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Further Discussion

2.6.15 As part of the River Wye catchment is permeable there is a risk that reliance on the statistical method may under-estimate flood flows. There is a risk that the right combination of antecedent conditions could prime the whole catchment, resulting in a more extreme flood response. Although the record at Ashford is of good quality and good length there are insufficient data to be confident that a full range of flood response have been captured – a much longer record would be needed.

2.6.16 Uncertainty in flood estimates in permeable/semi-permeable catchments is not specific to the River Wye and previous studies by Halcrow and JBA Consulting have attempted to address this issue through application of continuous simulation methods. In these studies, for Salisbury and Doncaster respectively, Probability Distributed Model have been calibrated and run for extensive periods using synthetic rainfall time series. Frequency analysis are then applied to the resulting predicted flow series. The results from both these studies have been promising but two important lessons have been learnt:

- The need for good quality rainfall data for calibration of the PDM model;
- The need to adjust results to be consistent with FEH Depth Duration Frequency statistics.

2.6.17 Application of continuous simulation methods was beyond the scope of this flood mapping study. Future application could be considered but the availability of flow data at Buxton would provide improved understanding of the differing response across the catchment and hence improved confidence in the performance of continuous simulation rainfall-runoff modelling.

2.6.18 A more detailed reconstructed flood history is recommended as a way of addressing uncertainty in the flood flows. For this study, uncertainty has been addressed through comparison of flood outlines for a range of flows. This analysis is described further below in addition to the importance of design hydrograph duration and volume.

2.7 *Design Storm Criteria*

2.7.1 An initial estimate of critical storm duration was produced using the automatic method in ReFH units for lumped catchments to Buxton and Ashford. An initial estimate of approximately 3.5 hours for Buxton and 9.5 hours were produced for Ashford in this way.

2.7.2 To further inform the selection of design events storm durations, observed events were plotted against dummy 1 % AEP flows and level time series generated from the hydraulic model for a range of storm durations. These comparisons are shown in Figures 2.28 and

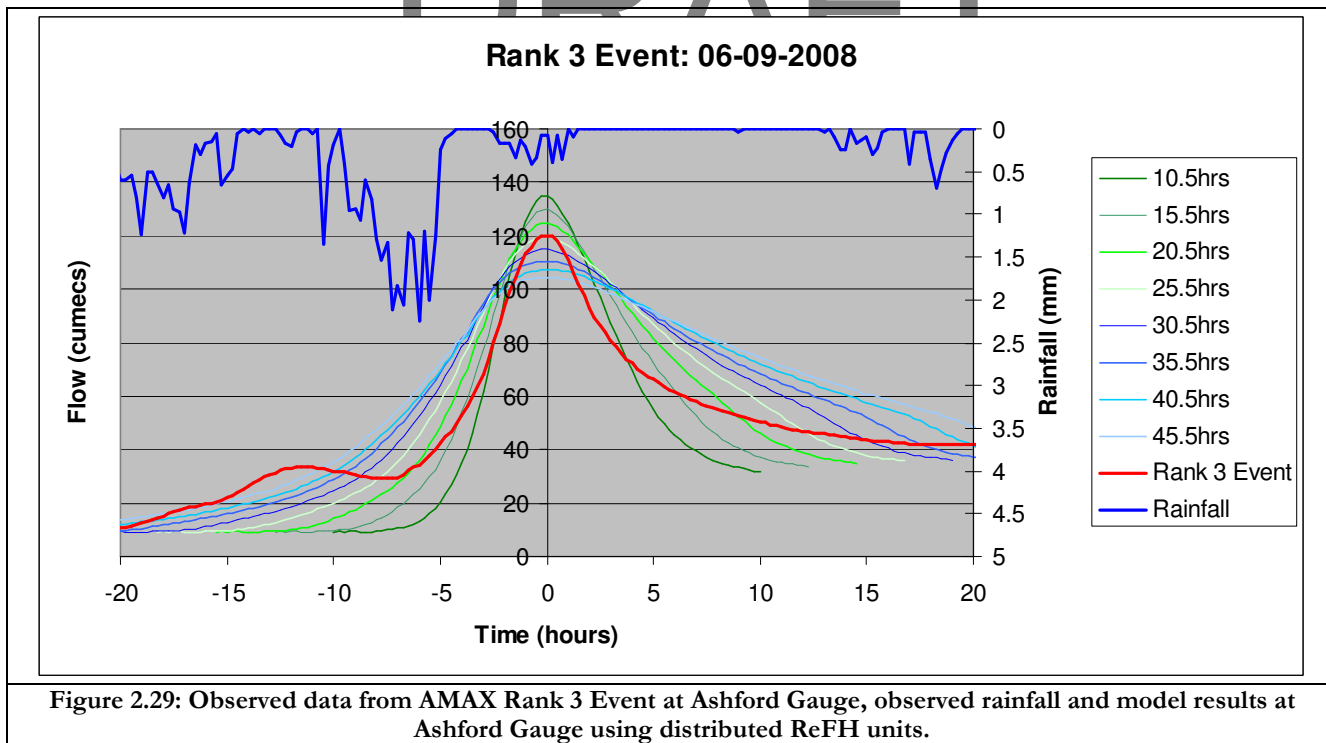
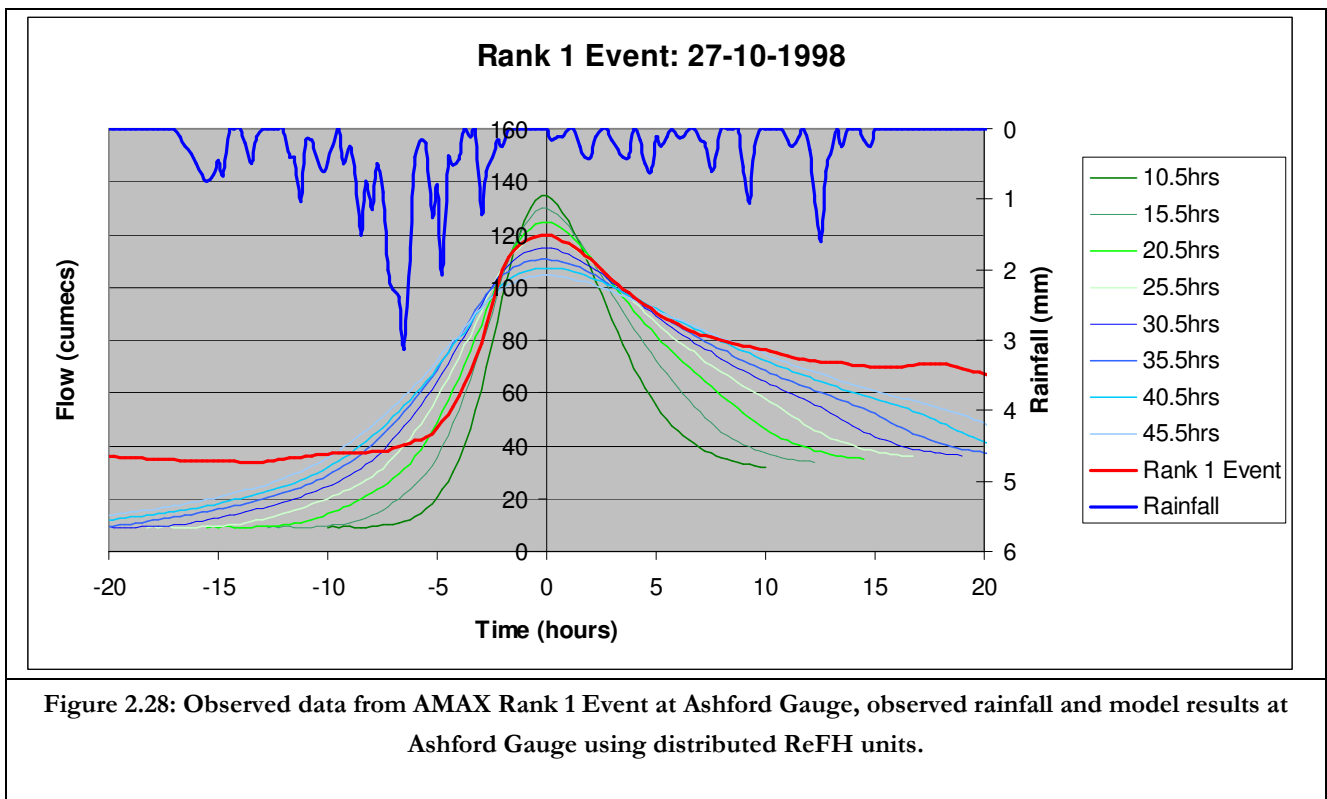
2.29 for Ashford and Figures 2.30 and 2.31 for Buxton. Dummy design flows are generated using unscaled inflows, prior to reconciliation with the preferred flood frequency curve at Ashford and are therefore significantly higher than adopted design flows. To facilitate comparison of hydrograph shape, observed flows have been scaled up to a similar magnitude to the dummy 1% AEP flows.

