysis was revisited.

The initial time-to-peak analysis was updated in January 2014 to include rainfall and flow data recorded between December 2012 and December 2013 at Needham Market and The Gulls respectively. The gauged flow records for Derry Brook and Cherry Tree Brook do not extend beyond 2001 and therefore have not been updated. Event lag times were extracted for this additional period and incorporated into the Tp analysis, adopting the same considerations as in section 1.7.1. The recalculated Tp value for the gauge at The Gulls is 4.30 hours, a decrease of 0.35 hours from the original estimate.

1.7.2.2 Event Seasonality

Each gauged record was analysed trends, assessing whether Tp varies between seasons. Historical photographic evidence and dated flood records supplied by the EA were used to identify the modal season(s) for large events. Eleven events between 1816 and 1993 could be ascribed a season, whilst the timing of four additional events were estimated but could not be confirmed. These are outlined below:

Season	Confirmed Events	Potential Events
Winter (DJF)	3	1
Spring (MAM)	1	1
Summer (JJA)	1	0
Autumn (SON)	6	2

Events within the gauged record were grouped by season and the geometric mean lag time calculated for each subset of data at each gauge. The resultant Tp calculated by season for The Gulls is given below. This pattern of a much shorter Tp for summer and similar values for autumn, winter and spring is reflected in the record at Derry Brook and Cherry Tree Brook.

Season	The Gulls - Tp (hours)
Winter (DJF)	4.58
Spring (MAM)	5.12
Summer (JJA)	2.58
Autumn (SON)	4.49

The seasonality trends indicate that a Tp value representing Autumn and Winter would reflect the majority of observed events at Debenham. Two Tp values were calculated, one equally representing the Autumn and Winter storm characteristics and the other weighted towards the Autumn Tp given the prevalence of events during this season in the historical record.

Watercourse		Derry Brook	The Gulls	Cherry Tree Brook
Tp (hours)	Equal weighting Autumn-Winter	4.13	4.53	4.83
	Weighted towards Autumn	3.98	4.52	4.77

1.7.2.3 Rainfall Intensity

The relationship between maximum rainfall intensity recorded during each event in the Tp analysis and the lag time was investigated. In small, steep catchments, extreme flood events often result from short duration convective storms and the rapid delivery of water to the channel predominantly via overland flow. The slower mechanisms of throughflow and groundwater flow are less likely to contribute to the flood peak in such events. The three catchments that meet in Debenham are relatively steep, with DPSBAR ranging between 10.8 and 22.1m/km.

Plots of the regression equation between maximum intensity and lag time illustrate that a relationship may exist in the three subject catchments (Figure 1-5 below). It can be construed that a number of flood events in Debenham result from rainfall events which have a high maximum intensity and subsequently result in a short lag time between rainfall and peak flows.

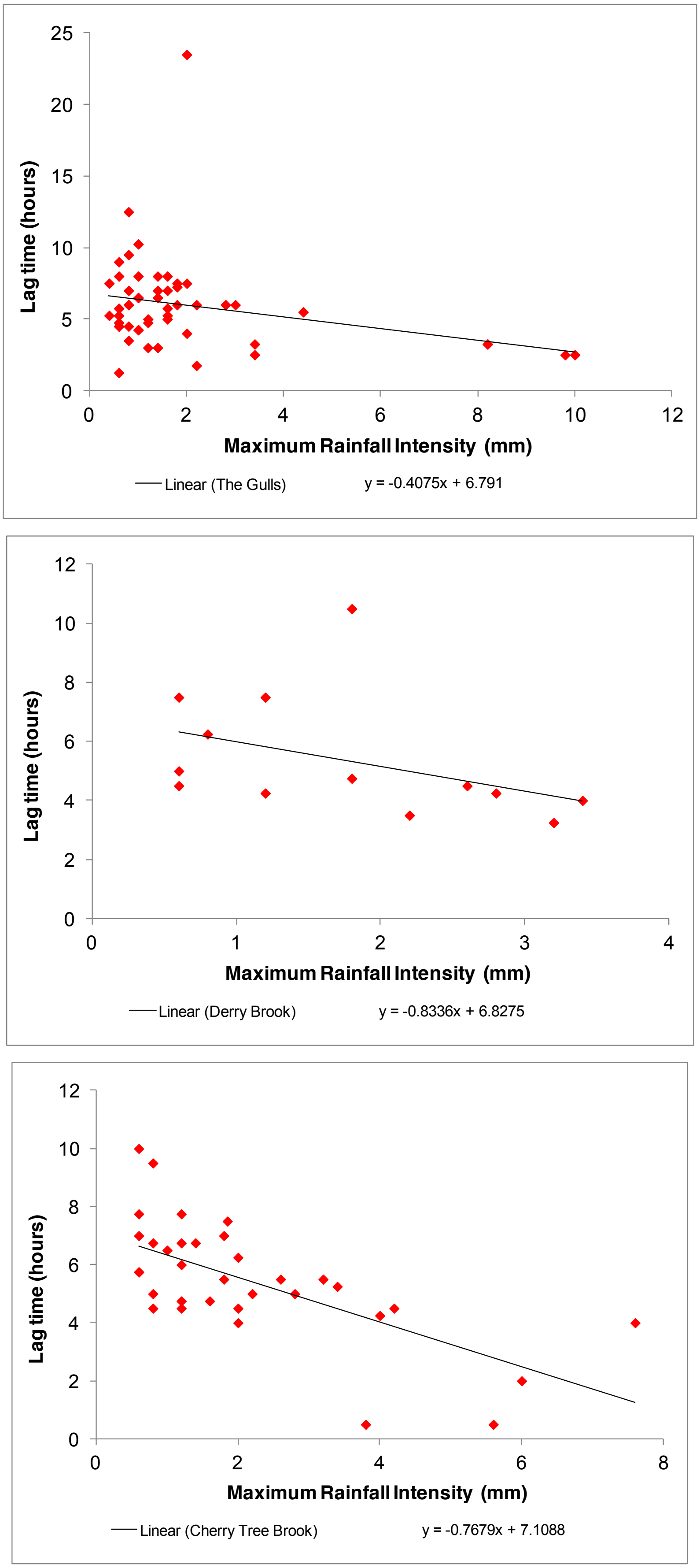


Figure 1-5: Relationship between maximum rainfall intensity and lag time

To represent this relationship in the model, the upper quartile (top 25%) of events, ranked by maximum rainfall intensity, were used to derive a new Tp. This was calculated using the same method as in Section 1.7.1. The results at the relevant gauge are given below:

Watercourse	Derry Brook	The Gulls	Cherry Tree Brook
Tp (hours)	3.26	3.53	2.18

These are substantially shorter than the existing Tp used in the model, particularly on Cherry Tree Brook, where Tp has been reduced from 3.95 hours. Please note an adjustment must be applied to these values to represent the upstream extent of the hydraulic model. These new values are likely to result in the peak flows being received more quickly in Debenham and may also result in a greater magnitude due to the coincidence of the peak flows from the three watercourses.

1.7.2.4 Conclusions

Inclusion of data from 2013 has resulted in a decrease in the Tp at The Gulls. The combined maximum stage is likely to increase from the original analysis due to the coincidence of these flows and may increase the modelled flood extent.

This seasonality analysis has supported the initial conclusions that peak levels on each watercourse occur at roughly the same time, with Tp at each of the gauges occurring within 0.8 hours, similar to the original analysis. The antecedent conditions are expected to be similar across all three catchments, varying primarily with season rather than in response to specific catchment characteristics. These new Tp values, whilst providing a more realistic representation of the catchments in response to extreme rainfall events, are unlikely to result in a significant change to the modelled flood extent from the initial model runs.

The clearest trend is evident in the rainfall intensity vs lag time analysis. The trend identified here suggested that catchment lag times decrease during particularly intense rainfall, which is responsible for a number of large flood events in Debenham. The shorter lag times associated with more intense rainfall events were used to derive a more representative Tp for the three watercourses, describing the more extreme fluvial events in the catchment. These new Tp values were between an hour and three hours shorter than the original estimates and are within 1.2 hours of each other. These may result in a larger modelled flood extent than the initial model runs and reproduces the fast rising water levels observed in Debenham in 1993.

1.8 Other data available and how it has been obtained

Type of data	Data relevant?	Data available?	Licence reference	Date obtained	Details
Check flow gaugings	Yes	Yes	Standard notice	20/02/2013	Check gaugings supplied by Environment Agency on Cherry Tree Brook and The Gulls. Information used in hydraulic model verification.
Historic flood data – give link to historic review if carried out.	Yes	Yes	Standard notice	20/02/2013	Photographs
Flow data for events	Yes	Yes	Standard notice	19/02/2013	Low flow data at Brandeston, full time series at Naunton Hall.
Rainfall data for events	Yes	Yes	Standard notice	17/06/2013	Needham Market, Stradebroke & Great Finborough.
Potential evaporation data	No	No	n/a	n/a	n/a
Results from previous studies	Yes	Yes	Z22442_CL _JBA	19/02/2013	2009 project
Other data or information (e.g. groundwater, tides)	No	No	n/a	n/a	n/a

1.9 Initial choice of approach

Is FEH appropriate (it may not be for very small, heavily urbanised or complex catchments)?	Yes.
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?	The main site of interest for the current study is Debenham village. Flooding is known to have occurred here on various occasions, particularly around Market Square and the south end of the High Street. Flooding of the settlement is likely to be driven by backing up of water behind hydraulic structures, a problem potentially exacerbated by coinciding peaks on the three watercourses in the village (River Deben, The Gulls and Cherry Tree Brook). The current study will also include rainfall inflows directly onto the 2D model domain to investigate the potential for surface water flooding in the village.
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	The upstream catchments of Derry Brook and Cherry Tree Brook are very similar in size, permeability, urbanisation and rainfall. The incoming Gulls tributary is a little larger, with a 14km^2 catchment at the confluence; all other catchment descriptors are roughly similar. Please note that the NRFA website highlights an area of highly permeable bedrock underlying the River Deben catchment downstream of Debenham. Whilst this will not impact flood levels in the village, it should be noted that flow records at both downstream gauging stations will include flows from permeable areas. There may also be some loss of water from the river to the aquifer as it flows over this permeable area.
Initial choice of method(s) and reasons Will the catchment be split into subcatchments? If so, how?	Both the FEH statistical methodology and the ReFH method have been considered. Flow estimates from each methodology are outlined in this document, and section 6 discusses the final choice of method and the justification for this choice.
Software to be used (with version numbers)	FEH CD-ROM v3.0[2], WINFAP-FEH v3.0.002[3] ReFH spreadsheet, ISIS

[2] FEH CD-ROM v3.0 © NERC (CEH). © Crown copyright. © AA. 2009. All rights reserved.
[3] WINFAP-FEH v3 © Wallingford HydroSolutions Limited and NERC (CEH) 2009.

2 Locations where flood estimates required

The table and map below lists the location of flood estimation points within the study reach. Lateral inflows are included in this section which account for any catchment area not modelled as a discrete inflow (i.e. a tributary). To avoid double counting, the catchments of these do not include any area where direct rainfall is to be applied in TUFLOW.

2.1 Summary of subject sites (main channel)

Site code	Watercourse	Site	Easting	Northing	FEH AREA (km^2)	Revised AREA if altered
River Deben and tributaries						
DEBN_001	Deben	U/S FEP at The Butts.	616850	263550	6.31	n/a
DEBN_002		Derry Brook level gauge.	617250	263600	6.60	n/a
DEBN_003		FEP at High Street.	617300	263400	20.38	n/a
DEBN_004		FEP at Priory Lane.	617500	263350	21.82	n/a
DEBN_005		FEP U/S of Cherry confluence.	617650	292900	23.66	n/a
DEBN_006		FEP D/S of Cherry confluence.	617600	262850	33.85	n/a
DEBN_007		FEP at Malthouse Farm.	618950	262150	36.50	n/a
DEBN_008		D/S FEP at Ashfield Place Farm.	620700	261250	39.80	n/a
PRIO_001	Unnamed drains	Priory Lane tributary inflow.	617500	263400	1.38	n/a
KENT_001		Kenton Road tributary inflow.	617800	263250	1.58	n/a
WINS_001		Tributary inflow near Winsford.	618300	262450	0.81	n/a
ASHF_001		Tributary inflow near Ashfield Place Farm.	619950	262050	1.29	n/a
The Gulls and tributaries						
GULL_001	The Gulls	U/S FEP at Red House Farm	616750	265100	2.85	n/a
GULL_002		D/S of Willowdene tributary.	617350	264650	12.65	n/a
GULL_003		Debenham level gauge.	617600	264200	13.18	n/a
GULL_004		U/S of R Deben confluence.	617300	263700	13.68	n/a
ASPD_001	Aspall Drain	U/S FEP	617450	266000	4.32	n/a
WILL_001	Unnamed drain	Willowdene tributary inflow.	617400	264700	1.59	n/a
Cherry Tree Brook						

Site code	Watercourse	Site	Easting	Northing	FEH AREA (km^2)	Revised AREA if altered
CHRY_001	Cherry Tree Brook	U/S FEP at White Hall Cottage.	615850	262100	6.63	n/a
CHRY_002		FEP at Poplar Farm.	616400	262100	8.02	n/a
CHRY_003		Cherry Tree Brook level gauge.	617550	262850	10.17	n/a
POPL_001	Unnamed drain	Poplar tributary inflow.	616450	262100	0.51	n/a

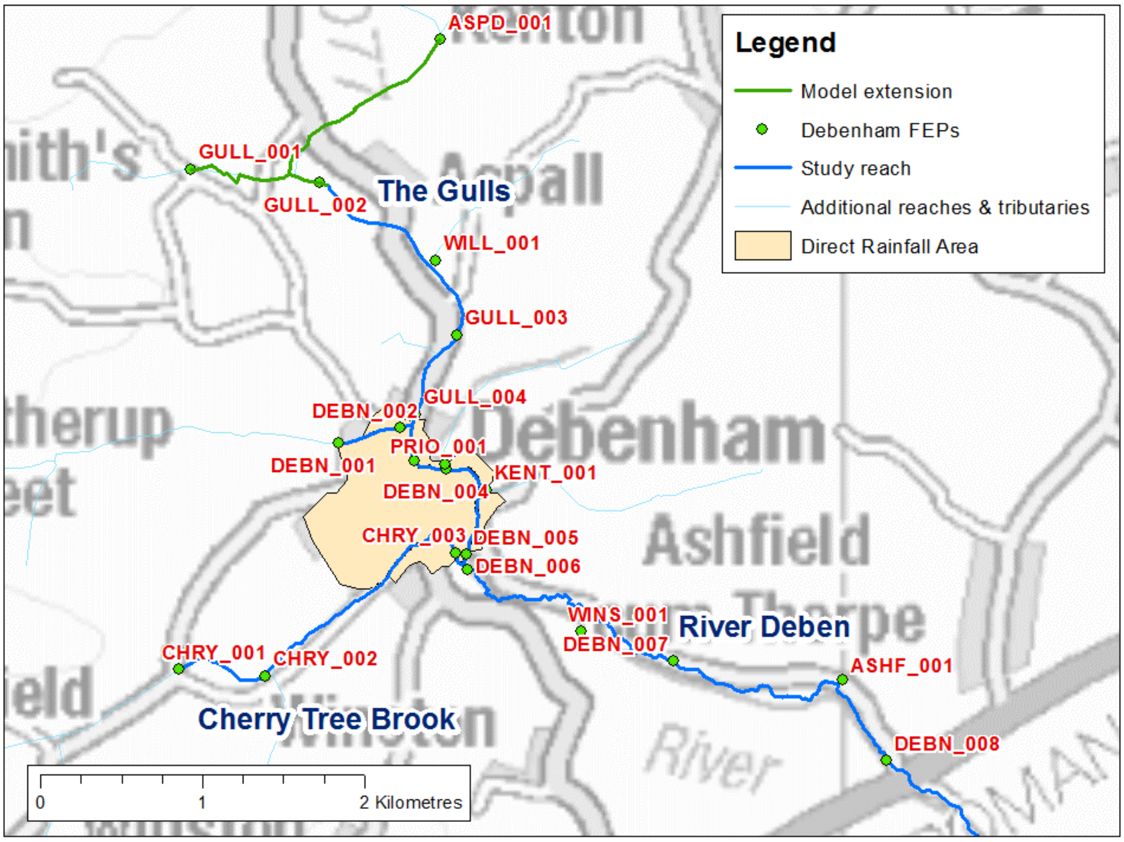


Figure 2-1: Debenham flood estimation points

Contains OS data © Crown copyright and database right 2014

2.2 Summary of subject sites (lateral inflows)

In addition to the main channel flow estimation points, there is a requirement for lateral inflows at Debenham. These account for any additional catchment area draining to each watercourse which is not already included as a direct inflow. It is important to include such “lateral” catchments to ensure all flow-generating areas are accounted for in the model.

For the current project two groups of lateral inflows are used, given the project’s requirement for a fluvial flood risk and a surface water flood risk model:

- Group 1: Lateral inflows which apply flows to the model outside of the direct-rainfall area.
- Group 2: Lateral inflows covering the area where direct rainfall will be employed.

The model constructed to represent the fluvial flood risk uses all lateral inflows - added directly to the ISIS model. The surface water model needs a direct rainfall input onto the TUFLOW domain; where this is the case, some lateral inflows (i.e. those in Group 2) are removed; otherwise these areas will be double counted.

