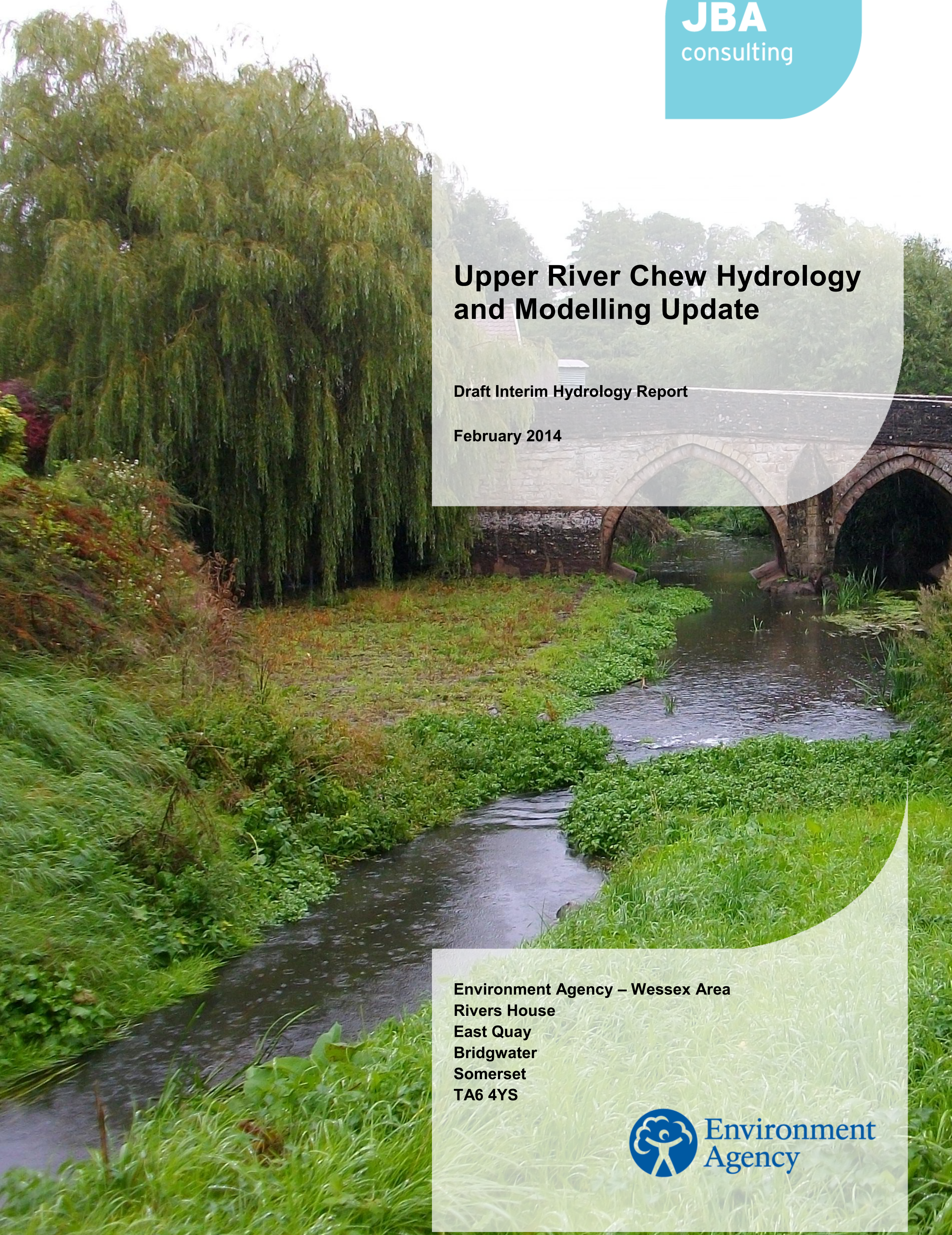

Upper River Chew Hydrology and Modelling Update

Draft Interim Hydrology Report

February 2014

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Revision History

Revision Ref / Date Issued	Amendments	Issued to
Draft Report v1.0 - February 2014		Tom Toogood

Contract

This report describes work commissioned by the Environment Agency by an email dated 13th August 2013. The Environment Agency's representative for the contract was Tom Toogood. Jenni Essex of JBA Consulting carried out this work.

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Purpose

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Acknowledgements

Environment Agency, Bristol Water, and Bath and North East Somerset Council staff are thanked for their assistance and support throughout this commission.