2.7.3 Through visual inspection of the results, the best match between observed and design hydrograph shape was identified as being 15.5 hr storm for Ashford, a little longer than the initial estimate of 9.5 hours produced using the automatic method. The original automatic method estimate of 3.5 hours at Buxton appeared to be reasonable in comparison with the observed events, so this figure was retained. It should be noted that although the 10.5 hr storm produced a higher peak flow at Ashford, and the 2.5hr storm produced a higher peak stage at Buxton, that design storm durations were chosen on the basis of hydrograph shape rather than peak, as peak flows would be based on statistical analysis. A comparison of peak flows for various storm durations at Ashford is given in Table 2.5. This table illustrates that peak flows are not particularly sensitive to storm duration at around the storm durations which match observed hydrograph shapes. Further details are provided in Appendix E.

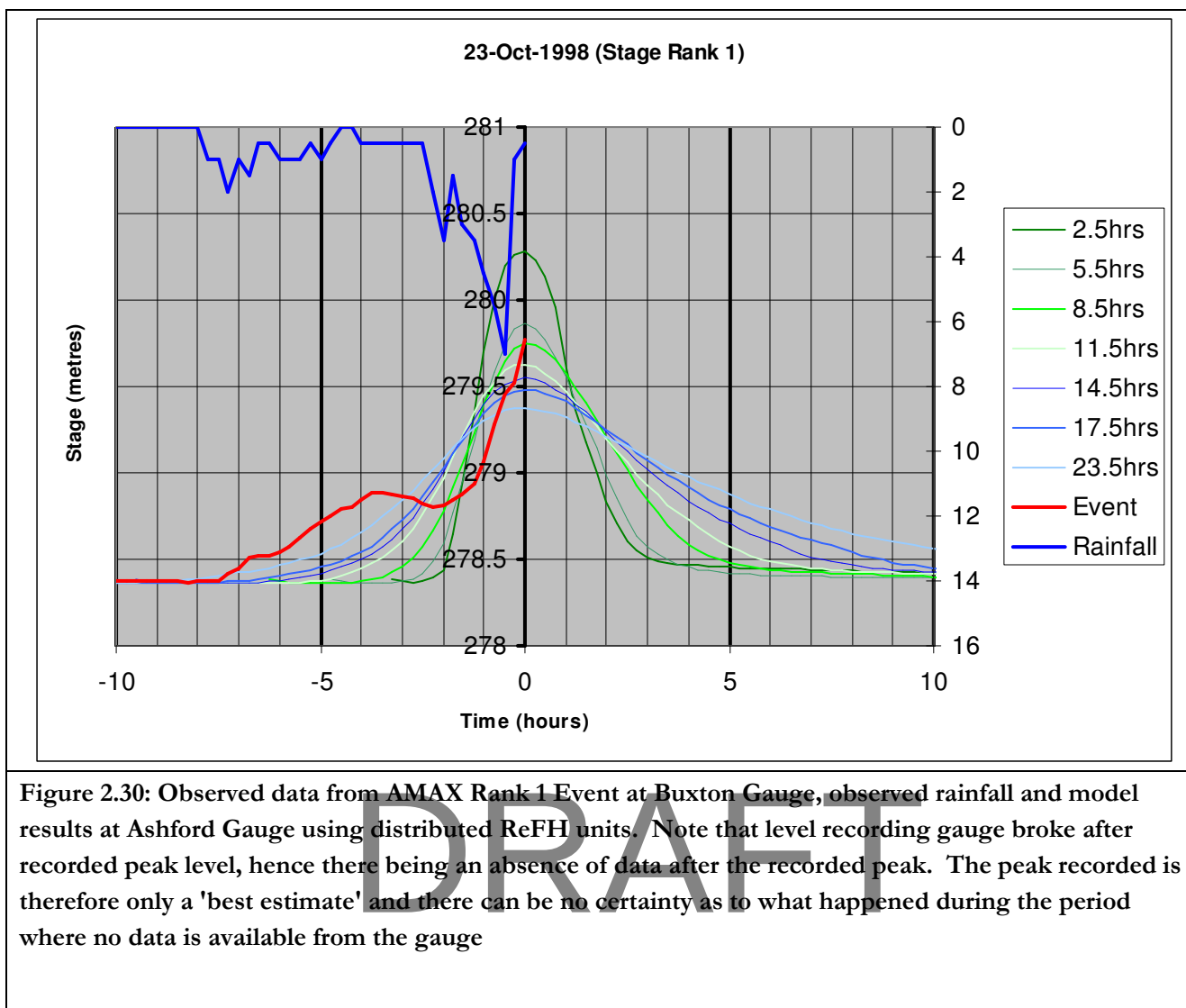
Table 2.5: Peak Flow for various storm durations at Ashford using distributed ReFH model.

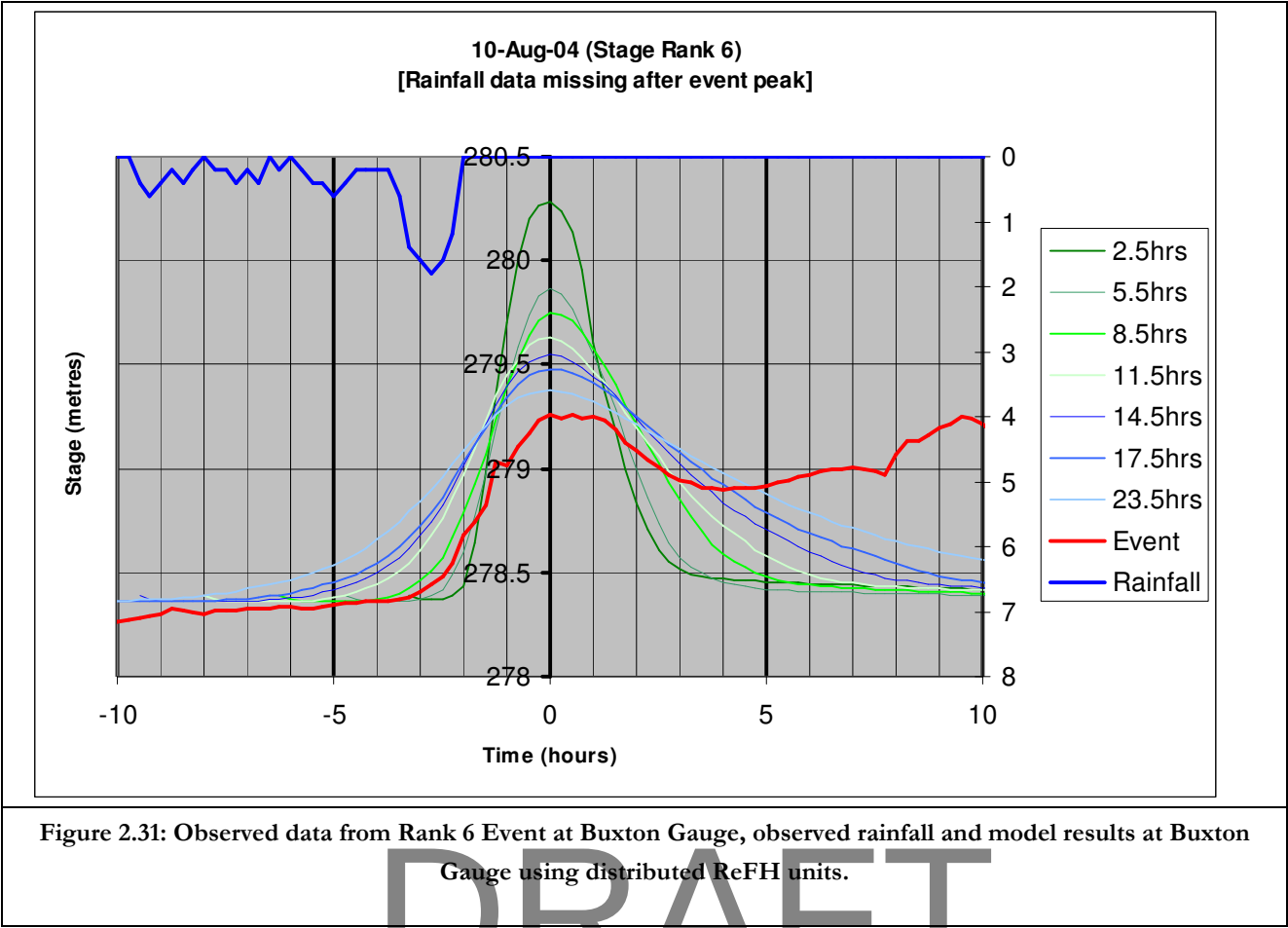
Storm Duration (hours)	Peak Flow (cumecs)
10.5	134.721
15.5	129.762
20.5	124.789
25.5	119.907
30.5	114.903
35.5	110.459
40.5	107.231
45.5	104.437

2.7.4 Design simulations were therefore undertaken for the 3.5 hour and 15.5 hour storm duration.



Buxton





2.8 Reconciliation to Single Site Statistical Analysis

2.8.1 Unscaled peak flows at Ashford, generated using ReFH boundary units with default catchment descriptors routed through the hydraulic models, are shown in Figure 2.32. In comparison with the preferred statistical flood frequency curve (single), the unscaled inflows can be seen to generate significantly higher flows. This result is consistent with results for the two hydraulic model verification events where scaling of inflows was also required to match the observed peak flow at Ashford and to provide a satisfactory fit the distributed wrack mark data.

2.8.2 To achieve a satisfactory fit with the preferred flood frequency curve, inflows generated using a 15.5 hr event, were scaled for each design simulation. The required scaling factors are shown in Table 2.6 with the final design flows generated using model shown in Figure 2.33. Calculations supporting this are in Appendix F – Scaling Calculations at Ashford.