Site code	Watercourse	Site	Easting	Northing	FEH AREA (km^2)	Revised AREA if altered
Group 1						
Deb_Lat	River Deben	Lateral inflow	n/a		n/a	0.21
Deb2_Lat	River Deben	Lateral inflow	n/a		n/a	0.07
Deb3_Lat	River Deben	Lateral inflow	n/a		n/a	3.90
Gul_Lat	The Gulls	Lateral inflow	n/a		n/a	0.69
Gul2_Lat	The Gulls	Lateral inflow	n/a		n/a	1.83
Aspd_Lat	Aspall Drain	Lateral inflow	n/a		n/a	2.27
Chy_Lat	Cherry Tree Brook	Lateral inflow	n/a		n/a	2.66
Group 2 (direct rainfall areas)						
Deb_DR	River Deben	Lateral inflow	n/a		n/a	0.08
Deb2_DR	River Deben	Lateral inflow	n/a		n/a	0.35
Chy_DR	Cherry Tree Brook	Lateral inflow	n/a		n/a	0.37

2.3 Important catchment descriptors at each subject site

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
River Deben and tributaries									
DEBN_001	1.000	0.28	0.324	2.52	13.8	589	42.82	0.002	0.132
DEBN_002	1.000	0.28	0.324	2.84	14.6	589	42.97	0.005	0.130
DEBN_003	1.000	0.28	0.318	3.85	14.3	592	41.33	0.007	0.174
DEBN_004	1.000	0.28	0.318	3.91	14.7	592	41.39	0.008	0.177
DEBN_005	1.000	0.28	0.319	4.37	15.1	592	41.50	0.009	0.177
DEBN_006	1.000	0.28	0.320	4.26	15.8	591	41.87	0.009	0.165
DEBN_007	1.000	0.28	0.322	5.59	17.2	591	42.19	0.009	0.158
DEBN_008	1.000	0.28	0.325	7.41	18.5	591	42.23	0.008	0.153
PRIO_001	1.000	0.28	0.320	1.70	18.5	596	41.87	0.005	0.218
KENT_001	1.000	0.28	0.320	1.57	17.8	595	41.82	0.005	0.163
WINS_001	1.000	0.28	0.343	1.12	29.8	591	46.95	0.028	0.037
ASHF_001	1.000	0.28	0.320	1.21	22.1	598	41.97	0.000	0.089
The Gulls and tributaries									
GULL_001	1.000	0.28	0.313	1.16	9.2	593	39.88	0.006	0.199
GULL_002	1.000	0.28	0.314	3.00	12.3	593	40.18	0.005	0.208
GULL_003	1.000	0.28	0.314	3.46	13.2	593	40.31	0.005	0.203
GULL_004	1.000	0.28	0.315	3.94	13.9	593	40.51	0.005	0.196
ASPD_001	1.000	0.28	0.312	1.65	8.7	591	39.71	0.000	0.249
WILL_001	1.000	0.28	0.313	1.53	10.8	594	40.06	0.000	0.233
Cherry Tree Brook									

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	SPRHOST	URBEXT	FPEXT
CHRY_001	1.000	0.28	0.318	2.45	13.7	587	41.36	0.001	0.178
CHRY_002	1.000	0.28	0.320	2.74	14.6	587	41.82	0.001	0.163
CHRY_003	1.000	0.28	0.323	3.70	17.4	588	42.76	0.008	0.137
Lateral Inflows (Group 1)									
Deb_Lat	1.000	0.28	0.324	0.41	44.2	589	42.82	0.000	0.132
Deb2_Lat	1.000	0.28	0.319	0.22	15.0	589	42.82	0.000	0.132
Deb3_Lat	1.000	0.28	0.323	2.20	17.5	591	42.82	0.000	0.132
Gul_Lat	1.000	0.28	0.314	1.36	44.4	593	40.51	0.000	0.196
Chy_Lat	1.000	0.28	0.320	1.72	33.6	587	42.76	0.000	0.137
Lateral Inflows (Group 2)									
Deb_DR	1.000	0.28	0.324	0.23	44.2	589	42.82	0.472	0.132
Deb2_DR	1.000	0.28	0.319	0.55	15.0	589	42.82	0.557	0.132
Chy_Deb	1.000	0.28	0.320	0.56	232.8	587	42.76	0.510	0.137

2.4 Checking catchment descriptors

Record how catchment boundary was checked and describe any changes (refer to maps if needed)	Catchment boundaries have been checked against OS mapping and LIDAR data. The FEH catchment boundaries match well with the topography of the area.
Record how other catchment descriptors (especially soils) were checked and describe any changes. Include before/after table if necessary.	Soils have been checked against UK soil maps. These were found to be acceptable. FARL values also appear reasonable based on OS mapping.
Source of URBEXT	URBEXT1990 – for ReFH URBEXT2000 – for Statistical method
Method for updating of URBEXT	CPRE formula from 2006 CEH report on URBEXT2000

3 Climate change

The Environment Agency published new advice on adapting to climate change in 2011[4]. The advice is specifically intended for projects or strategies seeking Government Flood Defence Grant in Aid. However, it notes that Risk Management Authorities in England may also find this information useful in developing plans and making flood risk management investment decisions even if there is no intention of applying for central government funding. It replaces Defra's Supplementary Note to Operating Authorities – Climate Change Impacts (2006), which has been used as the source of climate change adjustments in numerous flood studies in recent years.

The advice provides climate change factors for river flood flows, extreme rainfall, mean relative sea level rise and storm surges. These are based on the UKCP09 climate impacts study. As well as the change factors, upper and lower end estimates are provided to help represent the range of the future risks. These allow for uncertainties in climate modelling and in the amount of future greenhouse gas emissions.

3.1.1 River flow

The change factors for river flow vary geographically by river basin district. They are derived for a flood return period of 50 years but are expected to remain relatively constant with increasing return period. They are all relative to a 1961-90 baseline which is appropriate because flood estimates from the FEH are derived from data that corresponds roughly with this baseline period. The change factors for Anglian Region are given below.

	Potential change anticipated for the 2020s	Potential change anticipated for the 2050s	Potential change anticipated for the 2080s
Upper end estimate	30%	40%	70%
Change factor	10%	15%	25%
Lower end estimate	-15%	-10%	-5%

The range between the upper and lower end estimates indicates that there is a large amount of uncertainty over the impacts of climate change on flood flows in Anglian Region. This may be partly due to the conflicting effects of the impact of higher temperatures on the development of large soil moisture deficits over the summer period and the potential for more extreme rainfall. For the purposes of the present study we will apply the change factor for the 2050s, +15%, to represent the potential impact of climate change on flood flows.

3.1.2 Extreme rainfall

Although we are able to make qualitative statements as to whether extreme rainfall is likely to increase or decrease over the UK in the future, there is l considerable uncertainty regarding the magnitude of these changes locally.

It is recommended that where projection of future rainfall is required for events more frequent 5-year return period, information is taken from the UKCP09. Where rarer events are being considered, the figures below are recommended.

[4] Environment Agency (2011). Adapting to Climate Change: Advice for Flood and Coastal Erosion Risk Management Authorities

	Potential change anticipated for the 2020s	Potential change anticipated for the 2050s	Potential change anticipated for the 2080s
Upper end estimate	10%	20%	40%
Change factor	5%	10%	20%
Lower end estimate	0%	5%	10%

For the purposes of the present study we will apply the change factor for the 2050s, +10%, to represent the potential impact of climate change on flood flows.

4 Statistical method

This section of the document records the calculations undertaken using the FEH statistical methodology. The results from this analysis are compared against those produced by ReFH.

4.1 Search for donor sites for QMED

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.	There are two gauging stations downstream of the Debenham study area; the Brandeston ultrasonic gauge and Naunton Hall gauge. These gauges have been discussed previously in this document and it is believed neither offers a suitable improvement on the un-gauged FEH statistical estimates produced below. The earlier Debenham project found the flows recorded at Naunton Hall resulted in lower design peak flows than the un-gauged peaks, which in turn produced flood outlines which the Environment Agency believes are too small. There are no suitable donor sites on any adjacent catchments. The nearest HiFlows-UK gauge to Debenham is on the River Gipping at Stowmarket, some 10 miles to the west. The catchment draining to this gauge is 127km^2, significantly larger than the Debenham catchments and therefore not suitable for use as a donor site.

4.2 Donor sites chosen and QMED adjustment factors

NRFA no.	Reasons for choosing or rejecting	Method (AM or POT)	Adjustment for climatic variation?	QMED from flow data (A)	QMED from catchment descriptors (B)	Adjust-ment ratio (A/B)
35035	No information available regarding the calculation of out-of-bank flows.	n/a	n/a	n/a	n/a	n/a
35002	Not thought to be representative of subject catchment.	n/a	n/a	n/a	n/a	n/a

4.3 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 adj.	
River Deben									
DEBN_001	CD	1.20	n/a	n/a	n/a	n/a	n/a		1.20
DEBN_002	CD	1.25	n/a	n/a	n/a	n/a	n/a		1.25
DEBN_003	CD	3.36	n/a	n/a	n/a	n/a	n/a		3.36
DEBN_004	CD	3.57	n/a	n/a	n/a	n/a	n/a		3.57
DEBN_005	CD	3.82	n/a	n/a	n/a	n/a	n/a		3.82
DEBN_006	CD	5.14	n/a	n/a	n/a	n/a	n/a		5.14

DEBN_007	CD	5.46	n/a	n/a	n/a	n/a	n/a	5.46
DEBN_008	CD	5.84	n/a	n/a	n/a	n/a	n/a	5.84
PRIO_001	CD	0.35	n/a	n/a	n/a	n/a	n/a	0.35
KENT_001	CD	0.39	n/a	n/a	n/a	n/a	n/a	0.39
WINS_001	CD	0.21	n/a	n/a	n/a	n/a	n/a	0.21
ASHF_001	CD	0.33	n/a	n/a	n/a	n/a	n/a	0.33
The Gulls								
GULL_001	CD	0.64	n/a	n/a	n/a	n/a	n/a	0.64
GULL_002	CD	2.27	n/a	n/a	n/a	n/a	n/a	2.27
GULL_003	CD	2.35	n/a	n/a	n/a	n/a	n/a	2.35
GULL_004	CD	2.42	n/a	n/a	n/a	n/a	n/a	2.42
ASPD_001	CD	0.90	n/a	n/a	n/a	n/a	n/a	0.90
WILL_001	CD	0.39	n/a	n/a	n/a	n/a	n/a	0.39
Cherry Tree Brook								
CHRY_001	CD	1.25	n/a	n/a	n/a	n/a	n/a	1.25
CHRY_002	CD	1.47	n/a	n/a	n/a	n/a	n/a	1.47
CHRY_003	CD	1.81	n/a	n/a	n/a	n/a	n/a	1.81
POPL_001	CD	0.14	n/a	n/a	n/a	n/a	n/a	0.14
Lateral inflows (Group 1)								
Deb_Lat	CD	0.07	n/a	n/a	n/a	n/a	n/a	0.07
Deb2_Lat	CD	0.03	n/a	n/a	n/a	n/a	n/a	0.03
Deb3_Lat	CD	0.81	n/a	n/a	n/a	n/a	n/a	0.81
Gul_Lat	CD	0.41	n/a	n/a	n/a	n/a	n/a	0.41
Chy_Lat	CD	0.57	n/a	n/a	n/a	n/a	n/a	0.57
Lateral inflows (Group 2)								
Deb_DR	CD	0.04	n/a	n/a	n/a	n/a	n/a	0.04
Deb2_DR	CD	0.16	n/a	n/a	n/a	n/a	n/a	0.16
Chy_DR	CD	0.16	n/a	n/a	n/a	n/a	n/a	0.16
Are the values of QMED consistent, for example at successive points along the watercourse and at confluences?						Yes		

Important note on urban adjustment

The method used to adjust QMED for urbanisation is that published in Kjeldsen (2010)[5] in which PRUAF is calculated from BFIHOST. The result will differ from that of WINFAP-FEH v3.0.003 which does not correctly implement the urban adjustment of Kjeldsen (2010). Significant differences will occur only on urban catchments that are highly permeable.

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 below.

The data transfer procedure is the revised one 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 centroids of the subject and the donor catchments. 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.

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

4.4 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	Subject site treated as gauged?	**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
River Deben				
DEBN_a	DEBN_001	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years).	L-CV: 0.287 L-SKEW: 0.203
DEBN_b	DEBN_004	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years).	L-CV: 0.289 L-SKEW: 0.119
DEBN_c	DEBN_007	No	No changes.	L-CV: 0.305 L-SKEW: 0.105
The Gulls				
GULL_a	GULL_004	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years). Keer at High Keer Weir (73015) removed due to gap in record from 1982 – 1990. **Added:** Crimple at Burn Bridge (27051) added to increase record length.	L-CV: 0.270 L-SKEW: 0.139
Cherry Tree Brook				
CHRY_a	CHRY_003	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years).	L-CV: 0.284 L-SKEW: 0.180
Tributaries and laterals				
TRIBS_a	WILL_001	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years). **Maintained:** Despite discordance due to one extreme event in 1977, the Severn at Hafren Flume (54091).	L-CV: 0.256 L-SKEW: 0.203
TRIBS_b	ASHF_001	No	**Removed:** Flore Experimental at Flore (32029) removed due to short record length (five years).	L-CV: 0.227 L-SKEW: 0.229
Notes Pooling groups were derived using the revised procedures from Science Report SC050050 (2008).				

4.5 Derivation of flood growth curves at subject sites

Site code	Method (SS, P, ESS, J)	If P, ESS or J, name of pooling group	Distribution used and reason for choice	Note any urban or permeable adjustment	Parameters of distribution (location, scale and shape)	Growth factor for 100-year RP
River Deben and tributaries						
DEBN_001 DEBN_002	P	DEBN_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.295 Shape: -0.204	3.25
DEBN_003 DEBN_004 DEBN_005	P	DEBN_b	GL (for consistency with above)	Kjeldsen (v3)	Location: 1.000 Scale: 0.298 Shape: -0.120	2.83
DEBN_006 DEBN_007 DEBN_008	P	DEBN_c	GL (for consistency with above)	Kjeldsen (v3)	Location: 1.000 Scale: 0.315 Shape: -0.106	2.86
PRIO_001 KENT_001	P	TRIBS_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.261 Shape: -0.203	2.98
WINS_001 ASHF_001	P	TRIBS_b	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.227 Shape: -0.229	2.85
The Gulls and tributaries						
GULL_001 GULL_002 GULL_003 GULL_004	P	GULL_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.277 Shape: -0.139	2.78
ASPD_001 WILL_001	P	TRIBS_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.261 Shape: -0.203	2.98
Cherry Tree Brook						
CHRY_001 CHRY_002 CHRY_003	P	CHRY_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.291 Shape: -0.181	3.09
POPL_001	P	TRIBS_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.261 Shape: -0.203	2.98
All Laterals						
All	P	TRIBS_a	GL (recommended by WIN-FAP)	Kjeldsen (v3)	Location: 1.000 Scale: 0.261 Shape: -0.203	2.98

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 are all carried out using the v3 method: Kjeldsen (2010).

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

4.6 Flood estimates from the statistical method

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	20CC	75	100	100CC	1000	1000CC
River Deben and tributaries										
DEBN_001	1.2	1.8	2.2	2.6	3.0	3.6	3.9	4.5	6.6	7.5
DEBN_002	1.3	1.8	2.3	2.7	3.2	3.8	4.1	4.7	6.8	7.9
DEBN_003	3.4	4.9	5.9	6.9	7.9	9.0	9.5	10.9	14.1	16.3
DEBN_004	3.6	5.2	6.2	7.3	8.4	9.6	10.1	11.6	15.0	17.3
DEBN_005	3.8	5.5	6.7	7.8	9.0	10.2	10.8	12.4	16.1	18.5
DEBN_006	5.1	7.6	9.2	10.7	12.4	14.0	14.7	16.9	21.6	24.9
DEBN_007	5.5	8.0	9.7	11.4	13.1	14.8	15.6	18.0	23.0	26.4
DEBN_008	5.8	8.6	10.4	12.2	14.0	15.9	16.7	19.3	24.6	28.3
PRIO_001	0.3	0.5	0.6	0.7	0.8	1.0	1.0	1.2	1.7	2.0
KENT_001	0.4	0.5	0.7	0.8	0.9	1.1	1.1	1.3	1.9	2.2
WINS_001	0.2	0.3	0.3	0.4	0.5	0.6	0.6	0.7	1.0	1.2
ASHF_001	0.3	0.4	0.5	0.6	0.7	0.9	0.9	1.1	1.6	1.8
The Gulls and tributaries										
GULL_001	0.6	0.9	1.1	1.3	1.5	1.7	1.8	2.0	2.7	3.1
GULL_002	2.3	3.2	3.9	4.6	5.2	6.0	6.3	7.3	9.6	11.0
GULL_003	2.3	3.3	4.0	4.7	5.4	6.2	6.5	7.5	9.9	11.4
GULL_004	2.4	3.4	4.1	4.9	5.6	6.4	6.7	7.7	10.2	11.7
ASPD_001	0.9	1.3	1.5	1.8	2.1	2.5	2.7	3.1	4.4	5.1
WILL_001	0.4	0.6	0.7	0.8	0.9	1.1	1.2	1.3	1.9	2.2
Cherry Tree Brook and tributaries										
CHRY_001	1.3	1.8	2.2	2.7	3.1	3.6	3.9	4.4	6.3	7.2
CHRY_002	1.5	2.1	2.6	3.1	3.6	4.2	4.5	5.2	7.3	8.4
CHRY_003	1.8	2.6	3.2	3.9	4.4	5.2	5.6	6.4	9.0	10.4
POPL_001	0.1	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.7	0.8
Lateral inflows (Group 1)										
Deb_Lat	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.4
Deb2_Lat	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Deb3_Lat	0.8	1.1	1.4	1.7	1.9	2.3	2.4	2.8	4.0	4.6
Gul_Lat	0.6	0.8	1.0	1.2	1.4	1.6	1.7	2.0	2.8	3.3
Chy_Lat	0.4	0.6	0.7	0.8	1.0	1.1	1.2	1.4	2.0	2.3
Lateral inflows (Group 2)										
Deb_DR	0.0	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
Deb2_DR	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.6	0.8	0.9
Chy_DR	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.6	0.8	0.9

5 Revitalised flood hydrograph (ReFH) method

The ReFH method has also been utilised and the resulting flows compared to those produced using the FEH statistical method, and then applied to the hydraulic model. Information from the flood history of the catchment is used to infer which set of results is most applicable. Given the lack of suitable flow data there is significant uncertainty with FEH statistical peak flow estimates, although the availability of level and rainfall data allows lag analysis to be undertaken, improving the time-to-peak estimate for the ReFH model.

The ReFH model uses a triangular unit hydrograph for routing rainfall to the catchment outfall (this "triangle" is kinked, which offers a more flexible shape than that used in the superseded FSR/FEH method). This is obviously a simplified method, although it is important the time-to-peak (Tp) is estimated as accurately as possible; an overestimate will result in a lower peak with a longer hydrograph and vice versa for an underestimate. Tp is often estimated using catchment descriptors alone, but in this project we can estimate this based on data.

The technique used to produce Tp values for each watercourse is provided in section 1.7. The resulting estimates were then used in ISIS ReFH units to produce a calibrated inflow hydrograph.

5.1 Parameters for ReFH model

The table below only includes flow estimation points which make up the inflows to the hydraulic model.

Three of these (one on each main watercourse) are calculated using an adjusted time-to-peak value, taken from the level gauge location on each watercourse. The ratio between the catchment descriptors Tp and the calculated Tp at each gauge location was calculated. The resulting ratio was then applied to the catchment descriptor Tp for each relevant inflow below.

