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Modelling Report

Appendix TNA2 – 2012 Hydraulic Modelling Report

Notes:

✓Page 17 - The Model includes the area with 75% development (impermeable surface) and 25% (green open space) permeable

✓Page 24 - All significant structures were included in the model?

✓Page 31 - Model scenarios include summer & winter penning levels

✓Page 33 - Second culvert at New Bow sluice needs automation?

key sentence Page 60 - Flood Extent - 'Post Development there is significant reduction in the flood extents East of the M5 near West wick.

Page 68 - Conclusions.

Weston Villages Strategic Flood Solution
River Banwell Modelling Report

North Somerset Council

March 2013
Revised Final Report
9W7535

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CONTENTS

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1 INTRODUCTION

1.1 Background

Royal HaskoningDHV were commissioned in January 2012 by North Somerset Council, in partnership with the Environment Agency, to undertake the Detailed Design of the Weston Villages Strategic Flood Solution. The Strategic Flood Solution is based on the preferred options from the previous work undertaken in 2007 as part of the Weston Flood Management Study Phase II.

The Weston Villages Strategic Flood Solution aims to develop strategic solutions for delivering a comprehensive flood defence scheme in Weston-super-Mare to protect both existing properties, to facilitate future development in the proposed Weston Villages area and to provide amenity and biodiversity enhancements. The solution is split into two parts; a compound channel on the River Banwell and a 'superpond' on the area around the airfield site adjacent to Cross Rhyne and Hutton Moor Rhyne, which drain into the Uphill Great Rhyne. The two catchments (River Banwell and Uphill Great Rhyne / Cross Rhyne) are distinct catchments and have therefore been modelled separately. IDB water level management can transfer water from some areas of the catchment into either of these catchments, however these flows are limited by a number of structures on the numerous watercourses operated within their system.

This report focuses on the modelling undertaken for the River Banwell only. A separate report has been produced detailing the modelling work undertaken on the Uphill Great Rhyne catchment.

The River Banwell, some 9km in length, is situated north east of Weston-super-Mare. The river rises at a spring at Banwell Village and discharges through New Bow Sluice, a tidal defence structure.

1.2 This Report

This technical report comprises details of the hydrology and hydraulics calculations carried out both in terms of the baseline modelling and the preferred option model. Hydrological and hydraulic assumptions are discussed and model results presented. Details of model calibration and sensitivity are included with results of the existing situation and possible future changes.

1.3 Study Area

The River Banwell, some 9km in length, is situated north east of Weston-super-Mare. The river rises at a spring at Banwell Village (ST 39888 59192) and discharges through New Bow Sluice (ST 35307 66016), a tidal defence structure. The current sluice was constructed in 1990, replacing an earlier sluice of inadequate capacity, and is 0.8km upstream of the confluence with the Severn Estuary.

The River Banwell is largely an artificial channel constructed several hundred years ago to drain not only the spring that rises at the limestone Mendip Hills, but also to drain the surrounding land, which is at an approximate level of 4.7 to 5.5mODN, for agricultural purposes. The majority of the catchment is below the mean high water springs tide level of 6mODN. The gradient of the river channel is very shallow (approximately 2.7m over

9km) and the flow in the river is consequently dominated by daily tide locking at the New Bow sluice.

A location plan is shown in Figure 1-1. This also shows the area of the option modelling.

1.4 Previous Studies

A number of previous studies have been carried out in relation to the River Banwell catchment:

- River Banwell Pre-Feasibility Report, Posford Duvivier, May 1997
- River Banwell Flood Study, Mouchel, 1996
- Weston-super-Mare Flood Management Study (FMS) Phase II (River Banwell), Royal Haskoning, March 2007

These reports have been obtained and reviewed for their relevant content. The FMS study in particular has been referenced throughout this report.

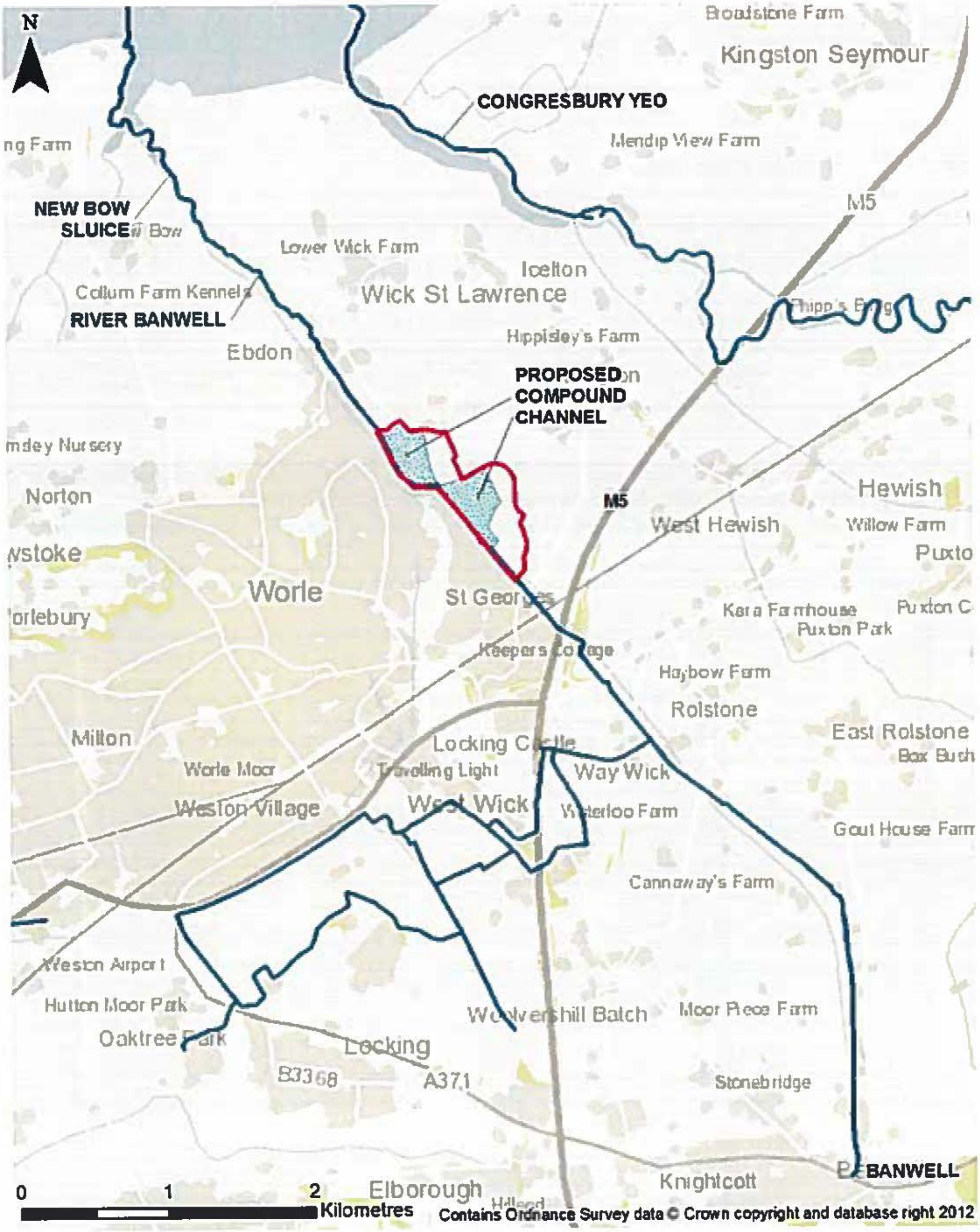


Figure 1-1 – Banwell location plan

2 CATCHMENT INFORMATION

2.1 Geology, topography and soils

The geology of the River Banwell catchment is characterised by Dinantian Carboniferous Limestone and estuarine alluvium (British Geological Survey, 1992). The River Banwell flows northwards from its source near Banwell village to the Severn Estuary. Banwell village is located on the north slope of Banwell Hill, a Dinantian Carboniferous Limestone feature that forms part of the Mendip Hills. Worlebury Hill, Birbeck Point and Middle Hope are limestone outcrops which protrude seawards to the west of the catchment, whilst the estuarine alluvium has been eroded to create Sand Bay and Woodspring Bay.

Topography within the catchment is strongly influenced by the geological character described above. While limestone outcrops, such as Worlebury Hill and Birbeck Point rise to over 100m above sea level, most of the catchment is estuarine alluvium floodplain less than 5m above sea level.

Soils also reflect the underlying geology. The dominant soil type within the floodplain of the River Banwell is loamy and clayey soil, which overlies the estuarine alluvium. This is found in coastal flats and has a naturally high groundwater. The soil is lime-rich to moderate and land cover is typically arable with some areas of grassland. The soil type changes where the estuarine alluvium becomes carboniferous limestone in the south and east of the study area. Two soil types more commonly found are: Lime rich and loamy clayey soils with impeded drainage; and slightly acid loamy and clayey soils with impeded drainage. The natural fertility is high and moderate to high respectively.

2.2 Hydrology and geomorphology

The River Banwell is a largely artificial channel, modified several hundred years ago for agricultural drainage purposes. The hydrology and geomorphology of the Banwell catchment are therefore strongly influenced by historic intervention and management practices, as well as the geology and low-lying topography of the catchment.

The river is a tide-locked watercourse with the majority of the catchment lying below the mean high water spring tide level (6mODN). At high tide, New Bow Sluice is closed to prevent the sea entering the watercourse, which prevents the river draining to the sea for 3 to 4 hours in each tidal cycle (Environment Agency, 1997). There is currently insufficient storage in the system during periods of flood flows to accommodate all the fluvial discharge during the tide locked period and flooding can occur.

Flows within the catchment are dominated by groundwater issuing from the Carboniferous Limestone of the Mendip Hills, which is a major aquifer. The gradient of the River Banwell is only approximately 1 in 3,300. These characteristics result in slow flow and low stream power along the river, together with an attenuated catchment response to rainfall events. The flow regime is further complicated by interactions with the extensive artificial drainage network, operation of flow control structures, pumping of surface waters from low-lying urban areas and the presence of balancing ponds to store surface water. Penning of water throughout the summer has been noted to be exacerbating problems of low flows downstream. Water level management for water resources must be considered in combination with flood risk management to ensure adverse effects from any flow control structures are limited.

The high degree of channel modification along the River Banwell has had an adverse impact on morphological conditions within the catchment. Much of the river has been realigned or re-sectioned, often resulting in a uniform channel with a straight platform and trapezoidal cross-sectional profile. Morphological diversity is consequently low, with limited variation in flow velocities and depths and limited presence of geomorphological features. The channel has been extensively over-widened / over-deepened and thus is effectively disconnected from the floodplain in many places. Stream power and sediment transport capacity is generally low and is further exacerbated by impoundment upstream of flow control structures. Disruption of river continuity has an adverse impact on both sediment transport and the migration of aquatic species, and is likely to result in sedimentation upstream. Direct sediment supply from agricultural runoff is facilitated through the drainage network and could contribute to sedimentation problems. Constriction of the river at some key structures is also creating pinch points that contribute to local flooding.

The morphological condition of the river is also affected by contemporary management practices. The Environment Agency undertake regular maintenance work along the River Banwell to clear weed growth and other obstructions. Maintenance of vegetation is undertaken according to two regimes, dredging and weed-cutting. The dredging is planned for once every 8 to 10 years depending on the siltation levels, which are monitored annually. Due to low siltation the channel has not been dredged for approximately 30 years. Weed-cutting is then undertaken on the right bank of the River Banwell annually. Both of these maintenance techniques have a high impact on marginal and riparian habitat conditions. Riparian vegetation is typically uniform along the River Banwell, predominantly comprising grassed banks that offer little shelter or shading for aquatic species. There is opportunity to reconsider alternative maintenance regimes as part of flood risk management.

2.3 Historical Flooding

Flooding of the Banwell catchment occurred in 1968 following a storm over the Mendip Hills. The flooded areas included Banwell Moor to the north of Banwell Village, part of St Georges Village and an area between St Georges and West Wick. It was noted that the River Banwell continued to rise for approximately six days after the storm, illustrating the slow response time of the catchment. It should be noted that at this time the catchment and watercourse were significantly different in both alignment and cross section. St Georges had very little development and the M5 motorway had not yet been built.

Flooding occurred more recently in January 2002 following heavy rain throughout the catchment. The flooded area was Banwell Moor between East Moor Rhyne and Middle Moor Rhyne. It is not thought that any property was flooded however flood waters rose close to Moor Dairy and Moorlands Farm.

Anecdotal evidence suggests that minor flooding events have occurred in low lying areas such as the airfield and the railway triangle in recent years.

3 DATA COLLECTION

3.1 Introduction

This section of the report describes the various data obtained and comments on the validity and use of the data in the modelling phase of the project.

3.2 Topographic Survey

The majority of the topographic survey information used as part of the modelling was from previous studies. This was then supplementary by a small survey in key locations commissioned as part of this investigation. The sections below provide details of the previous topographic surveys undertaken in this area, along with the new survey undertaken in May 2012.

3.2.1 Historic Survey Data

Survey information was provided by the Environment Agency from a survey carried out in 1978; however the information comprised cross section data only. Without the location points provided on a long section it was not possible to accurately geo-reference the location of the cross sections. Some of these cross sections had been resurveyed in 1984 to form part of a hydraulic model however this survey was not found.

3.2.2 Environment Agency Bridgwater (2005)

Since the original survey was twenty five years old it was considered appropriate as part of the previous FMS Phase II study to undertake a survey to provide current cross section data. Therefore a topographic survey of the River Banwell was undertaken in May 2005 by Environment Agency officers. This survey comprised river sections below the water level using an Acoustic Doppler Current Profiler. These sections were tied into the Global Positioning Network (GPS) by measurement of water level on the day of the survey.

3.2.3 Royal Haskoning (2005)

To supplement the Environment Agency river sections, surveys of the structures along the River Banwell were carried out by Royal Haskoning. This was done by a level survey with some details being taken from as built drawings of structures.

3.2.4 Land and Sea Survey (Various)

Land and Sea Surveys had undertaken topographic survey of part of the Rhyne system (Grumblepill Rhyne) for Mead Homes Ltd as part of their development proposals. This previous survey was provided for this commission.

During the modelling, there was a need for additional survey to provide certainty on the levels of key structures. Due to their knowledge of the area, Royal HaskoningDHV commissioned Land and Sea Surveys to survey several cross sections and structures on the Grumblepill, Wolvershill, Locking and Cross Rhynes. During this additional survey, Land and Sea Surveys were also able to provide clarifications regarding queries that arose from their previous surveys of the area.

3.2.5 LiDAR Data

Updated LiDAR data (filtered and unfiltered DTM and DSM), flown in 2011, was provided by the Environment Agency in January 2012. This data was used to extend model cross sections and for definition of the storages areas and flooded extents.

3.3 Hydrological Data

3.3.1 Stage Gauges

There are three Environment Agency gauges on the river Banwell which record stage, details of which are listed below:

- St Georges, record length 11 years, 15 minute level gauge;
- Waterloo Bow, record length 3 years, 15 minute level gauge;
- Banwell Spring, record length 6 years, 15 minute level gauge with rating curve.

The years of data given above are the number of water years the gauge has been active for. In all sets of data there are periods of time where the gauge was faulty or not recording, leaving gaps in the data.

3.3.2 Rain Gauges

There is one rain gauge within the catchment which is a Tipping Bucket Rain gauge (TBR). This provides readings after every 0.2mm of rainfall. This gauge is located at St. Georges.

In the absence of any other rainfall data in the catchment it was necessary for hydrological investigations to assume that rainfall is constant throughout the catchments. As the catchment is approximately 18km^2 and relatively flat this is thought to be a fair assumption. Where available, radar data for the events were received to confirm the assumption of catchment wide storms.

3.4 Tide Data

There is an Environment Agency stage gauge both upstream and downstream of New Bow sluice. This provides level data recorded every 15 minutes. Data for the entire record of this gauge, from March 1995 to 2005 was obtained. However there are large gaps in this data record. To fill these gaps, recorded tide levels at both Hinkley Point and Avonmouth were obtained from the National Tidal and Sea Level Facility and a level at the mouth of the River Banwell was developed based on the level relationship between these two stations along the River Severn.

Extreme tide levels for New Bow are reproduced from *Coastal Flood Boundary Conditions for UK Mainland and Islands report, SC060064/TR2: Design Sea Levels, February 2011*, published by the Environment Agency. This replaces the *Environment Agency, South West Region, Report on Regional Extreme Tide Levels, 2003*, which was used in the previous study. Estimates of tide levels and extreme water levels at the site are given in Table 3-1. These values are for a base year of 2008 with an increase of 3.5mm per year based on Planning Policy Statement 25 (PPS25) guidance to bring them up to 2012 levels. Note that ordnance datum is 6m below chart datum in this area.

Table 3-1 – Tide levels and Extreme Water Levels at Weston-super-Mare and New Bow Sluice (2012)

Return Period	Weston-super-Mare (mODN)	New Bow Sluice (mODN)
MLWS	-5.2	-4.91
MHWS	6	6.29
1 year	7.61	7.90
5 year	7.81	8.11
10 year	7.93	8.24
25 year	8.06	8.39
50 year	8.18	8.51
100 year	8.30	8.63
200 year	8.42	8.76
1000 year	8.75	9.08

*Ordnance Datum is 6m below Chart Datum

The prediction point for water levels at Weston-super-Mare is a short distance along the coast from the outlet of the River Banwell at New Bow sluice. Therefore in order to determine an appropriate tidal climate for the hydraulic model, the relationship of water levels along the coast (River Severn) from the latest tide levels released by the Environment Agency was used to depict a MHWS level. An additional 290mm was added to the prediction given in Table 3-1; a MHWS peak tide of 6.29mODN has been utilised in this study.

3.5 Climate change

Climate change has been considered using PPS25 guidance. To ensure a precautionary approach is taken the upper end estimate has been used for all assessments. Table 3.2 shows the predictions for sea level rise over time for the guidance.

Table 3.2 – Sea level rise predictions

Epoch	PPS25 (South West)
	Sea level rise (mm/year)
Present day up to 2025	3.5
2026 – 2050	8.0
2051 – 2080	11.5
2081 - 2115	14.5

Based on the information in Table 3.2, PPS25 guidance predicts that by 2112 sea levels will have risen by approximately 1.05m.

In terms of fluvial flows a standard increase of 20% will be applied, in accordance with PPS25 guidance.

3.6 Other Data

3.6.1 Royal Haskoning (2007)

As part of the FMS Phase II study, HECRAS models of the River Banwell and the West Wick Rhyne had been developed by Royal Haskoning using the data outlined in Sections 3.2. to 3.4. The data in these models were used to create the new ISIS model of the Banwell catchment, supplemented with the additional survey information obtained in 2012.

3.6.2 Drawing

A series of technical drawings have been received from the Environment Agency, IDBs and other parties regarding structure dimensions (such as New Bow Sluice), structure locations and types and typical drainage pathways (many of the IDB's Rhyne systems can flow in different directions depending on management practices / penning). These drawings have been utilised to help in the schematisation and detail of the hydraulic model.

3.6.3 Additional Information

Information from the Environment Agency, IDB and other parties was collected throughout the study. The data collected included information on Penning levels at New Bow Sluice (Environment Agency) and for different locations for both summer and winter within the IDB system.

Other data obtained for the use in this study include the Environment Agency Flood Zone 2 and 3 extents (Environment Agency). These extents were based on the outputs of the 2007 FMS Phase II modelling.

4 BASELINE HYDROLOGICAL ASSESSMENT

4.1 Catchment boundaries and sub-catchments

During the 2007 study much consideration was given to the catchments and sub-catchments for both the River Banwell and the Uphill Great Rhyne, particularly the split between the two catchments. This involved discussions with the Internal Drainage Board (IDB) and Environment Agency, utilising their local in depth knowledge. These catchments were, therefore, not adjusted as part of this revised hydrological assessment. However, part of the catchment that was previously thought to drain entirely into the Wessex Water system in Uphill has been added into the assessment to account for exceedence of the drainage system.

The catchment boundaries for the two catchments are shown in Figure 4-1.

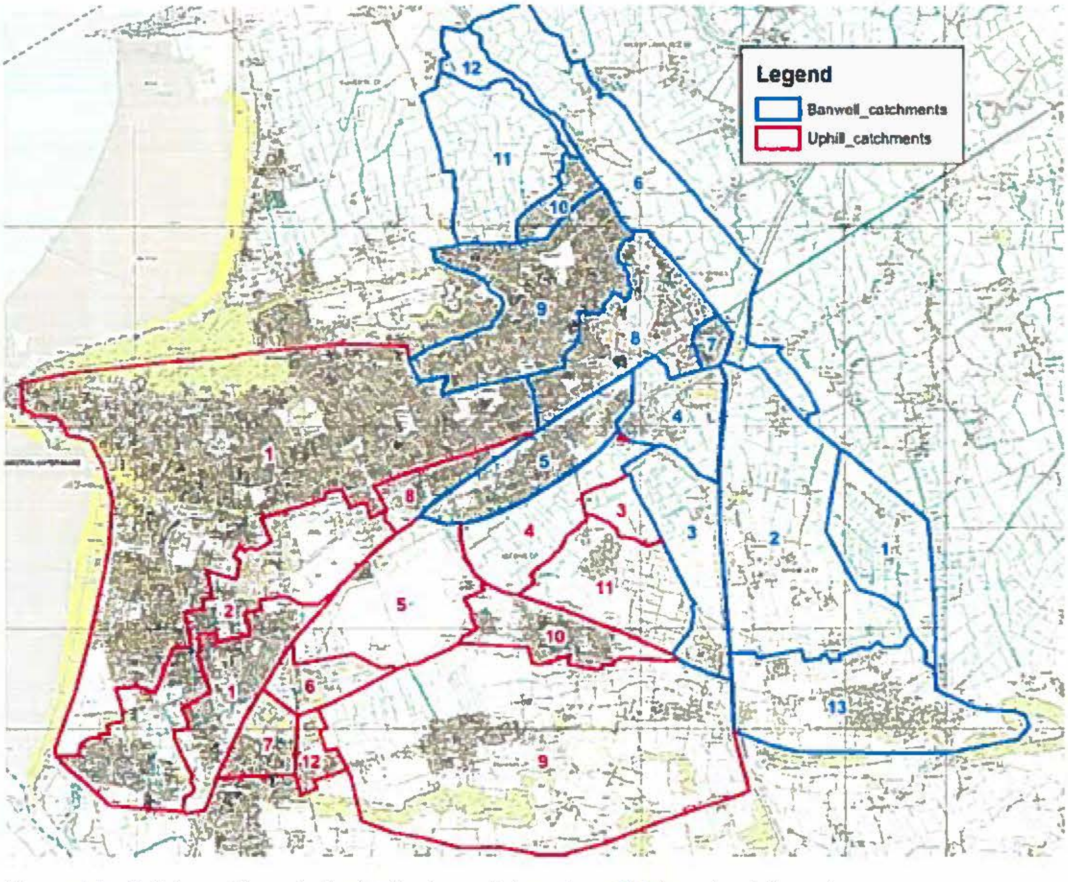


Figure 4-1 – Catchment boundaries for the two catchments, split into sub-catchments

4.2 Catchment Background

The River Banwell catchment is approximately 18km^2 and extends from the Mendip Hills, to the South of Banwell village, to the coast. The catchment comprises both rural and urban areas with the low lying rural areas of the catchment having an extensive Rhyne system.