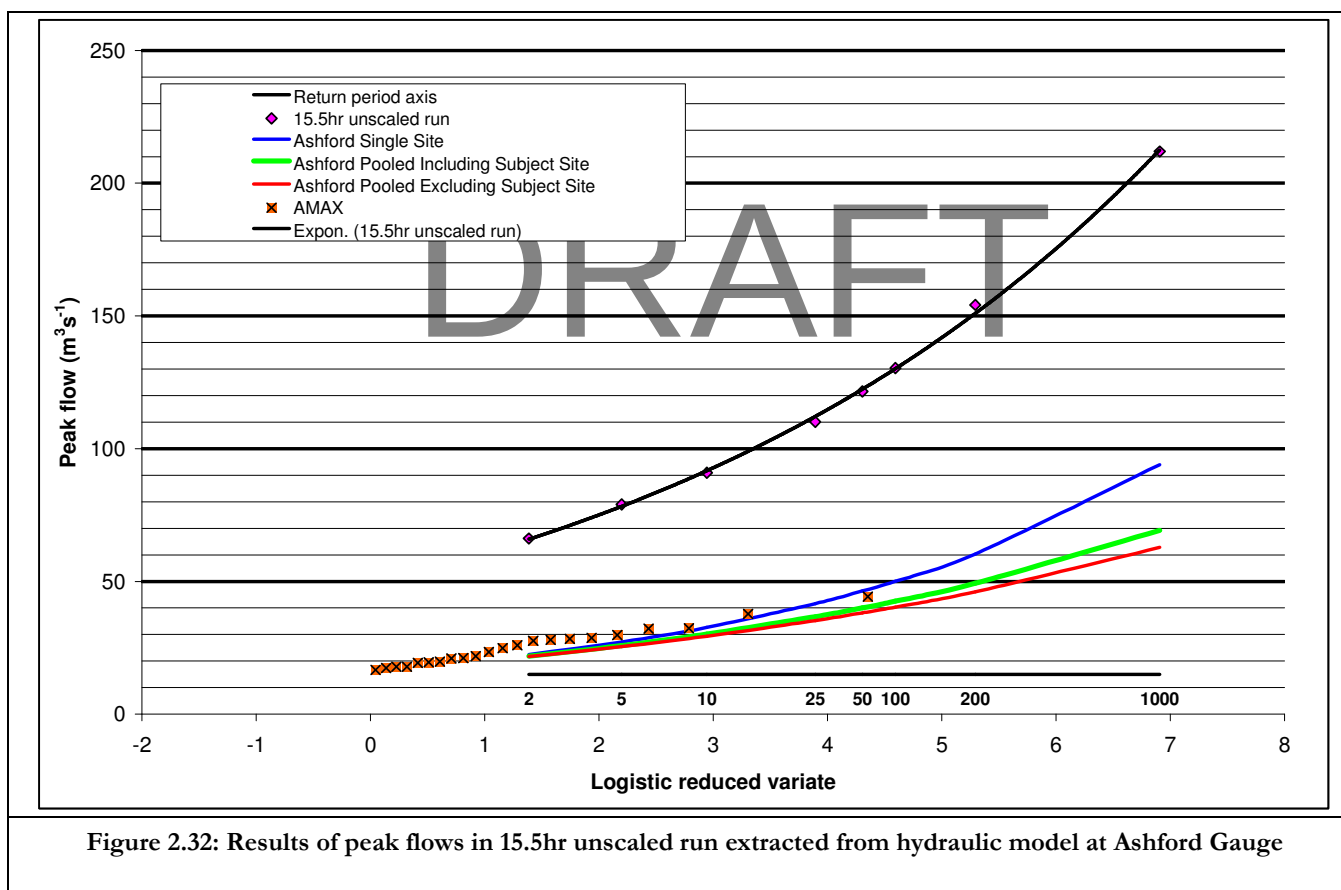


Table 2.6: Results of peak flows in 15.5hr unscaled run extracted from hydraulic model at Ashford Gauge and scaling factors required to match single site statistical results

Return Period	Logistic reduced variable	Target Curve (Ashford Single Site)	15.5 hrs	
			Original Model Results	Scaling Factor
5	1.39	22.44	66.218	0.339
10	2.20	27.20	79.002	0.344
20	2.94	32.71	90.98	0.360
50	3.89	41.65	110.076	0.378
75	4.30	46.38	121.575	0.381
100	4.60	50.08	130.375	0.384
200	5.29	60.37	154.164	0.392
1000	6.91	93.94	212	0.443

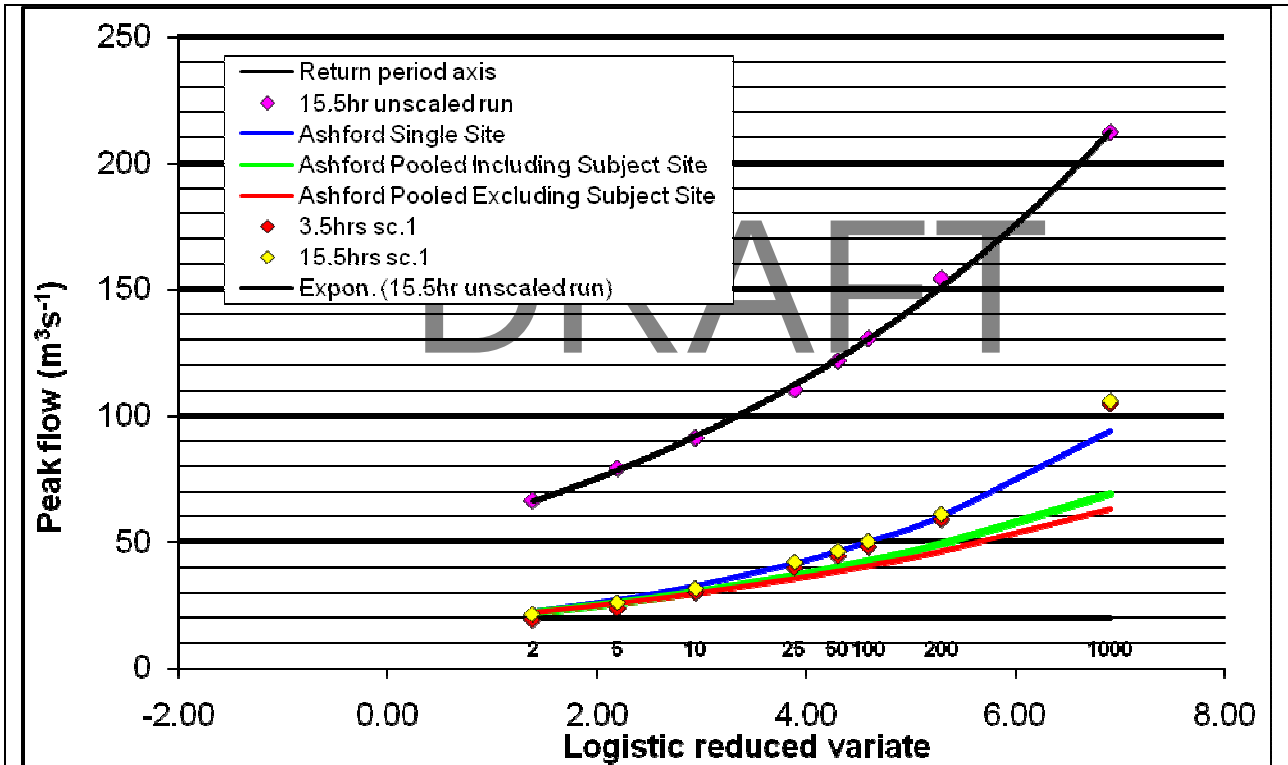


Figure 2.33: Results of peak flows in 15.5hr unscaled run extracted from hydraulic model at Ashford Gauge

3 Hydraulic Modelling

3.1 Introduction

- 3.1.1 This section outlines the hydraulic modelling undertaken as part of the River Wye SFRM2 flood risk mapping study.

3.2 Model Extents

- 3.2.1 The modelled extents of the combined 1D-2D ISIS-TUFLOW model constructed in this study are from Burbage in Buxton (SK 04144 72682) to downstream of Bakewell (SK 22828 67190). Two tributaries of the River Wye have also been modelled; the Hogshaw Brook (SK 05971 74262) and the Nunn Brook (SK 06457 74294) to their confluence with the River Wye (SK 06372 73570). A section of the model is 1D only, this extends from the Mineral Railway (SK 08688 72391) to The Rookery Bridge (SK 19047 69464), this section has been modelled in 1D only because the watercourse is contained within a very narrow steep sided valley with very little floodplain so a 2D floodplain is unnecessary in this area.

3.3 Model Background

- 3.3.1 The River Wye model was received as two separate models. The Upper Wye model, covering the area from the upstream of Buxton (SK 04144 72682) to Ashford gauge (SK 18268 69698), was built by HR Wallingford in 1999. The section from Ashford gauge to downstream of Bakewell (SK 22828 67190) was covered by the Parsons Brinckerhoff (PB) model from 2002. The Wallingford model originally went downstream as far as the confluence with the River Derwent but PB was commissioned to improve the modelling through Bakewell in 2002.