Site code	Method: BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details) LAG: Refined with lag analysis	Catchment descriptor Tp (hours)	Adjustment factor at gauge (Data Tp / CD Tp)	Final Tp
River Deben and tributaries				
DEBN_001	LAG	5.19	0.59	3.09
PRIO_001	CD	3.79	n/a	3.79
KENT_001	CD	3.66	n/a	3.66
WINS_001	CD	2.36	n/a	2.36
ASHF_001	CD	3.20	n/a	3.20
The Gulls and tributaries				
GULL_001	LAG	3.67	0.55	2.03
ASPD_001		4.61	0.55	2.55
WILL_001	CD	4.15	n/a	4.15
Cherry Tree Brook and tributaries				
CHRY_001	LAG	5.12	0.36	1.86
POPL_001	CD	2.43	n/a	2.43
Lateral Inflows (Group 1)				
Deb_Lat	CD	1.25	n/a	1.25
Deb2_Lat	CD	1.17	n/a	1.17
Deb3_Lat	CD	4.50	n/a	4.50
Gul_Lat	CD	3.07	n/a	3.07
Aspd_Lat	CD	3.98	n/a	3.98

Site code	Method: BR: Baseflow recession fitting CD: Catchment descriptors DT: Data transfer (give details) LAG: Refined with lag analysis	Catchment descriptor Tp (hours)	Adjustment factor at gauge (Data Tp / CD Tp)	Final Tp
Gul2_Lat	CD	1.58	n/a	1.58
Chy_Lat	CD	3.32	n/a	3.32
Lateral Inflows (Group 2)				
Deb_DR	CD	0.32	n/a	0.32
Deb2_DR	CD	0.61	n/a	0.61
Chy_DR	CD	0.32	n/a	0.32

Site code	C_{max} (mm) Maximum storage capacity	BL (hours) Baseflow lag	BR Baseflow recharge
NB: All values calculated directly from catchment descriptors			
River Deben and tributaries			
DEBN_001	278	35.5	0.7
PRIO_001	274	32.6	0.7
KENT_001	274	32.2	0.7
WINS_001	293	28.5	0.7
ASHF_001	279	31.7	0.7
The Gulls and tributaries			
GULL_001	269	29.9	0.7
ASPD_001	268	32.1	0.7
WILL_001	269	31.7	0.7
Cherry Tree Brook and tributaries			
CHRY_001	273	35.0	0.7
POPL_001	282	28.6	0.7
Lateral Inflows (Group 1)			
Deb_Lat	278	24.2	0.7
Deb2_Lat	274	21.2	0.7
Deb3_Lat	277	34.7	0.7
Gul_Lat	269	29.8	0.7
Gul2_Lat	281	25.9	0.7
Aspd_Lat	266	31.9	0.7
Chy_Lat	274	33.0	0.7
Lateral Inflows (Group 2)			
Deb_DR	278	8.5	0.7
Deb2_DR	274	8.7	0.7
Chy_DR	274	9.5	0.7

5.2 Design events for ReFH method

In order to ensure the critical storm is modelled at Debenham, a range of storm durations were tested in an early version of the hydraulic model. The water level results were extracted at a variety of critical locations and compared. As can be seen from the results table below, any of the design storms tested could be used, with millimetre differences in many of the results (well below the model's accuracy). However, for this study we have decided to use the 7.5 hour duration as this produced slightly higher water levels at the most locations.

Peak water levels at individual model nodes (mAOD)						
Duration (hrs)	DEBN_5745u	DEBN_5404u	DEBN_5297fu	DEBN_4651	CHRY_0452	CHRY_0340u
6.5	36.037	35.633	35.222	34.048	34.131	34.045
7.5	36.041	35.637	35.225	34.053	34.136	34.054
8.5	36.039	35.635	35.224	34.053	34.137	34.056
9.5	36.036	35.632	35.222	34.053	34.137	34.056
10.5	36.032	35.627	35.219	34.051	34.134	34.053
11.5	36.027	35.622	35.215	34.047	34.130	34.049

Site code	Urban or rural	Season of design event (summer or winter)	Storm duration (hours)	Storm area for ARF (if not catchment area)
All	Rural	Winter	7.5 hours (derived from critical duration testing)	33.85
Are the storm durations likely to be changed in the next stage of the study?			Durations have already been optimised.	

5.3 Flood estimates from the ReFH method

Please note that only those inflows applied directly to ISIS have been included in the table below, as the hydraulic model is used to route flow through the remainder of the study extent.

The flows listed below can be considered as the design event for Debenham, as the critical storm duration was tested at various points throughout the settlement. However, it must be considered that this is not the design storm for the tributary inflows – these catchments are smaller and therefore the design event would likely be generated by a shorter storm duration.

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	20CC	75	100	100CC	1000	1000CC
River Deben and tributaries										
DEBN_001	2.1	2.8	3.4	4.0	4.6	5.4	5.8	6.6	10.7	12.3
PRIO_001	0.4	0.5	0.7	0.8	0.9	1.0	1.1	1.1	2.0	2.3
KENT_001	0.5	0.6	0.8	0.9	1.0	1.2	1.3	1.5	2.4	2.8
WINS_001	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.6	1.9
ASHF_001	0.4	0.6	0.7	0.8	0.9	1.1	1.2	1.4	2.3	2.6
The Gulls and tributaries										
GULL_001	1.2	1.6	1.9	2.2	2.6	3.0	3.2	3.7	6.0	6.9
ASDP_001	1.6	2.1	2.5	3.0	3.4	4.0	4.3	4.9	8.0	9.2
WILL_001	0.4	0.6	0.7	0.8	1.0	1.1	1.2	1.4	2.2	2.6
Cherry Tree Brook and tributaries										
CHRY_001	2.8	3.9	4.7	5.5	6.4	7.5	8.1	9.3	15.0	17.3
POPL_001	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	1.0	1.2
Lateral inflows (Group 1)										
Deb_Lat	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.6	0.7
Deb2_Lat	0.04	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
Deb3_Lat	1.0	1.4	1.7	1.9	2.2	2.6	2.8	3.2	5.2	6.0
Gul_Lat	0.2	0.3	0.4	0.4	0.5	0.6	0.6	0.7	1.1	1.3
Gul2_Lat	0.8	1.1	1.3	1.5	1.8	2.1	2.2	2.6	4.1	4.7
Aspd_Lat	0.6	0.8	1.0	1.2	1.4	1.6	1.7	2.0	3.1	3.6
Chy_Lat	0.9	1.2	1.4	1.6	1.9	2.2	2.4	2.7	4.4	5.1
Lateral inflows (Group 2)										
Deb_DR	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.4
Deb2_DR	0.2	0.3	0.4	0.5	0.5	0.6	0.7	0.8	1.2	1.4
Chy_DR	0.3	0.4	0.4	0.5	0.6	0.7	0.7	0.9	1.4	1.6

5.4 Direct rainfall

In order to produce surface water flood maps, direct rainfall is input directly onto the TUFLOW model domain. Design rainfall hyetographs, shown below, have been obtained from ISIS ReFH units with the storm duration set to a variety of storm durations (0.75, 1.75, 2.75 and 3.75 hours). Each of these summer rainfall scenarios was run through the hydraulic model.

The seasonal correction factor for rainfall was set to 1.0 to ensure no losses (although use of the summer rainfall produces a correction factor of close to 1.0 regardless) and the areal reduction factor was also set to 1.0.

The shortest duration storm (0.75 hours) resulted in the greatest depths and velocities, although it should be noted that confidence associated with design rainfall estimation at durations less than one hour is very low. Therefore for the purposes of this project we propose modelling direct rainfall using a one hour duration storm.

For the direct rainfall model scenario, the fluvial inflows are derived using the same storm event (i.e. one hour duration). However, the seasonal correction factor and areal reduction factor are not set to 1.0; the ReFH unit within ISIS calculates this automatically. This ensures the same rainfall profile is not applied to both the watercourses and the direct rainfall area – an appropriate approach if it is considered short and intense convective summer storms are likely to be highly localised.

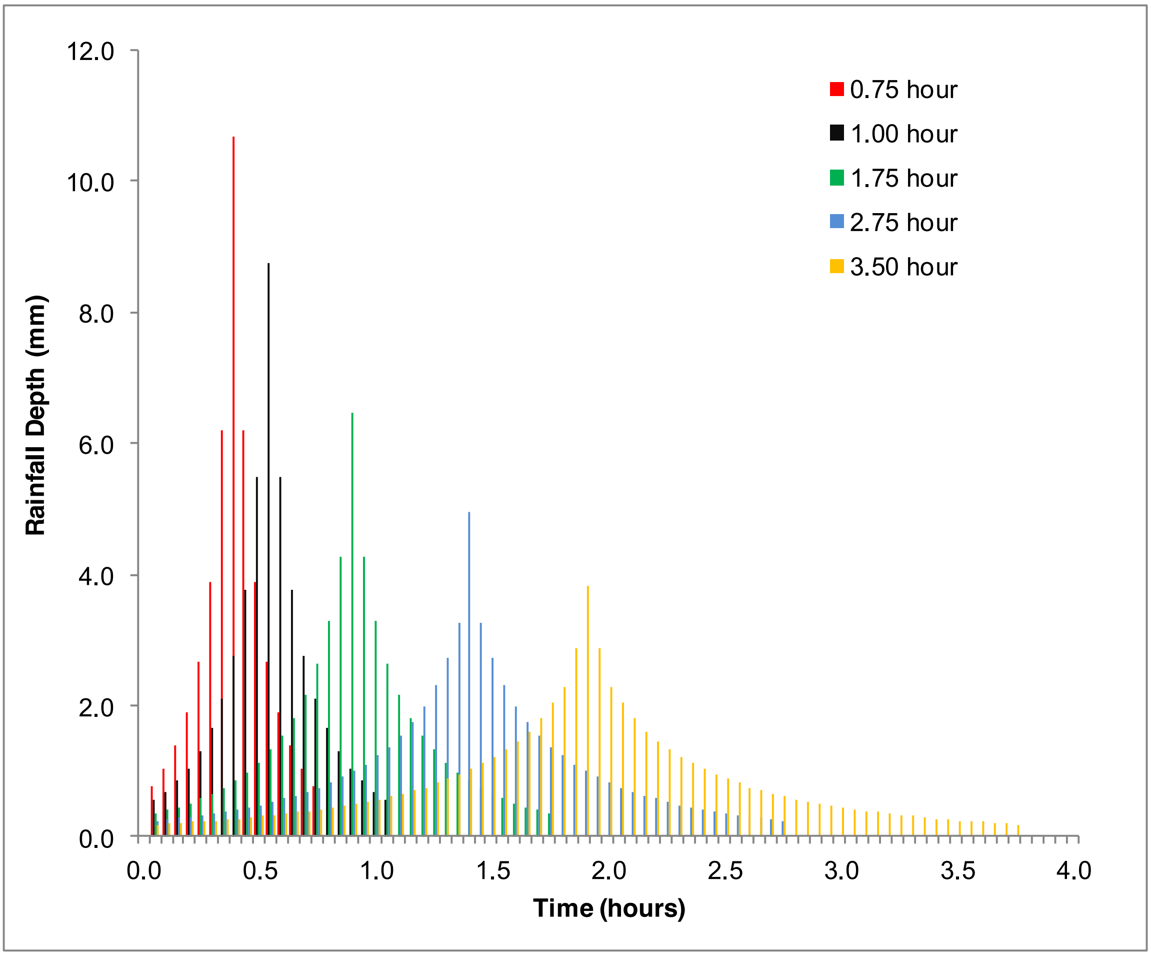


Figure 5-1: Debenham design rainfall depths for various storm durations (100-year return period)

6 Discussion and summary of results

6.1 Comparison of results from different methods

This table compares peak flows from ReFH with those from the FEH Statistical method at example sites for two key return periods. Only the sites used as direct inflows to the hydraulic model are used for this comparison.

Site code	Peak flow comparison					
	Return period 2 years			Return period 100 years		
	FEH Stats	ReFH	ReFH/Stats ratio	FEH Stats	ReFH	ReFH/Stats ratio
Main channels						
DEBN_001	1.2	2.1	1.8	3.9	5.8	1.5
GULL_001	1.9	3.5	1.8	5.3	9.5	1.8
CHRY_001	1.3	2.8	2.2	3.9	8.1	2.1
Tributaries						
PRIO_001	0.3	0.4	1.3	1.0	1.1	1.1
KENT_001	0.4	0.5	1.3	1.1	1.3	1.2
WINS_001	0.2	0.3	1.5	0.6	0.9	1.3
ASHF_001	0.3	0.4	1.3	0.9	1.2	1.3
WILL_001	0.4	0.4	1.0	1.2	1.2	1.0
POPL_001	0.1	0.2	2.0	0.4	0.5	1.3

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.	As shown in the table above, ReFH flows are consistently greater than those calculated using the FEH statistical method. Tributaries exhibit less change given the lack of time-to-peak calibration on these catchments. The earlier modelling results at Debenham used an even lower FEH statistical flow, resulting in flood outlines that appeared small when compared to historical evidence. Having input ReFH flow hydrographs into the hydraulic model, it is apparent that these produce significantly more extensive flood outlines than those from statistical. The results from ReFH flows also compare favourably when it is considered the Market Square has flooded on at least five occasions in 70 years (i.e. a return period of between 10 and 20 years). The new modelling results show flooding at this location at the 10 and 20-year return period. The ReFH also lends itself to using the available level data more so than the statistical method. As discussed, rainfall and level data allow Tp values to be refined and added to the hydraulic model.

6.3 Assumptions, limitations and uncertainty

List the main assumptions made (specific to this study)	It is assumed that the ReFH method produces suitable peak flows for the catchment in question. Available information from flood history suggests this is the case, although with no flow record it is difficult to conclude with certainty. It has also been assumed that the same design storm will produce the same event on each of the tributaries around Debenham. This assumption is justified by the similar nature of the catchments, although in reality there will be some difference in catchment response to a given rainfall event. The rainfall record at Needham Market has been used to estimate time to peak on the Debenham watercourse in conjunction with available level data; this is deemed an appropriate rain gauge for use due to its proximity. However, it is likely that in reality the storm profile at Debenham would be slightly different.
Discuss any particular limitations, e.g. applying methods outside the range of catchment types or return periods for which they were developed	ReFH is appropriate for this study given the availability of level data and rainfall records. The subject catchments are neither heavily urbanised nor very permeable.
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).	No confidence limits have been published for the ReFH model, although confidence in Debenham model results is increased by the availability of time-to-peak data and historical flood photographs.
Comment on the suitability of the results for future studies, e.g. at nearby locations or for different purposes.	Results recorded in this document are robust and may be used on future studies, although it should be considered that a project is in progress to re-calibrate ReFH against the revised FEH rainfall frequency statistics. It is recommended that the flow estimates provided here are reviewed and amended accordingly following a major flood event, or if any reliable flow data become available in the future.
Give any other comments on the study, for example suggestions for additional work.	n/a

6.4 Checks

Are the results consistent, for example at confluences?	Yes.
What do the results imply regarding the return periods of floods during the period of record?	The review of flood history at Debenham suggested the Market Square has flooded at least five times in 70 years of photographic record. Flows produced via ReFH and input into the hydraulic model produce flood outlines which reflect this observation.
What is the 100-year growth factor? Is this realistic? (The guidance suggests a typical range of 2.1 to 4.0)	Yes, the 100-year growth factors for the discrete inflows (i.e. non-laterals) range from 2.50 at POPL_001 to 3.00 at WILL_001.

If 1000-year flows have been derived, what is the range of ratios for 1000-year flow over 100-year flow?	1,000-year/100-year ratios range from 1.78 at WINS_001 to 2.00 at POPL_001.
What range of specific runoffs (l/s/ha) do the results equate to? Are there any inconsistencies?	Specific runoff estimates range from 7.55 l/a/ha to 11.11 l/s/ha at WILL_001 and WINS_001 respectively.
How do the results compare with those of other studies? Explain any differences and conclude which results should be preferred.	Flow estimates published here are greater than those produced for an earlier study at Debenham.
Are the results compatible with the longer-term flood history?	Yes – see main report and section 1.2.2 for further information.
Describe any other checks on the results	Draft model outlines were presented to the Environment Agency at a meeting in September 2013 and later at a community engagement meeting. It was considered that draft outlines were too small compared to the observed flood history and therefore hydrological analysis was re-visited, along with re-parameterisation of the hydraulic model.

6.5 Final results

Site code	Flood peak (m^3/s) for the following return periods (in years)									
	2	5	10	20	20CC	75	100	100CC	1000	1000CC
River Deben and tributaries										
DEBN_001	2.1	2.8	3.4	4.0	4.6	5.4	5.8	6.6	10.7	12.3
PRIO_001	0.4	0.5	0.7	0.8	0.9	1.0	1.1	1.1	2.0	2.3
KENT_001	0.5	0.6	0.8	0.9	1.0	1.2	1.3	1.5	2.4	2.8
WINS_001	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.6	1.9
ASHF_001	0.4	0.6	0.7	0.8	0.9	1.1	1.2	1.4	2.3	2.6
The Gulls and tributaries										
GULL_001	1.2	1.6	1.9	2.2	2.6	3.0	3.2	3.7	6.0	6.9
ASDP_001	1.6	2.1	2.5	3.0	3.4	4.0	4.3	4.9	8.0	9.2
WILL_001	0.4	0.6	0.7	0.8	1.0	1.1	1.2	1.4	2.2	2.6
Cherry Tree Brook and tributaries										
CHRY_001	2.8	3.9	4.7	5.5	6.4	7.5	8.1	9.3	15.0	17.3
POPL_001	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	1.0	1.2
Lateral inflows (Group 1)										
Deb_Lat	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.6	0.7
Deb2_Lat	0.04	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
Deb3_Lat	1.0	1.4	1.7	1.9	2.2	2.6	2.8	3.2	5.2	6.0
Gul_Lat	0.2	0.3	0.4	0.4	0.5	0.6	0.6	0.7	1.1	1.3
Gul2_Lat	0.8	1.1	1.3	1.5	1.8	2.1	2.2	2.6	4.1	4.7
Aspd_Lat	0.6	0.8	1.0	1.2	1.4	1.6	1.7	2.0	3.1	3.6
Chy_Lat	0.9	1.2	1.4	1.6	1.9	2.2	2.4	2.7	4.4	5.1
Lateral inflows (Group 2)										
Deb_DR	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.4
Deb2_DR	0.2	0.3	0.4	0.5	0.5	0.6	0.7	0.8	1.2	1.4
Chy_DR	0.3	0.4	0.4	0.5	0.6	0.7	0.7	0.9	1.4	1.6

7 Annex – supporting information

7.1 Pooling group composition

DEBN_a

Station ID	Watercourse and station name	Distance SDM
27073	Brompton Beck @ Snainton Ings	1.296
27051	Crimple @ Burn Bridge	1.511
44009	Wey @ Broadwey	1.528
26802	Gypsey Race @ Kirby Grindalythe	1.719
25019	Leven @ Easby	1.810
44006	Sydling Water @ Sydling st Nicholas	1.892
36009	Brett @ Cockfield	1.905
45816	Haddeo @ Upton	1.907
20002	West Peffer Burn @ Luffness	1.935
33045	Wittle @ Quidenham	2.074
28033	Dove @ Hollinsclough	2.103
203046	Rathmore Burn @ Rathmore Bridge	2.130
36010	Bumpstead Brook @ Broad Green	2.212
27010	Hodge Beck @ Bransdale Weir	2.219
29009	Ancholme @ Toft Newton	2.264
44008	Sth Winterbourne @ W'bourne Steepleton	2.260

DEBN_b

Station ID	Watercourse and station name	Distance SDM
33045	Wittle @ Quidenham	0.369
29009	Ancholme @ Toft Newton	0.540
20002	West Peffer Burn @ Luffness	0.613
36009	Brett @ Cockfield	0.751
33054	Babingley @ Castle Rising	1.373
36010	Bumpstead Brook @ Broad Green	1.515
33032	Heacham @ Heacham	1.518
41020	Bevern Stream @ Clappers Bridge	1.529
27073	Brompton Beck @ Snainton Ings	1.592
203046	Rathmore Burn @ Rathmore Bridge	1.592
36003	Box @ Polstead	1.626
26003	Foston Beck @ Foston Mill	1.643
34005	Tud @ Costessey Park	1.702