For the purpose of hydrological analysis the catchment was initially considered as a whole catchment and later divided into 13 sub-catchments. This sub-catchment division was undertaken with the IDB to ensure that their knowledge of the area was fully incorporated into the hydrology. This was done originally for the Phase II study and reviewed as part of this assessment. No changes were made as a result of the recent review. Table 4.1 provides a summary of the sub-catchments and their areas, the locations of which are shown on Figure 4-1.

Table 4.1 River Banwell Sub-Catchments

Catchment	Catchment Name	Drainage Path	Area km^2
1	Banwell Moor	West Moor Rhyne	1.20
2	Wolvershill	West Moor/Way Wick	3.31
3	Grumblepill	Way Wick	1.28
4	West Wick	Way Wick	0.79
5	Moor Lane Development	Way Wick	0.95
6	East of River Banwell	Lateral	2.63
7	St. Georges(Willow close)	D/S M5	0.13
8	St. Georges	Walford Avenue Outfall	1.36
9	Worle and North Worle	North Worle Pumping Station	2.13
10	Ebdon Grounds	U/S Ebdon Bridge	0.29
11	Northfield Rhyne	Northfield Rhyne	1.55
12	New Bow Sluice area	Lateral	0.32
13	Banwell Village	Banwell Spring	2.33

4.3 Connection of the two systems

Following discussions with the IDB and review of the topography it was clear that both Rhyne systems are heavily dependent on the penning structures and water can flow in both directions depending on the settings of key structures. This means that at times water can flow from the Uphill system towards the Banwell and vice versa. This would be particularly relevant during a blockage scenario. To account for this inflow CR2_5553 and Area 3 have been included in both models, therefore, ensuring the worst case scenario is considered.

4.4 Previous investigation (2007)

As part of the Phase II study a detailed hydrological assessment was undertaken. This looked at the two approved Flood Estimation Handbook (FEH) methods available at the time i.e. FEH Statistical Pooling Group Method and the FEH Rainfall Runoff Method. The FEH Statistical Method produced significantly lower flow estimates than the FEH Rainfall Runoff Method. In addition, due to the small size of some of the sub-catchments it was felt that the FEH Rainfall Runoff method was most suitable for the area as a whole, and gave the most precautionary flows. The preferred method was, therefore, the FEH Rainfall Runoff method. For the Banwell this included the use of observed data to improve the time to peak and baseflow estimates, however, due to a lack of site data these observed improvements were not possible for the Uphill and Cross Rhyne catchments.

Details of the previous hydrological analysis can be found in the 2007 River Banwell Modelling report and the 2007 Uphill Modelling Report.

The decision to use the FEH Rainfall Runoff Method was considered to be appropriate for this study and was, therefore, carried forward as part of the revised hydrological assessment. Use of the Revitalised Flood Hydrograph Method (ReFH) was also considered; however, it is not suitable for urban catchments and for situations where the duration used is much larger than the critical storm duration. This study focusses on the longer duration storms and therefore the ReFH method was not considered to be suitable for this study.

4.5 Summary of review process

The original hydrological assessment undertaken for the Weston Flood Management Study Phase I work was carried out in 2005 – 2006. This used version 1 of the Flood Estimation Handbook to determine the catchment characteristics in the form of catchment descriptors. Since that study, revisions have been made to the FEH catchment descriptors and the latest version is now version 3. As part of the detailed design of the Weston Villages Strategic Flood Solution the catchment characteristics have been reviewed as summarised below.

The catchment descriptors from version 1 and version 3 of the FEH CD-ROM for both catchments were compared. These showed only minor differences, all of which were within the typical range as specified in FEH supplementary report Chapter 3.3. The catchment descriptors for the sub-catchments were also reviewed, with a focus on the key catchment descriptors utilised as part of the FEH Rainfall Runoff Method.

Generally the only catchment descriptors which showed noticeable differences were the AREA and therefore the DPLBAR, along with the URBEXT. It is not a surprise that the urban extent has changed over time and therefore, this was investigated in more detail using aerial photography. In terms of AREA, the catchments had been discussed with the IDB as part of the previous study and so we are confident in the AREA values used. The only change in area relates to the Wessex Water flow as detailed above. DPLBAR is dependent on AREA and therefore, it follows that any changes in AREA has an impact on the DPLBAR. As for the whole catchment, all of the differences are still within the specified ranges quoted in the FEH supplementary report Chapter 3.3.

Key elements to this study are the urban extent and percentage runoff. A review of the SPR values used was undertaken which showed that generally the values had increased by 1.5% since the previous study. As part of the 2007 calibration process a 30% increase had been applied to the SPR values and therefore this increase has been included within that factor. No new information is available to confirm the SPR values and therefore, it was agreed that this would be investigated further as part of the model calibration process.

The urban extent was reviewed by considering the previous URBEXT value used and what classification it fell into according to FEH Volume 5 Chapter 6 (i.e. essentially rural, slightly urbanised, moderately urbanised, heavily urbanised, very heavily urbanised and extremely heavily urbanised). This was then compared to the Mastermap classifications for the area and aerial photography using Google Earth. Generally the Mastermap and aerial photography analysis provided the same urban category as the previous FEH values, with some places having a higher urban classification based on FEH than the other methods. No changes were therefore made to the baseline urban extent value.

4.6 Calibration

Due to the presence of gauges within the Banwell catchment, calibration of the model was possible. This was also thought to be the most suitable way to review and revise the hydrology for the catchment. The same calibration events were used as for the previous 2007 modelling i.e. 28th October 2000, 24th November 2000, 27th November 2001.

Based on the previous hydrology the resulting water levels for each of the calibration runs were significantly lower than observed at the gauges. Therefore tests were undertaken focussing on the time to peak of each catchment, the percentage runoff and the unit hydrograph to improve the calibration. The use of lowland shaped unit hydrograph was also considered; however, this was not found to provide a good match with the observed data. Following a meeting with hydrologists acting on behalf of the developers for the Weston Villages (29th August 2012), the Catchment Wetness Index (CWI) was also investigated. This process resulted in amendments to the above parameters, details of which are provided in Section 7.

4.7 Resulting design parameters

The section below summarises the hydrological parameters used for the River Banwell, and changes that have been made since the previous hydrological assessment in 2007. The justifications for these changes are documented in section, which deals with model calibration discussion in Section 7.

- No changes to catchment boundaries or areas;
- Rural time to peak changed from approximately 24 hours to 10 hours based on the model calibration;
- Urban time to peak changed from the catchment descriptor value of between 1-2 hours, to a set value of 2 hours based on the model calibration;
- Rural standard percentage runoff changed from FEH values to FEH values x1.3 based on model calibration;
- Urban standard percentage runoff changed from FEH values to FEH values x1.3 based on model calibration;
- Non-standard unit hydrograph shape adjusted to take into account the partly lowland nature of the area;
- Rural catchment wetness index (CWI) adjusted from FEH values of approximately 118 to an observed value of 160 from the October 2000 event. ; and
- Urban catchment wetness index (CWI) maintained at the FEH value of 116 – 118.

4.8 Design peak flow estimates and hydrographs

Using the parameters detailed above resulted in the peak flow estimates shown in Table 4.2 and the 100 year hydrographs in Figure 4.2. Note these estimates are based on the critical storm duration for the catchment, 51 hours. Other durations were tested (as detailed in Section 6) which resulted in slight changes to these values.

Table 4.2 – Peak flow estimates for the critical storm duration

Inflow	Peak flow (cumecs)							
	2yr	5yr	10yr	20yr	50yr	100yr	1,000yr	100yr+CC
1	0.45	0.55	0.62	0.69	0.79	0.86	1.23	1.04
2	0.88	1.17	1.34	1.54	1.81	2.02	3.01	2.42
3	0.31	0.42	0.49	0.57	0.68	0.76	1.14	0.91
4	0.20	0.27	0.31	0.36	0.42	0.47	0.71	0.57
5	0.25	0.34	0.39	0.45	0.53	0.59	0.88	0.71
6	0.63	0.85	0.99	1.15	1.36	1.52	2.29	1.82
7	0.03	0.04	0.04	0.05	0.06	0.07	0.10	0.08
8	0.32	0.43	0.50	0.58	0.69	0.77	1.15	0.92
9	0.50	0.68	0.79	0.92	1.08	1.21	1.81	1.45
10	0.07	0.10	0.12	0.13	0.16	0.18	0.26	0.21
11	0.09	0.15	0.18	0.22	0.28	0.33	0.56	0.39
12	0.08	0.11	0.13	0.14	0.17	0.19	0.29	0.23
13	0.42	0.48	0.51	0.56	0.61	0.66	0.87	0.79
CR2_5553	0.92	1.26	1.47	1.70	2.02	2.26	3.44	2.71
TOTAL	5.14	6.83	7.89	9.05	10.65	11.87	17.74	14.24

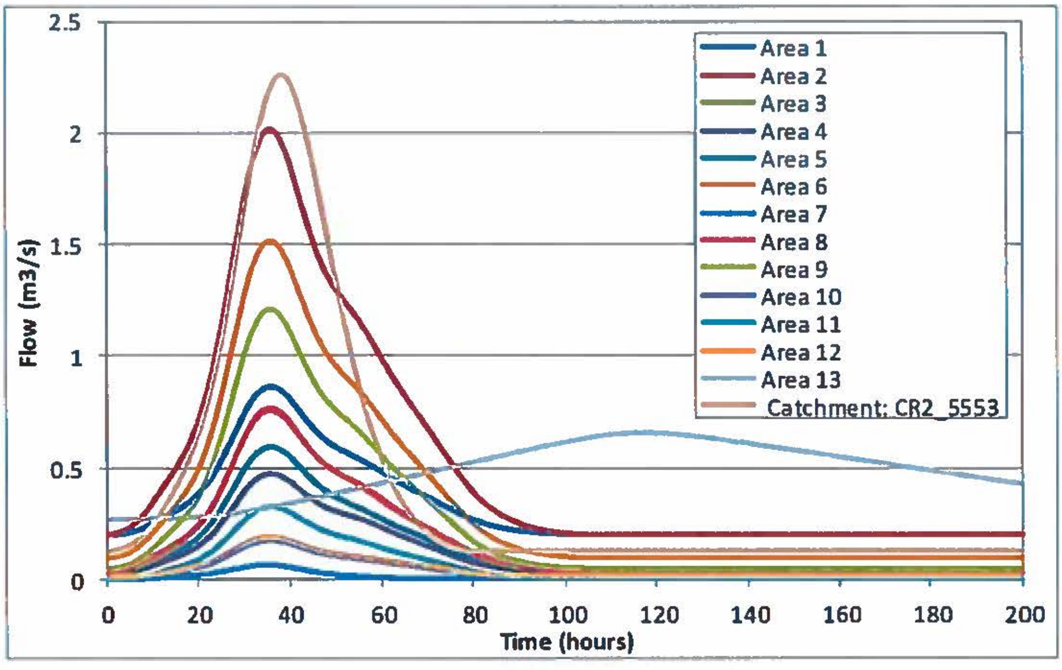


Figure 4-2 – 100 year pre development inflow hydrographs

5 POST DEVELOPMENT HYDROLOGICAL ASSESSMENT

5.1 Introduction

The development of fields / open grassland leads to an increase in the amount of impermeable land within a catchment. This results in less water infiltrating into the ground during a rainfall event and therefore more surface water runoff. The urbanisation of an area also increases the speed at which a catchment / area of a catchment reacts to rainfall.

An assessment has been undertaken to determine the impact of the proposed development on both the percentage runoff of the River Banwell and Uphill Great Rhyne catchments and the urban extent within the catchments. This method uses a hybrid approach of the Modified Rational Method and the FEH Rainfall Runoff Method.

The Modified Rational Method considers the proposed land uses of the development, and therefore the likely area of additional impermeable land that previously was permeable. It then uses land use coefficients to determine a post development percentage runoff for the catchment. This post development percentage runoff has then been input into the FEH Rainfall Runoff boundary units within the ISIS model to determine the impact this increased percentage runoff has on the flow estimated to be entering the watercourse for rainfall events for various magnitudes.

In addition, the urban extent has also been adjusted post development and input into the same FEH Rainfall Runoff boundary units.

5.2 Proposed development details

Figure 5.1 shows the areas of development highlighted by North Somerset Council in the adopted Weston Villages Strategic Planning Document (SPD), June 2012. This shows the key areas to be considered as part of this investigation.

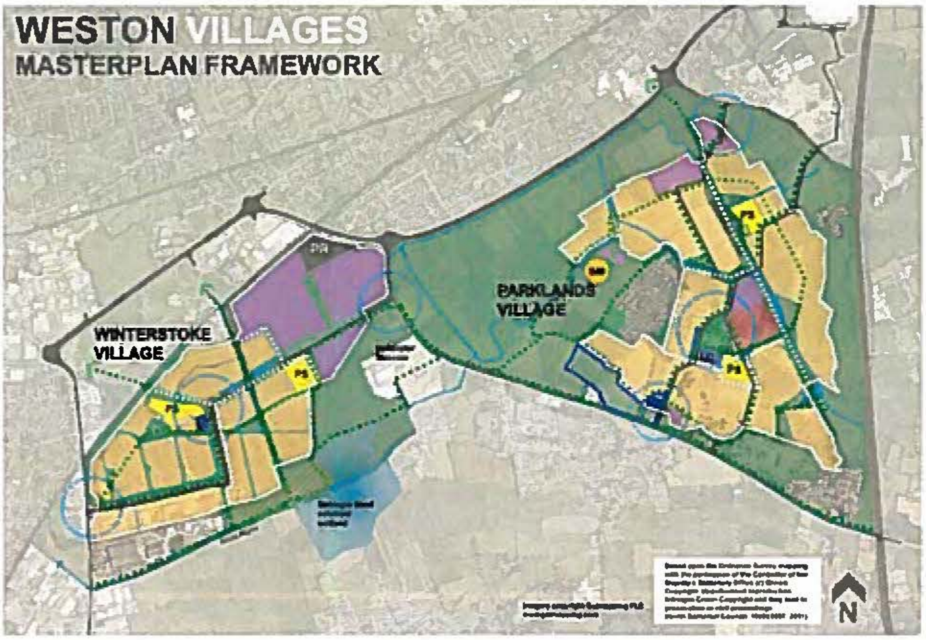


Figure 5.1 – Weston Villages Master plan as shown in the SPD, June 2012

The impact of development was dealt with in the previous FMS Phase II assessment by assuming that 75% of the development area would be impermeable, 25% remaining permeable. This assumption was applied for all developments. The master planning of a number of the developments has progressed since the Weston FMS Phase II study and therefore more detail is known regarding the proposed land use types and therefore the resulting impermeable areas. Master plans were obtained where available from the respective developers. Where developments are not yet at the Master plan stage the previous assumption of 75% impermeable was applied. Figure 5.2 shows the development plot boundaries. Details of the proposed land use within each development are provided in Appendix C.

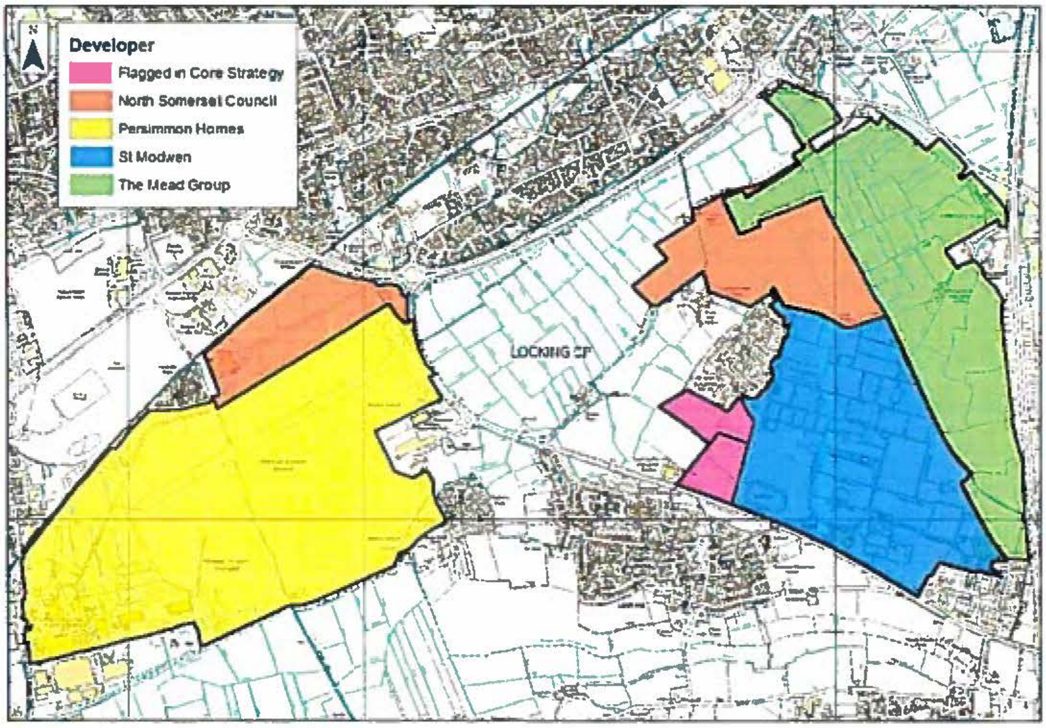


Figure 5.2 – Development plots

5.3 Impact of the development

5.3.1 Percentage runoff

Along with information regarding the land use plots proposed for each development, the developers also provided assumptions based on the percentage of impermeable area for each land use. These varied between the different developers, depending on their individual plans. Table 5.1 below summaries the values used.

Table 5.1 – Assumptions regarding impermeable areas for various land uses

Land use	Percentage impermeable area
Residential	50 – 65 %
Employment	70 – 75 %
School	70 – 85%
Highways / infrastructure	100%

Land use	Percentage impermeable area
Mixed use	75 - 85 %
Formal pitches	10%

The aim of this assessment was to consider the combined impact of all of the development on the response of the River Banwell and Uphill Great Rhyne & Cross Rhyne catchments. The impact of the development has therefore been taken into account by considering how the development alters the percentage runoff of the inflow catchments and the urban extent. This was assessed individually for each development, and in combination, so that the increase in flow and volume within each catchment can be attributed to each development in a consistent way. A standard proforma has been set up to record the relevant development information, assumptions made, and the resulting increases in flow and volume. A proforma has been produced for each development and is provided in Appendix C.

We have used a hybrid methodology to assess the impact of the development on the catchment flows. This hybrid method uses the land use coefficients from the Modified Rational Method and applies them to the pre-development runoff rates used in the FEH Rainfall Runoff Method.

The methodology is as below:

- Land use coefficients for impermeable land e.g. tarmac, roofing etc. have been assumed to be 0.9. This reflects the fact that the majority of the rainfall onto impermeable areas will become direct runoff.
- Usually green fields / open space are given a land use coefficient of 0.3 to reflect the fact that a high proportion of the rainfall can soak into the ground and therefore the direct runoff is relatively low.
- The percentages of each catchment that are impacted by the developments were calculated (Equation 5.1) based on the effective size of the development i.e. excluding the green open spaces, along with the percentage area of each development that will be impermeable. These calculations were based on the Master plans provided by the developers. Where Master plans were not available an assumption was made that 75% of the development would be impermeable and 25% green open space.
- The pre-development percentage runoff determined during the model calibration was then adjusted using Equation 5.2 to determine a post-development percentage runoff.
- The post-development percentage runoff was input into the FEH Rainfall Runoff boundary units in ISIS to determine the impact of the development on the flow for each catchment, and the resulting additional volume that needs to be stored by each scheme. This was tested for various duration storms and return periods.

$$\text{Proportion of the catchment impacted by development} = \frac{\text{Effective development area}}{\text{Catchment area}} \qquad (5.1)$$

* Effective development area is the development area minus any green open spaces

$$PR_{post} = (PCD \times I \times a) + (PCD \times (1 - I) \times b) + ((1 - PCD)PR_{pre}) \qquad (5.2)$$

Where:

- PR_{pre} = catchment percentage runoff pre-development
- PR_{post} = catchment percentage runoff post-development
- PCD = proportion of the catchment impacted by the development
- I = proportion of the development that will be impermeable (i.e. 0 – 1 where 0 is all permeable and 1 is all impermeable)
- a = land use coefficient for impermeable land
- b = land use coefficient for permeable land / green open spaces

For example, if the development covers 40% of a catchment, 75% of the development will be impermeable, and the pre-development percentage runoff is 30% then the post development percentage runoff would be 48% as shown below:

$$PR_{post} = (0.4 \times 0.75 \times 0.9) + (0.4 \times 0.25 \times 0.30) + ((1-0.4) \times 0.30) = 0.48 = 48\%$$

This was undertaken for each catchment based on two scenarios:

- All of the development is in place – this provides an indication of the impact of the proposed new villages as a whole and the required storage volumes for the schemes
- Each individual development separately – this provides details of how the total volume is split between the individual developments.

5.3.2 Urbanisation

In addition, the urban extent has also been adjusted post development and input into the same FEH Rainfall Runoff boundary units. URBEXT is the catchment descriptor used within the ISIS boundary unit to represent urban extent. This takes into account the intensity of the urbanisation as well as the size. URBEXT is therefore calculated based on Equation 3 below, using Land Classification Map 2000.

$$URBEXT_{2000} = URB_{EXT} + 0.5\ SUBURB_{EXT} + 0.8\ IBG_{EXT} \qquad (5.3)$$

Where URB_{EXT} is the extent of heavily urbanised areas e.g. town and city centres, whilst $SUBURB_{EXT}$ is the extent of moderately urbanised areas e.g. villages, residential area, some industrial areas, and IBG is the Inland Bare Ground.

Based on the vision of the villages in the Weston Villages Strategic Planning Document, and the Master plans provided by the developers, we have assumed that all of the area will be classified as moderately urbanised rather than heavily urbanised. The post development URBEXT value ($URBEXT_{post}$) has therefore been calculated using Equation 5.4, where the $URBEXT_{pre}$ is the pre-development URBEXT value.

$$URBEXT_{post} = URBEXT_{pre} + (0.5 \times PCD) \qquad (5.4)$$

For example, for a rural catchment ($URBEXT_{pre}$ of 0.004), where the development will cover 40% of the catchment area, the post development urban extent will be 0.204, which would be classified as a moderately urban catchment.

5.4 Post development design flows

The parameters for standard percentage runoff and URBEXT were calculated for all of the developments combined, shown in Table 5.2, and then input into the FEH boundary units in ISIS to determine the post development hydrographs. For the Banwell only Area 3 and CR2_5553 were affected by the developments. The catchment CR2_5553 has been included in this assessment as a worst case scenario to reflect the chance that based on certain IDB penning conditions, this catchment can flow into the River Banwell system. Table 5.3 shows the impact of the development on the peak flows for these sub-catchments whilst Figure 5.3 and Table 5.4 shows the impact on the hydrographs. These are based on the critical storm duration for the catchment of 51 hours. As stated previously, within the hydraulic modelling various storm durations were tested. Comparisons between the pre development and post development water levels were made using the same storm durations to ensure the comparison was like-for-like.

The time to peak values were not adjusted following the developments because the proposed developments still meant the catchments are relatively rural, with lots of grass areas. The changes to the URBEXT and percentage runoff were therefore sufficient to account for the development and the change to the response, as shown particularly by CR2_5553 inflow in Figure 5.3.