3.4 Data Availability

- 3.4.1 A number of topographic surveys have been carried along the River Wye as part of the earlier studies. This information was provided by the EA in either digital or hardcopy format, along with other useful data, and is summarised below:

- Detailed Aerial Survey through Bakewell covering the floodplain and urbanised areas, Parsons Brinckerhoff 2002;
- Aerial survey on the Wye from Buxton down to the confluence with the River Derwent. Mason Land Surveys, July 1997;
- LiDAR on the River Wye up to and including Buxton. Flown March 2008, Environment Agency Geomatics Group;
- River Wye Channel Cross Section Survey, Survey Operations Ltd, January 1999;

- River Wye Channel Cross Section Survey though Bakewell from Bridge Street Bridge (SK 21955 68651) to weir at SK 21989 68402, Cartographical Surveys Ltd., July 1999;
- River Wye Channel Cross Section Survey though Ashford and Bakewell from Ashford gauge (SK 18277 69704) to weir at SK 22848 67296, Mott MacDonald April 1998;
- Hogshaw Brook Channel Cross Section Survey and ISIS *.DAT file, Cartographical Surveys Ltd, March 2009;
- Nunn Brook Channel Cross Section Survey and ISIS *.DAT file, Cartographical Surveys Ltd., March 2009;
- Ashford in the Water property threshold level survey, Cartographical Surveys Ltd., January 2001;
- Environment Agency stop log survey (levels), January 2009, Flood Incident Management;
- River Wye, Bakewell Scheme ISIS Model, Parsons Brinkerhoff, June 2002;
- River Derwent – Wye Model and Floodplain Mapping, HR Wallingford, June 1999 (superseded by Parsons Brinckerhoff study in Bakewell);
- Culvert surveys and CCTV footage for culverts through Buxton;
- GIS layers with wrack mark data from historical flood events;
- GIS layer with model node locations for the 1999 Wallingford model;
- Photographs of historical flooding through Bakewell;
- NFCDD data for the River Wye through Buxton and Bakewell.

3.4.2 In addition to the above existing data, additional survey was undertaken as part of this commission; details of which are provided in Appendix G.

3.5 Model Reviews

3.5.1 Both the Wallingford (Buxton) and PB (Bakewell) models underwent a detailed review prior to use in this study.

3.5.2 Reviewing the Bakewell model was made difficult by the absence of any model visualiser or GIS / geo-referenced data and few labels or comments in the model itself. This made the

construction of a model schematic both more important, in order to understand the model, and more difficult, particularly given the complex nature of the watercourse. The schematisation of the PB model in Bakewell was found to be good apart from:

- Some 'glass-walling' taking place in a mill race near the Fire Station (near model node B206) - as well as over-estimating peak river levels in this area, this was causing some numerical non-convergence and instability when the model was run;
- A sluice gate and 110m section of channel was missing just downstream of Ashford (near model node A101).

3.5.3 The numerical performance of the Bakewell model was good.

3.5.4 The Buxton model was reviewed after the initial site visit and the following limitations were discovered, as follows:

- Many of the chainages in the Buxton area were incorrect in the model and many structures missing, making it more difficult to identify where river sections should be;
- Some of the missing structures would have a significant effect locally. These included a number of weirs and bridges identified during the walkover site visit, including a large weir just upstream of The Square culvert. Neither were they in the channel survey data that was used to construct the 1D Buxton model. Because Buxton lies in a steep catchment it was important to include in-channel weirs, not only to represent the real situation (and peak water levels) properly, but also for reasons of model stability across the 1D-2D link;
- Culvert entry and exit units were not in the Buxton model. This would have impacted negatively on the calculation of head losses due to different flow regimes as well as not including the entry and exit head losses;
- The Buxton model only ran in steady state, not unsteady. It was found to be very unstable at low flows – this needed to be addressed in order to run hydrographs through the model in order to perform the velocity and depth mapping needed;
- Structures in the Buxton model, especially culvert dimensions, did not match up with survey data. Bed levels were not set to avoid the model running dry at low flows.

3.6 1D Model Improvements

3.6.1 Prior to undertaking the velocity and depth modelling and mapping it was agreed with the Environment Agency that the existing limitations in the 1D models needed to be addressed. This included gathering further survey data for the missing structures through Buxton and

Ashford gauge (31 additional sections in total). Halcrow Geomatics were commissioned to survey the additional sections – copies of their reports can be found in Appendix G.

- 3.6.2 The two models were combined into one and a GIS visualiser constructed to assist in understanding the models. All modelled structures were checked against the most up-to-date survey data and many structures updated, including Bridge Street culvert. Manholes were added to the culverts ready for linking to the 2D domain – these were particularly important for velocity and depth mapping through Buxton. Bed levels were reviewed and updated to assist in model stability and correctly model the real hydraulics of the watercourse. Chainages were corrected with reference to OS tiles in the GIS. Culvert entry and exit units were added to the culverted structures and general loss units added where necessary in order to provide a realistic estimate of head losses through culvert bends / expansions and contractions. Finally, all additional survey data was added to the combined 1D ISIS model and the model stabilised.

3.7 Model Construction

Model Type

- 3.7.1 The River Wye was modelled using ISIS-TUFLOW, this software links ISIS (version 3.3) 1D hydraulic modelling software and TUFLOW (version 2009-07-AE-iSP) 2D hydraulic modelling software. ISIS-TUFLOW has been used to model the towns of Buxton and Bakewell while ISIS has been used to model the reach in between. The model has been constructed this way to model the more complex floodplain interactions in the towns more accurately while the reach in between the towns is very steep with very no complex floodplain and so is best suited to be modelled in ISIS. TUFLOW is a computer program for simulating depth-averaged, two-dimensional free-surface flows such as occurs from floods. A powerful feature of TUFLOW is its ability to dynamically link to the 1D network hydrodynamic program ISIS. ISIS-TUFLOW incorporates the full functionality of the ISIS 1D network based on the full one-dimensional free-surface flow equations.

Model Node Labelling Convention

- 3.7.2 The existing labelling convention from the two existing 1D models was retained in order to assist future users of the modelling in identifying the origins of the various parts of the model. Through Buxton this consisted of WY###, where:

- The first two letters denote the watercourse;
- The three numbers refer to the surveyed section done by Survey Operations Ltd. in January 1999.

- 3.7.3 New surveyed sections added to the River Wye have a similar naming convention of WYA###, where:

- The first two letters denote the watercourse, the 'A' denotes an additional section;

- The three numbers refer to the nearest surveyed section done by Survey Operations Ltd. in January 1999.

3.7.4 New surveyed sections on the Hogshaw Brook and Nunn Brook are labelled as YY-###, where:

- The first two letters denote the watercourse (HB for Hogshaw Brook, NB for Nunn Brook);
- The three numbers refer to the chainage from the downstream confluence with the River Wye (for the Hogshaw Brook) and with the Hogshaw Brook (for the Nunn Brook).

3.7.5 Surveyed sections on the River Wye through Ashford and Bakewell are labelled as A###, B###, C### or D### according to the labelling in the Mott MacDonald survey of April 1998.

3.7.6 Interpolates are labelled with an 'I'. Copies of surveyed sections have the same label as the surveyed section with a 'D' or 'U' added (to denote downstream or upstream).

3.8 1D Model Build

3.8.1 The model build involved remedial work on the two existing models prior to initiating the proposed model build activities. The two existing 1D models were combined to form one full hydrodynamic model and a GIS visualiser constructed for the entire model in order to understand better the schematisation and identify areas requiring improvement and updating. The Hogshaw Brook and Nunn Brook models were built separately and grafted onto the full River Wye model, with extra care given to the confluence junctions and the initial conditions. During the model build process the old models were checked thoroughly, using a set of manual checks i.e. chainage checks and coefficient checks as well as using an automated checking process developed specifically for ISIS to aid modellers in the QA of models. This has ensured the Wye model has been updated to current modelling best practice where necessary. An in-house interpolating tool was used to create additional interpolate sections to improve the stability of the 1D model and assist in the running of the 1D-2D combined model further on in the commission.

3.8.2 Prior to the model build several sections of the River Wye were surveyed where missing structures were identified during the initial site walkover. The new sections of survey are listed in Table 3.1.