DEBN_c

Station ID	Watercourse and station name	Distance SDM
33045	Wittle @ Quidenham	0.482
20002	West Peffer Burn @ Luffness	0.571
29009	Ancholme @ Toft Newton	0.626
36009	Brett @ Cockfield	0.703
33054	Babingley @ Castle Rising	0.747
33032	Heacham @ Heacham	0.821
36003	Box @ Polstead	0.944
26003	Foston Beck @ Foston Mill	0.952
34005	Tud @ Costessey Park	0.983
36007	Belchamp Brook @ Bardfield Bridge	1.099

36004	Chad Brook @ Long Melford	1.127
37003	Ter @ Crabbs Bridge	1.160

GULL_a

Station ID	Watercourse and station name	Distance SDM
27073	Brompton Beck @ Snainton Ings	0.873
33045	Wittle @ Quidenham	1.083
29009	Ancholme @ Toft Newton	1.130
20002	West Peffer Burn @ Luffness	1.280
36009	Brett @ Cockfield	1.368
203046	Rathmore Burn @ Rathmore Bridge	1.960
26802	Gypsey Race @ Kirby Grindalythe	1.998
36010	Bumpstead Brook @ Broad Green	2.046
33054	Babingley @ Castle Rising	2.101
41020	Bevern Stream @ Clappers Bridge	2.125
25019	Leven @ Easby	2.156
72014	Conder @ Galgate	2.190
203049	Clady @ Clady Bridge	2.261
33032	Heacham @ Heacham	2.266
27051	Crimple @ Burn Bridge	2.329

CHRY_a

Station ID	Watercourse and station name	Distance SDM
27073	Brompton Beck @ Snainton Ings	1.227
36009	Brett @ Cockfield	1.320
20002	West Peffer Burn @ Luffness	1.336
26802	Gypsey Race @ Kirby Grindalythe	1.430
33045	Wittle @ Quidenham	1.472
25019	Leven @ Easby	1.574
27051	Crimple @ Burn Bridge	1.591
44009	Wey @ Broadwey	1.620
29009	Ancholme @ Toft Newton	1.679
203046	Rathmore Burn @ Rathmore Bridge	1.720
36010	Bumpstead Brook @ Broad Green	1.737
44006	Sydling Water @ Sydling St Nicholas	1.779
27010	Hodge Beck @ Bransdale Weir	1.942
44008	Sth Winterbourne @ W'bourne Steepleton	1.967
41020	Bevern Stream @ Clappers Bridge	2.033

TRIBS_a

Station ID	Watercourse and station name	Distance SDM
76011	Coal Burn @ Coalburn	2.130
27073	Brompton Beck @ Snainton Ings	2.299
45817	Rhb Trib to Haddeo @ Upton (trib)	2.767
44009	Wey @ Broadwey	3.403
27051	Crimple @ Burn Bridge	3.423
45816	Haddeo @ Upton	3.479
28033	Dove @ Hollinsclough	3.718
54091	Severn @ Hafren Flume	3.853
44006	Sydling Water @ Sydling St Nicholas	3.887
54092	Severn @ Hore Flume	3.912

26802	Gypsey Race @ Kirby Grindalythe	3.956
25019	Leven @ Easby	3.997
33045	Wittle @ Quidenham	4.038
20002	West Peffer Burn @ Luffness	4.094
29009	Ancholme @ Toft Newton	4.094
36009	Brett @ Cockfield	4.109
25003	Trout Beck @ Moor House	4.142

TRIBS_b

Station ID	Watercourse and station name	Distance SDM
76011	Coal Burn @ Coalburn	1.215
45817	Rhb Trib to Haddeo @ Upton (trib)	1.619
44009	Wey @ Broadwey	2.780
27051	Crimple @ Burn Bridge	2.796
45816	Haddeo @ Upton	2.824
27073	Brompton Beck @ Snainton Ings	3.071
28033	Dove @ Hollinsclough	3.111
54091	Severn @ Hafren Flume	3.186
54092	Severn @ Hore Flume	3.188
44006	Sydling Water @ Sydling St Nicholas	3.410
25019	Leven @ Easby	3.578
26802	Gypsey Race @ Kirby Grindalythe	3.595
91802	Allt Leachdach @ Intake	3.709
25011	Langdon Beck @ Langdon	3.733
25003	Trout Beck @ Moor House	3.808
54022	Severn @ Plynlimon Flume	3.911
206006	Annalong @ Recorder 1895	3.937

7.2 Events used in lag analysis

Those highlighted in grey represent the upper quartile of rainfall intensity used in lag time vs rainfall intensity analysis.

The Gulls		River Deben		Cherry Tree Brook	
Event Date	Lag Time (hrs)	Event Date	Lag Time (hrs)	Event Date	Lag Time (hrs)
02/01/2004	12.5	19/12/1997	7.50	28/08/1997	0.50
31/01/2004	5.25	18/01/1998	7.50	02/12/1997	9.50
02/02/2004	6.50	15/04/1998	10.50	19/01/1998	7.75
29/04/2004	4.75	28/10/1998	3.25	06/03/1998	6.50
01/05/2004	4.00	10/12/1998	4.25	07/04/1998	5.00
03/05/2004	5.00	23/12/1998	4.50	24/10/1998	7.75
17/07/2004	2.50	12/12/1999	5.00	27/10/1998	5.50
23/08/2004	7.50	25/02/2000	6.25	31/10/1998	10.00
15/04/2005	7.00	25/09/2000	4.25	23/12/1998	7.00
24/08/2005	1.25	10/10/2000	4.50	12/06/1999	7.50
31/12/2005	10.25	21/10/2000	4.75	03/07/1999	4.50
15/02/2006	5.75	07/03/2001	4.00	16/08/1999	4.25
23/02/2006	7.50	07/04/2001	3.50	14/09/1999	4.00
25/09/2006	8.00			24/10/1999	6.75
24/10/2006	3.25			11/12/1999	5.75
18/11/2006	6.00			23/12/1999	7.00
26/11/2006	6.00			03/01/2000	5.75
07/12/2006	6.00			24/02/2000	6.75
06/01/2007	4.75			26/04/2000	4.75
18/01/2007	1.75			27/05/2000	6.75
14/02/2007	4.50			02/07/2000	0.50
27/05/2007	23.50			09/07/2000	4.00
25/06/2007	2.50			03/08/2000	2.00
10/10/2007	8.00			26/09/2000	5.00
17/10/2007	6.00			10/10/2000	5.50
06/12/2007	8.00			21/10/2000	5.50
25/12/2007	8.00			02/11/2000	4.50
11/01/2008	6.50			06/11/2000	6.25
16/03/2008	7.00			28/11/2000	4.75
27/05/2008	3.25			08/12/2000	4.50
03/06/2008	5.75			24/12/2000	5.75
12/08/2008	2.50			27/01/2001	4.50
05/10/2008	7.50			12/02/2001	6.00
01/11/2008	5.00			08/03/2001	5.00
10/11/2008	3.00			17/03/2001	5.25
13/12/2008	5.25				
19/01/2009	3.00				
10/02/2009	4.50				
02/12/2009	6.50				
29/12/2009	9.50				
16/02/2010	7.00				
28/02/2010	6.00				
26/08/2010	3.25				
26/09/2010	7.25				
09/11/2010	4.50				

11/01/2011	6.00				
03/05/2012	5.50				
04/11/2012	5.25				
14/12/2012	4.25				
26/01/2013	3.50				
14/02/2013	4.25				
09/03/2013	9.00				

B Appendix B - Hydraulic model check file

Introduction

This report provides a detailed record of information on the hydraulic model constructed for the Debenham flood mapping modelling project in early 2014, in addition to the model extension project undertaken in late 2014. Information on the results of QA and validation checks are also included here. It complements the information in the main report which gives more general information on the model.

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1. Contents

2. Model Overview and data summary

2.1 Summary of model requirements

Table 2-1: Summary of model requirements	
Give an overview which includes: **a)Purpose of study** **b) Number of return periods** **c) Study extent** **d) Specific areas of interest** **e) Broad scale or detailed model?** **f) Hydraulic outputs required** **g) Timeframe**	JBA were commission by the Environment Agency in early 2013, with a view to updating an existing ISIS only model of the River Deben in Suffolk (referred to as the "2010 model"). To provide the most suitable modelled representation of the study area, a 2D TUFLOW model domain was added to the existing 1D ISIS model. Both components of the model were updated using new survey data and LIDAR in the model hereafter known as the "2013-14 model". In addition to the fluvial model, a direct rainfall component was also added to the 2D domain in Debenham. This allows the assessment of surface water flood risk. Following production of draft model deliverables a community engagement meeting was held with residents of Debenham. It became clear during the course of the event that the 2013-14 draft outlines were considered to be underestimating flood risk to the settlement and as a result the Environment Agency commissioned further work to re-parameterise the hydrology and hydraulic modelling. Various changes were made, discussed in this document, and the resulting flood outlines provide an improved match to the observed flood history. Outputs from the 2013-14 project included an array of proposed flood alleviation options at Debenham, varying from two stage channels to storage reservoirs on the upstream tributaries. Following the completion of the 2013-14 project, the Environment Agency asked JBA to extend the hydraulic model upstream in order to assess the true impacts of any potential storage scheme. The opportunity was also taken to improve representation of the floodplain topography in the centre of the village, where topographic spot level survey was obtained adjacent to Water Lane. This document forms the technical reporting associated with both the 2013-14 hydraulic modelling and the current extension work, completed in October 2014.

2.2 Available data

Table 2-2: Summary of existing data	
Are there any existing models being incorporated into this study? If so summarise **a) Model type** **b) Model extent** **c) Broad scale or detailed model?** **d) Existing floodplain representation.** **e) When the model was built and by whom?**	Yes. An original ISIS only model of the River Deben was constructed by JBA in 2010. The 2010 model was broad scale, covering the entire reach of the River Deben. For the 2013-14 study we used the existing model as a starting point, building upon the work undertaken previously to produce a more detailed model, with an additional TUFLOW domain. The 2013-4 ISIS-TUFLOW model was used as the basis for the current study, extending this in an upstream direction. Updates to the 1D model domain included revised hydrological estimates, alterations to hydraulic structures, incorporating additional in-channel survey, re-defining hydraulic roughness among other changes.
What DTM data are available for this study? **a) LIDAR** **b) SAR** **c) Filtered/unfiltered**	Both unfiltered and filtered LIDAR are available for the study reach, available at 1m and 2m grid resolution. Topographic spot level survey is available in the centre of Debenham around Water Lane.

d) Resolution **e) Date of surveying and processing.**	
What mapping data is available? Are building footprints required?	OS Master Map, 1:10,000, 1:50,000 and 1:250,000 background maps have been supplied for use in the current project by the EA.

2.3 Modelling software

Both the hydraulic modelling software and release versions used to produce the final results should be recorded in this section. This should allow future users to replicate the results if required in subsequent modelling studies.

Table 2-3: Modelling software	
1D domain	ISIS v3.7 (double precision version used in direct rainfall model)
2D domain	TUFLOW 2013-12-AC-iSP-w64 TUFLOW 2013-12-AC-iDP-w64
Additional software?	n/a

2.4 Model schematisation

The schematisation of the ISIS-TUFLOW model domain will have an important impact on model run times and results. The following chapter should provide a log of the decisions made during this process as well as providing an overview of the final model schematisation.

Table 2-4: Model schematisation	
Are there multiple TUFLOW domains in the model?	No.
What is the geographical extent of the ISIS and TUFLOW domain(s), and why were these limits selected as boundaries to the 2D model?	Four upstream boundaries are specified for the Debenham model, one for each of the main watercourses (The Gulls, Derry Brook and Cherry Tree Brook) and one for an un-named drain which flows into the Gulls at Aspall (referred to hereafter as the Aspall drain); these are all located upstream of the town. The downstream boundary is located near the A1120 road bridge. Study extents are provided in Figure 1-1 of the main report. The River Deben ISIS channel is attached to TUFLOW on both banks as it flows through Debenham. Upstream of the town the watercourse is modelled in a combination of 1D and 1D-2D, the boundaries between domains defined by the valley topography. Downstream of the Cherry Tree Brook confluence only the left bank remains as a linked 1D-2D model, the schematic governed by the floodplain topography. The lower 3km of the watercourse is modelled only in ISIS. The two major tributaries, The Gulls and Cherry Tree Brook are both modelled in ISIS only in their upper reaches. This domain is linked to TUFLOW around the town of Debenham.
What is the total area of the TUFLOW model domain(s)?	1.1 km^2
What software has been used for the 1D component(s)? Why?	ISIS has been used given its suitability for representing complex 1D channels with various hydraulic structures. Other benefits include its ability to dynamically link to TUFLOW and also for straightforward conversion of the 2010 model.

2.5 Model folder structure

TUFLOW modelling requires the use of many different files; these are typically contained within an organised file structure making it easier for the user to manage the model files. This section should be used to summarise the folder structure used in this hydraulic model and to provide instructions on the required path of this folder (if absolute paths have been used in the model files).

Table 2-5: Model folder structure	
Instructions on required path for running model on another machine.	The entire modelling folder should be copied to the root of the C drive. All TUFLOW control paths will be correct with the exception of the results and checks locations. These are not relative to the TUFLOW control file and as such will need updating. ISIS event files (ief) will also require updating to link to relevant ISIS and TUFLOW files.

3. 1D model domains

3.1 1D domain summary

The study reaches shown in Figure 1-1 within the main report illustrates the extent of 1D ISIS model domain. Please note that some 2013-14 model parameterisation undertaken was as a result of the community engagement meeting held in November 2013, following supply of the draft deliverables. This was undertaken in order to better represent flooding in Debenham; residents at the meeting unanimously agreed that draft results underestimated flood risk.

Table 3-1: Overview of ISIS Model

	Model Ref/ Details	
	Model name:	DEBEN_ISIS_KRH_027.DAT
	Purpose:	Flood mapping and option appraisal
Upstream Boundaries:	ReFH boundary units	ReFH units have been used to apply ReFH derived flow hydrographs to the hydraulic model. The inflows have been stored in a separate ISIS ied file for each return period. Time-to-peak calibration was undertaken as part of the hydrological calculations; details of this can be found in the accompanying FEH Calculation Record. Each flow hydrograph is produced using a 7.75 hour design storm – this was obtained by running a variety of durations and recording which produced the greatest peak water levels in Debenham. Checks have been undertaken to ensure the design flows at Debenham are the same as those in the 2013-14 model.
Downstream Boundaries:	NCDBDY	Normal depth boundary based on channel slope; located 4.5km downstream of the River Deben-Cherry Tree Brook confluence near the A1120 road bridge. The distance between the boundary and Debenham significantly exceeds the backwater length, ensuring any assumptions made at the downstream boundary do not impact water levels at Debenham.
Total Number of nodes and structures:	375 nodes including: 8 orifices 18 bridges 12 culverts 1 weir 3 in-channel spill units	
Lateral Inflows	Two groups of lateral inflows have been added to the Debenham hydraulic model. The first of these include lateral catchments not within the TUFLOW model domain (Chy_Lat, Gul1_Lat, Gul2_Lat, Deb_Lat, Deb2_Lat and Deb3_Lat) and are used in both the fluvial and surface water design runs as ReFH inflows. The second group consists of lateral catchments included in the TUFLOW domain. These are included as ISIS ReFH units for the fluvial design event. For the surface water scenario, where direct rainfall is applied to the TUFLOW domain, the inflows are removed to prevent double-counting. Node labels associated with the second group are Chy_DR, Deb_DR and Deb2_DR.	

3.2 ISIS model update

3.2.1 Channel representation

For the 2013-14 project the 2010 Debenham model was updated to improve channel representation in places, firstly where new survey was available and secondly where improvements to the schematic were required to aid linking to the TUFLOW model. The list below illustrates any significant changes made to the model and the justification for such alterations:

- The current project includes extension of the 2013-14 model upstream using 2014 survey to include an additional 931m of the Gulls watercourse and the Aspall drain.
- The reach length has been changed between CHRY_0113d and CHRY_0000u at the downstream extent of Cherry Tree Brook for the 2013-14 model. This was previously recorded as 196m when in reality only 113m of channel exists.
- Section CHRY_0253d was narrowed based on the photograph below in 2013-14, illustrating a channel constriction downstream of the bridge not recorded at the upstream surveyed cross section. A further original width section is added 10m downstream at CHRY_0243 to prevent the narrow area being interpolated too far downstream.

Figure 3-2: Channel geometry at CHRY_0253d

- Various interpolates added to improve stability and allow linking to TUFLOW. A number of these have been produced using the HEC-RAS interpolation tool and added to the model as an ISIS cross section unit; where this is the case the suffix "hi" is added to the node label.

3.2.2 Floodplain representation

Much of the ISIS model is connected to TUFLOW, therefore negating the requirement for extended cross section geometry or reservoir floodplain storage units. However, the upstream extents of The Gulls, Cherry Tree Brook and the drain at Aspall are ISIS only, as is the downstream extent of the River Deben. In these locations the surveyed cross sections have been extended across the floodplain to high ground, parallel to the dominant flow direction. In doing so glass-walling of water is prevented.

3.2.3 Hydraulic roughness

Hydraulic roughness values used have been updated from those used in the 2010 model. Many of the values were updated in 2013-14 based on photographs taken during the site visit and those available from the surveyors. Further updates in Debenham were undertaken for the current project. These were initially calculated using the following formula known as Cowan's method:

$$n = (nb + n1 + n2 + n3 + n4)m$$

where:

nb = a base value of n for a straight, uniform, smooth channel in natural materials

n1 = a correction factor for the effect of surface irregularities

n2 = a value for variations in shape and size of the channel cross sections

n3 = a value for obstructions

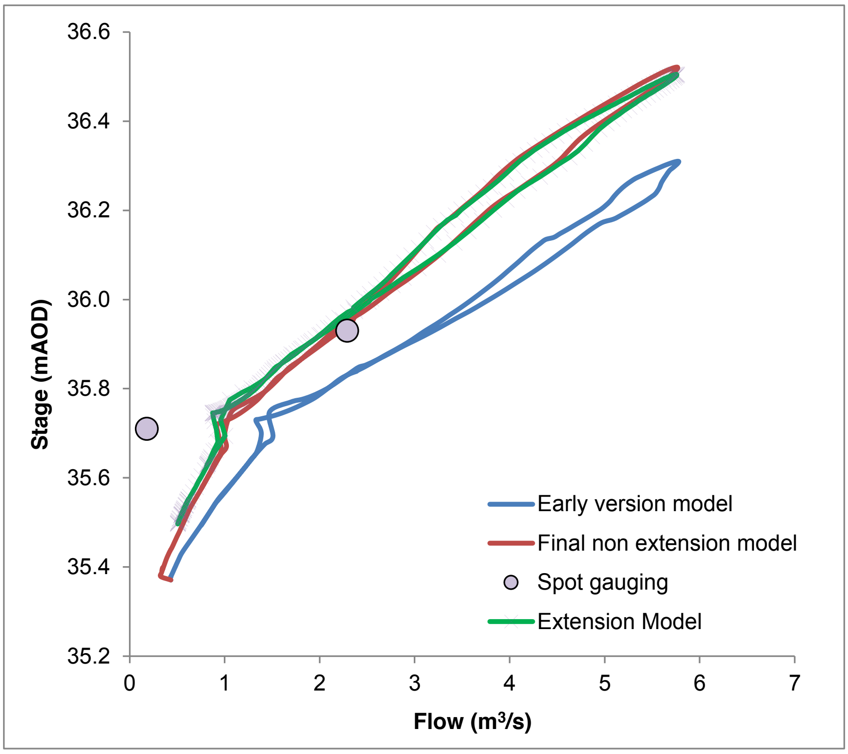
n4 = a value for vegetation and flow conditions

m = a correction factor for meandering of the channel

Upon presenting the 2013-14 draft results at the community engagement meeting, it was apparent that the resulting flood outlines were smaller than those observed in some of the largest events on record (particularly 1956). Following the meeting two spot gaugings were supplied by the Environment Agency, one at the level gauge site on The Gulls, and another at the Low Road bridge on Cherry Tree Brook. No level was supplied for Cherry Tree Brook; this was therefore inferred from the supplied photograph of water levels at the bridge. There is also significant hysteresis noted in the model results suggesting the gauging site is located within the backwater length of the River Deben confluence.