Table 5.2 – Impact of development on URBEXT and SPR

Inflow	Development	Percentage impermeable	Pre development		Post development	
			SPR	URBEXT	SPR	URBEXT
Area 3	Mead Homes	75%	34.4%	0.044	54.1%	0.286
CR2_5553	St Modwen, NSC and additional area in Core Strategy	81%	26.7%	0.11	41.2%	0.249

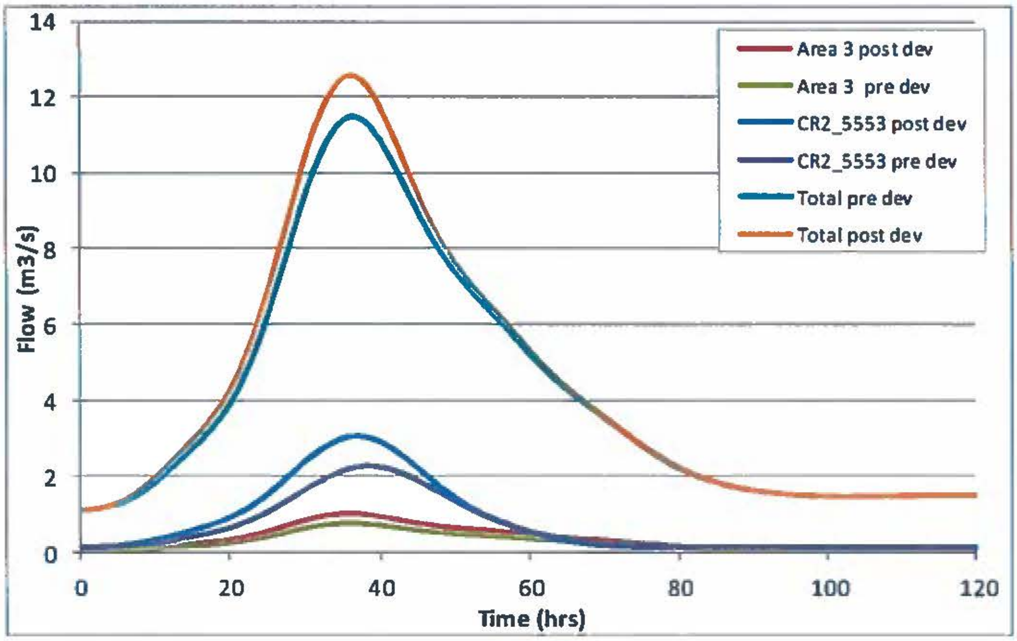
Table 5.3 – Impact of development on peak flows

Return Period (yrs)	Area 3 peak flow (cumecs)		CR2_5553 peak flow (cumecs)	
	Pre development	Post development	Pre development	Post development
2	0.31	0.42	0.92	1.27
5	0.42	0.57	1.26	1.73
10	0.49	0.66	1.47	2.01
20	0.57	0.76	1.70	2.31
50	0.68	0.90	2.02	2.73
100	0.76	1.01	2.26	3.05
1,000	1.14	1.51	3.44	4.56
100 +CC	0.91	1.21	2.71	3.66

Table 5.4 – Volume of 1 in 100 year hydrographs pre and post development (rounded to the nearest 100m^3)

Catchment	Volume (m^3)		
	Pre development	Post development	Difference
Area 1	160,100	160,100	0
Area 2	307,700	307,700	0
Area 3	107,900	139,800	31,900
Area 4	67,900	67,900	0
Area 5	83,600	83,600	0
Area 6	217,700	217,700	0
Area 7	6,600	6,600	0
Area 8	105,800	105,800	0

Catchment	Volume (m^3)		
	Pre development	Post development	Difference
Area 9	166,200	166,200	0
Area 10	25,600	25,600	0
Area 11	41,400	41,400	0
Area 12	26,800	26,800	0
Area 13	439,300	439,300	0
CR2_5553	251,200	314,300	63,100
TOTAL	**2,007,800**	**2,102,800**	**95,000**



Figures 5.3– Impact of development on 100 year hydrographs

6 MODEL SETUP

6.1 Introduction

This section describes the construction of an ISIS model of the River Banwell and outlines the decisions and assumptions made in the model construction process. ISIS version 3.5.0.135 was used.

6.2 Schematisation

The River Banwell and its tributaries (IDB system) can be characterised for most of its length as an embanked channel with a low lying, flat floodplain. These 'embankments' are not defined as regular 'raised' defences; instead they are relatively 'flat' and at a level similar to the surrounding ground in many places. The lowest parts of the floodplain are those towards the middle to upper third of the catchment with ground levels being higher downstream of the M5 compared to those upstream.

The channel has multiple interactions with ditches and Rhynes, which drain and maintain water levels in the floodplain. The levels in the floodplain are maintained by North Somerset IDB using a series of control structures throughout the system.

The schematisation of the model has evolved through the model calibration process, however the basis of the schematisation is that the main channels have been modelled in 1D as cross sections and the floodplain has been modelled using reservoir (area/elevation) relationships.

It is possible to model the floodplain in 2D using TUFLOW or similar software (connected to the model 1D elements); however this has not been utilised at this stage due to the length of time it would take to model multiple scenarios. Typically long storm duration events characterise the Banwell catchment therefore a model which takes a matter of minutes to run compared to many hours/days was seen as advantageous. The reservoir units also provide a satisfactory representation of how the floodplain fills up during a storm event, due to the flat nature of the catchment where water levels across the drainage system are typically identical across discreet areas.

It was necessary to consider the purpose of the modelling in terms of predicting flood risk under the future scenario of increased development. At the time of construction of the model, it was not known where the increased runoff (due to increased development) would discharge. However through time, the manner and location for which water will be discharged has been made available and therefore the model has been updated to incorporate such mechanisms.

It was therefore necessary to schematise the model so that the increased flood risk could be assessed in both the River Banwell and interconnecting Rhyne systems. Due to the interaction with the IDB system and the fact that the IDB system can flow in more than one direction depending on water level management practices, it was necessary to model a "regular practice" scenario rather than tens of different scenarios posed by the unlimited number of management practices that could be undertaken.

Figure 6.1 shows the location of the modelled reach and key locations (modelled reaches are shown in dark blue with IDB Rhynes in light blue)

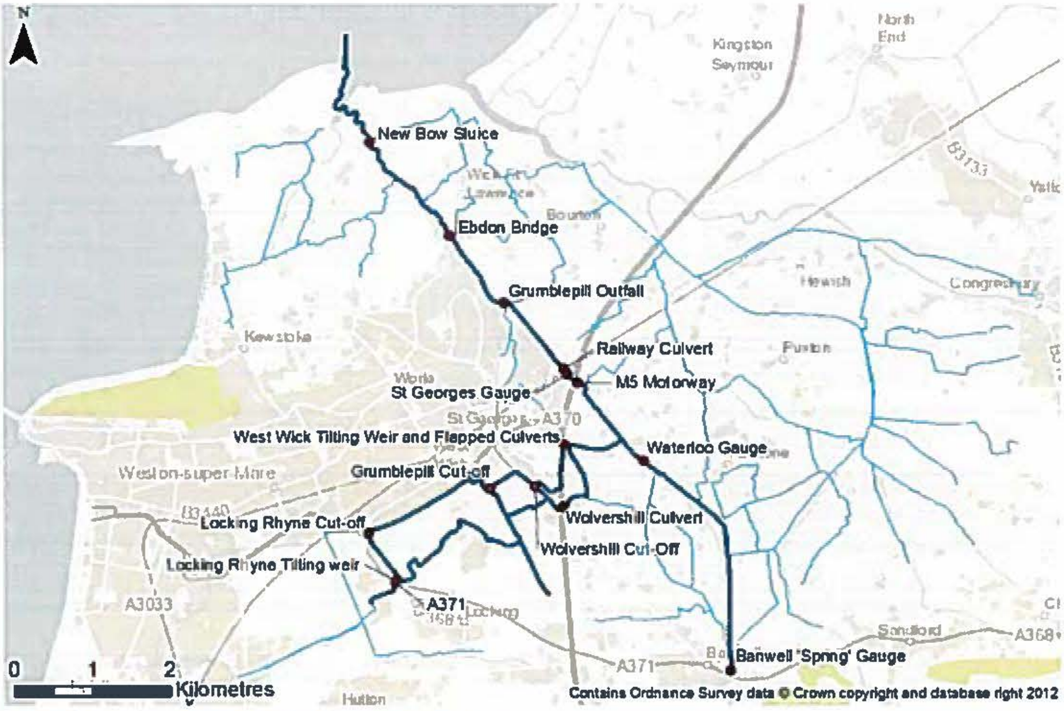


Figure 6.1: River Banwell Model Reaches (Dark Blue)

6.3 Cross Sections

Model cross sections for the River Banwell and West Wick Rhyne are based on those in the HECRAS models from the previous Royal Haskoning study (2007). The HECRAS models were converted to ISIS using HECTIC. Cross sections were reviewed, updated or deleted as required. Sections were extended using LiDAR data where the channel ran parallel to high ground or roads/embankments (such as the M5 or Somerset Avenue (A370)). The location of cross sections (and other model features) can be found in the model .gyx file. Figure 6.2 shows the location of the model cross sections.

Through the process of model development, the number and detail of sections downstream of New Bow Sluice (tidal reach) has been simplified to improve model stability.

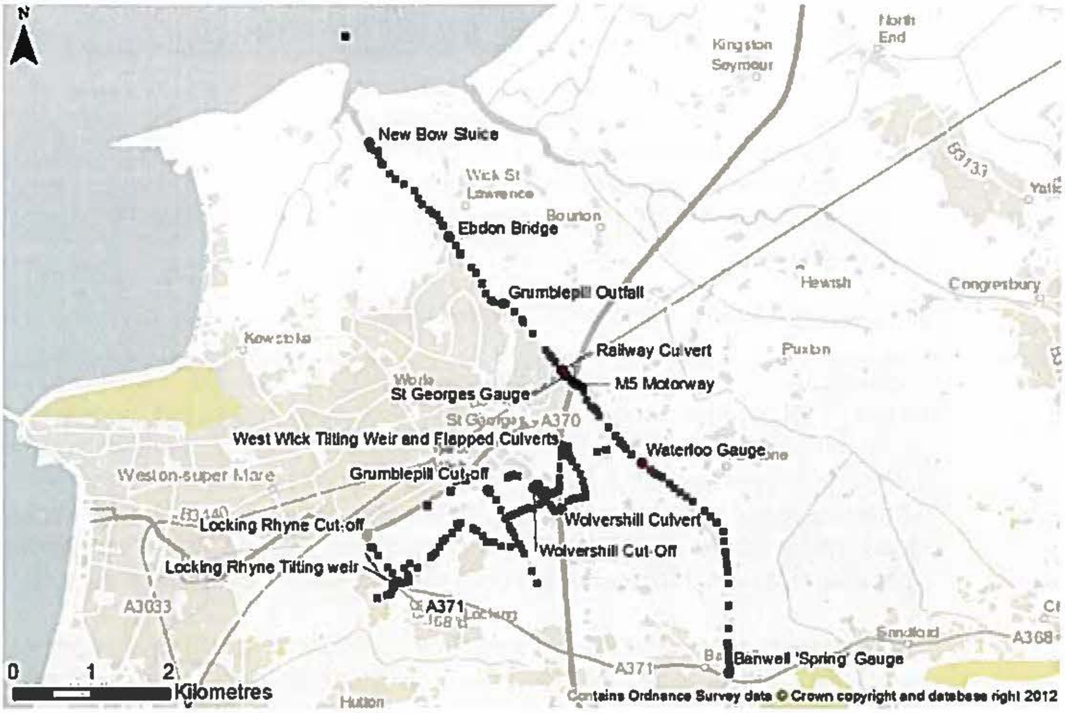


Figure 6.2: River Banwell Cross Section Locations (Source Google Earth)

6.4 Structures

All significant structures were included in the model. Structure coefficients were based on values recommended in the ISIS User's Manual. A list of structures in the present day and post development models are included in Appendix A.

Many of the structures that may be classified as culverts have been modelled as either bridge units or orifices. The hydraulic model became unstable and crashed when entrance and exit losses were applied to the culverts therefore different structure units were adopted. Sensitivity tests were performed to determine the difference in water levels these different units gave. This is discussed further in Section 8.4.

This was deemed an appropriate, conservative approach as the 'storage' capacity of the length of the culverts would not be included (increasing water levels by a minimal amount) and since the system gives a relatively flat water surface (due to ground levels and backing up at the tidal sluice) there is relatively little or no head loss across many of the structures.

The most significant structure on the reach is New Bow Sluice. The sluice stops the ingress of the tide. Correspondence with the Environment Agency suggests that:

On the upstream face there are two culverts to allow water to pass through to the estuary. Looking downstream the left culvert has a tilting weir, a penstock and a flap. The right culvert has a penstock and a flap valve. The right culvert is always closed and water levels are controlled by the tilting weir on the left culvert. The right gate could be opened if there was a big flood event and if failure/maintenance is required on the tilting weir. The tidal flap prevents high tides from coming up the Banwell.

The following alarm trigger levels for New Bow sluice are:

- Summer low level alarm = 3.7mODN
- Summer high level alarm = 4.8mODN
- Summer desirable level = 4.1mODN
- Winter low level alarm = 2.8mODN
- Winter high level alarm = 4.8mODN
- Winter desirable level = 3.2mODN

The sluice is mainly operated on summer settings.

The flapped outlet (left hand opening) and gated weir have been included in the baseline model. Both summer and winter desired levels have been used as control water levels in the model calibration as there was no record of actual operation for these events. The model operation rules for the tilting weir operates on a 1,800 second sample time where the sluice opens or closes by 0.05m depending on water levels upstream of the sluice being above or below the desired water level in the River Banwell. These operation rules were taken directly from the previous HECRAS model.

There are a number of tilting weirs within the system, many of which operate at different levels at different times of the year. In order to reduce the number of different combinations that were possible for these numerous structures, the penning levels of these structures was fixed at a position deemed to be worst case in order to assess flood risk. This is therefore a conservative approach.

6.5 Roughness

Channel and floodplain roughness has been represented in the model by the use of an appropriate Manning's 'n' value. These values were estimated based on a site inspection and photographs.

Typical roughness values for the channel are 0.050 to 0.055 with river banks and floodplains ranging from 0.025 to 0.100 depending on the land use material or density of vegetation involved.

The model had previously used higher roughness values than these stated above (the values were pushed to the boundary of credible values). This was necessary to calibrate the model. However, after further investigation of various hydrological inputs, it was discovered that the higher roughness values were no longer necessary. Instead other parameters were used to raised water levels to match observed levels.

6.6 Reservoirs and Spills

Out of bank flow was modelled by the use of reservoirs and lateral structures (spills). In ISIS a reservoir is defined using an elevation/volume relationship. This relationship was determined using the LiDAR data extracted using ArcView. The level of the lateral structures was determined from the cross section data. Where the banks were assessed to be too high to be overtopped (for example the downstream reach where bank levels are 1-2m higher than the upper reach), spills have not been included.

The location of areas modelled using reservoir units in the baseline model can be seen in Figure 6.3.

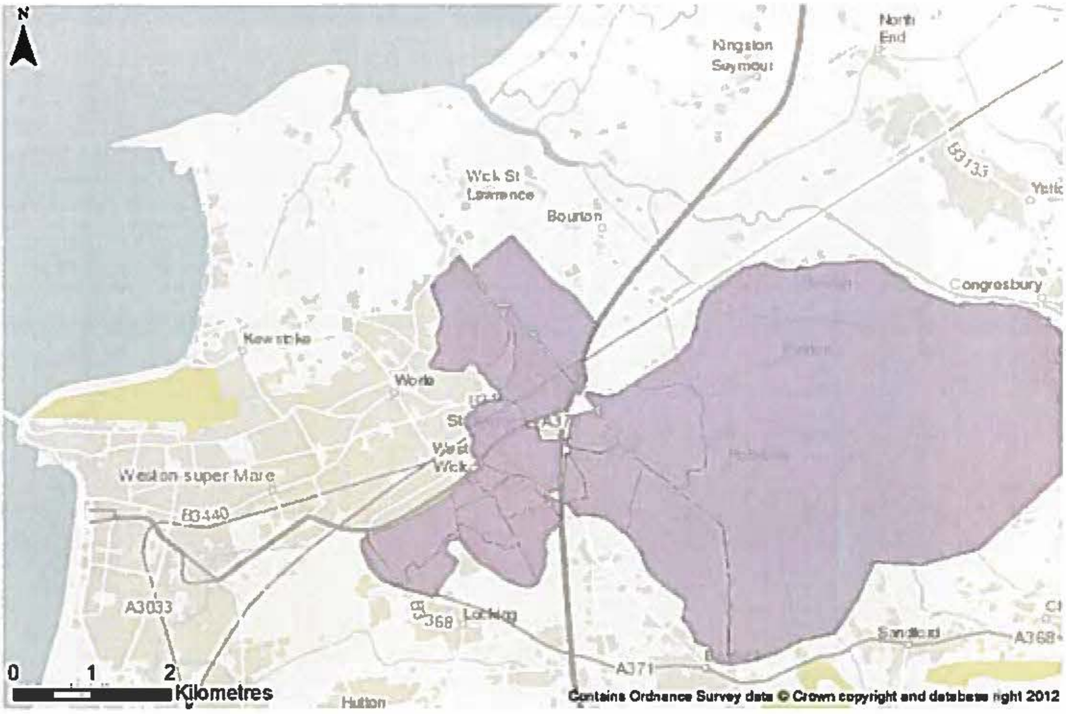


Figure 6.3: River Banwell Reservoir Unit Locations (Source Google Earth)

A small pond providing additional storage capacity has been installed on the River Banwell just upstream of New Bow sluice (shown in Figure 6.4). The design of the pond and embankment surrounding it allows water to flow in and out of the pond freely depending on the level of the tide, (tide locking of New Bow sluice) therefore as water levels rise in the main channel, they rise at a similar rate in the pond (although the pond slows the rate at which the water level would rise if it did not provide additional storage volume). This feature has not been modelled as a 'reservoir'; however its storage capacity has been included in the model cross sections. This is represented in sufficient detail using model cross sections. As the pond does not 'flow' or contribute to conveyance in the channel, a roughness coefficient of zero was given to the part of the section representing the pond. A section for the model including the pond is shown in Figure 6.5.

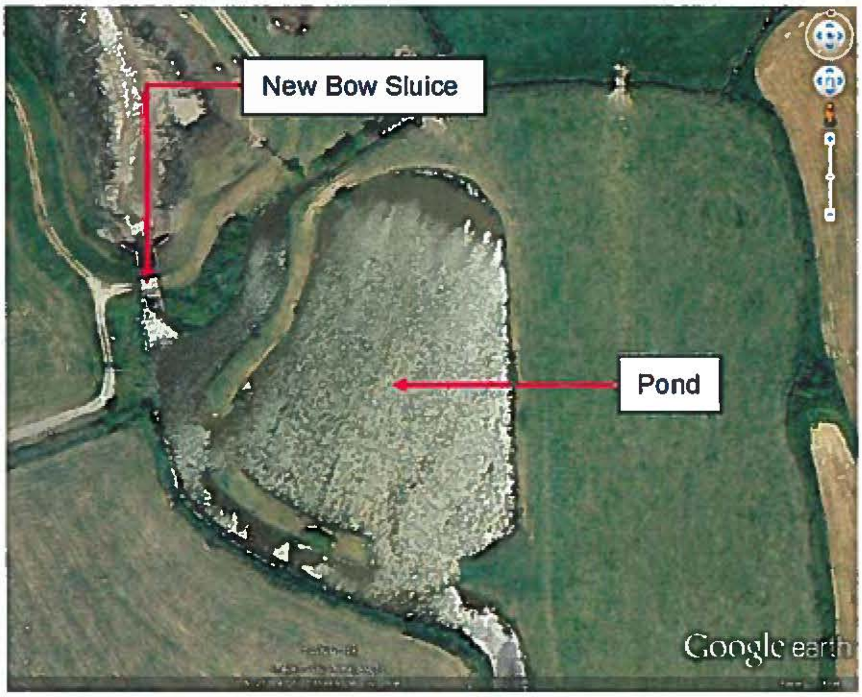


Figure 6.4: River Banwell Additional Storage Feature (Source Google Earth)

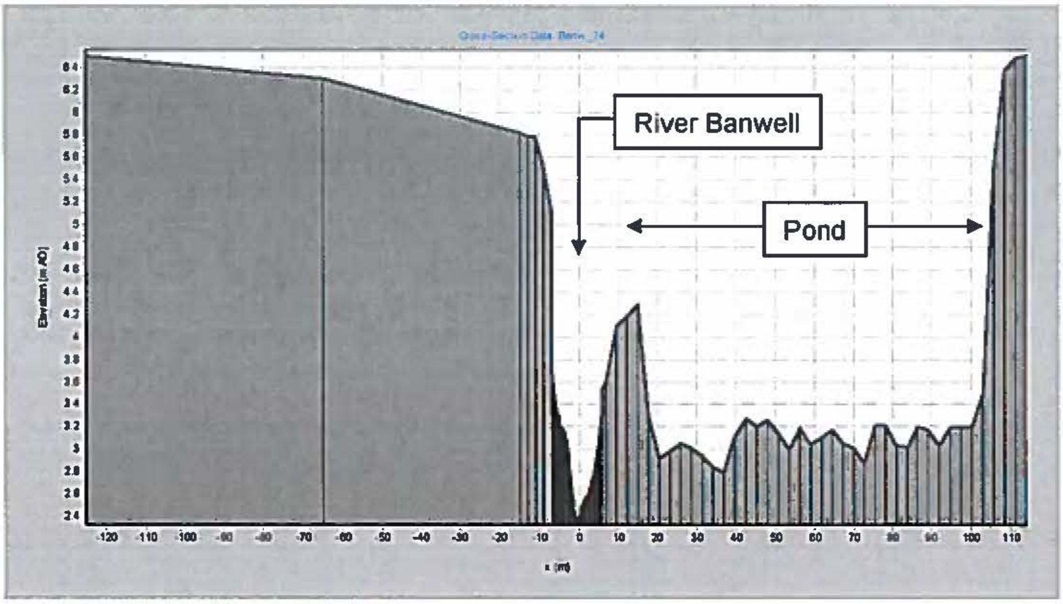


Figure 6.5: River Banwell Additional Storage Feature Typical Cross Section

A range of spill coefficients were considered to connect the river cross sections to the floodplain. This was based on the width of spills (ranging from 2-5m), roughness coefficient (ranging between 0.1 and 0.2 which is applicable for shallow depths of flow but is very low for some of the reaches in the model), average depth of flow (0.05 and 0.1m depth) and a range of modular limits (0.7-0.9). Of the 21 tests performed, an average spill coefficient of 0.37 was calculated. Given the geometry of the banks and surrounding ground elevations (inefficient at conveying water due to being relatively flat), the slope of the catchment being very shallow and the depth of water expected to flow over the spills is very shallow, a lower coefficient than the average has been adopted. A spill coefficient of 0.30 has therefore been used. Sensitivity testing has been performed to test variance in model results to this coefficient however, results of this are provided in Section 8.4.

Spill unit coefficients used in the model were questioned during a first review of the model however the nature of the catchment and the calculations performed suggested that they were representative for this catchment.

Spills representing passage of water over more defined structures or over structures back into the watercourse have been given higher spill coefficient values. These typically range between 0.5 and 1.5 depending on characteristics of flow as described above.