Table 3.1: Locations of newly surveyed sections

Node Labels	Location
WYA145 – WYA145Ii7	Golf Access Bridge, SK 0435 7325 (Surveyed section 3458)
WYA14412 – WYA14412D	Golf Access Bridge, SK 0458 7336 (Surveyed section 3151)
WYA143 – WYA143Di11	Golf Access Bridge, SK 0462 7337 (Surveyed section 3104)
WYA14212	Section through Ford, SK 0482 7338 (Surveyed section 2884 - channel only)
WYA141 – WYA141D	Footbridge, SK 0482 7337 (Surveyed section 2880)
WYA140I – WYA140IDi1	Footbridge, SK 0499 7334 (Surveyed section 2658)
WYA138 – WYA138D	Weir, SK 0515 7332 (Surveyed section 2501)
WYA138I – WYA138IDi3	Footbridge, SK 0515 7331 (Surveyed section 2491)
WYA137 – WYA137Ii2	Weir, SK 0521 7331 (Surveyed section 2433)
WYA134DI – WYA134DID	Footbridge, SK 0545 7334 (Surveyed Section:2180)
WYA134DI 1 – WYA134DI 1Di8	Weir, SK 0545 7333 (Surveyed Section: 2170)
WYA132I3 – WYA132I3D	Footbridge, SK 0553 7338 (Surveyed section 2050)
WYA132I2 – WYA132I2Di6	Weir, SK 0554 7338 (Surveyed Section: 2036)
WYA131 – WYA131Di6	Footbridge, SK 0562 7342 (Surveyed section 1951)
WYA131I – WYA131IDS1	Weir, SK 0567 7347 (Surveyed section 1879)
WYA129-1 – WYA129-D1	Upstream Face of Bridge Street Culvert, (Surveyed section 1303)
WYA129I2	Downstream Face of Bakewell Road Culvert, (Surveyed section 1123)
WYA129D-1 – WYA129D- 1Di5	Footbridge, SK 0640 7355 (Surveyed section 1072)
WYA129D-2 – WYA129D- 2Di4	Footbridge, SK 0643 7350 (Surveyed section 1013)
WYA127I – WYA127IDi7	Bridge, SK 0651 7323 (Surveyed section 738)
WYA126I – WYA126IDi5	Weir, SK 0650 7312 (Surveyed Section: 621)
WYA126I3 – WYA126I3Di4	Bridge, SK 0650 7307 (Surveyed section 569)
WYA125ID_1 – WYA125ID_1i3	Open section, level gauge, SK 0651 7294 (Surveyed section 469)
WYA123 – WYA123DS1	Open Section, SK 0672 7286 (Surveyed section 187)
WYA123-DI –	Footbridge, SK 0686 7286

Node Labels	Location
WYA123-DID	(Surveyed section 70)
AA5	Access Bridge, SK 1823 6968 (Surveyed section 2547)
AASUS2	Weir at Ashford gauge, SK 1825 6969 (Surveyed section 2525)
AA7	Base of weir, SK 1826 6969 (Surveyed section 2522)
AA101OU	Sluice gate, SK 1990 6961 (Surveyed section CS3.001)
CA110D	Open Section, SK 1990 6959 (Surveyed section CS2.002)
CA111U	Open Section, SK 1998 6954 (Surveyed section CS2.001)

Edits to old model in old survey

- 3.8.3 Cross section chainages and distances to next, in the remaining sections of the old model, were checked using OS MasterMap and known structure locations. Chainages which were found to be incorrect were altered to better reflect reality and the total length of the modelled watercourses. Any changes to the cross sections from the old model have been recorded in the comments section of the cross section unit in ISIS.

Manning's Roughness

- 3.8.4 Manning's 'n' values for the 1D channel model have been determined on a reach by reach basis based on the available survey data and site visits to the watercourse. Table 3.2 lists the Manning's 'n' values adopted throughout the model 1D channel.

Table 3.2: Roughness in ISIS channel

Reach	Channel Bed	Banks
Upstream extent to - Golf Access Bridge (SK 04577 73359)	0.040	0.060
Downstream of Golf Access Bridge – Downstream extent	0.035	0.055
Hogshaw Brook and Nuun Brook	0.030	0.055

Preparation for linking to 2D model

- 3.8.5 All of the cross sections throughout the model were trimmed back to the location of the HX line (the 1D-2D boundary line in the TUFLOW model) to allow the floodplain to be modelled accurately in 2D.

3.9 2D Model Build

- 3.9.1 The floodplain region was modelled in TUFLOW. The 2D grid data is comprised of elevation data derived from the Digital Terrain Model (DTM) created using filtered LiDAR data (topographic data for the model floodplain (2D domain) was provided by the Environment Agency in the form of filtered LiDAR data). Filtered LiDAR refers to LiDAR data that has been passed through a classification and filtering routine that attempts to strip out vegetation and buildings.

LiDAR Validation

- 3.9.2 A statistical comparison was carried out between two elevation datasets; Filtered LiDAR and Environment Agency Aerial Survey, based on photogrammetry, to check the integrity of using ground model elevations for flood mapping of the River Wye and tributaries. With the availability of LiDAR data, the comparison hoped to establish if LiDAR data can be used with confidence for the entire study area by cross-validating elevation values where Aerial Survey data co-exist.
- 3.9.3 Both datasets had to undergo an initial phase of processing and preparation, so that data points could be plotted together and coverage extents established in order to use a function built in to the 3D software add-on in MapInfo called Vertical Mapper 3.1 to extract the elevation values from the LiDAR data where Aerial Survey data points co-exist. The elevation values of both LiDAR and Aerial Survey were then statistically analysed and compared.
- 3.9.4 Results produced suggested that there was potentially a degree of uncertainty in using LiDAR data for this mapping study as differences in elevation of +/- 5metres between the two datasets were identified for a number of data points. Consequently, it was recommended that further investigation and elevation validation was needed in order to restore confidence in the use of the LiDAR ground model.
- 3.9.5 GPS ground truthing data was requested from the Environment Agency and used in the same way as the Aerial Survey data to compare elevation values with the LiDAR data. The findings of this statistical analysis clearly showed an excellent correlation between the elevations of the two datasets confirming that the integrity of the LiDAR data was fine and could be used for this flood mapping study.

Model schematisation

- 3.9.6 In order to model flow between the 1D ISIS model and the 2D TUFLOW floodplain model, it is necessary to link the two model elements together. Linking is carried out by spatially representing the ISIS river sections in GIS, and linking these river sections to the 2D grid using points and polylines. The polyline representing the edge (bank) of the ISIS channel is recognised as a head-time boundary in TUFLOW. ISIS results are used to determine water levels at the location of the river sections, and TUFLOW interpolates water levels along the 2D boundary polyline linking the ISIS river sections. The water levels and ground elevation of the 2D grid cells intersecting the connection lines determine whether there is net flow into or out of the 2D domain. It should be noted that momentum effects are not considered in the transfer of flow from 1D to 2D, though this is not considered to be important for this study. Flow across the 1D/2D boundary is also accounted for within the ISIS flow computations.
- 3.9.7 Based on the size of the area to be modelled and the local topography two different grid sizes were chosen; a 5m grid was chosen for Buxton and a 7.5m grid was chosen for

Bakewell. These grid sizes allow the areas to be modelled accurately and quickly, the modelled extents and grid extents are shown in Figure 3.1.

- 3.9.8 It was noted during the model build that there are several gaps in the LiDAR data where lakes exist. This is due to the LiDAR not penetrating the surface of the water. Where this has occurred the grids have been modified based on the surrounding ground level data.
- 3.9.9 Several changes have been made to the basic geometry of the TUFLOW grid using z- lines (These lines allow specific geometry to be entered into the floodplain). In Buxton the channel geometry of the tributary of the Wye has been reinforced using a z-line and the banks of one of the ponds have been reinforced as well. In Bakewell several z-lines have been added to improve model stability and ensure that water spills out of the channel as it would do in reality, these z-lines have been added near Sheepwash Bridge, Ashford Lake, near Brookside and downstream of Bakewell.
- 3.9.10 There is a formal Derbyshire Dales District Council maintained defence located in Bakewell. To ensure that this defence was properly represented within the model, it was represented using a z-line. The defence is located at the Crescent in Bakewell

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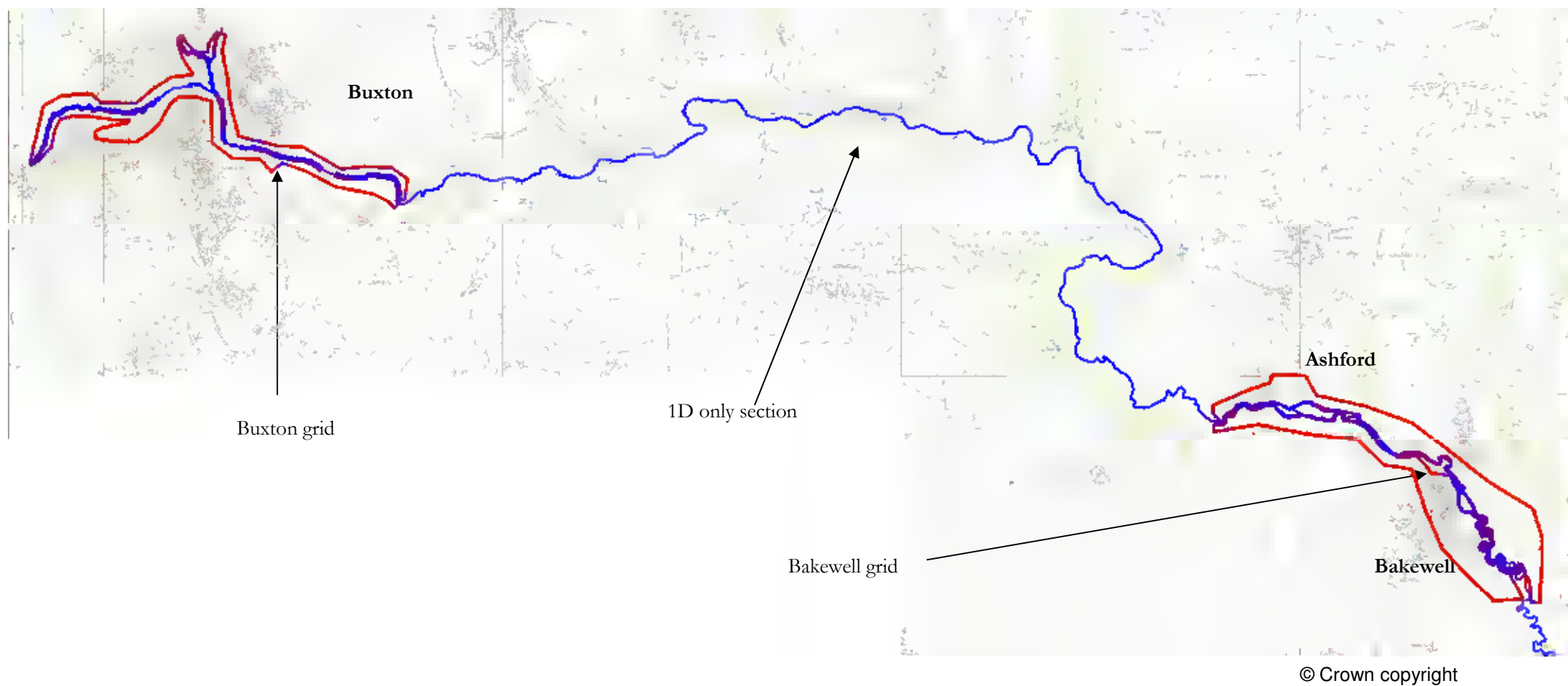