It was evident from these gaugings that the model under-predicted levels for a given flow (around 0.08m on the Gulls and 0.20m on Cherry Tree). Despite only one gauging at each site, it suggested the early modelling may not be behaving in the manner expected. The low water level and the aforementioned small outlines suggested an increase in hydraulic roughness may better represent the area. Roughness was increased for the original project on a trial and error basis until the results matched the spot gauging; the example below is from The Gulls. During the current model development JBA were supplied with a second spot gauging, this time at a much lower flow.

Figure 3-3: Rating curves on The Gulls



It is clear from the graph above that the final stage predicted by the new hydraulic model fits the observed higher flow gauging significantly better than the early development model version. However, there is a poor fit to the low flow gauging. This is likely due to the greater influence of small scale features and vegetation during low flows; such characteristics will not be well accounted for in the model given the relatively coarse cross section spacing. Whilst this implies the model does not perform as well at very low flows, the purpose of this study is to assess the flood risk at Debenham and therefore it is encouraging the results fit the higher spot gauging. If the model were to be used in future for low flow analysis is it recommended a finer resolution topographic survey is undertaken.

A similar trend is seen on Cherry Tree Brook where a steady flow of 1.77m^3/s results in a stage of 33.04mAOD, only around 0.03m lower than that recorded. It is not possible to illustrate this above due to the hysteresis present in the hydrodynamic model caused by the backwater effect of the River Deben.

This approach makes best use of the available data although should be re-visited if additional check gaugings become available. It is our belief that the resulting outputs better represent the flood risk to the village. Roughness values remain within bands regularly used in hydraulic models, although these are relatively high it should be considered that the river channels around Debenham are narrow, irregular and contain significant vegetation. Please note that some local alterations have been made to hydraulic roughness within Debenham since the development of the 2013-14 model, most notably downstream of Priory Lane where flow is constrained within wooden banks.

It is strongly recommended that the model is re-visited if local flow data become available. The presence of a high quality flow gauging station on any of the main channels would significantly improve confidence in flow estimates, and also allow further calibration of the modelled levels against know flows.

The model has been split into a number of reaches, each with a differing Manning's n value. A number of example sections are shown below, each with associated roughness values.

Table 3-2: Final hydraulic roughness values

Cherry Tree Brook	
	Section: CHRY_1938 Channel: 0.079 Banks and floodplain (grass and ploughed fields): 0.093
	Section: CHRY_0288u Channel: 0.046 Near channel banks (trees): 0.124 Floodplain modelled in TUFLOW
The Gulls	
	Section: GULL_2735 Channel: 0.065 Near channel banks (trees): 0.116 Floodplain: 0.102

Section: GULL_1433

Channel: 0.065

Near channel banks (trees): 0.116

Floodplain: 0.102

Section: GULL_0277d

Channel: 0.065

Near channel banks: 0.087

Floodplain modelled in TUFLOW

River Deben

Section: DEBN_5863

Channel: 0.075

Banks and floodplain modelled in TUFLOW

Section: DEBN_5354

Channel: 0.030

Near channel banks: 0.072

Floodplain modelled in TUFLOW

Section: DEBN_5182d

Channel: 0.038

Near channel banks: 0.116

Floodplain modelled in TUFLOW

Section: DEBN_4045

Channel: 0.065

Near channel banks: 0.087

Floodplain modelled in TUFLOW

3.2.4 Structures

1D hydraulic structures in the model were updated in 2013-14 from the 2010 model where new 2013 survey data were available. Where this was not available structures were updated using the supplied 2007 topographic survey data; most structures were originally modelled as orifice units. Although this is a suitable approach in some situations, structures are better represented by the relevant ISIS unit.

Cherry Tree Brook

Table 3-3: Structure CHRY_2161o	
Structure description	3 barrel culvert at Bush Corner.

Included in model?	Yes – taken from 2007 survey.
How has structure been modelled?	Modelled simply as an orifice unit to represent all three barrels, with an invert level of 40.56mAOD and a bore area of 1.12m^2. This structure is located away from any area of flood risk and therefore the simplistic representation is appropriate, whilst maintaining model stability. A parallel spill is included over the section with a spill coefficient of 1.4 to represent the metal railings.
	
Upstream face	Downstream face

Table 3-4: Structure CHRY_0959c	
Structure description	Multiple culverts beneath road
Included in model?	Yes – taken from 2013 survey
How has structure been modelled?	A complex structure with five pipes beneath the road, three of which are located at a higher level. The two lower barrels and one upper barrel have a trash screen located at their inlet. To model the structure the lowest two barrels are included as circular culverts units with a diameter of 0.4m and a length of 12.59m. These include orifice units to represent the inlet and outlet flow constriction/expansion, with a bore area of 0.1m^2 (based on a diameter reduced by 0.04m to account for trash screen). The use of orifice units allows the drop from outlet to channel bed at the downstream face to be represented as weir flow. These culverts are given a Manning's n value of 0.03. The upper pipes are included only as orifice units; the left structure has a bore area of 0.06m^2 to account for the trash screen, the other two set to 0.07m^2. These features cannot be included as culvert units given they do not convey water at low flows (ISIS allows orifice units to be dry, but not culverts). The inconsistency in approach is unlikely to have a major impact on flood flows as during these events the majority of flow will pass over the road deck. The structure is also located significantly upstream of any areas of flood risk. A parallel spill is included over the section with a spill coefficient of 1.0 to account for the hedge visible below.
	

Upstream face	Downstream face

Table 3-5: Structure CHRY_0926B	
Structure description	Track bridge
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 35.23mAOD and an opening width of 3.28m. Geometry taken from 2013 survey; structure not included in 2009 model. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the smooth deck.
	
Upstream face	Downstream face

Table 3-6: Structure CHRY_0841B	
Structure description	Track bridge
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 34.85mAOD and an opening width of 2.41m. Geometry taken from 2013 survey; structure not included in 2009 model. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the smooth deck.
	
Upstream face	Downstream face

Table 3-7: Structure CHRY_0658B	
Structure description	Arch bridge carrying driveway.
Included in model?	Yes – taken from 2013 survey.

How has structure been modelled?	Included in the model as an arch bridge unit with a soffit level of 34.52mAOD and a springing level of 33.72mAOD. Geometry taken from 2013 survey; structure not included in 2009 model. The cross section immediately downstream of the structure has been given a bed level 0.28m higher than that upstream, as recorded on the long profile supplied with the topographic survey. A parallel spill is included over the section with a spill coefficient of 1.2 to represent the railings.
	
Upstream face	Downstream face

Table 3-8: Structure CHRY_0340B	
Structure description	Road bridge
Included in model?	Yes – taken from 2007 survey.
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 33.25mAOD and an opening width of 3.89m. Originally modelled as an orifice unit. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the well spaced railings.
	
Upstream face	Downstream face

Table 3-9: Structure CHRY_0288B	
Structure description	Footbridge
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 33.44mAOD and an opening width of 3.21m. Originally modelled as an orifice unit. Although the structure itself is unlikely to result in major head-losses during a flood event, the channel constriction associated with the concrete banks needs to be represented in the model. A parallel spill is included over the section with a spill coefficient of 1.4 to represent

	the smooth deck with sparse railings.

Upstream face

Downstream face

Table 3-10: Structure CHRY_0253B	
Structure description	High Street road bridge.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 33.55mAOD. The upstream open channel section is notably different from the downstream section (see photograph below). The downstream constriction was not surveyed although this is likely to impact upstream flood levels. The bridge cross section is modelled at its full width, although the downstream section is narrowed based on the photograph below. Originally modelled as an orifice unit in 2010. A parallel spill is included over the structure, including the parapet wall, with a spill coefficient of 1.7 to represent the smooth material.

Upstream face

Downstream face

Table 3-11: Structure CHRY_0112o	
Structure description	Road bridge.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Road bridge with two parallel barrels beneath. A weir is present at the downstream face, behind which water backs up (see the photograph with upstream still water below). It is thought this weir has a significant impact on low return period flood peaks on Cherry Tree Brook and therefore its inclusion is vital. This structure has been modelled with two orifice units. The structure is short enough to have limited barrel losses (i.e. no need for a culvert unit) and the orifice uses the weir equation as water flows over the invert to the lower downstream channel. The left hand barrel has been given an invert level of 32.05mAOD and a bore area of 3.59m^2. The right hand barrel is set to 32.05mAOD and 3.26m^2 respectively.

	The drop in channel bed from upstream to downstream is represented by the adjoining channel sections, although the downstream bed level is inferred from site photography. Water levels recorded by the surveyors in early 2013 have been used as a method of calibrating this structure (albeit during a low flow scenario). This gives confidence in the model performance at this location. A parallel spill is included over the structure, including the parapet wall, with a spill coefficient of 1.5.

Looking upstream – note the still water as a result of backing up behind the weir.

Downstream face

The Gulls

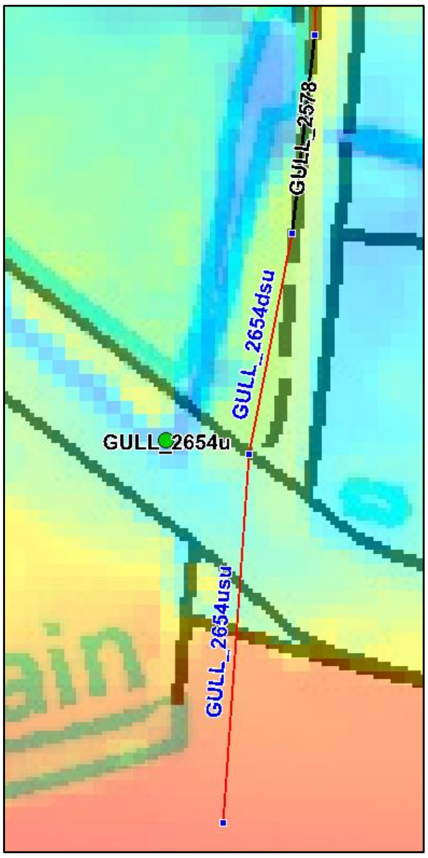
Table 3-12: Structure GULL_2735c	
Description	Culvert beneath road embankment
Included in model?	Yes – taken from 2014 survey.
How has structure been modelled?	Included in the model as a sprung culvert unit and has an invert level of 45.68 mAOD, is 6.06m long and 1.6m in width. The structure has a springing height of 0.24m and crown height of 0.77m. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the relatively smooth road deck.

Upstream face

Downstream face

Table 3-13: Structure GULL_2578c	
Description	Culvert beneath road embankment
Included in model?	Yes – taken from 2014 survey.
How has structure been modelled?	Included in the model as a sprung culvert unit and has an invert level of 44.8 mAOD, is 5.66m long and 1.5m in width. The structure has a springing height of 0.43m and crown height of 0.52m. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.2 to represent the vegetated embankment; this is extended to represent the left bank spill, although the right hand spill is captured in a separate unit as described below. A 90° left-hand bend in the channel upstream of GULL_2578c has necessitated altered representation of floodplain flows. The schematic depicts three separate spill units over the road between GULL_2729d and GULL_2578d. The spill GULL_2654usu is an in-line spill unit representing out of bank flows upstream of the river bend flowing across the road junction with the potential to re-join the channel downstream of the culvert at GULL_2572c. The spill GULL_2654dsu is a lateral spill unit representing out of bank flows flowing over the road between the river bend and the culvert downstream. These flows are also connected to the channel downstream of the culvert at GULL_2572c. The spill GULL_2578su is an in-line spill representing flows overtopping the culvert embankment at GULL_2578c.

Upstream face



Schematic

Table 3-14: Structure GULL_2262c	
Description	Culvert beneath road.
Included in model?	Yes – taken from 2014 survey.

How has structure been modelled?	Included in the model as a rectangular culvert unit and has an invert level of 43.44 mAOD and is 7.1m long. The structure is 2.37m in width and 0.91m in height. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.6 to represent the relatively smooth road.
	
Upstream face	Road deck and left bank

Table 3-15: Structure GULL_1645B	
Description	Arch bridge carrying track.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as an arch bridge unit with a soffit level of 41.3mAOD and a springing level of 40.2mAOD. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the relatively smooth deck.
	
Upstream face	Downstream face

Table 3-16: Structure GULL_1211c	
Description	Sprung arch culvert carrying dismantled railway.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Modelled as a sprung arch culvert with an invert level of 37.25mAOD (averaged from surveyed bed level). Culvert barrel is 1.77m in width, with a springing level 0.95m above the bed and the soffit level 0.76m above this. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.0 to represent vegetation between the channel and the road. This has been modelled so only a width

	of 10.5m is available to flow; this accounts for the road which cuts through the railway embankment (see photograph below).

	
Upstream face	Road and embankment (channel shown by arrow). Photograph from Google Streetview

Table 3-17: Structure GULL_0969o	
Description	Two small culverts beneath field entrance track.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Modelled as two parallel orifice units with dimensions taken from 2013 survey; not included in earlier model. The bore area of each barrel is set to 0.44m^2. The use of orifice units aids model stability; although not representing the barrel losses associated with the 5.51m long structure, this is not within a critical area of the model and is therefore deemed suitable. Most flow is conveyed downstream over the parallel spill in high flow conditions. The spill is included over the section with a spill coefficient of 1.7 to represent the very smooth concrete deck.

	
Upstream face	Downstream face

Table 3-18: Structure GULL_0644B	
Description	Brick arch.
Included in model?	Yes – taken from 2007 survey.
How has structure been modelled?	Included in the model as an arch bridge unit with a soffit level of 36.66mAOD and a springing level of 35.66mAOD, with a 1.92m wide opening. Cross sectional geometry taken from 2013 survey at section immediately downstream although this survey did not include the structure. Width and arch levels are therefore taken from 2007 survey. A parallel spill is included over the section with a spill coefficient of 1.2 to represent the vegetated deck.

	
Upstream face	Downstream face

Table 3-19: Structure GULL_0487B	
Description	Wooden footbridge above channel constriction.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Modelled as a USPBR bridge unit with a flat soffit level of 36.08mAOD. Impact on upstream water levels is more driven by the channel constriction (only 3m wide at this point) rather than the bridge deck. Geometry from 2013 survey; not included in earlier model. The cross section immediately downstream of the structure has been given a bed level 0.17m higher than that upstream, as recorded on the long profile supplied with the topographic survey. A parallel spill is included over the section with a spill coefficient of 1.2 to represent the vegetated banks and the wooden railings on the deck.
	
Upstream face	Downstream face

Table 3-20: Structure GULL_0277B	
Description	Arch bridge carrying Gull Farm driveway.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Included in the model as an arch bridge unit with a soffit level of 36.32mAOD and a springing level of 35.13mAOD. Geometry taken from 2013 survey; structure not included in 2009 model. A parallel spill is included over the section with a spill coefficient of 1.7 to represent the smooth wall.

	
Upstream face	Downstream face

Table 3-21: Structure GULL_0011c	
Description	Rectangular culvert beneath The Flats road.
Included in model?	Yes – taken from 2007 survey.
How has structure been modelled?	Located at the confluence between The Gulls and the River Deben. Included in the model as a rectangular culvert unit with an invert level of 33.72mAOD and a length of 10.80m. The structure is 2.71m wide and 1.23m high. It was included in the earlier model as an orifice unit and has therefore been updated. No new 2013 was commissioned for this structure. The spill over the deck is included in TUFLOW.
	
Upstream face	Downstream face

River Deben

Table 3-22: Structure DEBN_5985c	
Description	Sprung arch culvert.
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Modelled as a symmetrical culvert to represent the unusual shape. The base of the barrel is 1.48m wide at 35.37mAOD, tapering to a soffit level at 36.75mAOD. The culvert is 19.4m in length. Geometry is taken from 2013 survey; this structure was not included in the original model. The spill over the deck is included in TUFLOW.

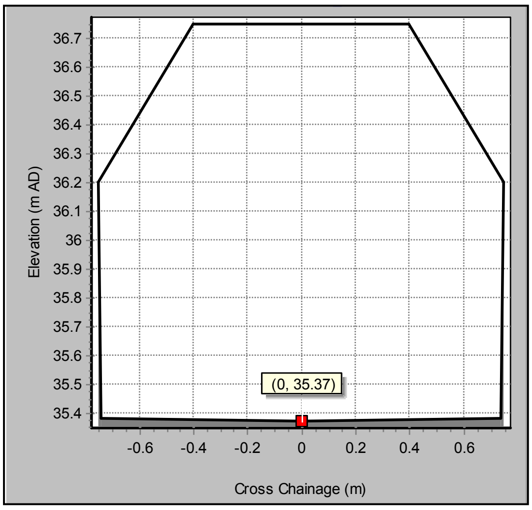
	
Upstream face	ISIS geometry

Table 3-23: Structure DEBN_5745c	
Description	Culvert conveying flow beneath The Butts/Chancery Lane junction
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	Dual barrel rectangular culvert modelled as such. Both barrels are similar shapes although the dimensions vary slightly. Both inverts are set to 34.28mAOD and the culverts are 26m long. The left barrel is 1m wide compared to 1.25m of the right barrel. Barrel heights are 0.76m and 0.78m respectively. Geometry is taken from 2013 survey; this structure was included in the original model as an orifice unit. No inlet and outlet units are defined as the bounding channel is not constrained when entering the structure. The spill over the deck is included in TUFLOW.
	
Upstream face	Downstream face

Table 3-24: Structure DEBN_5447B	
Description	Footbridge adjacent to Aspall Road.
Included in model?	Yes – estimated from channel geometry and photography.
How has structure been modelled?	Footbridge modelled as a USBPR bridge unit with a flat soffit of 34.75mAOD. Width matched to upstream channel width. A parallel spill is included over the section with a spill coefficient of 1.1 to represent the wire mesh present.