6.7 Downstream Boundary Conditions

The River Banwell is tidally affected. It was therefore important to reflect the impact of the tide in terms of preventing the discharge of water due to "tide locking". The model was therefore run using a downstream level against time boundary (HT boundary).

New Bow sluice prevents inundation by the tide and limits the discharge from the river at low tide. This structure was modelled to assess the influence of tide locking on the system. The 'design' and 'post development' model runs have been undertaken utilising a sinusoidal tide curve with a peak water level of MHWS for the appropriate chainage along the coastline. The MHWS at the mouth of the River Banwell in the River Severn is 6.29mODN. Figure 6.6 shows a representation of the tidal boundary condition used in the hydraulic model.

Tests were performed at an early stage of the modelling to determine if coincident or non-coincident tidal and fluvial peaks would cause the highest water levels in the River Banwell model. Fluvial tests were performed to determine the travel time of different storm durations i.e. the time at which the peak of the storm reaches New Bow Sluice; these models were run with a normal depth boundary. The timings of the peaks were then used to offset a tide curve so that the peak of the tide occurred at the peak of the fluvial event. Further tests were performed with the peak of the fluvial event occurring at 3 and 6 hours before and after the tidal peak. The results from this analysis showed that there was very little difference between all of the model runs and therefore the coincident peaks was taken forward into the design and post development model runs.

It should be noted that although higher tides could arise when fluvial events occur, it is probable that the number of tidal peaks of a level of 6.29mODN or above are less likely to occur than that included in the model boundary. However this is included in the analysis as a precautionary approach.

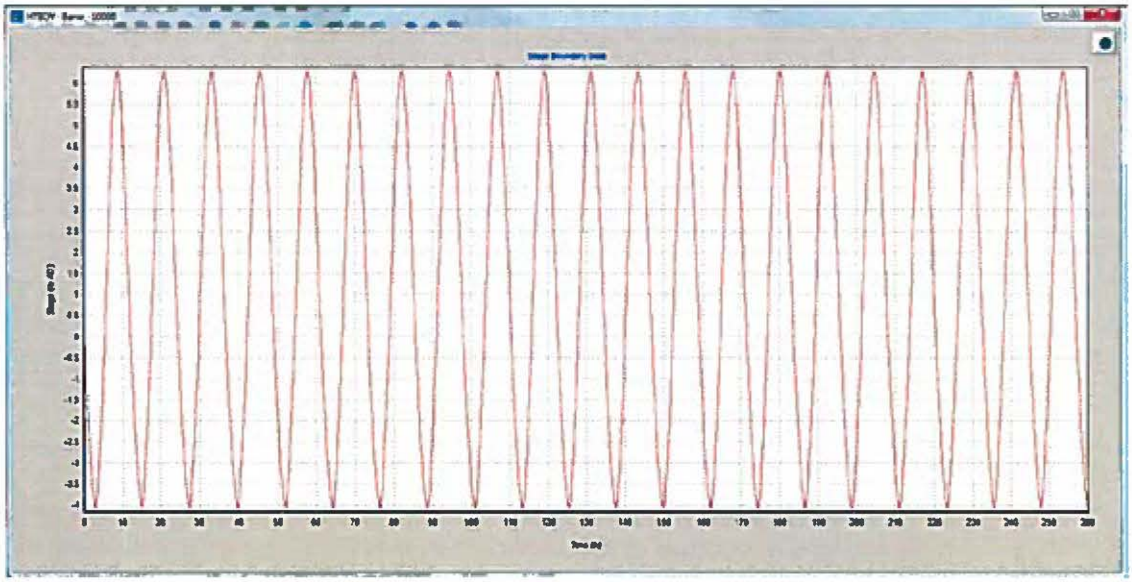


Figure 6.6: Hydraulic Model MHWS Tidal Curve

6.8 Inflows

Inflows into the River Banwell comprise a mixture of Rhynes and surface water discharges. These inflows are set out in Table 6.1 below with the sub-catchments that drain to each inflow. The methodology for determining the hydrographs for the 13 sub-catchments is presented in Section 4 of this report. Sub-catchment 6, which extends along a large portion of the right bank was split between different inflows to reflect the different drainage paths of the catchment. Figure 6.7 shows the location of the extent of each of the catchments.

Table 6-1 Model Inflows

Inflow Chainage	Watercourse	Catchment Names (s)	Catchment No(s)
8900	River Banwell	Banwell Spring	13
5881	River Banwell	Banwell Moor Wolvershill	1 2
4129	River Banwell	St Georges (Willow Close)	7
2696	River Banwell	Worle and North Worle	9
1980	River Banwell	Ebdon Grounds	10
633	River Banwell	Northfiled Rhyne New Bow Sluice Area	11 12
5337 4701 4262 3074 1741 1114	River Banwell	East of River Banwell	6
4310	West Wick Rhyne	Moor Lane Development	5
2020	West Wick Rhyne	West Wick	4
707	West Wick Rhyne	Grumblepill	3
Direct to Reservoir	Grumblepill Rhyne	St Georges	8

The direction in which water flows in the IDB system is dependent on structure operation and water level management practice. Different approaches have been taken into account in the model calibration process in order to obtain a sufficient calibration. Therefore the way in which the inflows have been included in the model include direct input into the rivers or into reservoir units in the floodplain which are linked to the channel through flapped outlets (the Grumblepill Rhyne).

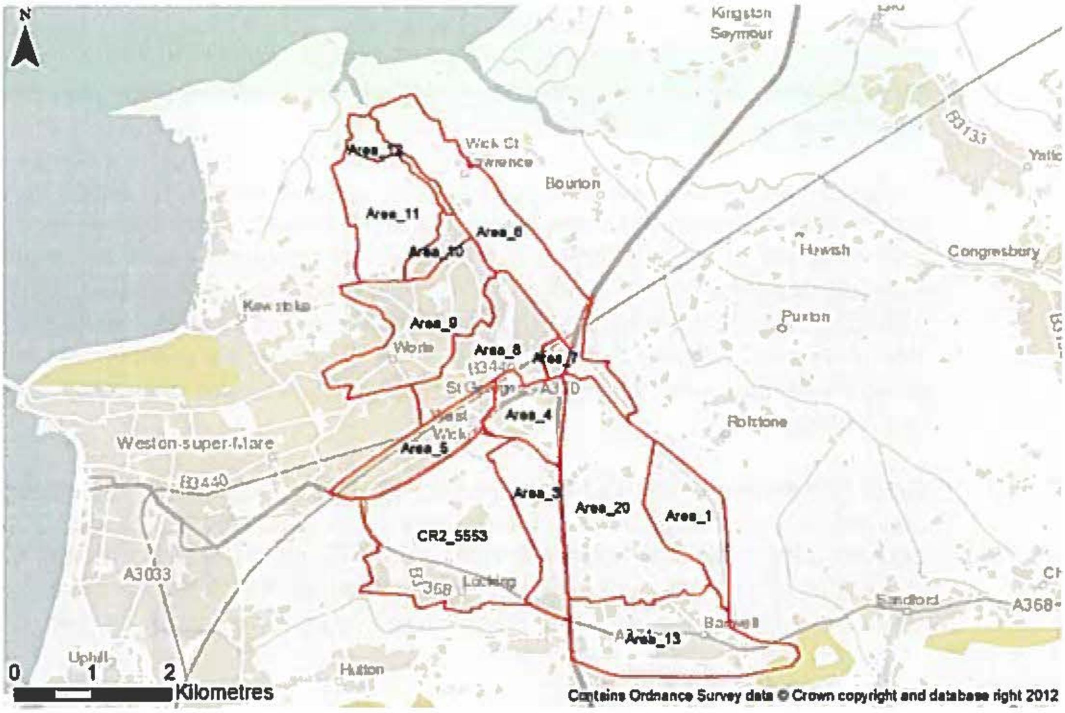


Figure 6.7: River Banwell Catchment Boundaries (Source Google Earth)

6.9 Critical Storm Duration

During an initial stage of the modelling, analysis of the results was undertaken to determine the critical storm duration for the River Banwell model. A series of storm durations had initially been run from 7 to 95 hours at four hour intervals.

The results from all of the different storm durations for a range of return periods were entered into a spreadsheet to determine the critical storm duration. It was discovered that there was not one critical storm duration from the results, but several; it varied in different parts of the catchment.

Of the 23 different storm durations that were run, it was evident that 7 of these gave the maximum water level in all locations to within 0-10mm. Therefore in order to minimise the number of model runs that would need to be run, it was determined that only these 7 storm durations should be run for any scenario for any return period. These 7 storm durations are:

- 19 hours
- 23 hours
- 47 hours

- 51 hours
- 67 hours
- 83 hours
- 95 hours

6.10 Model Scenarios

A variety of model scenarios have been considered as part of this study however there are two overarching scenarios, the 'present day' baseline and a 'post development' scenario. Both of these scenarios have been run through the model with winter and summer penning levels at New Bow Sluice to determine the effect of seasonal water level management practices on the River Banwell.

A variety of post development options were considered for the River Banwell. This report does not detail the numerous options considered, it does however describe and give evidence of the changes to water levels for options 2A and 2D. Additional runs were also undertaken using options 2E and 2F, however only as part of the sensitivity testing. Option 2 considers lowering of ground elevations downstream of the railway and M5 as shown in Figure 6.8. The area shown in Figure 6.8 shows the extent of the ground lowering, which increases floodplain storage volumes during high flow or severe tide locking events.

Option 2 has been modelled by extending the cross sections within the reach shown in Figure 6.8 on the right hand bank as there is no formal raised spillway into these two defined area. As it is expected that water will pond in these areas, a roughness coefficient of zero has been used for the extended length of the cross section. The elevations at which the extended cross section has been modelled at are:

- Option 2 A – 4.30mODN
- Option 2 D – 4.75mODN
- Option 2 E – 4.9mODN (sensitivity test)
- Option 2 F – 5.2mODN (sensitivity test)

Figure 6.9 shows a typical cross section from Option 2A within the reach length. In order to ensure the correct floodplain storage was modelled for these options, the baseline model was updated to include new cross sections and junctions so that the distinct change in channel geometry would be represented correctly.

A check was performed to ensure that the surface area of the Option (measured using GIS) was correctly represented by the model geometry. It was necessary to modify some of the cross section widths by a few metres to ensure that the modelled surface area and actual land area available matched (the surface area in the model was calculated by summing the area of floodplain between consecutive cross sections in the model based on distances between the sections and the width of the floodplain). There was less than 1% difference in the surface areas of the modelled and actual land surface that was available.

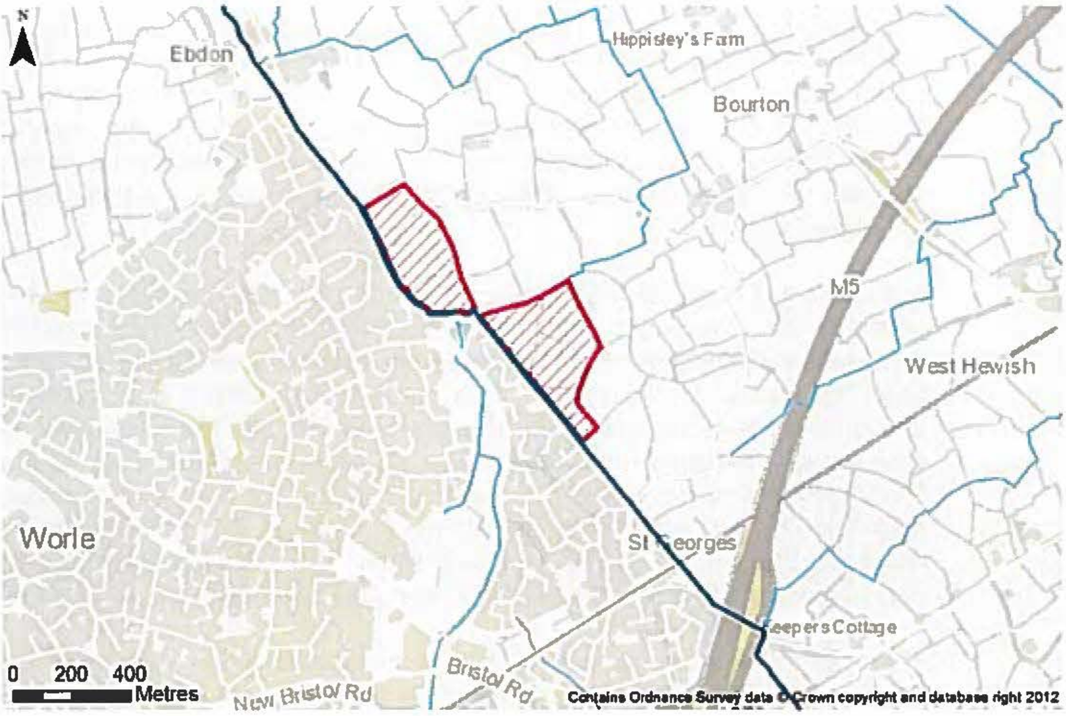


Figure 6.8: Option 2 Location of Ground Lowering (Source Google Earth)

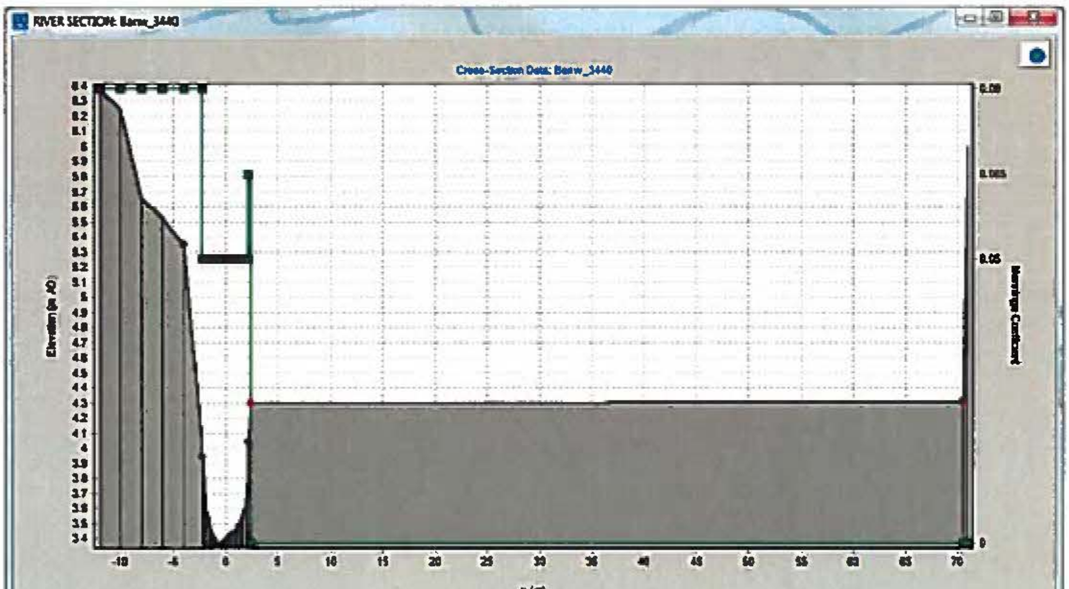


Figure 6.9: Option 2A Model Ground Lowering Schematisation

The modelled cross section has a 0.01m elevation difference between the right hand bank and the extremity of the floodplain prior to rising to higher ground. It was assumed that the lowering would be bounded by an embankment, shown here at an elevation of 6.0mODN so that water would not flow across the floodplain beyond the lowered ground should water levels rise above normal ground levels.

A further inclusion in the post development option is operating rules for the second New Bow Sluice opening. Currently, the second opening at New Bow Sluice can be opened during a flood event but may not be opened unless (it is believed) to be manually operated. Under the post development scenario, it is assumed that automation of the second opening will be included. The 'rules' for this automation in the model are based on opening of a new sluice once the existing 'alarm' elevation of 4.8mODN is reached at New Bow Sluice.

The sluice has been modelled with an invert of 2.7mODN (the same invert as the tilting weir) and the same width as the flapped culvert immediately downstream (1.9m). The water levels at the sluice are sampled every 1800 seconds (every 30 minutes). If the water level raises the alarm, the sluice opens to a maximum of 1.5m (which would be above the soffit of the culvert) to allow water to flow into the culvert (and out to sea if water levels downstream of the tidal flap are below those upstream). Once the water level upstream of the sluice falls back below the alarm level (but above a level of 4.2mODN) the sluice begins to shut slowly at a rate of 0.02m per half hour. Once the water level falls to 4.2mODN the sluice shuts automatically to ensure this opening is only utilised at high flows so that normal everyday water level control is not affected.

The reasons for these operating rules are to optimise the use of the second culvert in draining the River Banwell. Analysis of the results shows that it takes several days to drain water from the Banwell following a large event therefore having both the gated weir 'flat' and the new sluice fully open will allow an optimum volume of water to be released from the system during low tide. Although there are desired penning levels through the IDB system, it is shown that the operational rules cause a limited time period in which levels drop below preferred penning levels.

It is possible that other structures may be employed to provide the same benefit such as a gated weir; however these have not been modelled within this study. The rules could be optimised to provide greater benefits.

7 MODEL CALIBRATION

7.1 Introduction

This section describes the calibration of the hydraulic model against observed historic flood events.

The model was calibrated using historic events recorded on a level gauge at Waterloo Bridge, St Georges gauge and the flow gauge in Banwell village. The events used were predominantly within bank and therefore it is presumed no bypassing of the gauge would have taken place.

The limitations of this calibration are therefore that it has only been calibrated for in bank flows and that the gauges used were all in the top half of the catchment.

The calibration process was undertaken three times during the study. The initial calibration was reviewed and assessed to be suitable. Following the first calibration, the model was updated and extended and therefore it was considered necessary to re-calibrate to understand if the changes to the model made differences to the way in which the model simulated flood flows.

Following the second calibration, a question was raised regarding the techniques used for determining design and post development inflows; the techniques for which were derived from the calibration process. The question raised regarded the high SPR values used in the calibration; a value of 65% was used for rural catchment and 75% was used for the urban catchments. These concerns were discussed at a meeting at the Environment Agency in Bridgwater on 29th August 2012 by a number of parties (Environment Agency, North Somerset Council, Royal HaskoningDHV, JBA Consulting, BWB, Black & Veatch, WSP, Mead Homes and Persimmon Homes). This led to a final calibration of the model using the hydrological techniques discussed and agreed at the meeting.

This section of the report therefore reflects on the history of the calibration process so that the reader is given a background into the processes undertaken in order to understand the final parameters taken forward for design and post development model runs.

7.2 Calibration Approach

The approach to the calibration was to calibrate the model at known level/flow gauging points. The previous FMS Phase II study had achieved limited success in calibrating the HECRAS model for the River Banwell. However a more rigorous calibration has been undertaken for the ISIS model.

Three locations have been used to calibrate the model, whereas the former study only used one point at Waterloo Bridge. In addition, the model is more sophisticated than the previous model by including rules for the tidal sluice to help improve the match of the peak water levels from the events.

Finally, the way in which the IDB system interacts with the River Banwell has been scrutinised and modelled in several ways to determine the sensitivity to the model schematisation and therefore guide our assumptions.

7.3 Calibration Events

The model was calibrated against the following three in bank events:

- 28^{th} October 2000;
- 24^{th} November 2000; and
- 27^{th} November 2001.

The rainfall for these events was used to recreate the model inputs and tide data was taken from the tide gauge at New Bow Sluice. The data for the October 2000 event was also analysed against FEH DDF modelling. This event is believed to be approximately 1 in 6 year rainfall event, given the depth of rainfall over the duration of the event. This is the largest return period rainfall event that was recorded by the rainfall gauges within the catchment.

7.4 Calibration Parameters

To calibrate the model both the shape of the flood peak and the peak level were considered. The following parameters were investigated during the model calibration process:

- Time to peak (Tp);
- Standard percentage runoff (SPR);
- Roughness Coefficient;
- Baseflow; and
- Catchment Wetness Index (CWI).

7.4.1 Time to Peak

The previous study used a time to peak (Tp) of 22 hours, which was longer than the FEH catchment characteristics value of 11 hours at Waterloo Bridge. The LAG analysis undertaken as part of the FMS Phase II study was therefore reviewed, highlighting that some double peak events had been included within the assessment. These were revised, focussing on the first peak only, which suggested that a lower Tp was more applicable. During the calibration process it was also found that generally decreasing the Tp (other than for catchment 13) brought the flood peak more in line with the observed peak at Waterloo Bridge. Catchment 13 is a baseflow led catchment and therefore the best match for this catchment was found with a very large time to peak. Therefore, the Tp values that gave the best calibration were approximately 1 hour for urban catchments, 7 – 8 hours for rural catchment (excluding catchment 13) and 90 hours for catchment 13.

It is possible that other characteristics could be changed to help improve the 'peakyness' of the modelled flood hydrograph however this was deemed appropriate based on observations while undertaking the calibration process.

7.4.2 Standard Percentage Runoff (SPR)

In the first two stages of calibration of the model, high SPR values were adopted, 65% for rural areas and 75% for urban areas. These values were based on a review of the Weston-super-Mare Surface Water Management Plan maps and knowledge of the ponding that often occurs across the catchment, therefore limiting infiltration during an event. Following a meeting held at the Environment Agency on 29th September 2012 it was agreed that the high SPR values used on the Banwell catchment were both very high and vastly different from the values that were to be adopted for the adjacent Uphill catchment (which uses FEH SPR values). Therefore a series of different SPR values were tested to try and obtain sufficient calibration for the River Banwell model starting at FEH catchment characteristics SPR values. It was necessary to increase the SPRs by 30% for catchments 1 to 13 in the model, along with alterations to CWI as detailed below, to obtain a more reasonable calibration for the largest event seen on the catchment (October 2000).

7.4.3 Roughness Coefficient

The value of Manning's roughness coefficient was adjusted by +50% in order to improve the calibration of the model in the first stage of the calibration. However as this pushed the values to the upper bounds of credibility, these were later reduced to more justifiable values.

7.4.4 Baseflow

The initial baseflow for the upper part of the catchment (Area 13) was changed to match the recorded initial flow rate at the Banwell (Spring) gauge. This equated to values of between 0.1 and 0.375 m^3/s depending on the event. Baseflow for the other catchments was not adjusted from FEH catchment descriptor values.

7.4.5 Catchment Wetness Index

Following a review of the hydrological approach and results of the initial calibration, it was agreed that the catchment wetness index (CWI) using real event data should be investigated further to help calibrate the model in conjunction with lowering SPR values. This was generated by calculating the catchment wetness due to the previous 5 days rainfall prior to the rainfall that caused the events used in the calibration. This was undertaken using the equations and information in FEH Volume 4.

It was assumed that the soil was saturated for all three events due to the seasonality and volume of rainfall over the 5 day and extended period; therefore a soil moisture deficit (SMD) of 0 was used in calculating CWI.

The following CWI was adopted for the three calibration events:

- 28th October 2000 – 160.781;
- 24th November 2000 – 142.328; and
- 27th November 2001 – 149.703.

7.4.6 Other Calibration Parameters

Although not classed as a parameter, the interaction with the IDB system was seen as a very important aspect of the model schematisation and calibration process. The HECRAS model included a tilting sluice and culverts on the West Wick Rhyne just upstream of the M5 crossing, however close inspection of Google Earth historic aerial photographs shows that these structures were not in place in 2004 therefore they have been removed from the original ISIS model for the calibration process. This is also the case for several new culverts that link drainage ditches on the left hand bank of the West Wick Rhyne further upstream; these are more recent features since a new development has been constructed.