Figure 3.1: Modelled Extents

Manning's Roughness

- 3.9.11 Manning's 'n' values for the 2D floodplain model have been determined using OS MasterMap data to inform land use. This was undertaken using the polygons in this dataset and assigning a roughness value to them. Table 3.3 lists the Manning's 'n' values adopted throughout the model 2D floodplain.

Table 3.3: Roughness in TUFLOW floodplain

Manning's 'n' Value	Land Use
0.020	Open Water
0.030	Roads
0.040	Channel (Drains etc.)
0.045	Grass/Gardens/Yards
0.050	Scrubby Grass
0.060	Dense Scrub
0.090	Trees
1.000	Buildings

Boundary Conditions

- 3.9.12 The Wye catchment has been divided into twenty four sub catchments (see Section 2). The inflows from these catchments have been derived using ReFH units and have been entered into the model at the relevant locations. One of the boundary conditions has been entered directly onto the 2D floodplain as this represents the channel emerging from under Macclesfield Road. This has been achieved by extracting the flows derived by the ReFH unit and then entering them into a TUFLOW bc_dbase format.
- 3.9.13 The downstream boundary of the model in both the 1D and 2D sections are normal slope boundaries which have been derived from the channel slope for the 1D section and the slope of the local LiDAR data for the 2D section.

Instability during model construction

- 3.9.14 There were several areas of instability during model construction; these were located mostly around the Hogshaw and Nunn Brooks as well as the long culverted sections through Buxton. The instabilities were being caused by the very steep channel drying out and the modelled water flowing supercritically through the channel and culverted sections. The channel drying out was solved by entering minimum flows into the upstream boundary units to keep the channels wet. At the entrances to the culverts the culvert entry parameters were examined and adjusted where viable, and inline spill units added to smooth the transition between open channel and culverted flow. The Manning's 'n' values upstream of the culverts have also been slightly increased in some locations to reduce model instability.

3.10 Hydraulic Calibration

- 3.10.1 Section 2.3.12 describes why it is not possible to calibrate the model hydrologically, this does not preclude calibrating the model hydraulically. To do this the inflows were scaled to the

gauged flow at Ashford gauge then the flood depths throughout the floodplain in Ashford and Bakewell were compared to observed wrack marks. This analysis was performed on two events, the October 1998 and the November 2000 events. Generally the modelled depths and the observed were a good match, with a couple of anomalous results however it is believed that the data for these wrack marks was either from surface water flooding or erroneous (one wrack mark level is 0.5m below that of the ground level).

3.10.2 Calibration in Buxton was performed against observed data as described in Chapter 6.

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4 Model Outputs

4.1 Model Runs

Design Runs

- 4.1.1 Nine design storms have been run through the model these are the 20%, 10%, 5%, 2%, 1.33%, 1%, 1%+20%Q, 0.5% and 0.1% AEP. Each design storm was run for two storm durations, the storm durations are 3.5 hours and 15.5 hours.

Undefended

- 4.1.2 The model was run without defences in Bakewell for the 1%, and 0.1% AEP events with a 15.5 hour storm duration.

- 4.1.3 The defences in Bakewell are bypassed for all events greater than the 20% AEP and overtopped for events greater than the 1% AEP. As a result there are no ABD's for these defences.

Blockage

- 4.1.4 Blockage scenarios have been run for the Spring Gardens and Bridge Street culverts. These culverts were each blocked by 25%, 50% and 75% and run with the 1% AEP 3.5 hour storm.

- 4.1.5 Blocking the Spring Gardens culvert by 25% caused a 57mm rise in water level, blocking the culvert by 50% caused large instabilities in the model and blocking the culvert by 75% caused a 196mm rise in water level.

- 4.1.6 Blocking the Bridge Street culvert by 25% caused a 110mm rise in water level, blocking the culvert by 50% caused a 265mm rise in water level and blocking the culvert by 75% caused a 594mm rise in water level.

Trash Screen Sensitivity

- 4.1.7 The effects of installing a trash screen on the Bridge Street culvert were modelled using the 1% AEP 3.5 hour storm. The trash screen modelled was 2m high by 2.8m wide with 20 10mm wide bars spaced at 140mm centres.

- 4.1.8 Using the trash screen described there was a maximum rise in water level of 43mm at peak flow. However the installation of a trash screen in will increase the risk of blockage and would therefore have to be maintained regularly.

4.2 Flood Extent Mapping

- 4.2.1 The flood extent maps have been created by extracting maximum depth grids from both the 1D results, using ISIS Mapper, and 2D results, using TUFLOW utilities. The maximum extent of these grids has then been contoured.

Design Runs

- 4.2.2 For the design runs flood extents have been produced by combining the maximum extents for the storm durations of a given event.

Undefended

- 4.2.3 The undefended outlines have just been mapped for Bakewell as they only affect this area of the model.

Blockage

- 4.2.4 The blockage scenario outlines have just been mapped for Buxton as they only affect this area of the model.

4.3 Depth and Velocity Mapping

- 4.3.1 Depth and velocity have been mapped by combining the maximum outputs for the different storm durations from the 2D model runs, for each return period.

4.4 Hazard Mapping

- 4.4.1 Hazard maps have also been produced for the modelled events and have been included digitally on the accompanying DVD. The flood hazard outputs are based on the UK Flood Hazard methodology and include a grid of Flood Hazard derived from the flood depth and velocity outputs and a debris factor. The Hazard and its associated classification are calculated within TUFLOW. The UK Flood Hazard is calculated by using the following equation from Defra's Flood Risks to People – Phase Two Document (FD2321/ TR2) (2006). Hazard is calculated as follows:

$$\text{Hazard} = d \times (v + 0.5) + DF$$

Where **d** = depth (m)

V = velocity (m/s)

DF = debris factor

- 4.4.2 Based on the value of the hazard for a given area, a Hazard Classification is then assigned. The Flood Hazard classifications are divided into four classes of risk:

Table 4. 1: Flood Hazard Rating and Associated Category

Flood Hazard Rating	Category
0.0 – 0.75	Low
0.75 – 1.25	Moderate
1.25 – 2.5	Significant
2.5 +	Extreme

4.4.3 These classes of risk then translate into the following Flood Hazard classification (Figure 4.1):

- Class 1: Danger for some – Flood zone with deep or fast flowing water that presents a hazard for some people (i.e. children);
- Class 2: Danger for most – Flood zone with deep or fast flowing water that presents a hazard for most people;
- Class 3: Danger for all – Flood zone with deep or fast flowing water that presents a hazard for all people.

4.4.4 For example, if peak water depths are 1.0 m, for velocities less than 1.0 m/s, the flooding is considered to present 'Danger for some'. For velocities between 1.0 m/s and 2.0 m/s the flooding is considered to present 'Danger for most'. For velocities greater than 2.0 m/s the flooding is considered to present 'Danger for all'.



Figure 4. 1: Flood Hazard Classification

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5 Interim Conclusions

- 5.1.1 The following details the conclusions that were presented to the Environment Agency mid-way through the project.
- 5.1.2 The catchment was schematised taking into consideration major inflows, gauged data stations and areas of specific interest. The modelled rating and EA rating at Ashford were investigated and compared. Both were found to be satisfactory. Statistical analyses (pooled and single site) were performed at Ashford. The single site analysis was found to give slightly higher results than pooled. Flood extents were not found to be very sensitive to peak flows. The decision was therefore taken to reconcile to the single site method, as this provides robust, conservative but reasonable flood outlines. Appropriate storm durations were determined by comparing observed data with results for distributed ReFH models at Buxton and Ashford. These storm durations were used for the draft model runs, and the worst case outline obtained by combining results.
- 5.1.3 The original River Wye models have been combined and improved upon to create a stable model which is a good representation of the river. The River Wye is a very steep watercourse which is heavily culverted through the town of Buxton. This can cause model instabilities which have been accounted for in the River Wye model.
- 5.1.4 The draft 1% AEP flood outline was compared with modelled outlines of observed events of a similar magnitude. The draft 1% AEP outline is considered to compare well with the observed outlines.
- 5.1.5 The defences in Bakewell are bypassed at a low return period reducing their effectiveness. Blocking the Spring Gardens culvert for the 1% AEP event does not cause any significant rise in water level, however blocking the Bridge Street culvert for the 1% AEP event can cause up to 594mm rise in water level for a 75% blockage. Putting a trash screen on the Bridge Street culvert does not cause a significant increase in water levels. However putting a trash screen in will increase the risk of blockage and would therefore have to be maintained regularly. (Note that the results from the blockage scenarios quoted in this paragraph are from the updated finalised model).