Upstream face

ble 3-25: Structure DEBN_5404c	
Description	Culvert beneath Market Square
Included in model?	Yes – taken from 2013 survey.
How has structure been modelled?	This culvert is of particular importance to the current study as floodwater is known to spill into Debenham Market Square upstream of the structure. The barrel is at around 60° to the upstream flow direction due to a bend in the channel immediately upstream of the inlet. However, to also account for the bend losses associated with the right angle bend immediately upstream of the structure, it has been decided to set this to 60°. In order to account for this the width of the structure is reduced from 4.44m to 2.61m (= cosine(60) x width). This effectively models the skew angle and upstream bend losses, reducing the area available to flow. The invert of the structure is set to 33.58mAOD in accordance with the 2013 survey data. The barrel is 12.69m in length with a height of 1.54m. The spill over the deck is included in TUFLOW.

Upstream face

Downstream face

Table 3-26: Structure DEBN_5297f	
Description	Ford at Water Lane
Included in model?	Yes – taken from 2013 survey
How has structure been modelled?	Modelled as a spill unit representing the drop from road level back into the natural channel at the eastern extent of Water Lane. A low spill coefficient of 0.8 is used. Despite the smooth crest surface this value is appropriate in order to represent the losses associated with the flows changing direction. The spill crest has been manually narrowed from that shown on the upstream survey section, which was located on the road, not the actual crest. This was highlighted during the low-flow calibration undertaken, where the initial modelled headloss was 0.28m compared to a recorded headloss of 0.36m The new crest width has been inferred from

	OS MasterMap data. During the modelling phase it was noted that a negative water surface gradient was present upstream of this small weir on Water Lane. The effect is created by the weir as water flows from Water Lane into the straight channelized section. Upstream, the water is trying to force a normal depth profile, but the weirs acts as a discontinuity, forcing levels to be increased locally. The momentum of water is therefore enough to create a localised negative gradient.
	
Spill crest	

Table 3-27: Structure DEBN_5182b	
Description	Priory Lane road bridge
Included in model?	Yes – taken from 2007 survey
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 34.40mAOD. Geometry taken from 2007 survey and updated from the earlier model which included the structure as an orifice unit. A skew angle of 20° is used to represent the angle of the bridge opening to the predominant flow direction. A parallel spill is included over the section with a spill coefficient of 1.4 to represent the sparse railings.
	
Downstream face	

Table 3-28: Structure DEBN_4787b	
Description	Maltings House bridge
Included in model?	Yes – taken from 2007 survey
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit level of 34.37mAOD. Geometry taken from 2007 survey and updated from the earlier model which included

	the structure as an orifice unit. A parallel spill is included over the section with a spill coefficient of 1.7 to represent the smooth wall.
	
Upstream face	Downstream face

Table 3-29: Structure DEBN_4491b

Description	Road bridge with old wooden bridge and weir crest immediately downstream
Included in model?	Yes – taken from 2013 survey
How has structure been modelled?	The upstream bridge is included in the model as a USPBR bridge with a flat soffit level of 33.19mAOD. Geometry taken from 2013 survey and updated from the earlier model which included the structure as an orifice unit. The second bridge is not included, as the first represents the hydraulic control. The in-channel weir crest is positioned 1m downstream of the bridge, with a crest level of 31.65mAOD and a spill coefficient of 1.4. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the sparse railings.

Downstream face

Table 3-30: Structure DEBN_3532w

Description	Weir structure
Included in model?	Yes– taken from 2013 survey
How has structure been modelled?	This feature consists of two crests with a flat concrete channel between. The upstream crest is the control on water levels due to the higher elevation and the channel constriction shown below. For this reason only the upper crest is included within the model. The crest level is set to 29.721mAOD, with a width of 1.51m and a general purpose weir unit is used.

	Any water flow around the crest on either the left or right bank is accounted for by a parallel spill with a spill coefficient set to 1.0. The centre of the channel (where the weir is located) is set to 35mAOD to block flow and therefore preventing double counting.
	
Weir crest	

Table 3-31: Structure DEBN_3319w	
Description	Weir structure
Included in model?	Yes– taken from 2013 survey
How has structure been modelled?	This feature consists of two crests with a flat concrete channel between. The upstream crest is the control on water levels due to the higher elevation and the channel constriction shown below. For this reason only the upper crest is included within the model. The crest level is set to 29.01mAOD, with a width of 1.39m and a general purpose weir unit is used. Any water flow around the crest on either the left or right bank is accounted for by a parallel spill with a spill coefficient set to 1.2. The centre of the channel (where the weir is located) is set to 33mAOD to block flow and therefore preventing double counting.
	
Weir crest	

Table 3-32: Structure DEBN_2882b	
Description	Track bridge
Included in model?	Yes – taken from 2007 survey
How has structure been modelled?	Included in the model as a USPBR bridge with two openings. The left opening has a flat soffit of 29.72mAOD and a width of 2.22m compared to the right hand soffit of 29.73mAOD and 5.45m width. Geometry taken form 2007 survey and updated from the

	earlier model which included the structure as an orifice unit. A parallel spill is included over the section with a spill coefficient of 1.4 to represent the relatively smooth deck with some vegetation.
	
Upstream face	Downstream face

Table 3-33: Structure DEBN_1222b	
Description	Track bridge
Included in model?	Yes – taken from 2007 survey
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit of 27.11mAOD and a width of 5.52m. Geometry taken form 2007 survey and updated from the earlier model which included the structure as an orifice unit. A parallel spill is included over the section with a spill coefficient of 1.4 to represent the relatively smooth deck.
	
Upstream face	Downstream face

Table 3-34: Structure DEBN_0888b	
Description	Road bridge
Included in model?	Yes – taken from 2007 survey
How has structure been modelled?	Included in the model as a USPBR bridge with a flat soffit of 25.56mAOD and a width of 6.10m. Geometry taken form 2007 survey and updated from the earlier model which included the structure as an orifice unit. A parallel spill is included over the section with a spill coefficient of 1.5 to represent the smooth deck.

Upstream face

Downstream face

Aspall drain

Table 3-35: Structure ASPD_1004c	
Description	Culvert beneath track
Included in model?	Yes – taken from 2014 survey
How has structure been modelled?	Included in the model as a circular culvert with an invert of 47.61mAOD and length of 3.4m. The structure has a diameter of 1.05m. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.6 to represent the smooth road.

Upstream face

Downstream face

Table 3-36: Structure ASPD_0512b	
Description	Road bridge
Included in model?	Yes – taken from 2014 survey
How has structure been modelled?	Included in the model as an arch bridge unit with a springing level of 46.36 mAOD and a soffit level of 47.54 mAOD. This structure is outside the study extent of the earlier model and was therefore not included previously. A parallel spill is included over the section with a spill coefficient of 1.4 to represent the vegetation and track surface.

	
Upstream face	Downstream face

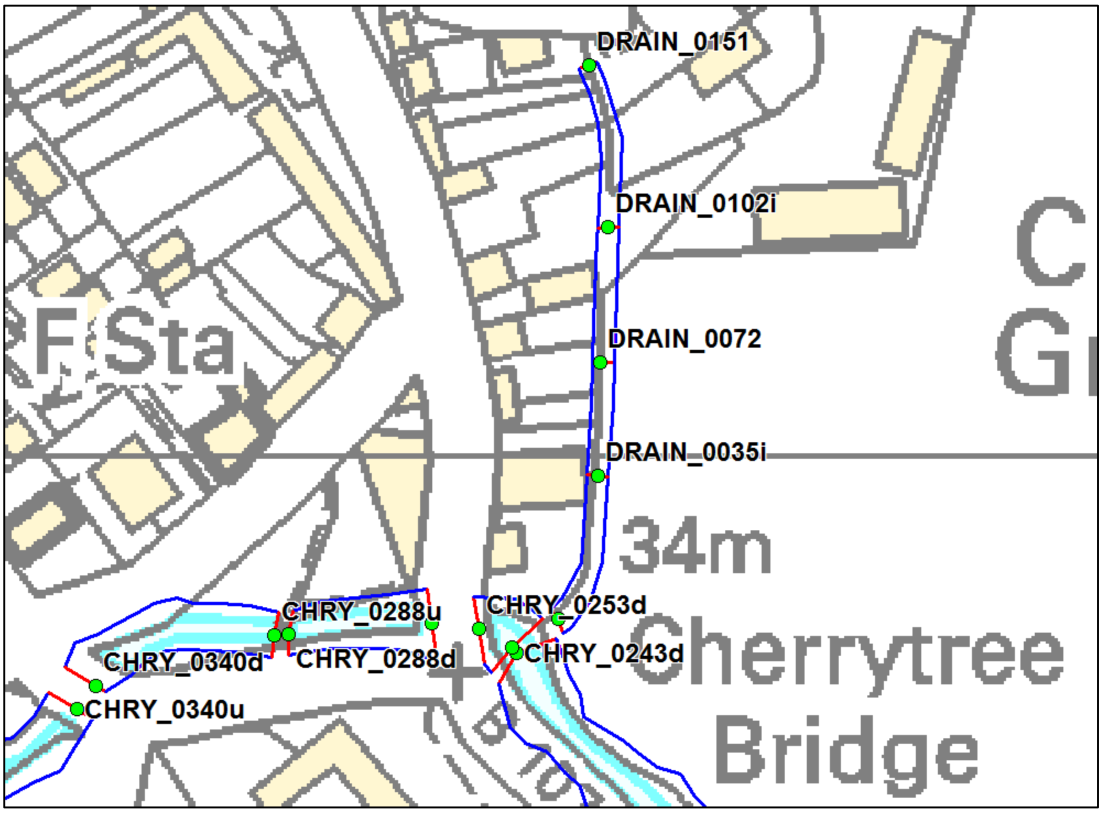
3.2.5 Additional channels

All channels previously discussed in this document were included in the 2010 model and simply updated for the 2013-14 project. Only the small drain at the southern extent of the High Street (upstream extent notated as DRAIN_0151) and the drain adjacent to Priory Lane (PRIO_0215) were developed within ISIS specifically as part of the 2013-14 commission. The locations of these small channels are discussed below.

The channel at the south end of the High Street is constructed using surveyed cross sections with interpolate units added to improve model stability. At the confluence with Cherry Tree Brook a spill unit is used to represent the sharp drop in bed level. No distinct inflow hydrograph is added to this drain; sewer mapping provided by Anglian Water shows there is no discharge directly into the drain. For the purposes of hydraulic modelling, a sweetener flow of $0.05m^3/s$ is added to prevent the channel running dry.

Roughness values have been set to 0.060, representing the vegetated channel as seen below. Similarly, the downstream spill unit has been given a relatively inefficient weir coefficient of 1.0.

Figure 3-3: Location of newly constructed channel



Upstream extent of DRAIN channel, looking downstream

ISIS – TUFLOW HX Boundary

ISIS – TUFLOW CN Line

ISIS Cross Section Location

The drain flowing adjacent to Priory Lane was not surveyed as part of either the 2010 project or the 2013-14 project. Despite this, it is apparent from the figure below that the channel is well defined; it is well represented in the 1m LIDAR and therefore cross sections have been taken from this dataset.

The reach was added to the ISIS model in 2013-14, 215m in length, attached to a 23m culvert discharging directly into the River Deben. In reality the culvert links back to a short section of open channel prior to the confluence, although at high flows this is always at capacity with flood water backing up from the River Deben. Therefore there is no available storage for additional flood water and the use of the longer culvert is suitable.

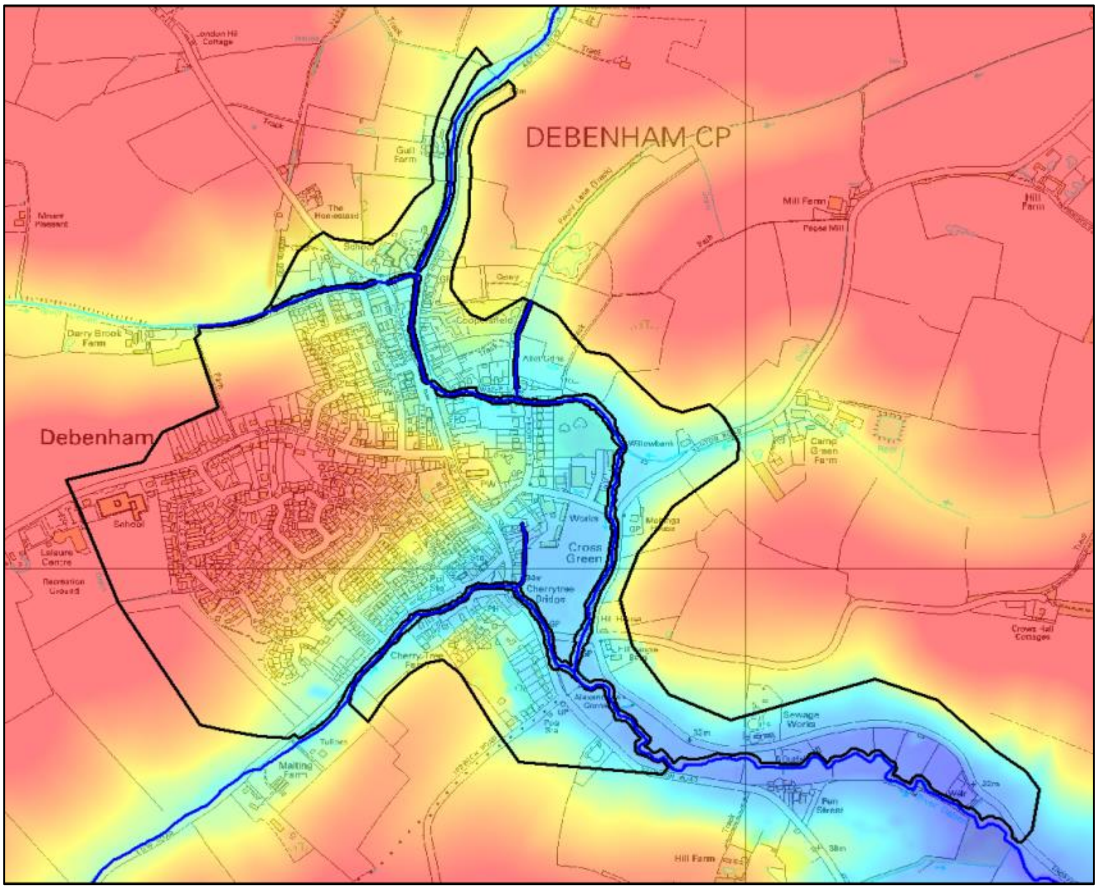
Figure 3-4: Priory Lane photographs

4. TUFLOW model domain – fluvial model

4.1 TUFLOW domain summary

Table 4-1: TUFLOW domain summary	
What is the cell size and why has it been chosen?	A 2m cell size has been chosen for the Debenham model. This allows flow paths in the village to be well represented and also is sufficiently small to represent many minor topographic features which are important when using direct rainfall inputs.
What is (are) the grid orientation(s)?	The grid is orientated north-west to south-east, in line with the dominant flow direction.

Figure 4-1: Diagram of TUFLOW model schematisation



DTM Elevations

30.0 m AOD

35.0 m AOD

40.0 m AOD

45.0 m AOD

50.0 m AOD

TUFLOW external boundary

ISIS Domain

4.2 Roughness coefficients

Table 4-2: Bed and floodplain resistance				
Approach to building bed and floodplain resistance		Roughness values have been defined based on OS MasterMap data and aerial photography. There are no specific guidelines for setting floodplain roughness values and this often calls for modelling judgement. The categories shown below were considered for use in the current model; only the ones marked "yes" have been applied. In order to model direct rainfall inputs to the model, depth varying Manning's roughness is used. At shallow flows a greater Manning's value is specified given the relatively greater impacts of micro topography (e.g. grass). This is both realistic and also improves model stability. The roughness categories are deliberately duplicated in the TUFLOW materials file (tmf). The materials file is altered for the direct rainfall version of the model – this is discussed in greater detail later in this document.		
Manning's n		**Land Cover Category**	**Land Type**	**Used?**
0 – 0.02m depth	**+0.02m depth**			
0.200	0.060	1	Grass	Yes
0.500	0.120	2	Dense trees/woodland	Yes
0.300	0.100	3	Shrubs, gardens with fencing	Yes
0.150	0.035	4	Gravel roads	Yes
0.100	0.030	5	Roads, paved areas and footpaths	Yes
0.250	0.060	6	Hard surface, work yards	Yes
0.040	0.040	7	Open car parks	No
0.200	0.200	8	Buildings	Yes
0.085	0.085	9	Roughness patches (for stability purposes)	No
0.300	0.080	10	Ploughed fields	Yes
0.030	0.030	11	Water surfaces	Yes
0.300	0.080	12	Bridge railings	Yes
0.500	0.080	14	Gully line north of Cherry Tree Brook	Yes

4.3 Modifications to ground model

Table 4-3: Modifications to ground model		
ID Layer Name	**Command (e.g. "Read MI Z Shape ADD")**	**Purpose of terrain modification and source of elevation data**
DTM_1M.txt	Read GRID Zpts	Reads in the ground model directly from an exported filtered LIDAR grid.
2d_zsh_drain_stamp_002.mif	Read MI Z Shape	Fills in drain channels along the study reach, effectively assuming these are full prior to the arrival of the flood wave.
2d_zsh_drains_001.mif	Read MI Z Shape	Stamps down the drain to the west of Debenham, draining into Cherry tree Brook.
2d_zline_banks_010.mif	Read MI Z Line THICK	Sets bank levels along the entire model domain based either on surveyed levels or filtered LIDAR.
2d_zsh_bridge_decks_004.mif	Read MI Z Shape	Sets the elevation of bridge decks when these are modelled in TUFLOW.
2d_zsh_topo_001.mif	Read MI Z Shape	Helps to smooth areas of poorly filtered LIDAR to

		improve direct rainfall stability.
2d_z_buildings_surveyed_002.mif	Read MI Zpts	Sets the surveyed threshold levels of buildings where available.
2d_z_buildings_averaged_002.mif	Read MI Zpts	Sets an average LIDAR level for buildings with non-surveyed thresholds. Please note that one building on the High Street (located between the Angel public house and the Vanilla cafe) has been removed to account for the archway present.
2d_z_buildings_stamped_002.mif	Read MI Zpts ADD	Stamps non-surveyed building thresholds up by 0.3m. Please note that one building on the High Street (located between the Angel public house and the Vanilla cafe) has been removed to account for the archway present.
2d_TIN_toposurvey.txt	Read GRID Zpts	Sets ground elevations based on a TIN (in ascii format) interpolated between surveyed spot levels. This topographic survey is at variable resolution, ranging from 2-10m, and is located immediately south-west of the junction between Water Lane and Priory Lane. The survey was commissioned to ensure a ridge of high ground between Water Lane and Priory Lane was well represented in the hydraulic model; this is covered in vegetation and therefore filtered LIDAR of this area is deemed unreliable.

4.4 Representation of buildings

The detail used to represent buildings in a 2D hydraulic model should be fit for the purpose the model is designed for. This may range from an average urban roughness suitable for crude large scale modelling to the application of specific threshold levels for each property in more detailed modelling studies.

There are a number of modelling options available within TUFLOW for modelling buildings; these are discussed in detail in Syme (2008)[1]. For the current project we have opted for using a threshold level with Manning's roughness values increased to 0.200 to represent increased flow resistance. As discussed above, threshold levels are either set to a surveyed level (where available) or based on averaged LIDAR elevations stamped up by 0.3m.