7.5 Calibration Results

7.5.1 Original Calibration

The calibration results from the original calibration are shown in Table 7.1 below and full plots of stage and flow against time for each event are shown in Figures 7.1 to 7.3. Figures 7.1 to 7.3 show the original calibration results using FEH boundaries without any adjustment (Ban_043_DATE.zzn shown in red) and the results of calibration using New Bow Sluice summer and winter penning levels (yellow and light blue). A set of results which includes the connection of the River Banwell to the floodplain at West Moor Rhyne is also displayed for summer and winter penning at New Bow Sluice (blue and green).

Table 7.7-1 River Banwell Calibration Results

Event	Bow Sluice Penning	Difference between model and calibration peaks (m)			
		Banwell Flow	Waterloo Level	St Georges Level	Best Fit
October 2000	Summer	-0.087	-0.072	0.119	
	Winter	-0.087	-0.077	0.107	*
November 2000	Summer	-0.027	-0.021	-0.017	*
	Winter	-0.027	-0.061	-0.118	
November 2001	Summer	-0.033	0.059	N/A	
	Winter	-0.033	0.019	N/A	*
Average	-	-0.049	-0.025	0.023	
Average Best Fit	-	-0.049	-0.026	0.045	

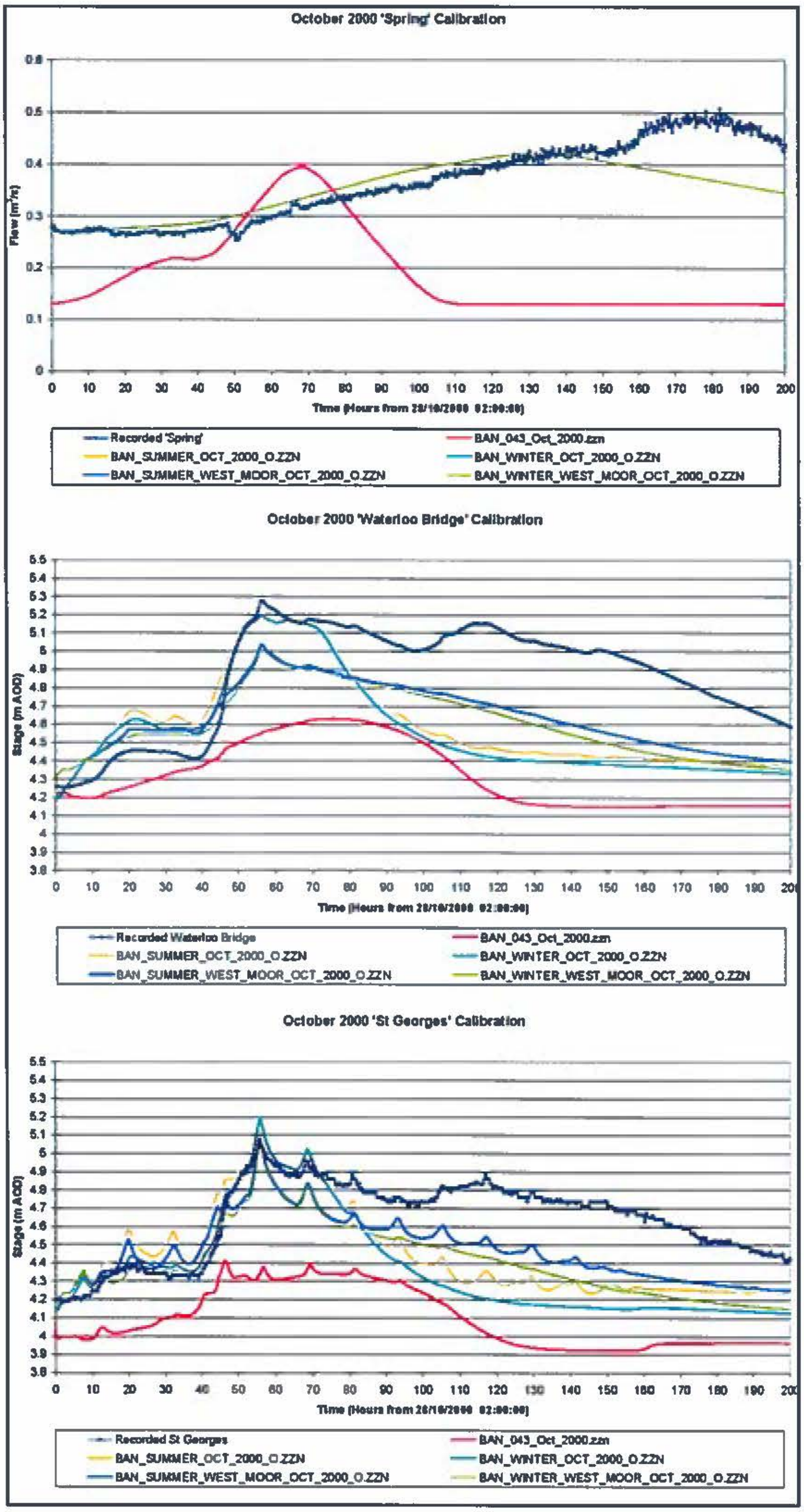


Figure 7.1: October 2000 Original Calibration Plots

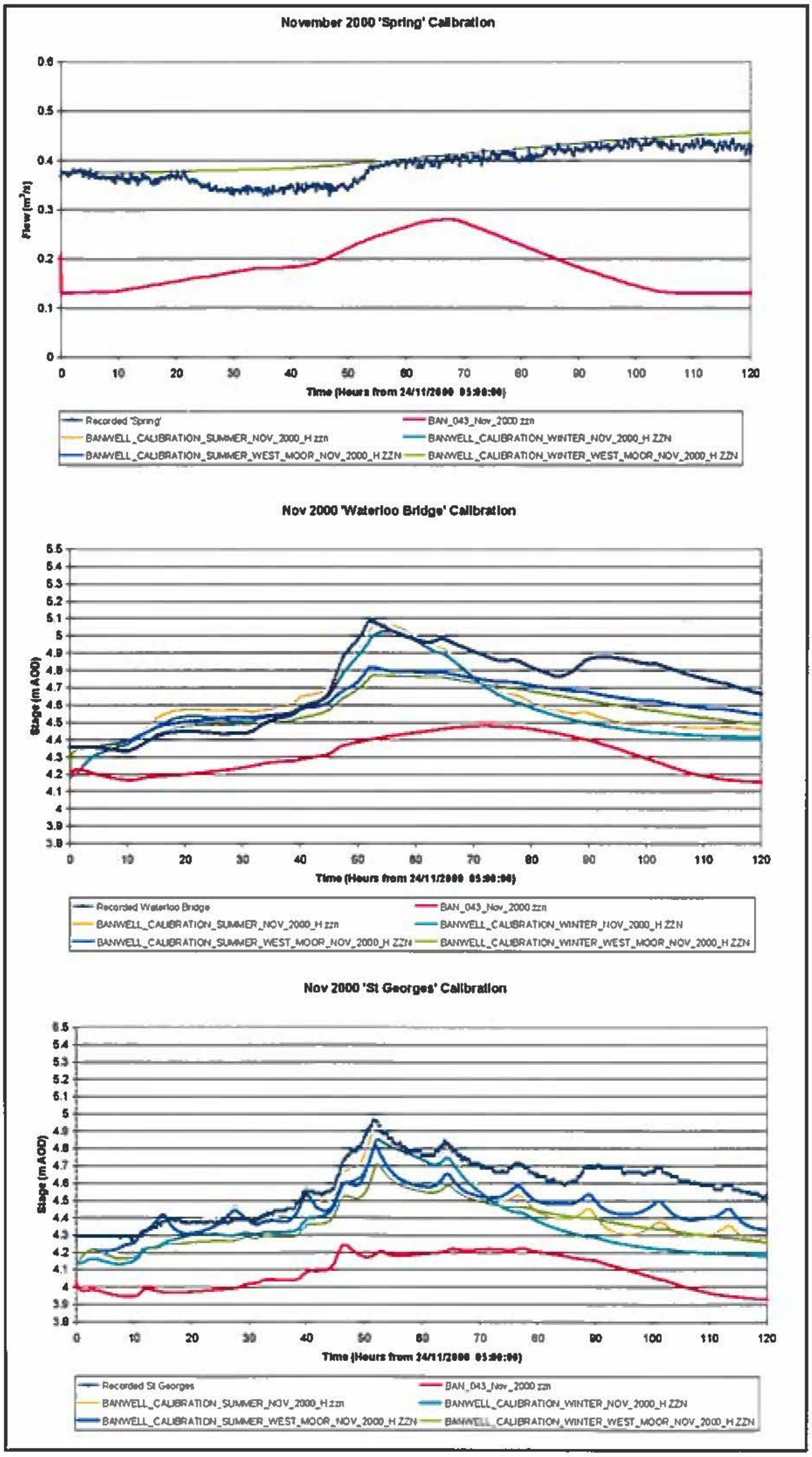


Figure 7.2: November 2000 Calibration Plots

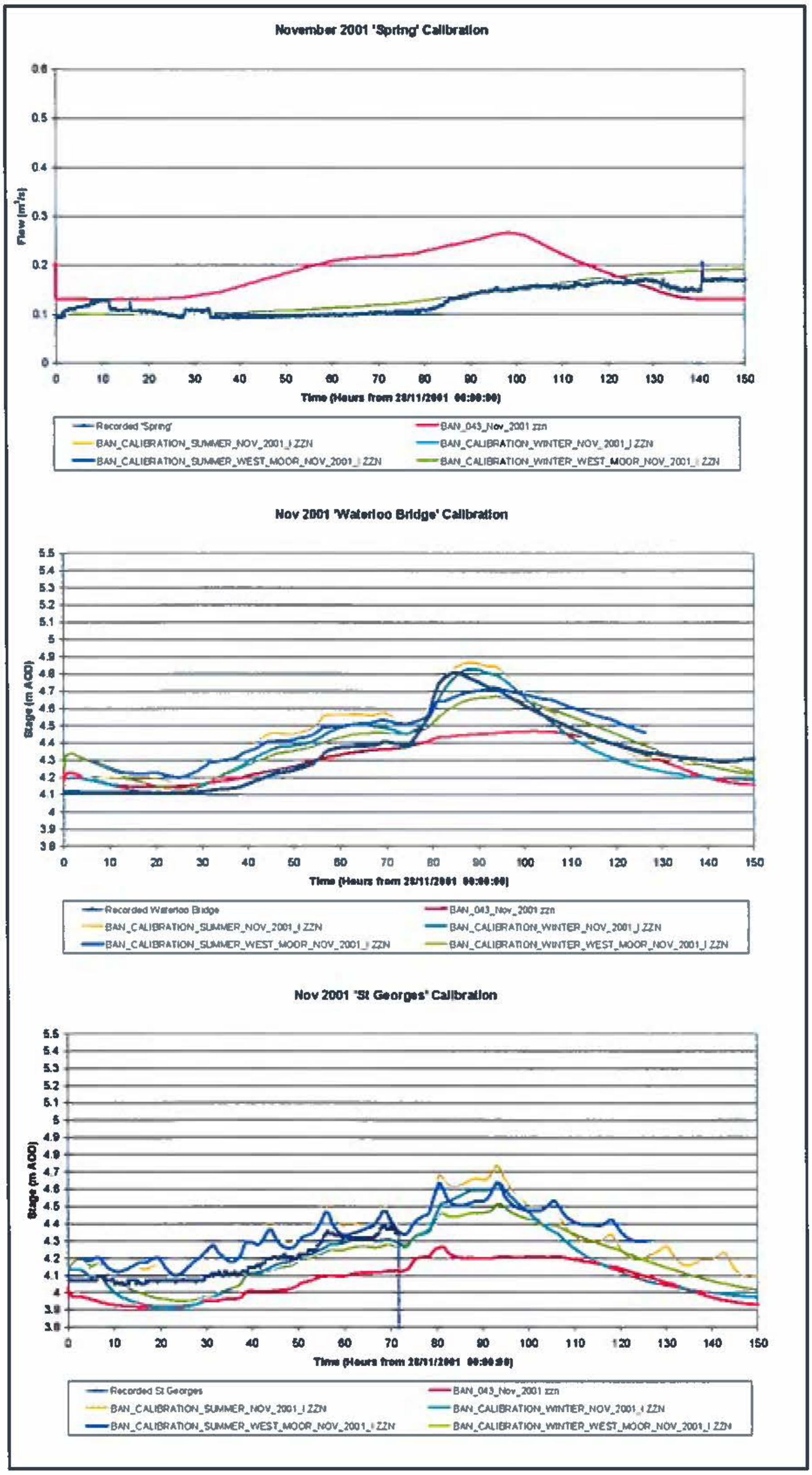


Figure 7.3: November 2001 Calibration Plots

The results from the original calibration show that the New Bow Sluice penning condition changes the result of the calibration. However, the maximum difference in recorded and model peak water levels is within 120mm. As the recorded data set is incomplete for November 2011, it has not been included in the analysis in Table 7.1.

The results from Table 7.1 and Figures 7.1 to 7.3 show that, in order to produce a satisfactory fit in terms of hydrograph shape and peak level, it is necessary to use results without the interaction with West Moor Rhyne. This is due to the fact that the model over predicts flow from the Banwell through the structure into the floodplain (modelled as a reservoir unit) which overly draws down water levels in the River Banwell.

This may be due to the oblique angle that the bridge is actually at in relation to the River Banwell, which is not modelled, or the fact that the Rhyne system is not included in the 1D element of the model and modelled as a reservoir unit. It is also possible that the rainfall in areas 1 and 2 of the hydrological investigation could increase water levels in the floodplain/reservoir unit and stop water flowing from the River Banwell into this area.

Although there are many arguments for and against the inclusion of this opening, it is clear that this should not be included in the model in order to produce a satisfactory calibration and therefore should also not be included for design runs. This will at least be a precautionary approach for the next stage of modelling.

Figures 7.1 to 7.3 show that the model closely predicts water levels, flood hydrograph shape and flood volume at the given calibration points. Both events in 2000 are much larger than the November 2001 event and therefore show a more marked set or results between the models with and without the West Moor interaction.

It should be noted that no out of bank calibration has been undertaken due to the lack of calibration data.

7.5.2 Further Calibration

The second calibration was undertaken following the updating of the hydraulic model to include further watercourses (Rhynes) to the west of the M5. The findings from the original calibration such as calibrating without the opening to West Moor were carried forward into this phase of calibration.

Previously the FEH and ReFH Unit Hydrograph had been used to try and derive flows for the calibration process. It was discovered that the ReFH boundary shape gave a better fit in terms of the shape of the hydrograph seen from real data at both Waterloo and St George's gauges, particularly for the tail of the hydrograph. This second phase of the study attempted to improve the calibration by therefore adjusting the shape of the Unit Hydrograph for the sub catchments.

Due to the nature of the low lying, predominantly flat catchment, a more trapezoidal 'lowland' Unit Hydrograph was first adopted for analysis to see how extending the 'peaky' Unit Hydrograph would change calibration results. A Unit Hydrograph was produced based on methods outlined in 'Pumped Catchments – Guide for Hydrology and Hydraulics' Environment Agency February 2012 (SC090006). Although the catchment is not pumped, the flat nature of the catchment means that the hydrological processes for much of the catchment may be better simulated using such methods, than those for upland rivers (as derived using FEH). The Unit Hydrograph; although changed in its appearance from a 'peaky' to a 'flat' trapezoidal shape, importantly still kept the same area (Qp) to that using any other method, it was only the shape that changed.

The results from changing the Unit Hydrograph shape did offer some betterment in terms of the shape of the calibration outputs however the peak stage results were very low due to reduction in peak runoff rates (related to the change in shape). Therefore analysis was undertaken to modify the shape of the unit hydrograph, keeping its area (Qp) the same but changing the ordinates to be more peaky (like that of a FEH or ReFH derived Unit Hydrograph) but with the longer duration of a lowland catchment Unit Hydrograph. It would have been possible to derive a Unit Hydrograph using real data if flow data had existed. The possibility of deriving a rating at either of the gauges (Waterloo or St Georges) to convert stage into flow was considered, however due to the tidal influence on the reach it was deemed that such a rating would be inaccurate and potentially misleading.

Several iterations of modifying the shape of the Unit Hydrograph were tested. The findings of this analysis still showed that a high SPR value was needed in order to generate the peak stage seen at Waterloo and St Georges gauges. The only ways in which to increase volume would be to increase the volume of the Unit Hydrograph and hence runoff (which was assessed not to be an acceptable approach), increase CWI to levels above that calculated using observed data (again an approach that was not deemed acceptable) or utilise high SPR values (which would be increased to the upper bound of credibility).

Other possible ways which could have caused the volume to be under estimated for the calibration were considered. It was considered possible that the rainfall record did not pick up the full or peak intensity of the events (i.e. there was a greater rainfall depth elsewhere in the catchment) therefore more rainfall was actually present to produce the observed water levels. Other reasons may lie in the operation of New Bow Sluice or the numerous structures under the IDB jurisdiction or simple event specific blockage of structures.

The results of the calibration were presented to a number of parties at a meeting held at the Environment Agency's offices in Bridgwater on 29th September 2012. The findings had also been reviewed in greater detail by a 3rd party prior to the meeting. The consensus of the persons present at the meeting were that normal best practice and more unconventional methods had been used to try and calibrate the model however the methods used were sound.

It was also considered that the high SPR values for the River Banwell system were too high. This was partly due to the values themselves being high and also that the SPR values used to calibrate the adjacent Uphill catchment were much lower. Although the Uphill catchment had no flow or stage gauges, flood outlines for the 1 in 5 and 1 in 10 year return periods were generated using a series of SPR values; these ranged from FEH values (~30% compared to those used in the Banwell calibration of 65% for rural catchments and 75% for urban catchments).

There was no evidence from Environment Agency or Council staff that the floodplain had been inundated to any significant degree or that any residents had complained of flooding within the last 20 years within the Uphill catchment. Using the Banwell calibration SPR values for the 1 in 5 and 10 year design runs caused extensive flooding to be shown from the model outputs, however lower SPR values (derived from FEH) showed only minimal flooding of the floodplain and no flooding of residential areas. It was considered that these lesser extents were therefore more realistic, suggesting that the FEH SPR values were most appropriate for that area.

There was further evidence that low SPR values should be adopted through calibration of the Uphill model for the October 2000 event. The same set of SPR values were adopted as in the design runs and a similar outcome was seen in terms of the flood outline. Analysis of the rainfall record against FEH DDF rainfall modelling suggested that the October 2000 event was an event of approximately 1 in 6 year rainfall return period. As expected, the model showed minimal flooding, giving more evidence that the SPR values from FEH gave a better calibration in the adjacent catchment to the River Banwell.

These findings therefore suggested that SPR was too high for the Banwell calibration given that it is adjacent to the Uphill catchment, however this would mean that adopting a lower SPR value would reduce modelled peak levels at the gauge locations due to less volume of runoff. However it was evident that all other acceptable means of increasing the volume and hence water levels during the calibration events had been undertaken. A final set of calibration runs were undertaken to determine what affects the change to SPR would have on the calibration. Figures 7.4 to 7.6 show the results from the calibration prior to and after the meeting held at Bridgwater. Table 7.2 summarises the inputs for the calibration events seen in Figures 7.4 to 7.6.

Table 7.2 – River Banwell Second and Final Calibration Run Parameters

Calibration Event	Run	Parameters
October 2000	BAN_95_Oct_2000_BD	Second calibration using model 95. Hydrology with user defined UH ordinates and 65/75% SPR for rural/urban areas and FEW CWI
October 2000	BAN_95_Oct_2000_BD+40	Second calibration using model 95. Copy of BD but calibration event run from hour 40 to test change to peak of hydrograph at Waterloo and St Georges
October 2000	BAN_95_Oct_2000_BF	Calibration changes following meeting. Copy of BD with all SPR values set to FEH values and CWI refined to observed event (160.781)
October 2000	BAN_95_Oct_2000_BG	Calibration changes following meeting. Copy of BD with all SPR values set to FEH x1.3
October 2000	BAN_95_Oct_2000_BH	Calibration changes following meeting. Copy of BG with urban inflows (Area 7 to 10) CWI changed to FEH values
November 2000	BAN_95_Nov_2000_X	Second calibration using model 95. Hydrology with user defined UH ordinates and 65/75% SPR for rural/urban areas and FEW CWI (same as Oct_200_BD other than rainfall)
November 2000	BAN_95_Nov_2000_X+40	Second calibration using model 95. Copy of X but calibration event run from hour 40 to test change to peak of hydrograph at Waterloo and St Georges
November 2000	BAN_95_Nov_2000_Y	Calibration changes following meeting. Copy of X with all SPR values set to FEH values x1.3 and CWI refined to observed event (142.328)
November	BAN_95_Nov_2000_Z	Calibration changes following meeting. Copy of Y with urban

Calibration Event	Run	Parameters
2000		inflows (Area 7 to 10) CWI changed to FEH values
November 2001	BAN_95_Nov_2001_M	Second calibration using model 95. Hydrology with user defined UH ordinates and 65/75% SPR for rural/urban areas and FEW CWI (same as Oct_2000_BD and Nov_2000 other than rainfall)
November 2001	BAN_95_Nov_2001_M+70	Second calibration using model 95. Copy of M but calibration event run from hour 70 to test change to peak of hydrograph at Waterloo and St Georges
November 2001	BAN_95_Nov_2001_N	Calibration changes following meeting. Copy of M with all SPR values set to FEH values x1.3 and CWI refined to observed event (149.703)
November 2001	BAN_95_Nov_2001_O	Calibration changes following meeting. Copy of N with urban inflows (Area 7 to 10) CWI changed to FEH values