6 Improvement of Flows at Buxton

6.1 Introduction

6.1.1 After review of the draft deliverables, the Environment Agency expressed concerns that flood outlines in Buxton may not be sufficiently large and that adopted design flows were lower than those previously identified. The Environment Agency queried whether adoption of catchment wide scaling of inflows to reconcile flows at Ashford did not pay sufficient regard for the higher runoff characteristics in the upper catchment.

6.1.2 The remainder of this chapter describes the additional work undertaken to investigate the flood history through Buxton and to discuss and agree revised design flows. It is noted that this additional work was restricted to Buxton, with revised flows being adopted for generation of flood maps through Buxton to the Mineral Railway (SK 08685 72392).

6.1.3 Full details and calculations are included in Appendix I.

6.2 Flow Distribution

6.2.1 In the preliminary model results, flows were distributed through the model using ReFH inflow boundaries derived from catchment descriptors (see Section 2.5). Figure 6.1 illustrates the distribution of default SPR values overlain on the bedrock geology. The default values mirror the geology, with higher SPR values in the upper catchment. The adopted scaling of the inflows used in the preliminary results retains this distribution although implied SPR value are lower than those shown in Figure 6.1.

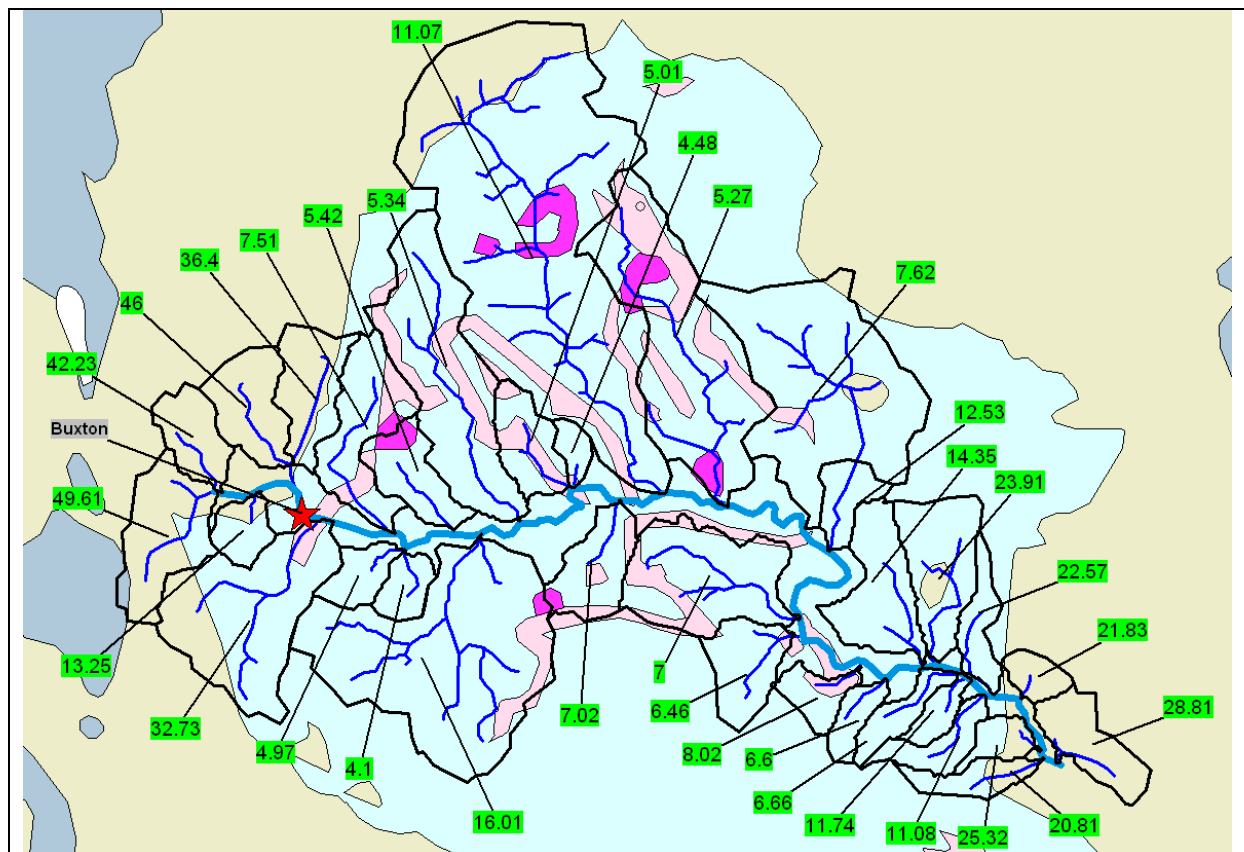


Figure 6.1: Bedrock geology of the study catchment with FEH SPR values

Pale Blue: Sedimentary Rocks - Tournaisian and Visean (Carboniferous Limestone Series)

Pale Pink: Igneous Intrusive Rock - Basalt and spilite

Dark Pink: Igneous Intrusive Rock - Basalt dolerite, camptonite and allied types

Pale Green: Sedimentary Rock - Namurian (Millstone Grit Series)

- 6.2.2 Flow distribution was verified through the simulation of two observed events, October 1998 and November 2000, and comparison with observed wrack marks. Inflows were scaled consistently across all sub-catchments, as with the design events. For October 1998 a scaling factor of 0.74 was used, with 0.55 being adopted for November 2000. Note that inflows were derived using observed rainfall (from the nearest rain gauge to each sub-catchment) but with default catchment antecedent conditions (C_{ini}). Analysis to derive C_{ini} values from observed rainfall and potential evapo-transpiration data was not considered justified given the primary purpose was to verify the performance of the hydraulic model against a known flow at Ashford. Adoption of actual, rather than design, C_{ini} values not being expected to change to distribution of inflows.
- 6.2.3 Comparison with observed wrack marks showed no strong spatial trends with average errors of -0.07 m and -0.12 m for the two events respectively. This performance was considered satisfactory. Most of the wrack marks were located in Bakewell and Ashford.
- 6.2.4 Closer examination of November 2000 results for three available wrack marks in Buxton showed an average of -0.2 m, which was considered borderline satisfactory/unsatisfactory.

Using the adopted scaling factor of 0.55, peak flows at the Buxton gauge were estimated to be $6.5 \text{ m}^3\text{s}^{-1}$. There are no records at the gauge to confirm whether this flow estimate is reasonable.

- 6.2.5 There are no formal wrack mark data available for the October 1998 event. Following discussion of the November 2000 event's borderline result, investigation of any additional evidence from the October 1998 was explored and additional photographic evidence was provided by the Environment Agency in June and July 2010. This data was used to try to estimate the peak flow of the October 1998 event

6.3 Estimating flow for the 1998 flood from observed data

- 6.3.1 A flow estimate for the October 1998 flood event was generated by comparing estimated water levels in photographs with the preliminary design levels and flows at appropriate model cross sections. The comparison is acknowledged to be subject to inaccuracies associated with:

- Determining the location photographs were taken (locations provided by EA);
- Determining the nearest model node (undertaken by Halcrow);
- Estimating water levels in the photographs by comparison with model cross-sections (undertaken by Halcrow);
- Likelihood that photographs were not taken at the peak of the event;
- Possibility that flooding was exacerbated by blockage of structures, thought to be particular issue on Hogshaw Brook;
- Possibility that some flooding was attributable to surface water flooding.

- 6.3.2 The comparison of estimated observed water levels and the preliminary design water levels (with associated peak flows) is shown in Table 6. 1. The table includes observed flow levels from the time series data at the Buxton gauge. The results indicate a wide variation in implied flows, with results ranging from $10 \text{ m}^3\text{s}^{-1}$ to more than $30 \text{ m}^3\text{s}^{-1}$. To try to reduce this range of flows, the reliability was assessed by consideration of:

- Whether model node results were derived from observed cross-section data or interpolates;
- Whether all smaller localised structures were included within the model;
- Whether implied water levels were consistent with LIDAR data.

- 6.3.3 By consideration of the reliability of flow estimates, higher flow estimates were discarded and a best estimate of $15\text{m}^3\text{s}^{-1}$ identified, with a likely range of $13 - 16 \text{ m}^3\text{s}^{-1}$. This estimate was used to indicate the severity of the event using new flood frequency analysis at the Buxton gauge (see Section 6.4).