4.5 Observation features

No observation features were used in this model.

[1] Syme W. J. (2008). Flooding in Urban Areas – 2D Modelling Approaches for Buildings and Fences. *Engineers Australia, 9th National Conference on Hydraulics in Water Engineering.*

4.6 Other geometry controls

Table 4-4: Other geometry controls	
Have initial water levels been set in the TUFLOW model domain. If so what commands have been used and why?	No
Have any z points been interpolated using the interpolate commands?	Yes. The “Interpolate ZUV” command has been used to smooth the ground model for direct rainfall modelling. All other geometry modifications are applied after the interpolation; ensuring bank crest levels etc are not interpolated.
Have any default values/coefficients been adjusted, if so why?	No

4.7 Floodplain structures

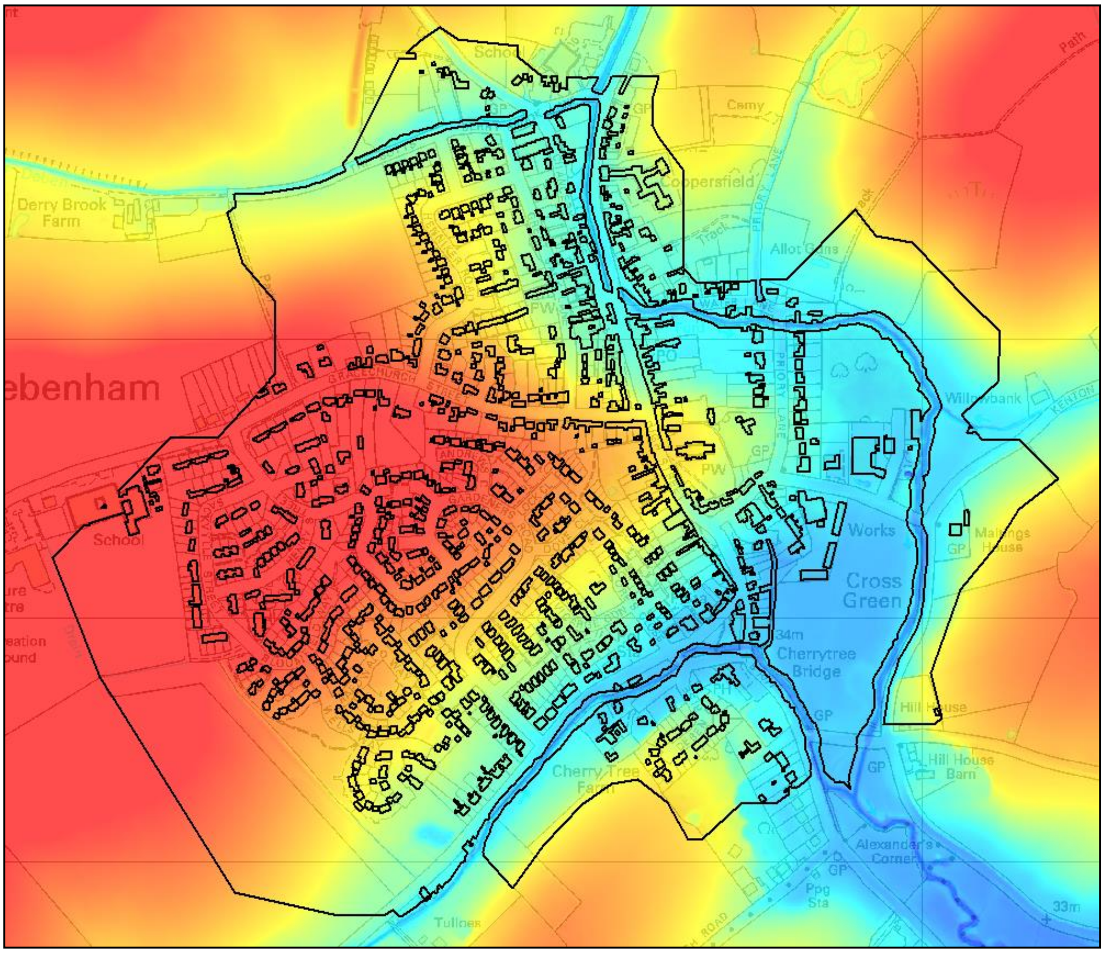
No floodplain structures are included in the Debenham model.

5. TUFLOW model domain – direct rainfall model

5.1 TUFLOW domain summary

Table 5-1: TUFLOW domain summary	
What is the cell size and why has it been chosen?	A 2m cell size has been chosen as for the direct rainfall version of the model.
What is (are) the grid orientation(s)?	North-west to south-east, in line with the dominant flow direction.

Figure 5-1: Direct rainfall input area



DTM Elevations		Direct rainfall input area
30.0 m AOD	45.0 m AOD	
35.0 m AOD	50.0 m AOD	
40.0 m AOD		

Please note that buildings have been removed from the direct rainfall input area shown above. Including this within the area would ensure rain falls directly onto the stamped up building thresholds. As a result, water would spill form these elevated areas onto the surrounding ground; at such shallow flows it is likely such a process would result in significant model instability.

Removal of the building footprints results from the direct rainfall input area results in an underestimate of the rainfall depth applied to Debenham (i.e. no rainfall is applied to buildings). To counter this effect, the rainfall depths applied to TUFLOW are increased based on a ratio of the TUFLOW area and building area as follows:

5.2 Roughness values and infiltration

Table 5-2: Roughness values and infiltration	
Approach to building bed and floodplain resistance	In order to model direct rainfall inputs to the model, depth varying Manning's roughness is used as discussed above and shown in Table 4-2. In addition to hydraulic roughness, the .tmf file is also used to apply initial and continuing losses to the direct rainfall area of the TUFLOW domain. For this model we have run two test scenarios; the first of these assumes no infiltration and the second has a continuing loss if 10mm/hr throughout the model run. This infiltration was tested to establish the likely impact of both soil infiltration and also loss to the surface water sewer system. The results illustrated that outlines and depths were very similar between the no infiltration and infiltration model version. Therefore the no infiltration scenario has been adopted for design model runs.

6. Model boundaries

6.1 Upstream boundaries

The upstream model boundaries have been included as ReFH inflow units within the ISIS model. Full details on the estimation of flood flows are available in the accompanying FEH Calculation Record (Appendix A).

Please note that three inflows are added to the model which account for the contributing area covered by the direct rainfall area. In the fluvial model these are input directly to ISIS are ReFH flows. In the direct rainfall version of the model the inflows are de-activated in ISIS to ensure no double counting occurs.

6.2 Downstream boundaries

A normal depth boundary has been used for the Debenham model. This unit automatically generates a flow-head relationship based on the upstream section data. Given the uncertainties associated with such a boundary type, it is important to locate this sufficiently downstream of the site of interest. The Debenham boundary is located 4.5km downstream of the Cherry Tree Brook – River Deben confluence and therefore is greater than one backwater length (around 0.6km) downstream of the town. This ensures its influence, and therefore any inherent uncertainties associated with it, do not impact the modelled levels at Debenham. This is confirmed in the sensitivity testing (see previous reporting from May 2014).

6.3 Rainfall boundaries

A direct rainfall boundary is applied in the relevant model versions covering the whole of the settlement. This is applied using TUFLOW bc-database files.

6.4 1D-2D boundaries

Table 6-1: Model boundaries

Check	Answer	Comments
Is there provision for floodplain flow to both enter and leave the TUFLOW model without being forced in-channel?	Yes	The TUFLOW domain is connected to the ISIS channel through Debenham. No additional 1D-2D boundaries exist, ensuring fluvial flood water can only enter/exit the domain via these connections.
Do the channel widths in ISIS match the width of the inactive area in the TUFLOW domain?	Yes	The channel width represented in ISIS has been removed from the TUFLOW domain to prevent doubling counting of available flow area.
Are there any instances of double counting conveyance?	No	
What boundaries have been used between the ISIS and TUFLOW domains? Why was this boundary type preferred?	HXI	The 1D-2D link has been applied using HXI lines. These take water level from the ISIS model and convert this into a flow entering the TUFLOW domain. The "a" parameter has been set to 0.25 in order to account for losses associated with water transferring from 1D to 2D and vice versa. In a number of locations (e.g. left bank along Aspall Road) this is increased to 0.50 to represent the fence railings present.
How were the locations of the boundary lines defined?	At the 1D-2D interface	1D-2D boundaries were placed where it was deemed 1D flow characteristics gave way to 2D flow. A good example of this is a bank crest, where flood water from ISIS spills into the TUFLOW domain. The 1D-2D link has been improved by the use of zlines, ensuring the boundary is located on the high point of the channel bank.

7. Stability fixes

7.1 ISIS

Unfortunately the reality of using hydraulic modelling software to represent real world situations is that some assumptions and modifications must be made in order to minimise numerical instability occurring in the model. Whilst these modelling decisions are important when developing a reliable model they are often not recorded. This chapter should be used to record some of the key processes that have been undertaken to limit instability in the model.

The Debenham ISIS model generally has good stability, with the exception of occasional instability spikes associated with the change of mode at orifice units. These instabilities are very minor and have no impact on peak water levels.

Table 7-1: Stability fixes

Instability **Summarise the location, cause and effect of the instability**	**Solution** **Discuss the methods used to reduce the instability and summarise the effect of the measures.**
A slight instability still exists in the model at culvert CHRY_0959c1 and CHRY_0959c2. However, the water level at which it occurs is far below the peak levels for all return periods. There is limited impact on the stage hydrograph and the site is located significantly upstream of the town.	Slot added to the top of the culvert which aids transition between free flow and surcharged flow through the culvert. Any remaining period of non-convergence is deemed suitable given the limited impact this has on results.

7.2 TUFLOW

Table 7-2: Stability fixes

Instability **Summarise the location, cause and effect of the instability**	**Solution** **Discuss the methods used to reduce the instability and summarise the effect of the measures.**
Mass balance issues associated with direct rainfall modelling	Use of “Interpolate ZUV” command smoothes the base topography helping remove erroneous micro topography.
Mass balance issues associated with direct rainfall modelling	Cell wet/dry depth set to 0.0002m. Instabilities often occur when shallow sheet flow is present, which is a regular occurrence when modelling direct rainfall. By setting the wet/dry depth to 0.0002m, TUFLOW assumes cells which contain very shallow flow are counted as dry, removing the instabilities.
Mass balance issues associated with direct rainfall modelling	Various alterations have been made to the ground model to remove mass balance problems. These are associated with areas of poor LIDAR filtering; where the terrain undulates unrealistically this can often cause stability problems.

8. Model runs

8.1 Design runs - fluvial

Table 8-1: Parameters of design runs	
Summarise the purpose of this group of model runs.	Flood mapping
Return periods modelled (yrs)	2-year, 5-year, 10-year, 20-year, 20-year+CC, 75-year, 100-year, 100-year+CC, and 1,000-year+CC
Model start time (hrs)	0 hours
Model run time (hrs)	20 hours
CPU time (hrs)	~ 5 hours
Initial conditions files	Initial conditions stored in ISIS .DAT file.
Map save interval (TUFLOW)	600 seconds
Model names	ISIS: DEBEN_ISIS_2014_009.DAT TUFLOW: Debenham_2014_2m_009_Q****.tcf

8.2 Design runs – direct rainfall

Table 8-2: Parameters of design runs	
Summarise the purpose of this group of model runs.	Flood mapping
Return periods modelled (yrs)	2-year, 5-year, 10-year, 20-year, 20-year+CC, 75-year, 100-year, 100-year+CC, and 1,000-year+CC
Model start time (hrs)	0 hours
Model run time (hrs)	20 hours
CPU time (hrs)	~ 5 hours
Initial conditions files	Initial conditions stored in ISIS .DAT file.
Map save interval (TUFLOW)	600 seconds
Model names	ISIS: DEBEN_ISIS_2014_009_DR.DAT TUFLOW: Debenham_2014_2m_009_Q****_DR.tcf

8.3 Sensitivity runs

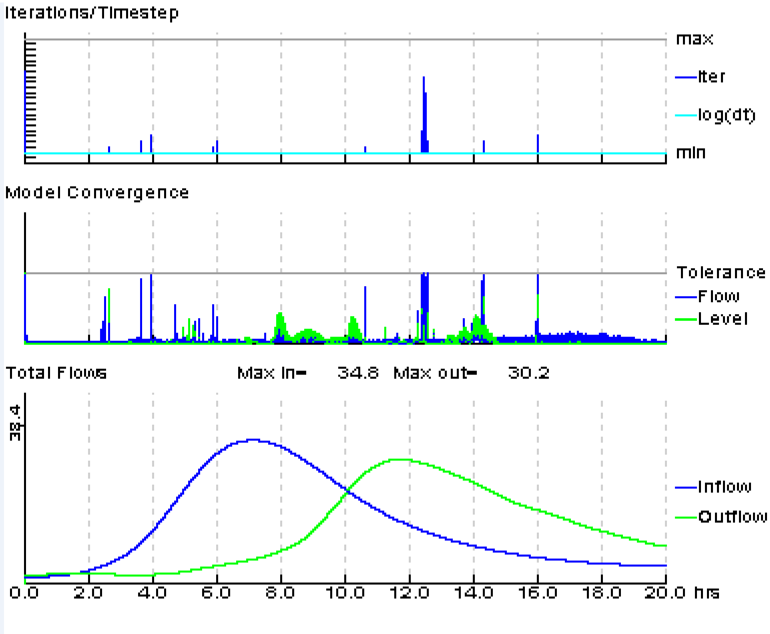
No new model sensitivity runs have been completed for the current project; sensitivity in and around Debenham was assessed as part of the 2013-14 project. Please refer to the relevant reporting for details.

9. Model results

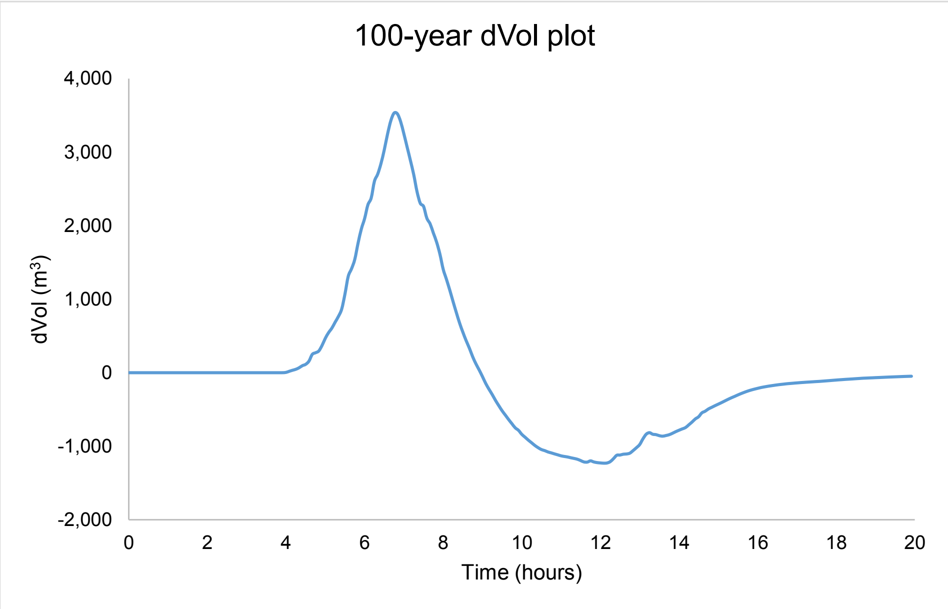
9.1 Summary of the model stability

Table 9-1: Summary of model stability		
Check	**Yes/No**	**Comments**
Is there any non convergence in the ISIS model? If yes where and what steps have been taken to minimise it?	Yes	As discussed in the earlier section of this document, there are a number of very brief periods of non-convergence associated with the Debenham model. These are related to culvert and orifice units, but are not located in the town and do not impact peak water levels.
Are there any warnings and/or checks generated by the model?	No	No

Table 9-2: Model stability plots



The above plot shows the 100-year ISIS run window, indicating non periods of non-convergence.



The plot above illustrates the change in volume (dVol) in the TUFLOW domain throughout the 100-year model run. The smooth dVol plot indicates a stable TUFLOW model, backed up by the mass balance remaining between the recommended +/- 1% throughout the model run.

10. Sensitivity

No new model sensitivity runs have been completed for the current project; sensitivity in and around Debenham was assessed as part of the 2013-14 project. Please refer to the relevant reporting for details.

C Appendix C - Damage estimate check file

Debenham Damage Estimate Check File

Contract

This report describes the damage assessment work commissioned by the Environment Agency, by commission reference AN169. The Environment Agency's representative for the contract was Rebecca Brown. Kevin Haseldine of JBA Consulting carried out this work.

Prepared by .... Kevin Haseldine BSc MSc

Analyst

Reviewed by .... Colin Riggs BSc MSc MCWIEM C.WEM

Chartered Senior Analyst

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1 Introduction

In addition to the production of flood outlines and depth/velocity grids, the Debeham hydraulic model outputs are also required in order to estimate economic damages within the study area. To do so, JBA's own FRISM software has been utilised. This is a geographical information system (GIS) based impact analysis software that computes a range of flood risk metrics, including property damages, based on the techniques outlined in the Multi-Coloured Manual (M-CM, see section 4). Annual average damages (AADs) are also calculated by the software, i.e. the average economic damage which can be expected as a result of flooding in a given year.

2 Available data

In order to calculate damage estimates, the following datasets were used.

- **Hydraulic modelling results**. Water levels were extracted from the ISIS-TUFLOW model discussed above and trimmed to match the flood outlines. FRISM requires these datasets in ArcMap format and therefore these are saved as raster grids.
- **MasterMap data**. Property polygons are used to calculate the building footprint area where no other information is available. These are also essential in defining when a property is flooded; for the purposes of this project if flood water overlaps any part of the polygon the property is adjudged as flooded. Further information is available in section 5.
- **National Receptor Dataset (NRD).** This dataset of national properties was trimmed to the study area and includes information on building type, footprint area, floor level and M-CM code.
- **Threshold survey.** Maltby Land Surveys Ltd was commissioned to undertake a threshold survey for buildings at significant risk of flooding in Debenham. In doing so, accurate elevation data are obtained for properties most at risk of flooding. Outside this zone LIDAR data are used to ascertain average elevation, conservatively assuming no threshold for these properties. The use of online photography to define thresholds was discounted; in order to do so major assumptions would have to be made, such as all properties within a street having the same threshold level.

3 Data preparation

Prior to using FRISM, the NRD data had to be spliced together with the threshold survey (or LIDAR levels where no survey was available) to ensure all properties within the study area were attributed with the threshold value. This was completed using ArcMap.

Secondly, the NRD dataset was trimmed to remove those data points not required for damage calculations. We have used the Environment Agency's Flood Map for Surface Water Property Count[1] guidance document to define which features to remove, which is suitable for use with fluvial events. A record of property types recommended for exclusion is shown below.

[1] Environment Agency., 2010. Flood Map for Surface Water - Property Count Method. Published by Environment Agency.