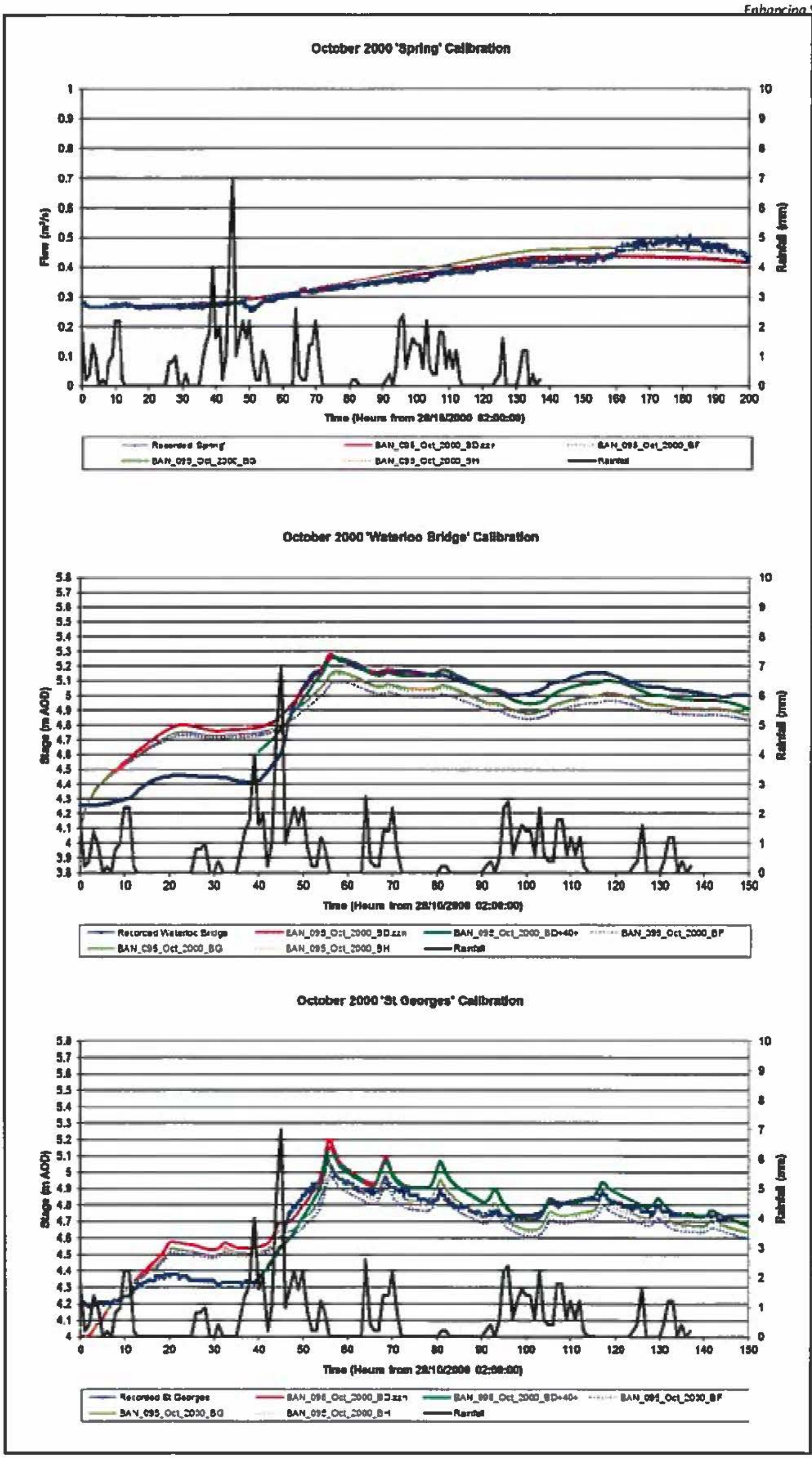


Figure 7.4: October 2000 Final Calibration Plots

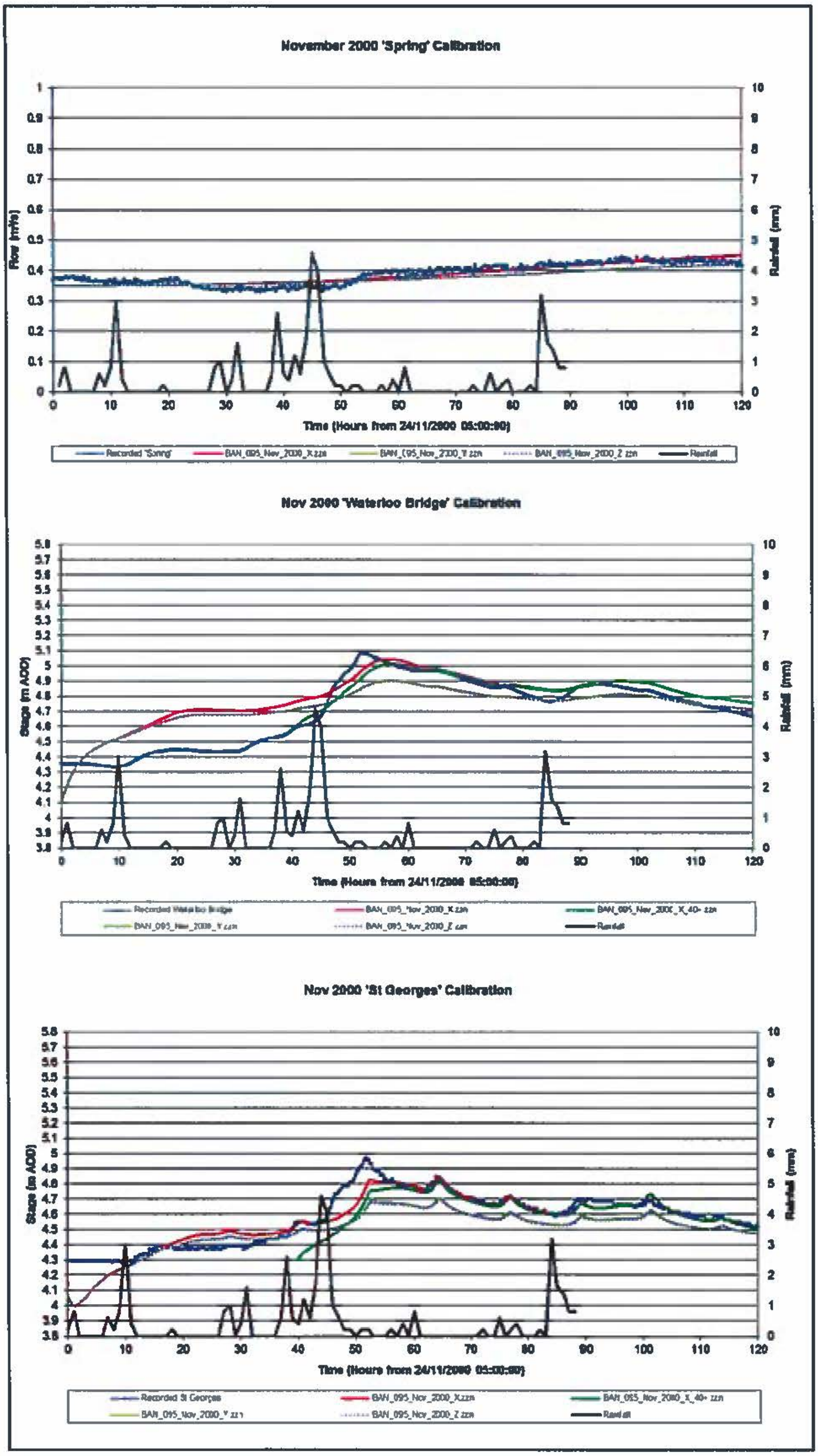


Figure 7.5: November 2000 Final Calibration Plots

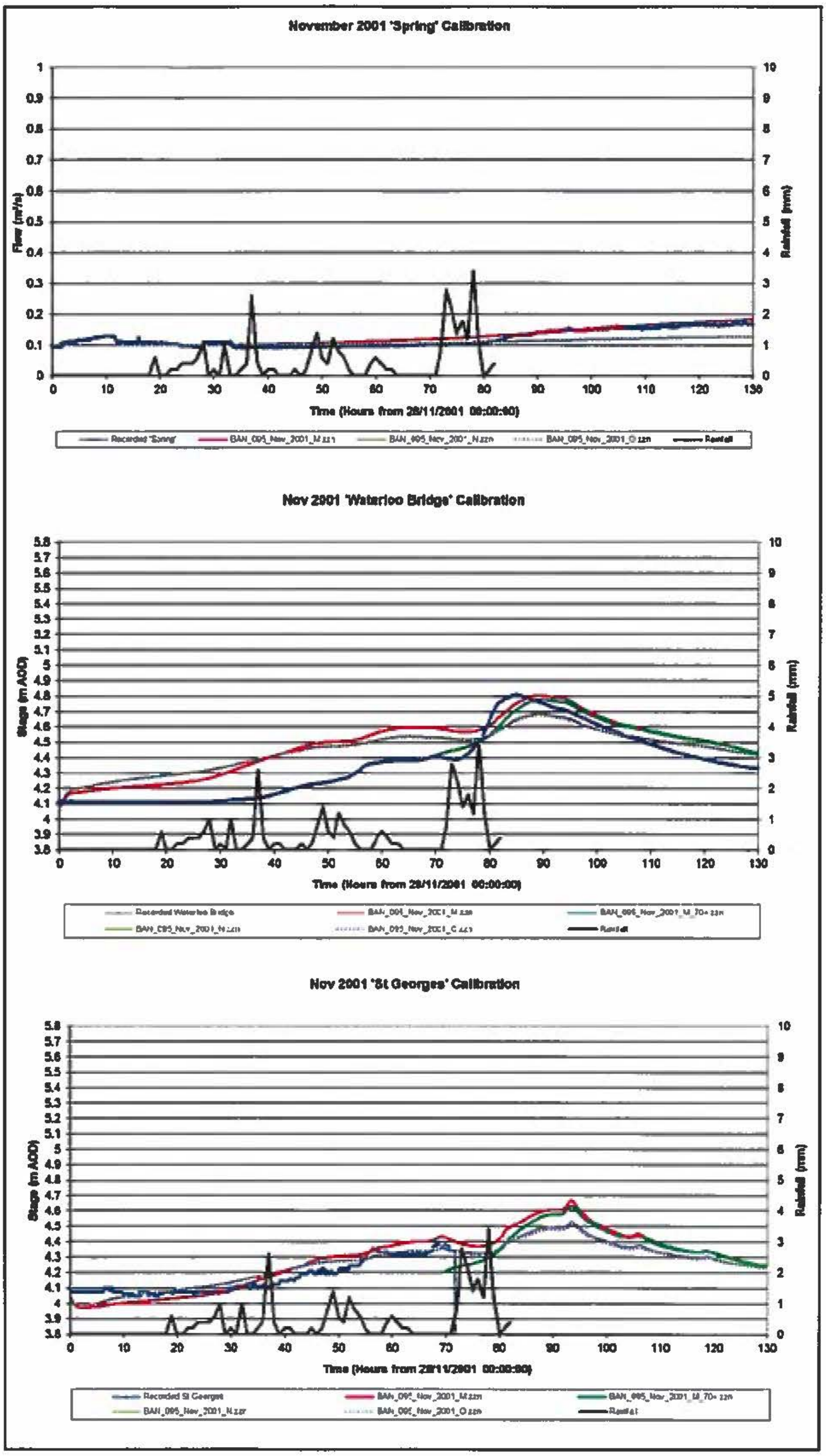


Figure 7.6: November 2001 Final Calibration Plots

The results of the final calibration show that a similar shape is maintained to the hydrographs however the modelled peak stage at Waterloo and St George's gauges are lower than when a higher SPR value is used. Table 7.3 summarises the difference in peak water levels for the final calibration.

Table 7.3 – River Banwell Final Calibration Peak Level Summary

Event	Gauge	Recorded Peak (m AOD)	Modelled Peak (m AOD)	Difference in Peaks (m)
October 2000	Waterloo Bridge	5.277	5.147	0.130
October 2000	St Georges	5.078	5.024	0.054
November 2000	Waterloo Bridge	5.086	4.9	0.186
November 2000	St Georges	4.968	4.693	0.275
November 2001	Waterloo Bridge	4.806	4.677	0.129
November 2001	St Georges	4.392*	4.509**	0.033

* Peak of the event not picked up however value indicates peak of recorded data

** Peak of modelled water levels for same period of recorded data

The difference between modelled and recorded peak water levels for the three events vary depending on the event considered. The calibration process attempted to ensure that the largest of the three events fell within a 150mm difference between modelled and peak stage, hence applying a 1.3 factor to the FEH SPR values of the model inflows. The same parameters were then transposed into the remaining two calibration events. The results of the November 2000 event were outside what would be desired, with the difference in peak levels being greater than 150mm however two of the three events fall within this confidence limit. The November 2000 event is also the smallest of the three events and our focus is primarily on extreme flooding rather than every day water levels.

inland up the river system as the head of water 'trapped' downstream of the railway culvert builds. The head builds due to relative high bank/ground elevations downstream of the railway, confining water to the channel compared to levels above the M5 Motorway, where water readily spills to the floodplain over the low bank crest elevations, which draws down the water level in the upper part of the system.

Figure 8.2: Railway Bridge Constriction (looking at upstream face)

The seasonality of flood events for the catchment suggests that higher magnitude flood events occur more commonly during winter than in summer, therefore it is more likely that winter penning rules would be in operation during the time of a large flood event (this however may not be the case with climate change). Therefore analysis was undertaken to determine the effect of the operating rules at New Bow Sluice on the same design event.

Figure 8.3 shows that for low magnitude events (1 in 2 years) the backwater effect of a summer penning level at New Bow Sluice compared to a winter penning level reaches upstream of Waterloo Bridge, almost 6 kilometres upstream. However, in higher magnitude events (1 in 20 years and above) the penning level of New Bow Sluice makes no difference to peak water levels. These findings may be partly due to having used a MHWS tide curve for the duration of the model run (approximately 10.5 day therefore 21 tidal cycles) however it is a fair assumption that the capacity of New Bow sluice is not sufficient to drain a large volume of water in sufficient time for a catchment with a very shallow gradient.

8 BASELINE MODEL RESULTS

8.1 Introduction

This section of the report describes the results and findings of the various baseline present day scenario model results.

8.2 Maximum Water Levels

The maximum water levels for the baseline model runs (both summer and winter penning at New Bow Sluice) can be found in Appendix B.

The analysis of the peak water levels from the various model runs shows that the catchment under certain conditions behaves like that of upland river systems; however under other conditions it behaves in a much different manner. Figure 8.1 shows the maximum water level along the River Banwell for a selection of different return periods.

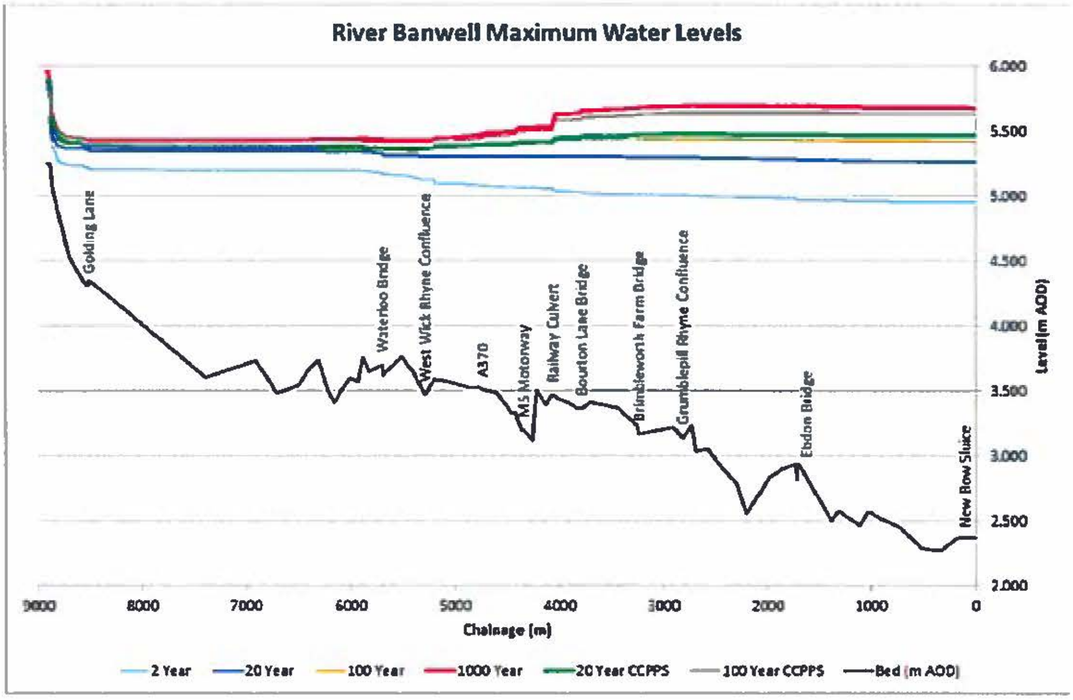


Figure 8.1: River Banwell Maximum Water Levels

For low order events (up to 20 year return period) the River Banwell results appears like any upland river system with maximum water levels decreasing with distance downstream. However, for larger magnitude events the combination of a large flood volume and tide locking causes a change in the pattern of flooding.

The flooding mechanism in larger magnitude events is that water flows into the River Banwell along its reach, approximately 50% entering upstream and 50% downstream or the Railway/M5 Motorway. As the water levels rise due to tide locking, the reach downstream of the Railway rises at a faster rate than that upstream of the railway. This is due to the backwater effect of the railway culvert which has a limited flow capacity (see Figure 8.2). At the timing of the peak of the flood event, water attempts to flow

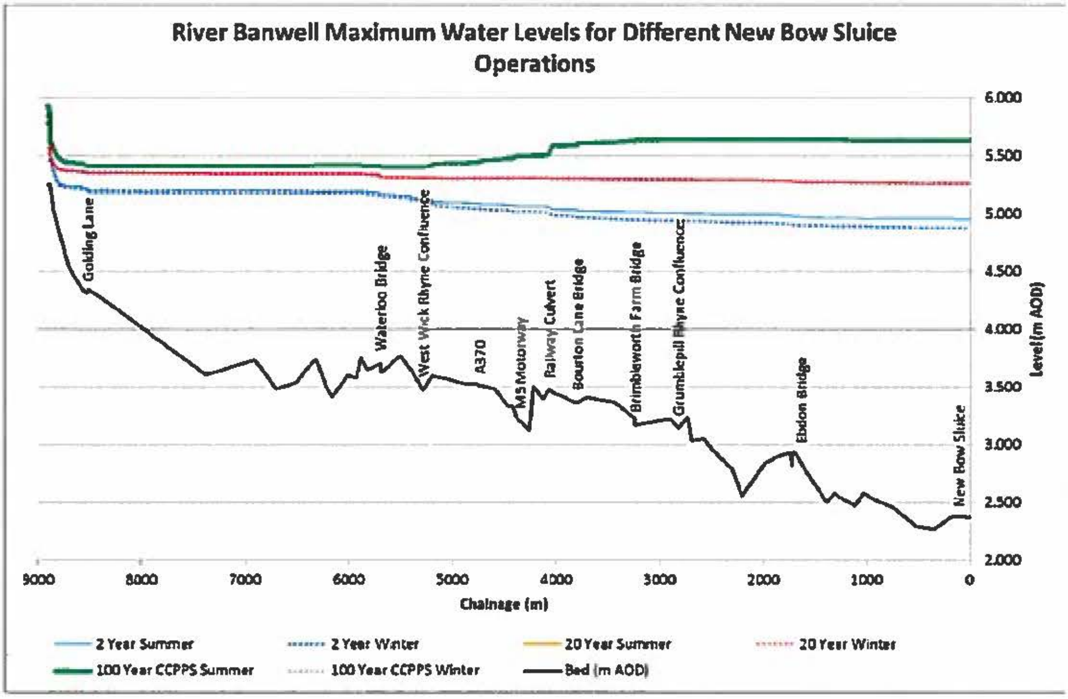


Figure 8.3: Effects of New Bow Sluice Penning Operations

8.3 Flood Extent

The flood extents for the baseline model can be found in Appendix A, Figures 1 to 11.

In the 1 in 2 year event, a low magnitude event the flood extents are mainly focussed within the drainage network to the east of the M5, a small area near between the A370 and Whorle train station and north of Locking (within an area enclosed by the A370, A371 and M5). Flooding also occurs in the low order events to an area of parkland, north-west of Walford Avenue.

As the magnitude of flooding increases, the flood extents become more prominent at Way Wick and north of Locking. Outside of the main concentrations of flood extents, it is evident that the main drainage network continues to be utilised. It is only in events of a greater magnitude than a 1 in 100 that the wider drainage network is significantly over capacity and the flood extents greatly increase.

The high order events (greater than 1 in 100 years) would result in prominent flood extents along the left and right banks of the River Banwell upstream of the M5, across the Great Rhyne network between the A371 and A370 and an area between the Worle train station and the A370. There are no significant flood extents downstream of the M5 along the River Banwell, only within the small area of parkland on the left bank which occurs in the low magnitude events and on the edge of Ebdon.

8.4 Sensitivity

8.4.1 Introduction

A series of sensitivity tests have been performed on the baseline scenario hydraulic model. These tests have been performed in order to gain confidence that changes in value of a key parameters will not significantly affect peak water levels.

The results for the 1 in 100 year 83 hour storm duration model has been used as a baseline for which to compare the results of the various sensitivity tests that were performed.

8.4.2 Roughness

Two sensitivity tests were performed on model roughness coefficients within the model. The ISIS 'Global' roughness tool was used to change all roughness by +/- 20% including within bridge units within the model. The results of these two model runs are compared with the baseline model results in Appendix A, Figures 24 and 25.

The results show that the majority of water levels from Banwell Village to the confluence with West Wick Rhyne vary by approximately 0.015m from that of the baseline, with a maximum variance of +/- 0.060m being seen. From West Wick Rhyne to New Bow Sluice, the results are gradually reversed with a variance of up to 0.010m being seen in maximum water level.

These findings are a result of water entering a tide locked system. The animation of the results shows that when the system is not tide locked the increase in roughness coefficient gives higher levels throughout the system as would be expected. Despite these complex interactions, the results show that the change in maximum water levels due to the change in roughness coefficient is not significant in the River Banwell.

A more marked change comes in the change in peak water levels in the floodplain. The change in water levels within the reservoir units varies between -0.735 and +0.553m from the baseline condition. However these changes in water level should be viewed in relation to the change in volume that spills to the floodplain; a change in water level of 500mm may only relate to a fractional change in inundation of the floodplain. There is a very steep incline to the area/elevation volume relationship at shallow depths of flooding of the reservoir units due to the capacity of drains and Rhynes within the floodplain, therefore Figures 24 and 25 in Appendix A gives a better representation of these changes.

The -20% in roughness results in smaller flood extents compared to the 100 year baseline in section 8.3. Most notably the main areas of flood extent (east of Way Wick, an area between the A370 and Worle train station and north of Locking) reduce and other areas of flooding are not experience. This is shown from there being no flood extents in the areas south of West Wick, adjacent to the M5 and right of the M5 (south west of Way Wick).

The flood extents for the +20% roughness show less notable differences compared to the baseline 100 year event. There are no new areas of flooding, only the extents increase slightly.

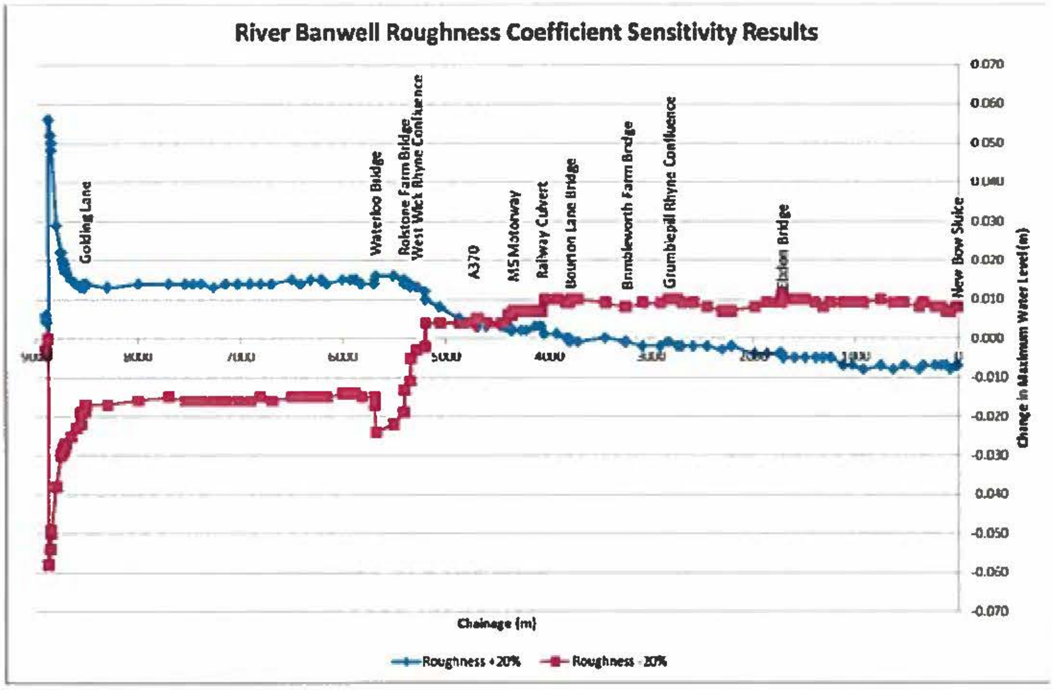


Figure 8.4: River Banwell Changes in Peak Water Level due to Change in Roughness Coefficient

8.4.3 Spill Coefficient

Tests were performed to determine the sensitivity of the model results to an increase in lateral spill coefficients. Spill coefficients were increased by 100% for this test (from 0.3 to 0.50) for lateral spills to the floodplain only.