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Executive Summary

Study scope

JBA Consulting was commissioned by the Environment Agency to undertake the Upper River Chew Hydrology and Model Update study under the Water and Environment (WEM) Framework. The study was required to review and update the existing hydrological assessment, update and improve the existing 1D-2D hydraulic models, and test a series of scenarios.

The existing hydrological assessment and hydraulic models were developed by Mott MacDonald (2010-2012) as part of a comprehensive review and update of the hydrology of the River Chew model, for flood mapping purposes. Additional 1D-2D hydraulic modelling was also undertaken. Following the completion of the study two large floods occurred in the River Chew catchment, in September 2012 and November 2012. The outputs from the modelling study did not represent the flood extents and flooded properties seen during the floods, particularly for the Winford Brook in Chew Magna.

The main uncertainty of the previous study was the hydrological assessment due to the complicated nature of the catchment response and limited confidence in some of the hydrometric data. The current study was commissioned to reassess the hydrology using new data and information which became available following the completion of the previous analysis. The 2012 floods highlighted the problem of design flow for the Winford Brook being too low and improving this aspect of the hydrology was the key focus of the current commission.

Methodology

A number of analyses were undertaken at the preliminary stage of the hydrological assessment to utilise the new hydrometric data and information available for the catchment gauges and to assess the influence of the two reservoirs on downstream flow:

- Routing calculations were performed using a rainfall runoff approach to determine the outflow hydrographs from Chew Valley Lake and Chew Magna Reservoir.
- Analysis of the Compton Dando gauge data was undertaken to determine ReFH model parameters from observed data to improve the hydrograph shape and peak flow estimates.
- Lag analysis was carried out on the Chew Magna Reservoir Weir and Chew Stoke Flood Warning gauge level data to improve the estimate of Tp(0) (instantaneous time to peak) and hence hydrograph shape for the ReFH method.
- Assessment of the most appropriate flow at Compton Dando for the July 1968 flood was undertaken.
- An alternative approach to deriving QMED and the flood frequency curve was applied to the Chew Magna Reservoir Weir gauge data. This is the FSR POT approach which makes best use of the numerous peak flow events recorded in 2012.
- FEH Statistical method analyses using the available flood peak data from the catchment at the key gauge locations. This utilised the new rating for the Chew Magna Reservoir Weir gauge and recent data for the Compton Dando gauge to determine QMED and to undertake single-site analyses.
- Pooling group composition was investigated and the merits of using a 'permeable' pooling group assessed.

Results

The results generated by the application of these methods and approaches to six initial key flow estimation locations were assessed and the preferred design peak flow estimates selected for each location. The most appropriate method to derive the outflow hydrograph from Chew Valley Lake is by routing a ReFH hydrograph through the reservoir using a routing model. Results are not provided for this location (CVL) at this stage of the study as the reservoir routing model is being incorporated into the main hydraulic model. The inflow hydrograph and hyetograph for all design events will be run through this model to derive the outflow hydrograph for the events. The preferred design peak flow estimates are:

Return period (yrs)	COMP (m^3/s)	WINF_A (m^3/s)	CMRW (m^3/s)	CHST_A (m^3/s)	CSFW (m^3/s)	CVL (m^3/s)
2	18.9	8.2	7.2	4.3	4.0	
5	28.9	12.4	10.9	5.7	5.4	
10	38.2	15.5	13.6	6.8	6.4	
20	50.2	18.6	16.4	7.9	7.5	
30	58.9	20.5	18.0	8.7	8.2	
50	72.2	22.8	20.0	9.7	9.3	
75	84.9	24.6	21.7	10.7	10.2	
100	95.4	25.9	22.8	11.4	10.9	
200	127	29.0	25.6	13.6	12.9	
1000	248	36.3	32.0	20.9	20.1	

Note: COMP - Compton Dando gauge; WINF_A - Winford Brook at confluence with River Chew; CMRW - Chew Magna Reservoir Weir gauge; CHST_A - Chew Stoke Stream confluence with River Chew; CSFW - Chew Stoke Flood Warning gauge.

The preferred method for deriving the COMP flow estimates generates a very steep growth curve. This has been carefully considered and, at this stage, is believed to provide the best design flow estimates at this location.

Additional flow estimation locations were selected to provide point inflows to the upstream extents of the Upper River Chew modelled watercourses, to provide lateral inflows to the model and to act as check nodes to ensure that modelled flows approximate preferred flow estimates. The design peak flow estimates for these locations were determined based on the decisions made for the key flow estimation locations.