Table 3-1: Property type exclusions

List of recommended property types for exclusion (from the Environment Agency guidance)		
Adventure playground	Jetty	Public emergency telephone
Aeration	Kiln	Polo
Agricultural showground	Landfill	Pond
Allotment	Landing stage	Pontoon
Apiary	Leisure pier	Post box
Aqueduct	Lime kiln	Public car park
Arboretum	Limestone extraction	Public garden
Armonatic garden	Lock	Public telephone
Ash disposal	Maze	Rabbit farming
Bandstand	Memorial gardens	Reservoir
Basin	Meteorology	Roller skating
Basketball	Mine	Rugby
Bird observatory	Mineral and fuel extraction	Sea fishing
BMX racing	Mineral water factory	Sewage filtration
Boating	Model boating	Sewage outfall
Brine reservoir	Mooring	Sewage pump house
Burial ground	Mussel bed	Sewage pumping
Bus shelter	Nature garden	Sewage recycling
Butterfly farm	Netball	Sewage storage
Chimney	Ornamental garden	Sewage treatment
Coal storage	Osier bed	Shaft
Commemorative garden	Oyster bed	Showground
Crane	Paddling	Skateboarding
Crazy golf	Park	Skiing
Croquet	Pets memorial garden	Slag heap
Dock	Pheasentry	Slate extraction
Dock basin	Pier	Spoil heap
Electricity sub-station	Piggery	Tree nursery
Emergency telephone	Pitch and putt	Vapour stack
First aid post	Play area	Vineyard
Flare stack	Playing field	Watercress bed
Garden of rest	PO box	Waterwheel
Hydraulic power	Point to point racing	Weighbridge

Properties were also removed if it was noted that the building no longer exists. A number of further assumptions have been made and agreed with the Environment Agency:

- All properties designated "potential upper floor" are not included in the damage calculations, but maintained for the purposes of flooded property counts.
- Where the building use is not clear from available photography an assumption is made (i.e. if at the bottom of a garden this is assumed to be a domestic shed).
- The floor area stated in the NRD is used, unless this is missing in which case area is calculated from building footprints.

4 Multi-Coloured Manual[2] (M-CM)

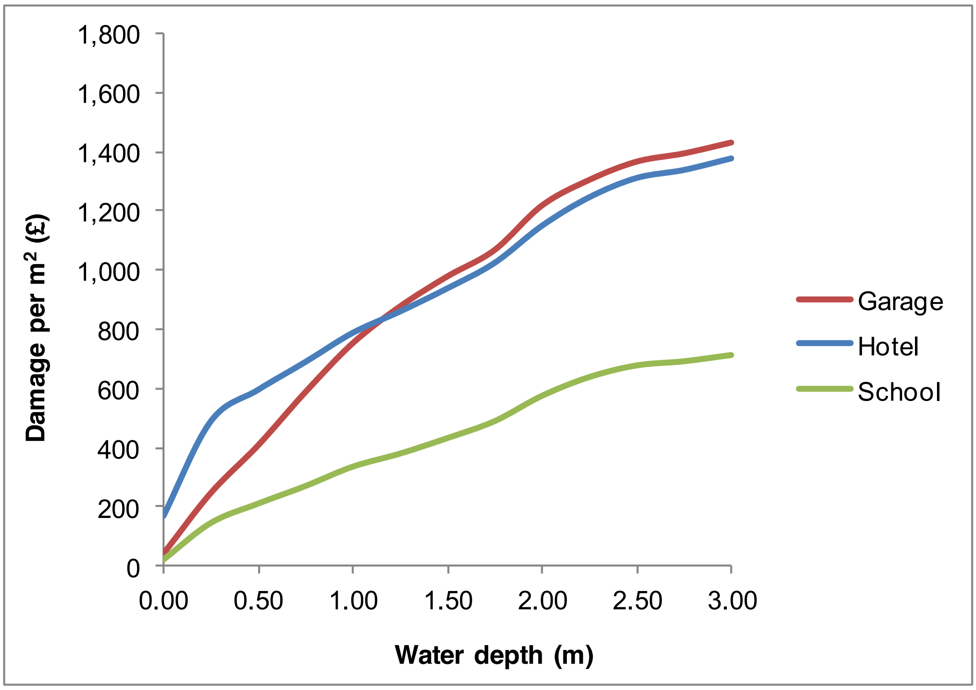
The M-CM was produced by the Flood Hazard Research Centre at Middlesex University to outline the techniques recommended for undertaking evaluations of the benefits for risk management projects. This includes a methodology for calculating expected damages at an individual property for a given flood event. The techniques outlined form the basis of our in-house FRISM tool, used in the current study.

4.1 Non-residential properties

Prior to use of FRISM, an M-CM code value is attributed to each building type within the NRD. This code number relates to a given depth-damage curve, a different curve exists for each individual property type. For example, a 3m deep flood would result in damages of £1,435/m^2 in a garage and £1,376/m^2 in a hotel. Curves are based on national average damages; a selection is illustrated below.

In the current project we have employed the most up-to-date 2013 curves. Non residential properties do not explicitly include basements, although damage is given for 0m depth; therefore a value is often calculated below 0m as results are interpolated between -0.25m and 0m depth.

Figure 4-1: M-CM non-residential depth/damage curves



The assigned M-CM codes have been checked against the building types specified in the NRD to ensure the correct depth/damage curve is applied. In a number of locations the building type is missing; where this is the case types are manually input using photography from surveyors and freely available online photography.

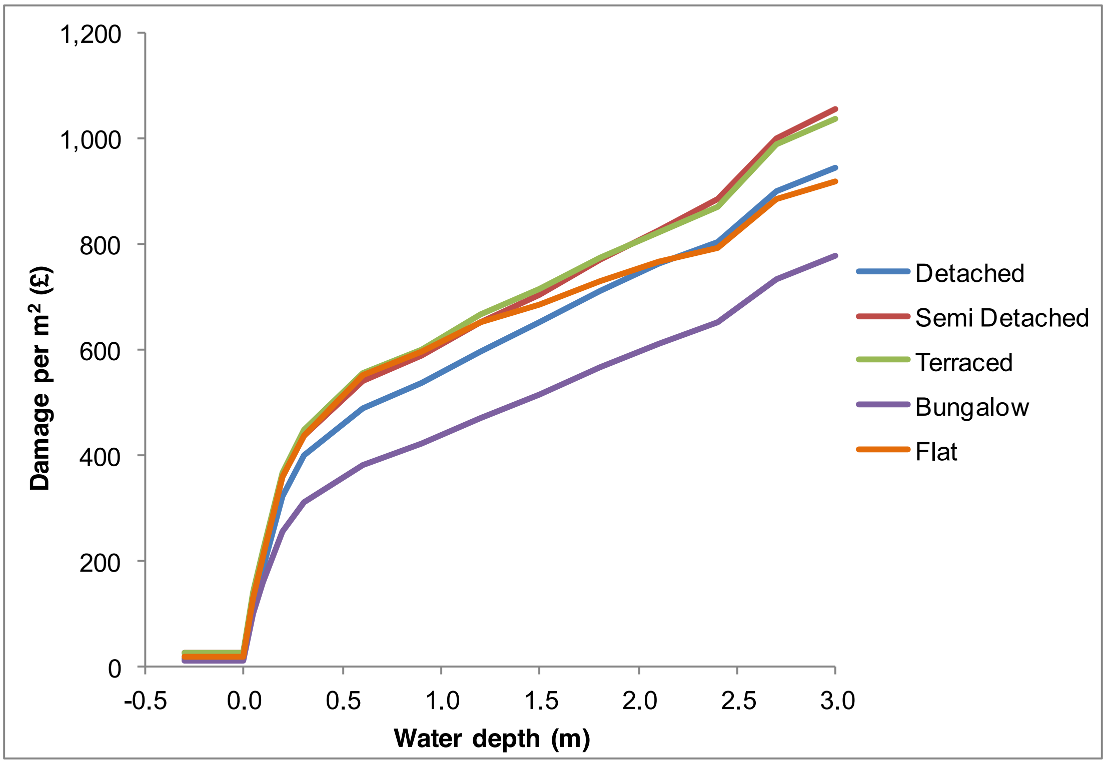
4.2 Residential properties

Residential properties are treated independently to commercial premises, in that the category is further sub-divided to terraced, bungalows, semi-detached, detached and flats. The depth-

[2] Penning-Rowsell, E. *et al.*, 2010. *The Benefits of Flood and Coastal Risk Management: A Handbook of Techniques - 2010 (The Multi-Coloured Manual).* Flood Hazard Research Centre.

damage curves employed are more detailed than for non-residential properties; these are reproduced in Figure 4-2, taken from the M-CM.

Figure 4-2: M-CM residential depth/damage curves



The residential curves used in JBA's FRISM software include an allowance for basements (highlighted by the -0.3m depths shown above), a standard approach outlined in the M-CM. Residential depth-damage curves are representative of national average curves and take account of the fact many properties include basements. For the current project we have not recorded which properties have basements and which do not. It is therefore appropriate to assume the average curve for the UK is suitable for application, so in some instances estimated damages are recorded when the depth of water is between -0.3m to 0.0m.

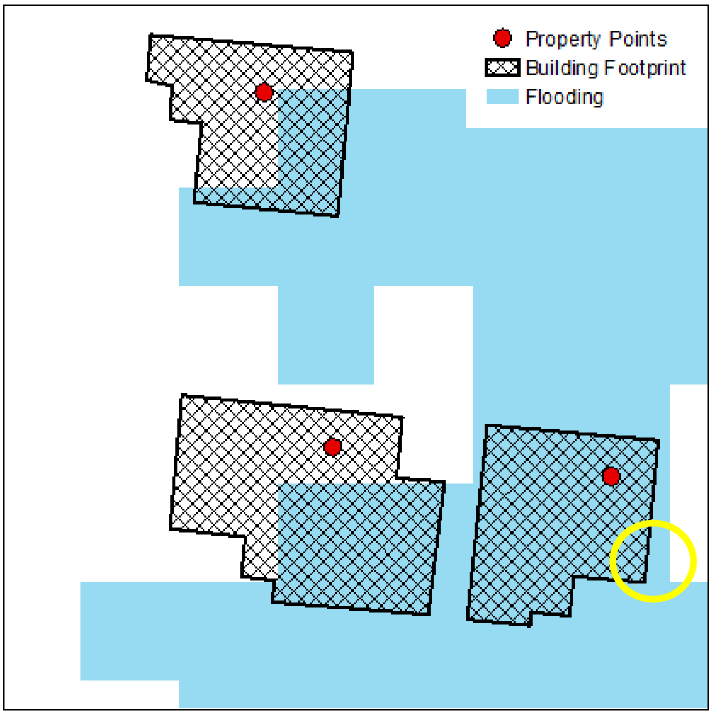
5 FRISM

FRISM computes a variety of metrics by combining flood modelling results together with a range of receptor data discussed above. The metrics that can be calculated depend on both the geometry type of the receptor data and the type of modelling results used. As water level grids were produced for this project, detailed property level analysis was computed and included minimum, maximum and mean depths and damages at each property (based on either the survey property threshold or that extracted from LIDAR). Property level analysis was then summarised over user defined reporting units to determine the total impact e.g. total damages for a particular flood event. As multiple events were modelled, the long term AADs were also computed for each metric.

Firstly, the provided NRD and the MasterMap polygon dataset enabled flooded property counts to be undertaken. Points (which contain attributes to identify their property type, threshold level and floor area) were linked to the building footprint data based on their spatial relationship. A property point was counted as flooded if any part of its associated building footprint intersected with the flood outline.

The detailed count is an accurate counting method and enables properties on the edge of flood datasets to be included resulting in a higher number of properties being counted as flooded. Figure 5-1 demonstrates the principle; in this case a simple count would only result in one property being included (circled) whereas the detailed count results in all three properties whose footprints intersect the flooding dataset being counted.

Figure 5-1: Illustration of detailed property count



5.1 FRISM depth damages

FRISM was used to calculate depth values for each receptor falling within the flood outline. FRISM calculates the maximum, minimum and mean depth of flooding within each building footprint by subtracting the threshold level from the water level grid (produced by the hydraulic model) and attributes these depth values to the property point. As an example, if the mean value of the water level grid at a property was 20.60m and a property threshold of 20.25m was set, the resultant mean property flood depth would be 0.35m.

Damages were calculated at the property level by using the well established methods set out in the M-CM, assigning depth-damage curves for each property type. These methods were implemented in the software as per the HR Wallingford (2008) Technical Note on National Flood Risk Assessment (NaFRA) Economic Calculations[3] and use the look-up tables published in the most recent M-CM update (2013).

The depth was then used in conjunction with property type and the M-CM depth-damage curves to obtain a damage value per metre squared (£/m^2) for the property. This was multiplied by the floor area of the property to obtain a property damage value. The damage estimates were then summarised across each reporting polygon to give a total damage sum for each flood compartment.

Please note:

1. Damage calculations for upper floors were not undertaken, although these were included in flooded property counts.
2. Damages were not capped to maximum property price or valuation.

5.2 FRISM annual average damages

As a range of modelled return periods were available, FRISM was used to apply a probability of occurrence to those results to calculate AAD estimates. Annual average values represent the

[3] Panzeri, M. and Mauz, J., 2008. *NaFRA 2007 Technical Note; Economic Calculations.* HR Wallingford

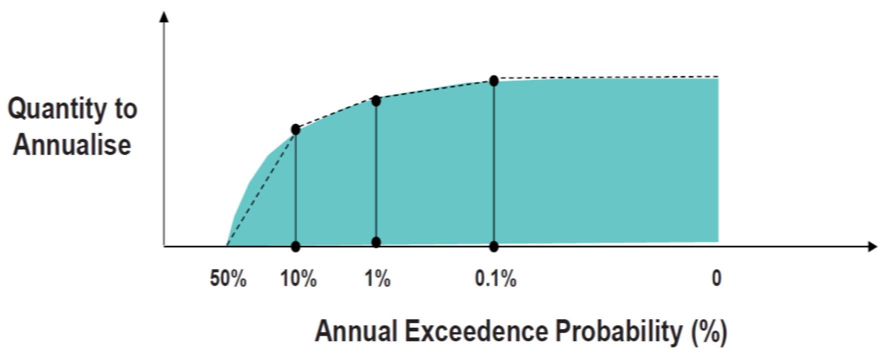
notional long term average or expectation of consequence in any given year and are a useful way of comparing flood risk between different areas.

A simple trapezium based rule was used to calculate these according to return period probability values (Annual Exceedance Probabilities (AEPs)). The equations used to generate the AAD values are recorded in Figure 5-2 below. These account for the fact that greater damages will occur at longer return periods, although the probability of these events occurring is significantly lower than for shorter return periods. The model was run for seven different return periods (2-year, 5-year, 10-year, 20-year, 75-year, 100-year and 1,000-year) to ensure the accuracy of results, as AADs can be overestimated particularly if few high probability events have been modelled.

The principle of the equations is illustrated in the example below, with the area under the curve integrated to give the AAD metric value. Figure 5-2 assumes that the onset of flooding (or zero damages) is the 1 in 1-year event and that the damages for rarer events do not increase beyond those incurred at the 1 in 1,000 (0.001%) AEP event. The 1-year return period (RP) was assumed to be the zero-damage return period for the current study, as some properties were found to flood at the 2-year event.

Figure 5-2: Approximation of annual average risk based on a limited number of events

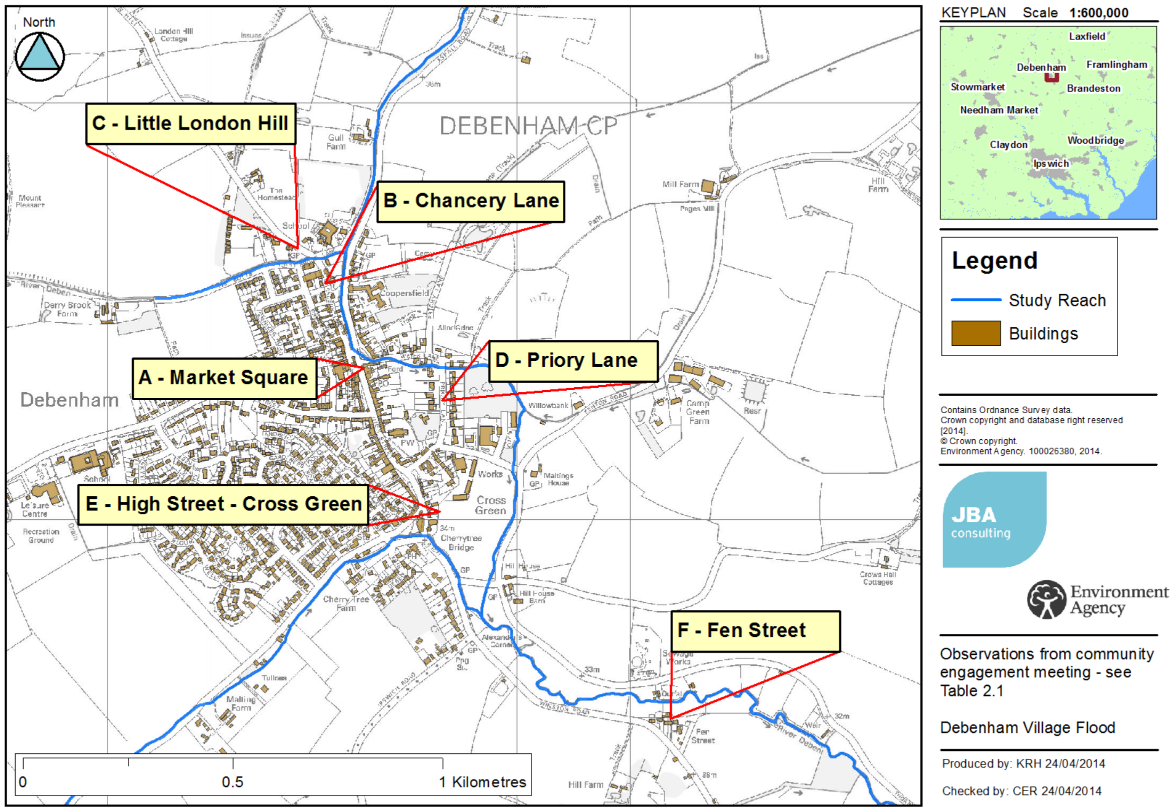
RP (Years)	AEP	Damage (£)	Contribution (£)
1	1.000 (a)	0 (b)	$=(a - a^1) * ((b + b^1)/2)$ = (1.000 - 0.500) * ((0 + 500)/2) = 125.00
2	0.500 (a^0)	500 (b^0)	$=(a^0 - a^1) * ((b^0 + b^1)/2)$ = (0.500 - 0.200) * ((500 + 1,000)/2) = 225.00
5	0.200 (a^1)	1,000 (b^1)	= 150.00
10	0.100 (a^2)	2,000 (b^2)	= 315.00
100	0.010 (a^3)	5,000 (b^3)	= 67.50
1,000	0.001 (a^4)	10,000 (b^4)	= 10.00
AAD = Sum of contributions			**= £892.50**



D Appendix D - Construction costs

E Appendix E - Maps

Figure A-1: Observations from community engagement meeting - see Table 2.1



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