The result from this test shows that the maximum water level along the River Banwell is decreased by up to 0.020m. This is due to raising the efficiency of the spills which decreases the head of water flowing over the spill and therefore the maximum stage in the channel. The knock on effect of lower levels in the Banwell is that the rest of the IDB system water levels (in channel) are also lower as the backwater levels is reduced in the Banwell. The resultant change in flood outline can be seen in Figure 26 of Appendix A.

The change in spill coefficient has increased the volume of water that spills to the floodplain from the watercourses in the model.

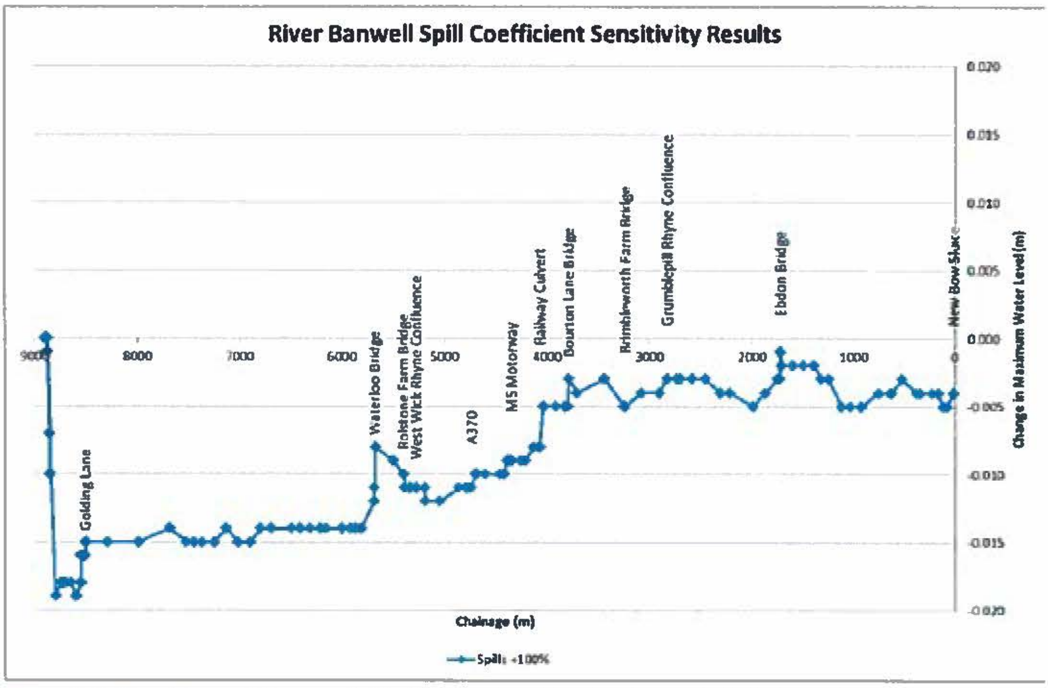


Figure 8.5: River Banwell Changes in Peak Water Level due to Change in Spill Coefficient

8.4.4 Structure Types

Some simple analysis was undertaken on the model in order to determine the impact that using different structure units in the model on water levels. The analysis focused on the water levels along the reach from upstream of the A370 slip road to downstream of the M5 and railway. It was realised that this area may potentially be sensitive due to the constriction on the watercourse caused by the size of the railway culvert.

The reach on the River Banwell from the A370 slipway to the railway has been modelled either as bridge or culvert units (without inlet and outlet losses due to instability problems) in order to determine the impact of using different approaches to structures on peak water levels. The results of this analysis (see Figure 8.6) show that the maximum difference in peak water level is less than 10mm. Levels downstream of the confluence with the West Wick Rhyne have increased by approximately 5mm while the levels upstream of this point in general have decreased by up to 5mm. Peak water levels in the floodplain are relatively unchanged with a peak decrease in water level of 0.124m.

This is further shown by Figure 23 in Appendix A, where the overall flood extents have decreased slightly in the structure sensitivity test.

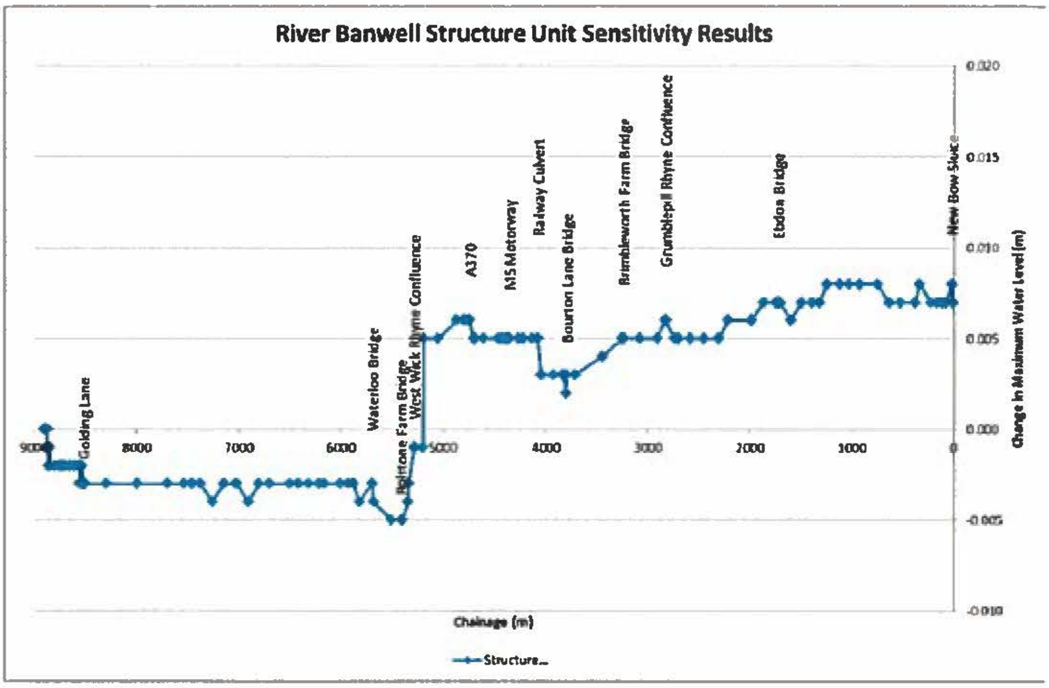


Figure 8.6: River Banwell Changes in Structure Schematisation

9 POST DEVELOPMENT RESULTS

9.1 Introduction

This section of the report discusses the results of the post development modelling and compares the water levels to the baseline model results.

9.2 Maximum Water Levels

The River Banwell maximum water levels for Options 2A and 2D with both winter and summer penning at New Bow Sluice can be found in tabulated form in Appendix B.

Figure 9.1 shows a comparison of peak water levels for Options 2A and 2D for selected return periods.

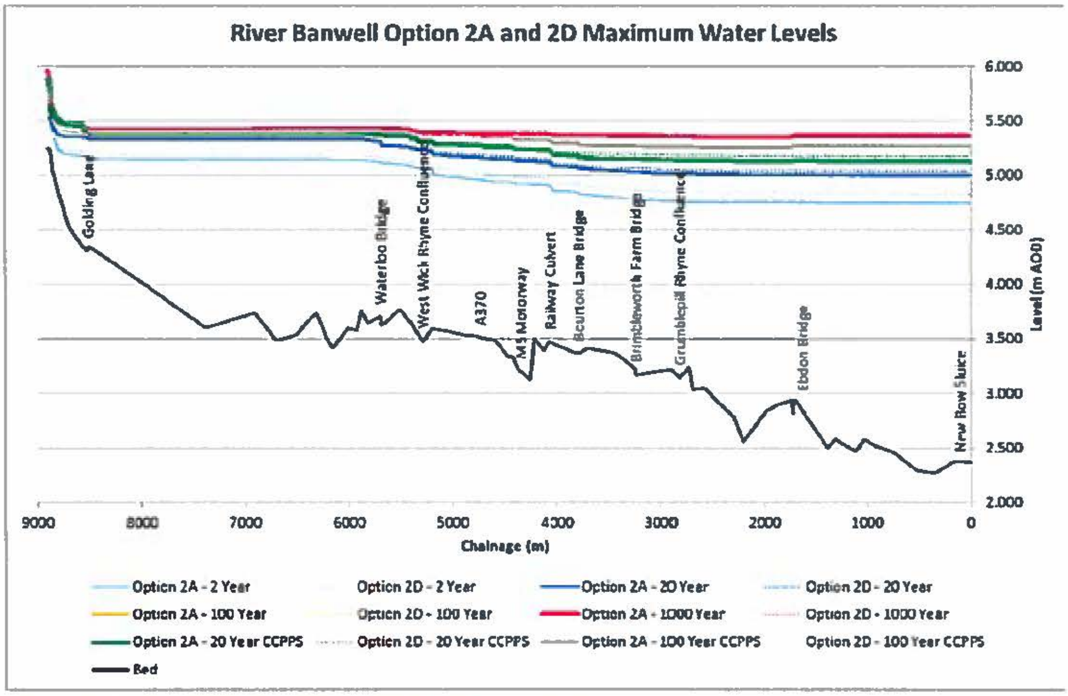


Figure 9.1: River Banwell Option 2A and 2D Peak Water Levels

Figure 9.1 shows that the level at which the floodplain is lowered will affect the peak water level along the River Banwell. The difference in peak water level of the two options is more marked at lower return periods.

Figure 9.2 shows the difference in peak water levels for summer and winter penning at New Bow Sluice for Option 2A for selected return periods. The results show that the peak water level is only affected in low magnitude events. For events greater than 1 in 20 years, the water levels in the channel are almost identical, irrespective of penning level.

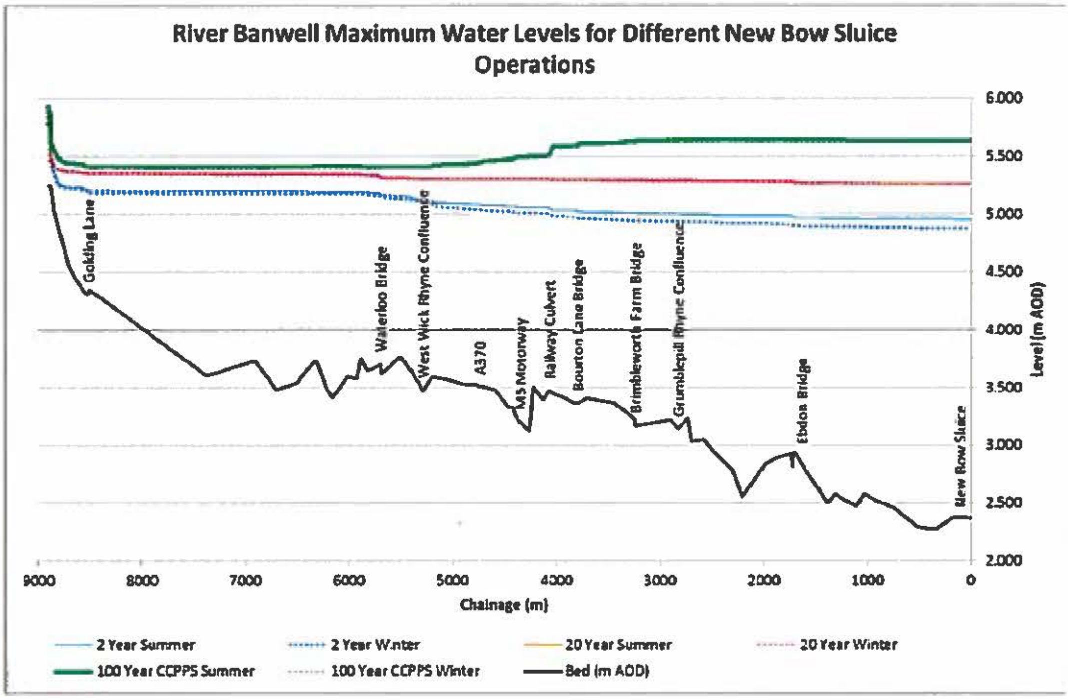


Figure 9.2: River Banwell Option 2A Summer and Winter Penning Peak Water Levels

Figure 9.3 shows a comparison of peak water levels in the River Banwell for the baseline and Option 2A scenarios.

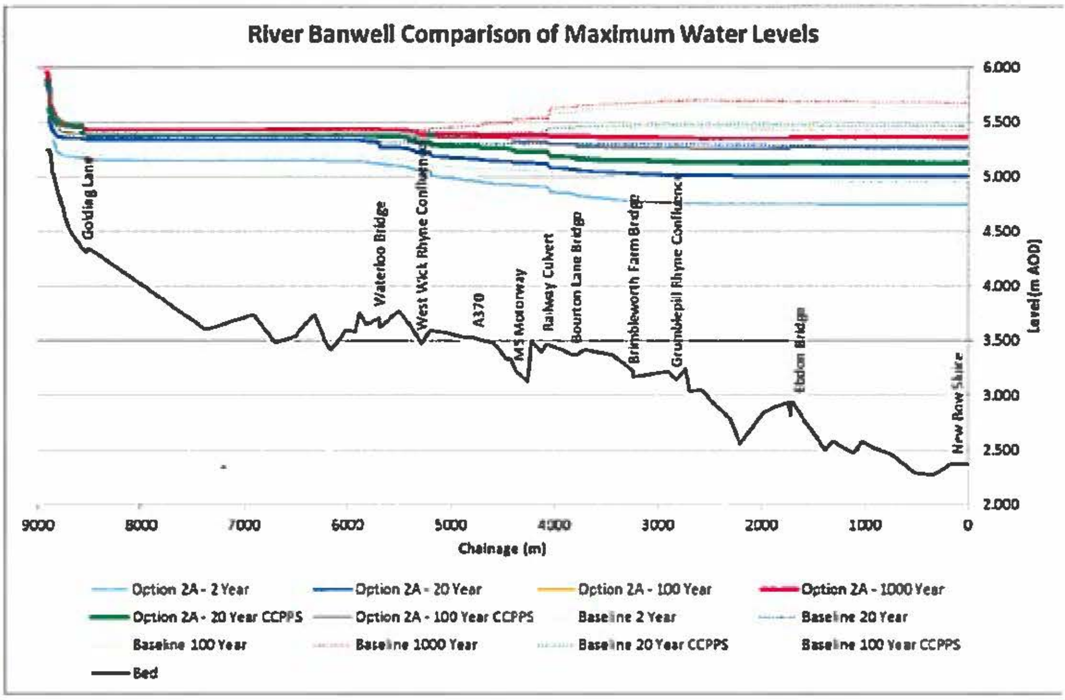


Figure 9.3: River Banwell Option 2A and 2D Peak Water Levels

Water levels in other parts of the system fluctuate depending on proximity to the lowered floodplain and increase (if any) in development runoff. Figures 9.4 and 9.5 show the long section results for the 1 in 100 year return period inclusive of climate change in Cross and West Wick Rhynes.

Cross Rhyne peak water levels increase in the Option scenarios by approximately 70mm due to the increased runoff from the new development, and the loss of floodplain storage in the area. This is not a significant increase and minor rhyne improvements within this area will help to mitigate this. This also has a knock on effect in the upper reaches of the West Wick Rhyne. West Wick Rhyne water levels are lower downstream of the M5 flapped outlets for the Option as the water levels in this reach are directly affected by levels in the River Banwell. Upstream of the M5 flapped outlets water levels are higher with the option due to the increased runoff from the new developments.

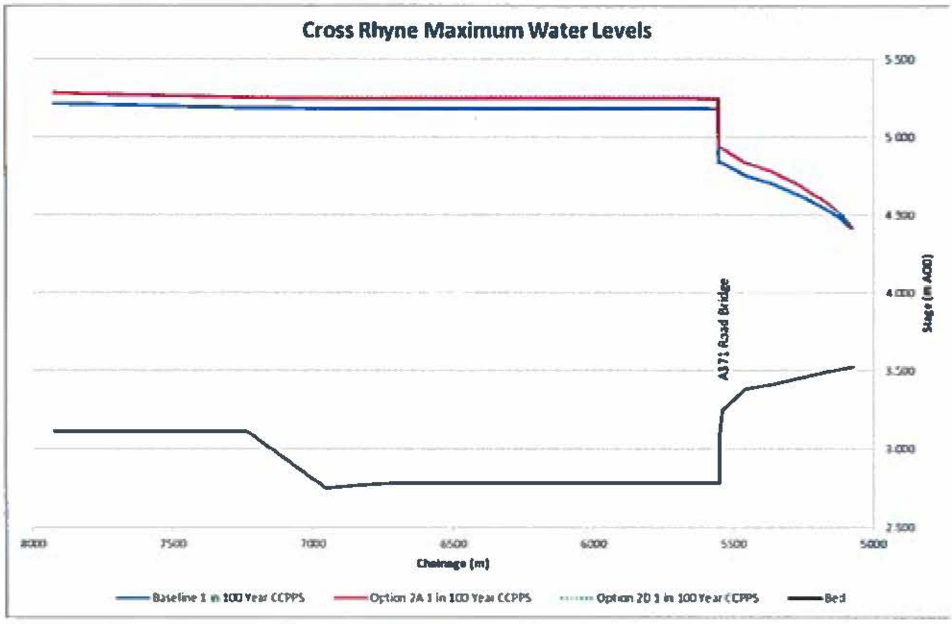


Figure 9.4: Cross Rhyne Option 2A and 2D Peak Water Levels

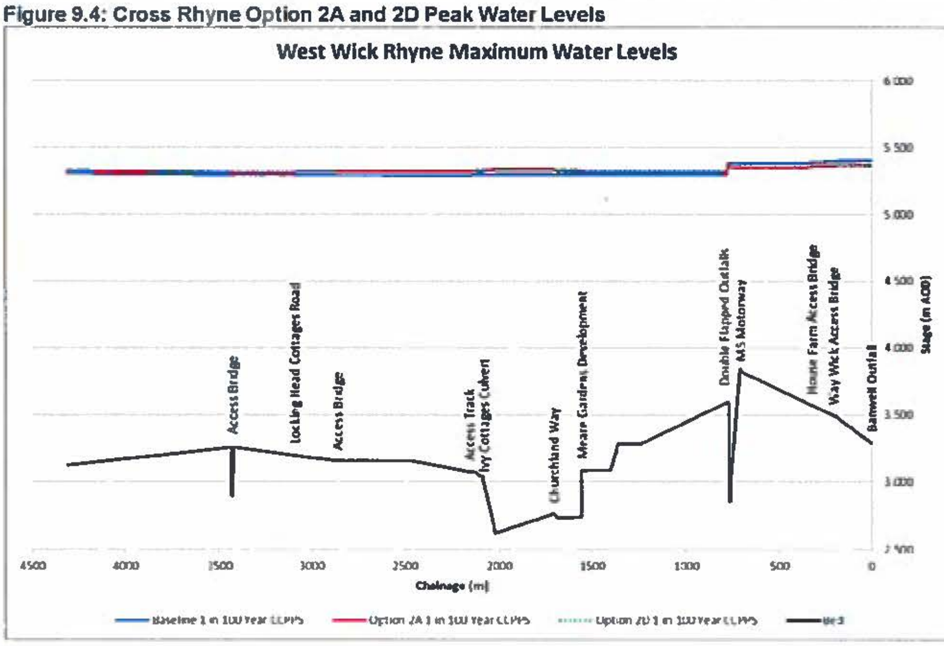


Figure 9.5: West Wick Rhyne Option 2A and 2D Peak Water Level

9.3 Flood Extent

In the 1 in 2 year event the flood extent shows flooding along the drainage network with the main concentration of flooding around the A370 and A371 boundary. The main flood extent areas to the east of Way Wick (right bank of River Banwell), south east of the Whorle train station and the area between the A370 and A371 increase as the magnitude of flood events increase.

Post development there is a significant reduction in the flood extents east of the M5, near West Wick. This is shown in Figure 9.6, which compare the baseline against the post development flood extents.

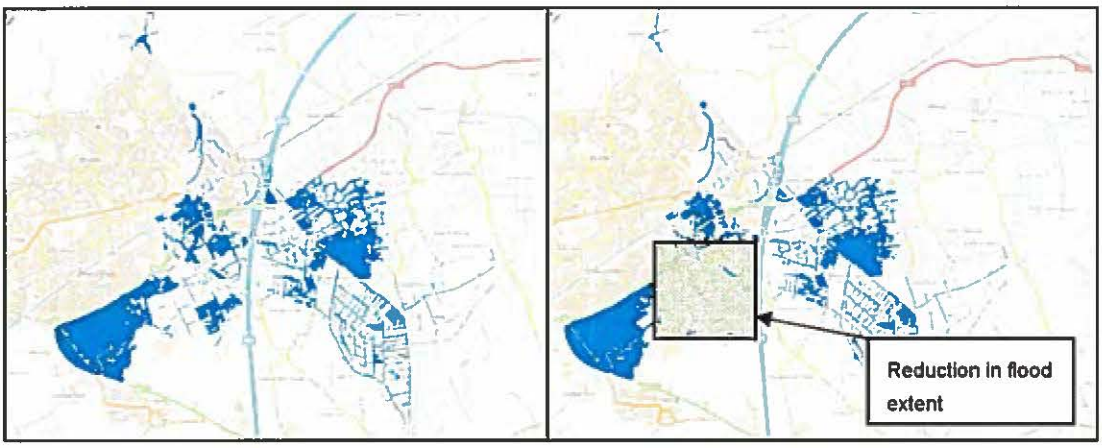


Figure 9.6: Flood extents for 1 in 100 year event; left: baseline, right; post development

During flood events greater than the 1 in 100 year event, there is a significant increase in flood extent across the whole area. In particular along the left bank of the River Banwell, upstream of the M5 and a smaller area of flooding becomes more prominent south of West Wick.

9.4 Frequency of Flooding

It is hard to determine how often the ground that has been lowered as part of the option will be utilised as it is not possible to predict future weather events. However it is possible to analyse historical data to obtain an idea how often it may occur in the future if similar weather patterns occur.

Analysis has been undertaken on the frequency of inundation of the lower floodplain utilising the St Georges level data collected from 1992 to 2005. St Georges gauge has been used rather than Waterloo Bridge as it is much closer to the site where ground lowering will occur. Levels above 4mODN are plotted in Figure 9.7. Figure 9.7 only shows data from 1997 onwards due to no data being recorded above 4.0mODN prior to this date.

The hydraulic model suggests that as water levels rise in the River Banwell, there is an approximate 100mm difference in water level between nodes Banw_4129 (St Georges Gauge) and Banw_3440 (upstream limit of the ground lowering). Therefore it is believed that a recorded water level of 4.5mODN or 4.85mODN would be sufficient to cause water to flow onto the lowered ground in Options 2A and 2D respectively.