Further work

The preferred design peak flow estimates and hydrographs generated during this hydrological assessment are the best estimates based on the data and information available. However, as with all flow estimation, there is some uncertainty in these results, particularly for those using the Chew Magna Reservoir gauge data.

Design hydrographs applied to the hydraulic model will initially be based on the ReFH method, due to the requirement for hydrographs to be routed through reservoirs, with adjustments to model parameters derived using gauge data used. The flood extents and depths generated by these hydrographs will be compared to recent and historical flooding and the modelled flows compared to the preferred flow estimates at the key locations. Consideration will then be given to revision of the preferred flow estimates and / or amendments to the ReFH model parameters.

Further flow estimation locations will need to be considered for the River Chew catchment model which extends to the confluence with the River Avon at Keynsham. The design peak flow estimates will be generated using a broadscale assessment and will utilise the work undertaken in this detailed hydrological assessment where possible.

Contents

List of Figures

List of Tables

Abbreviations

AEP .... annual exceedance probability
AMAX .... annual maximum
ARF .... areal reduction factor
FEH .... Flood Estimation Handbook
FRIS .... Flood Reconnaissance Information System
FSR .... Flood Studies Report
GEV .... Generalised Extreme Value (distribution)
GL .... Generalised Logistic (distribution)
HyDB .... Hydrometric Database (JBA)
JBA .... Jeremy Benn Associates
NRFA .... National River Flow Archive
P3 .... Pearson Type III (distribution)
POT .... peaks over threshold
PR .... percentage runoff
QBAR .... mean annual flood
QMED .... median annual flood
ReFH .... Revitalised Flood Hydrograph
REBDY .... (ISIS) Rainfall / Evaporation boundary unit
REFHBDY .... (ISIS) ReFH boundary unit
SoW .... scope of works
TBR .... tipping bucket raingauge
WEM .... Water and Environment (Framework)

Conversion between return period and AEP

Return period (yrs)	2	5	10	20	30	50	75	100	200	1000
AEP (%)	50	20	10	5	3.33	2	1.33	1	0.5	0.1

1 Introduction

1.1 Project background

JBA Consulting was commissioned by the Environment Agency to undertake the Upper River Chew Hydrology and Model Update study under the Water and Environment (WEM) Framework. The study was required to review and update the existing hydrological assessment, update and improve the existing 1D-2D hydraulic models, and test a series of scenarios.

The existing hydrological assessment and hydraulic models were developed by Mott MacDonald (2010-2012) as part of a comprehensive review and update of the hydrology of the River Chew model, for flood mapping purposes. Additional 1D-2D hydraulic modelling was also undertaken. Following the completion of the study two large floods occurred in the River Chew catchment, in September 2012 and November 2012. The outputs from the modelling study did not represent the flood extents and flooded properties seen during the floods, particularly for the Winford Brook in Chew Magna.

The main uncertainty of the previous study was the hydrological assessment due to the complicated nature of the catchment response and limited confidence in some of the hydrometric data. The current study was commissioned to reassess the hydrology using new data and information which became available following the completion of the previous analysis. The 2012 floods highlighted the problem of design flow for the Winford Brook being too low and improving this aspect of the hydrology was the key focus of the current commission.

1.2 Study objectives

A summary of the main uncertainties associated with the existing hydrological assessment was provided in the project scope of works (SoW). These are:

- Comparatively high flows appear to be generated at the top of the catchment compared with the flow at Compton Dando gauging station; the mid-catchment does not appear to attenuate significantly.
- The July 1968 flood flow at Compton Dando gauging station is uncertain due to rubble filling the flume; the value of this outlying extreme flow significantly influences flow estimates generated using the gauge data.
- The standard recommended flow estimation techniques produce flood flow estimates which are too low; the FEH (Flood Estimation Handbook) Rainfall Runoff method proved the most suitable approach for the previous assessment.
- There are datum issues with some of the gauges. In particular the Environment Agency lake level gauge at Chew Magna Reservoir differed to the Bristol Water gauge record. The Environment Agency gauge has since been moved to a more suitable location.
- The rating for the gauge on the Winford Brook downstream of Chew Magna Reservoir was found to be underestimating the spill from the reservoir. Bristol Water recently (June 2013) completed a project to rerate the weir to improve confidence in estimating flood flows downstream of the reservoir.