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Table 6. 1: Comparison of estimated observed water levels and design water levels

						Oct'98	Preliminary Design simulation results (3.5 hr storm)						Estimate of observed level from photograph	Comment	Implied flow (at gauge). No blockage	Reliability
						Peak flow estimate at Buxton Gauge (m ³ s ⁻¹)										
Location	Node Label	Chainage	Bed Level	Left Bank	Right Bank	10.0	9.7	12.6	14.1	15.2	18.3	30.8				
Serpentine Walks	WY139D	99	295.56	296.72	296.50	296.51	296.64	296.74	296.78	296.81	296.89	297.11	297.20	Estimate probably out as bank levels at bridge are not typical of reach	>30	Poor
	WY138	204	295.52	296.10	296.37	296.25	296.35	296.43	296.45	296.48	296.54	296.76	296.37		10-12	Good
	WYA137I	379	294.21	295.11	295.55	295.03	295.09	295.16	295.18	295.20	295.25	295.36	295.20		15	Good
Pavilion Gardens	WY132i3	222	289.95	292.49	292.46	291.26	291.40	291.53	291.59	291.63	291.75	292.78	292.57	Interpolate section. Estimate for photograph in Wallingford 2000 report, 292.4 mAOD - model node WY132i2	25-30	Poor
	WY131	311	289.96	292.42	290.81	290.78	290.92	291.04	291.10	291.14	291.41	292.59	292.54	Possibly missing structure	approx 30	Poor
	WYA131I	393	289.96	294.21	291.35	290.67	290.79	290.90	290.95	290.99	291.31	292.58	291.35		approx 20	Good
			181				293.53	293.59	293.60	293.61	293.62	293.63	293.98	295.00	Comparison with LiDAR	>30
Pavilion Garden Floodplain channel		241				292.33	292.45	292.47	292.51	292.49	292.53	292.68	293.10	Comparison with LiDAR	>30	Poor
Ashwood Park	WY129D-1	1191	283.03	284.15	284.12	283.79	283.98	284.13	284.24	284.30	284.44	284.76	284.15		13	Good
Buxton Gauge	WYA125ID_1i2	1841	278.00	282.31	281.97	278.77	279.22	279.55	279.70	279.82	280.13	280.39	279.77		15	Good
Ashwood Dale Quarry	WY118i3	2836	268.27	269.26	272.68	269.43	269.57	269.72	269.79	269.83	269.94	270.36	272.13	Interpolate section	>30	Very poor
	WY114	3398	258.28	259.78	260.17	259.19	259.39	259.58	259.67	259.72	259.83	260.21	259.78		15-18	Good
	WY113U-1	3487	256.96	257.96	259.43	258.06	258.25	258.44	258.53	258.64	258.78	259.25	258.36	US Culvert	10-12	Good
	WY112	3523	256.85	263.60	260.27	257.63	257.76	257.89	257.96	258.03	258.16	258.62	258.25	DS Culvert	approx 20	Good

The yellow highlighted cells indicate where the modelled water levels are below the estimated observed level

6.4 Statistical Analysis at Buxton

6.4.1 To improve confidence in design flows through Buxton, a statistical analysis was conducted using the pooling group approach with a range of methods applied for derivation of QMED.

6.4.2 The WINFAPv3 software and HiFlows v3.02 database were used to generate a growth curve for Buxton, with results shown in Table 6. 2. Details of the pooling analysis are provided in the FEH Calculation Record in Appendix H..

Table 6. 2: Growth Curve at Buxton

Return Period (years)	Growth Factor
2	1
5	1.38
10	1.65
20	1.96
50	2.41
75	2.64
100	2.81
200	3.28
1000	4.66

6.4.3 Three approaches to estimation of QMED were evaluated based on:

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- (a) Use of observed level, and estimated flows, from observations at Buxton gauge;
 - (b) Catchment descriptors improved through donor transfer based on 1999 FEH guidance (FEH volume 3 p17). The use of this previous FEH donor transfer method could be justified if the donor catchment is very similar to that of the subject catchment, warranting a direct transfer. This could occur if the donor and subject catchments were on the same watercourse or perhaps in the very near vicinity and have very similar hydrological properties;
 - (c) Catchment descriptors improved through areal weighted donor transfer based on 2009 FEH guidance (940_08 Improvements to the FEH statistical method pages 4 & 5). The use of this FEH transfer method should always be considered as it is current best practice.

6.4.4 Using these three methods values of 10.0, 8.8 and 11.2 m³s⁻¹ were identified. The 10.0 m³s⁻¹ estimate was provided by the Environment Agency, whilst 8.8 and 11.2 m³s⁻¹ values were computed by Halcrow using data from station 28033 as a donor site. The range of values gives a general indication of the plausible range of acceptable values.

6.4.5 Combining these QMED estimates with the growth curve allows estimation of likely range of event severity for the 1998 event. Event severities estimates are:

- (a) QMED = 8.8 m³s⁻¹. October 1998 event severity between 1 in 10 and 1 in 20 years
- (b) QMED = 11.2 m³s⁻¹ October 1998 event severity of approximately 1 in 5 years

6.4.6 This range of estimates shows acceptable agreement with the local flood history, the 1998 event being notable in recent years but not resulting in significant property flooding. Following discussion, a QMED value of 10.0 m³s⁻¹ was agreed with an implied severity for October 1998 of 1 in 7 or 8 years. This is the average value of the two catchment descriptor estimates with the different methods of donor transfer. It is felt that either of the two transfer methods could be fully justified for this study, therefore taking an average value is viewed as providing a conservative, yet realistic approach. The average value of 10.0m³/s also matches the QMED value estimated directly from the level gauge, therefore increasing confidence that 10.0m³/s is a reliable estimate.

6.4.7 The final flood frequency curve for Buxton is shown in Table 6. 3. Inflows for catchments upstream of the Buxton gauge have been scaled to achieve a satisfactory fit with the adopted flood frequency curve which has resulted in the adoption of the scaling factors also shown in Table 6. 3. A design storm duration of 3.5 hours has been used.

Table 6. 3: Final Flood Frequency Curve for Buxton

Return Period (years)	Growth Factor	Flow (m ³ s ⁻¹)	Scaling Factors
2	1	10	0.923
5	1.38	13.8	0.943
10	1.65	16.5	0.723
20	1.96	19.6	0.777
50	2.41	24.1	0.840
75	2.64	26.4	0.820
100	2.81	28.1	0.786
200	3.28	32.8	0.745
1000	4.66	46.6	0.720

6.5 Results and Discussion

6.5.1 The revised design flows for Buxton are notable higher than the preliminary results provided in February 2010. Higher flows are primarily justified by:

- More realistic event severity estimate for the 1998 event;

- Consistency with the anecdotal evidence of frequent flooding of the A6 downstream of Buxton;
- Larger flood extents for extreme return periods (100 yr and 1000yr), which better reflect the expected response given the underlying geology.

6.5.2 In the February 2010 submission, design flows were reconciled to the preferred flood frequency curve at Ashford. The revised flows for Buxton replace the catchment wide scaling although it is noted that the preferred flood frequency at Ashford is unchanged. Consideration was given to whether adoption of higher flows through Buxton required consequent reductions in inflows for sub-catchments downstream of Buxton. Whilst more detailed evaluation of sub-catchment inflows would be desirable availability of data for intervening rural reaches is limited to wrack mark data for the two verification events. Data from these two events generally supports the overall distribution of preliminary flows and therefore detailed re-evaluation of design flows was not considered further. For Development and Flood Risk Purposes a site's location relative to the mineral railway should determine which model is used for assessing flood risk

6.5.3 Revised design flows for Buxton have therefore been used to generate new flood maps for reaches as far as the Mineral Railway crossing over the Wye. Downstream of the Mineral Railway, maps generated in February 2010 are retained. The Mineral Railway location was selected because it is coincident with the change in underlying geology between the less permeable upper catchment and more highly permeable catchment further downstream.

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7 Comparison of Statistical and Modelled Flows at Bakewell

7.1 Introduction

7.1.1 After review of the February 2010 draft deliverables, the Environment Agency queried the possibility of producing a flood frequency curve for Bakewell and comparing this with model results.

7.1.2 Full details and calculations are included in Appendix J.

7.2 Generation of Flood Frequency Curve at Bakewell

Growth Curve

7.2.1 The single site growth curve generated for the Ashford site was used to provide a growth curve for the Bakewell site. This is shown in Table 7.1.

QMED

7.2.2 Two methods of producing a QMED estimate for the Bakewell site were considered. Firstly, a QMED estimate was produced from catchment descriptors using Ashford as a donor site, using the adjustment equation from OI 940_08: 'Improvements to the FEH statistical method.' This method produced a QMED estimate of $21.5\text{m}^3\text{s}^{-1}$, approximately 30% larger than the QMED produced from the AMAX series at Ashford of $16.5\text{m}^3\text{s}^{-1}$. This was considered too large, since the increase in catchment area between Ashford and Bakewell is only approximately 13%. A second approach was therefore taken of calculating a pro-rata QMED for Bakewell based on the increase in catchment area between the two sites. This approach produces a QMED estimate of $18.7\text{m}^3\text{s}^{-1}$. 95% QMED confidence intervals generated for Ashford were also subjected to scaling on the basis of area. The flood frequency curve for Bakewell based on this QMED estimate is shown in Table 7.1. It is noted that the use of Ashford as a donor for transfer using the previous FEH method as outlined in FEH volume 3 p17, results in a QMED estimate at Bakewell of $18.2\text{m}^3/\text{s}$. This is relatively close to the value estimated through the pro-rata of the catchment size and therefore gives additional confidence in the QMED estimate of $18.7\text{m}^3/\text{s}$ taken forward for the design flows.