The relationship between the St Georges gauge and the northern limit of the ground lowering described above may be affected by tide locking and therefore the assessment of how often the lowered ground will be flooded has included a 100mm bound i.e. considering a level of 4.4mODN to 4.5mODN for Option 2A and 4.75mODN to 4.85mODN for Option 2D. This is shown in Figure 9.7.

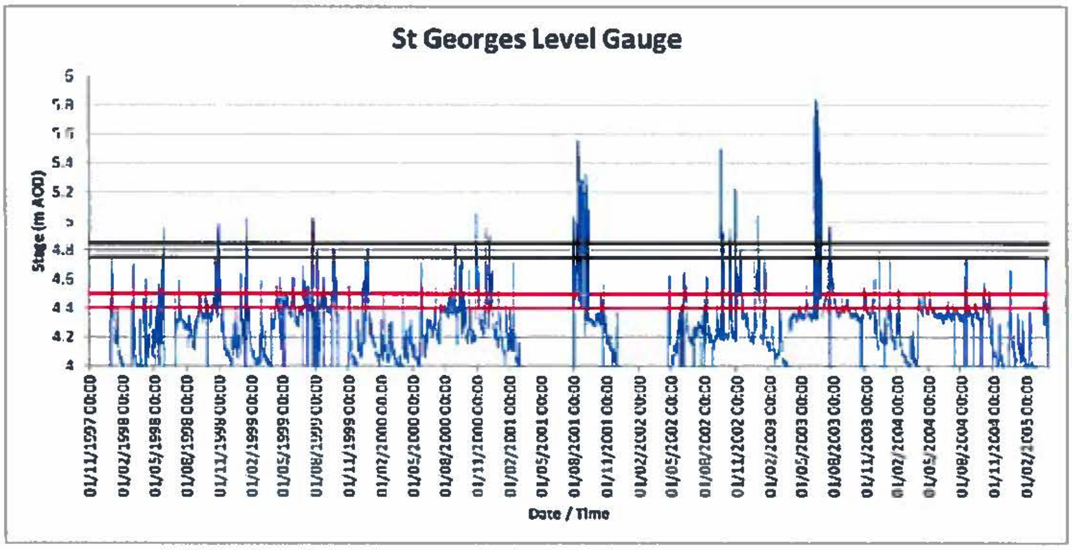


Figure 9.7: St Georges Gauge Historic Level Data

There are some issues with the data for August 2001 and July 2003, where the water levels appear to be very 'spikey'. The analysis was undertaken assuming these periods of the record was only one large event as it was not possible to differentiate if they were several events or 'noise' within the data.

If Option 2A is considered, the historic record suggests that the water level rose above 4.4mODN on approximately 150 occasions and above 4.5mODN approximately 100 times within the 7.5 years of record analysed. This suggests that the lowered floodplain would flood 13 to 20 times a year on average.

If option 2D is considered, the historic record suggests that the water level rose above 4.75mODN on approximately 25 occasions and above 4.85mODN approximately 15 times within the 7.5 years of record analysed. This suggests the lowered floodplain would flood 2 to 4 times a year on average.

9.5 Duration of Flooding

The change in geometry of the channel in Options 2A and 2D not only cause a change in the peak water level in the River Banwell, but a change in the shape of the hydrograph. Figures 9.8 to 9.11 show a series of hydrographs (47 hour storm duration) at a series of model node locations. Table 9.1 gives reference to the location of the model nodes in Figures 9.8 to 9.11.

Table 9.1 – Model Node Locations

Model Node	Location
Banw_3440	River Banwell at the upstream end of the reach where floodplain lowering will be undertaken.
lhb_2904	Series of ponds upstream of the outfall of the Grumblepill into the River Banwell
We_4310	Right hand bank floodplain between the West Wick and Cross Rhyne upstream of the A371 crossing
Wolv_0570	Wolvershill Rhyne between the golf course off Churchland Way and the Grumblepill Rhyne

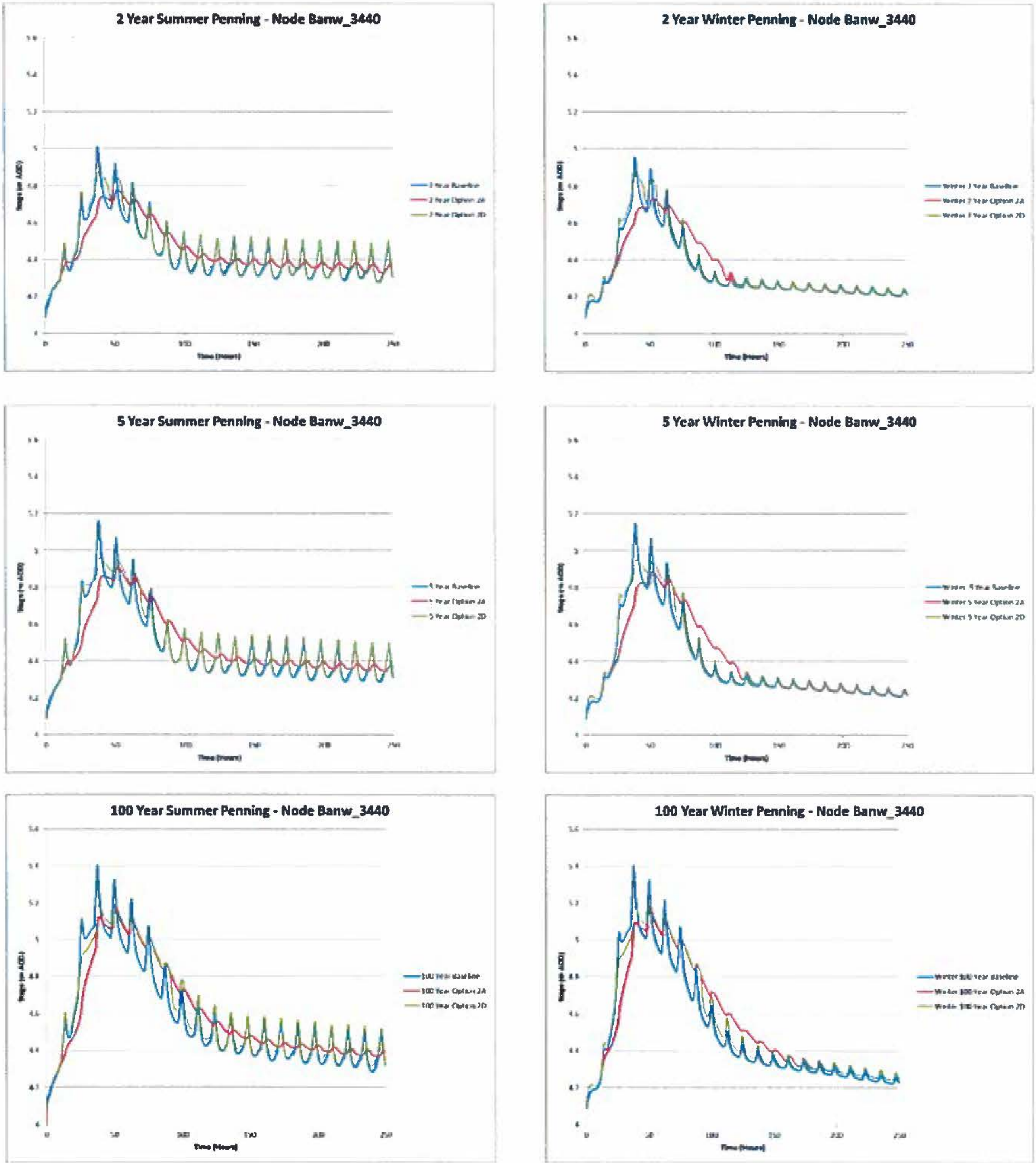


Figure 9.8: Duration of Flooding at the Proposed Lowered Floodplain

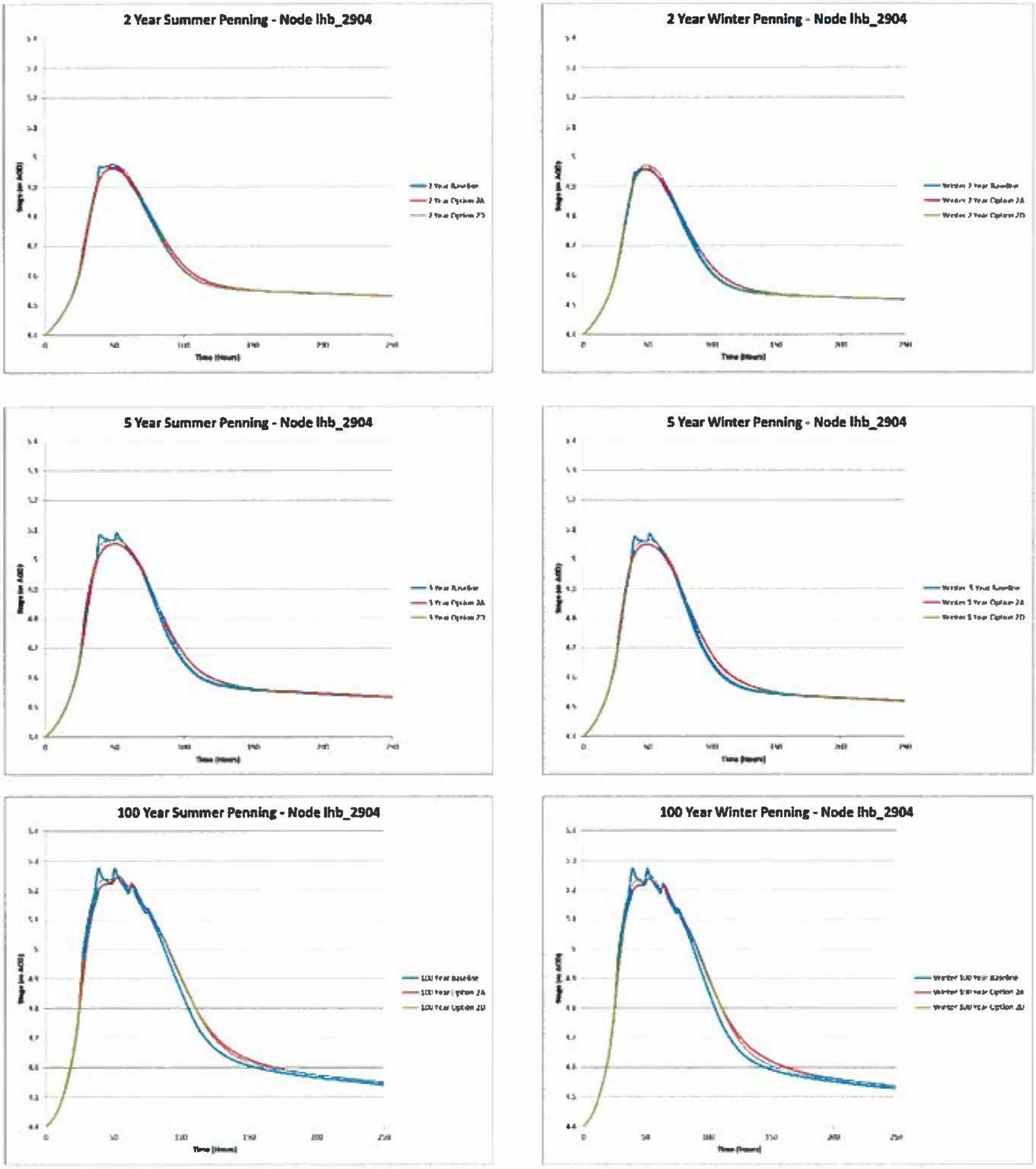


Figure 9.9: Duration of Flooding in the Grumblepill Rhyne (Outfall)

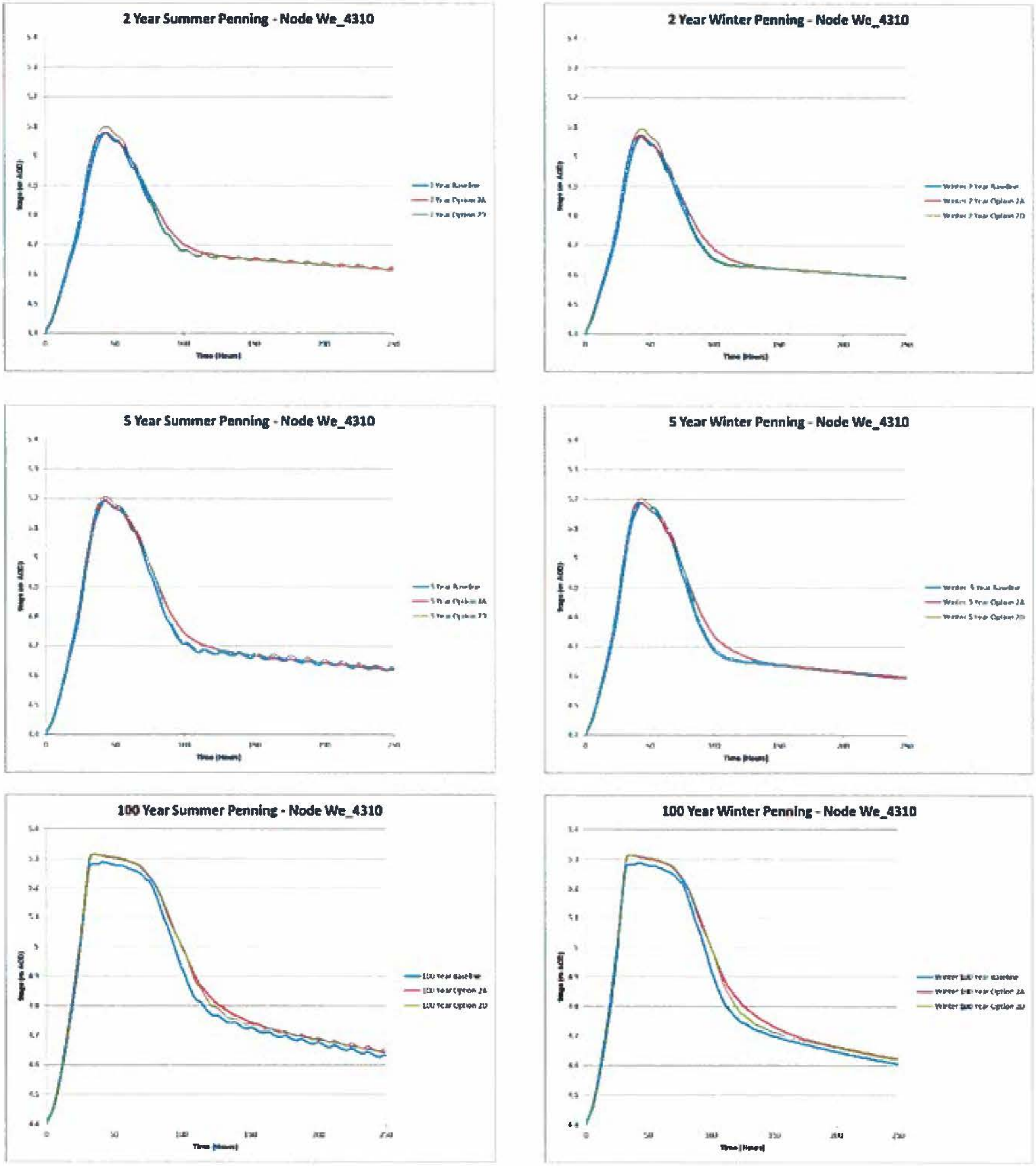


Figure 9.10: Duration of Flooding in the Floodplain Upstream of the A370 on Cross Rhyne

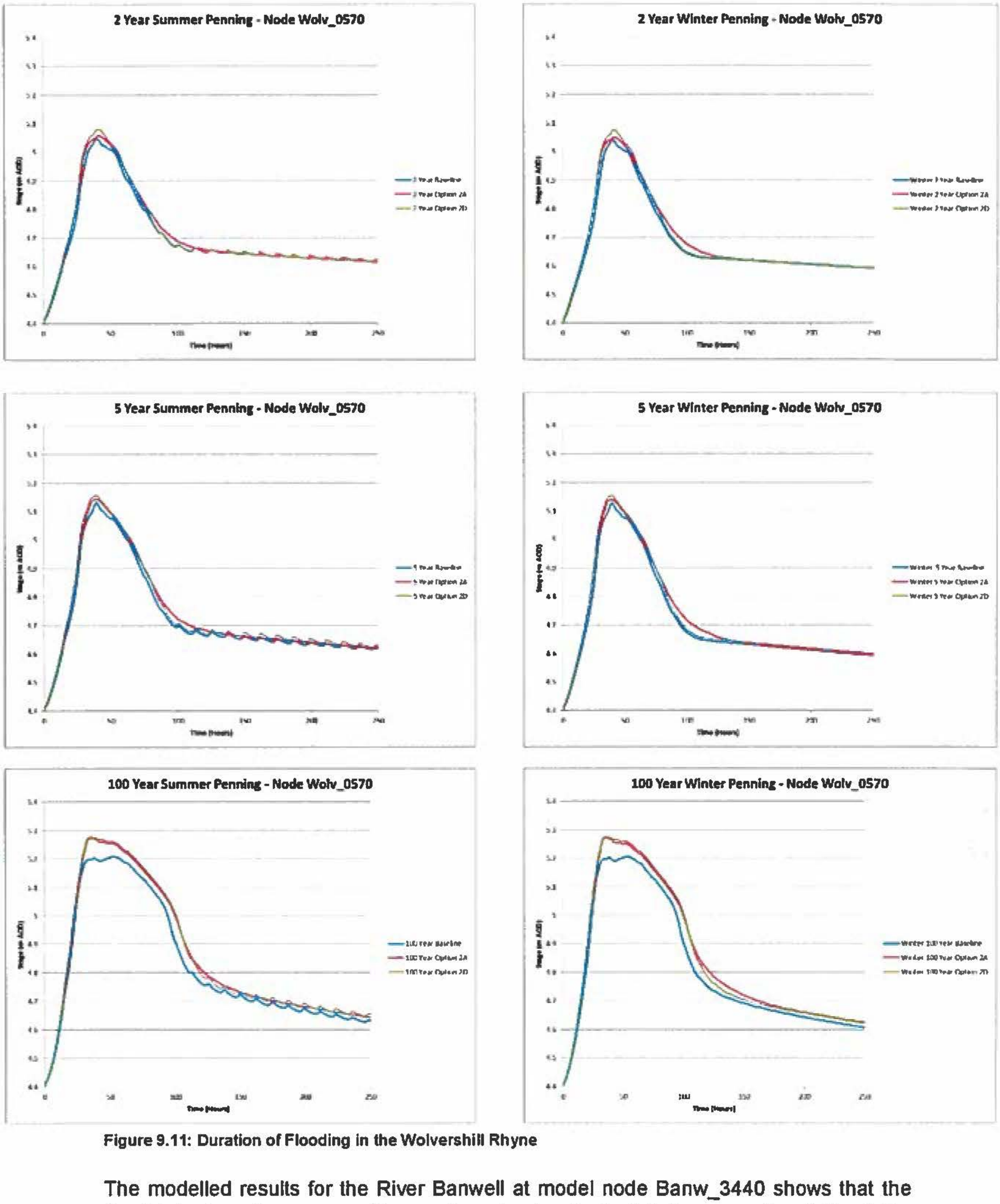


Figure 9.11: Duration of Flooding in the Wolvershill Rhyne

The modelled results for the River Banwell at model node Banw_3440 shows that the floodplain lowering causes the peak water levels to fall however the 'tail' of the hydrograph is extended as there is a greater volume of water stored in the reach downstream of the railway culvert compared to the baseline condition. The elevation at which the floodplain lowering is taken has a small impact on the peak level but more of an impact at the rate the floodplain drains and the duration of flooding experienced.

The penning level at New Bow Sluice also determines the time it takes to drain the lowered floodplain. For example in a 1 in 100 year event, water will drain from the floodplain for option 2A at approximately 200 hours for summer penning but at 150 hours for winter penning; therefore there is a difference of 2 days in inundation. From historical evidence it is more likely that larger fluvial events will be observed in winter therefore drainage of the floodplain in the winter penning scenario is most likely to be experienced.

It is expected therefore that under winter penning conditions Option 2A would inundate the lowered floodplain i.e. compound channel, for approximately 180 hours for a 1 in 100 year 47 hour storm event, while Option 2D would inundate the floodplain for approximately 70 hours for the same event. This is because the River Banwell starts to spill into the compound channel later in Option 2D compared to Option 2A, due to the difference in spill height. The duration of flooding is therefore very sensitive to the depth at which the compound channel is set at, however due to the flat nature of the area the peak water level at this location will be similar for both options.

In the Grumblepill Rhyne, water levels are reduced by both options but it does take a slightly longer period of time to drain the system than in the baseline condition. This is mainly due to increased runoff from the proposed development.

Ian Sturdy.

Levels in the floodplain of the Cross Rhyne and Wolvershill Rhyne are increased and remain higher for longer under the post development scenarios. This is not a result of the Banwell scheme. It is purely because the development will generate increased runoff and will reduce the floodplain storage volume as ground levels will be raised out of flood risk.

There is little means of conveyance from these areas to the River Banwell other than via four 300mm flapped outlets (two on the Wolvershill cut-off and two on the Locking cut off) and the Wolvershill culvert. The Wolvershill culvert often backs up from high water levels in the River Banwell and West Wick Rhyne; however the water level drops with the lowered floodplain in place. The extra volume of runoff however is greater than that of the capability to transfer water from these areas to the River Banwell and therefore water levels are increased. It should be noted that the scenarios modelled are the worst case scenario to determine maximum water levels in terms of ground raising for the new development. A further parallel study into a flood scheme on the Cross Rhyne Uphill catchment would help to reduce the increased water levels seen in these areas.

10 CONCLUSIONS

A hydraulic model of the river Banwell and part of the IDB Rhyne system has been constructed using an existing HECRAS model and additional survey data. A number of structures have been included in the model in order to help calibrate the model and predict current and future flooding conditions within the system.

The structure with the most significant effect on water levels in the entire system is New Bow Sluice at the outfall of the River Banwell which prevents tidal ingress but also 'tide locks' fluvial flows from being released during high tide.

New Bow Sluice operates with a summer and winter penning control where water levels are managed at a desired level of 4.1 and 3.2mAOD respectively. For events greater than a 1 in 20 year return period, the penning levels used at New Bow Sluice have no effect on the peak water levels within the river channel although they do have an effect on how quickly the system drains.

The hydraulic model has been calibrated to a number of historic events. Through this process, it became evident that the calibration criteria used to calibrate the model were different to that needed to calibrate the adjacent Uphill Cross Rhyne catchment. Therefore the final calibration parameters that have also been taken forward into the design runs have been tailored so that both sets of parameters are more similar.

The model has been used to determine flood risk throughout the catchment for a present and post development scenario.

The post development scenario includes an additional volume discharged into the watercourses due to increased runoff from the change in land use and a decrease in floodplain storage in the area of the development as ground levels will be raised above levels of modelled flood risk.

The post development scenario also includes a scheme along the River Banwell, downstream of the Railway. This includes lowering of the floodplain on the right hand bank of the River Banwell to allow additional storage within an extended two stage channel. This, along with automation of the second sluice at New Bow Sluice (currently only used for maintenance and on an ad-hoc basis for large flood events) reduces the peak levels in the River Banwell in the post development scenario.

Although the additional storage helps to reduce water levels in the River Banwell, the system takes longer to drain with the storage in place as a larger volume of water is held within the river.