The key objectives of the study, outlined in the SoW and later communications, were to:

- Review and reassess the existing hydrology, including the July 1968 flood flow estimate at Compton Dando gauging station, and propose the most appropriate method for improving flow estimates. This should utilise the new data and information which has become available since the previous study was completed.
- Run the revised flow estimates through the Upper River Chew hydraulic model for all design events and the 2012 verification flood events. The Compton Dando model and River Chew catchment model should also be updated with the new hydrology and rerun to ensure consistency between the models.
- Review the existing models and identify any areas where improvements could be made. Update the existing models, applying any improvements /amendments identified in the review. Input the new bank survey commissioned by the Environment Agency to improve the representation of the 1D-2D boundary.
- Undertake scenario testing of flood mechanisms using the updated modelling, and assess impacts and possible solutions. Scenarios include assessing the influence of

Chew Magna Reservoir and Chew Valley Lake, and the impacts of channel maintenance.

1.3 Report structure

This report provides a technical account of the review and update of the hydrological assessment. It is an interim report to be supplied to the Environment Agency for discussion and agreement on the representative design flow estimates to be taken forward for use in the hydraulic modelling. This report will subsequently be supplied as a technical appendix to the modelling report. The key details of the assessment are summarised in the Flood Estimation Calculation Record (Appendix A), which provides an audit trail of the calculations undertaken and information to allow the work to be reproduced.

The following elements are reported:

Element	Report section
Introduction	1
Catchment Overview	2
Methodology Overview	3
ReFH Method & Analyses	4
Statistical Methods & Analyses	5
Comparison of Results	6
Catchment-wide Application (Upper Chew)	7
Summary & Further Work	8

2 Catchment Overview

2.1 Catchment characteristics

The River Chew rises near Chewton Mendip to the west of Midsomer Norton in Somerset. It flows north-west from this location into Chew Valley Lake and from the lake north-east to join the River Avon at Keynsham. The key tributaries of the River Chew are the Winford Brook and Chew Stoke Stream which join the River Chew at Chew Magna (358040, 162950) and Chew Stoke (357070, 162090), respectively. The River Chew catchment boundary and key watercourses is shown in Figure 2-1.

Chew Valley Lake is the most significant surface water feature in the River Chew catchment. Construction of the lake started in 1950 and took just over five years to complete. The lake was built to provide water for Bristol and has a surface area of 4.9km^2 which is approximately 8% of the catchment area draining to the spillway of the lake. Flow passing out of Chew Valley Lake consists of an environmental compensation flow (0.17m^3/s May to November; 0.08m^3/s December to April) and spillway flow which occurs when the reservoir level exceeds the spillway level.

Chew Magna Reservoir is located on the Winford Brook at the upstream extent of Chew Magna village. The reservoir was constructed in the 1930s to supply water for villages in the Chew Valley and has a surface area of about 0.02km^2; this is approximately 0.1% of the Winford Brook catchment area to the spillway of the reservoir. Flow passing out of Chew Magna Reservoir consists of an environmental compensation flow (0.05m^3/s May to September; 0.03m^3/s October to April) and spillway flow which occurs when the reservoir level exceeds the spillway level. Due to the size of the reservoir, and in relation to the size of the catchment draining to it, it will empty and fill very quickly resulting in limited attenuation potential.

The River Chew catchment is predominantly rural. The main settlements in the catchment are Chew Magna, Chew Stoke, Pensford and Keynsham downstream of Chew Valley Lake and East Harptree and Litton upstream of the lake.

The catchment topography is generally steep although it flattens slightly towards the lower extent at Keynsham. The Winford Brook and Chew Stoke Stream catchments are particularly steep with limited floodplain.

The British Geological Survey website[1] 1:625,000 geology mapping shows the River Chew catchment to be predominantly underlain by mudstone, siltstone and sandstone. There are areas of limestone in the upper catchment above Chew Valley Lake and in the upper Winford Brook catchment. In the upper catchment soils are predominantly well-drained, fine silty and clayey types, and well-drained fine loamy soils. In the lower catchment soils are fine and coarse grained loamy types, with slowly permeable subsoils and slight seasonal waterlogging, and slowly permeable seasonally waterlogged fine loamy and clayey soils.

The physical characteristics of the River Chew catchment suggest that it is likely to respond reasonably rapidly to rainfall, particularly the Winford Brook and Chew Stoke Stream catchments. The more permeable geology and soils in the upper parts of the catchment may moderate this response to some extent. Chew Valley Lake will have a significant impact on flow downstream of the spillway. In this area the response of the catchment is more likely to be influenced by the Winford Brook and Chew Stoke Stream. The size of Chew Magna Reservoir suggests that it is unlikely to have a substantial impact on catchment response or attenuate the runoff hydrograph to any significant degree.

[1] http://mapapps.bgs.ac.uk/geologyofbritain/home.html

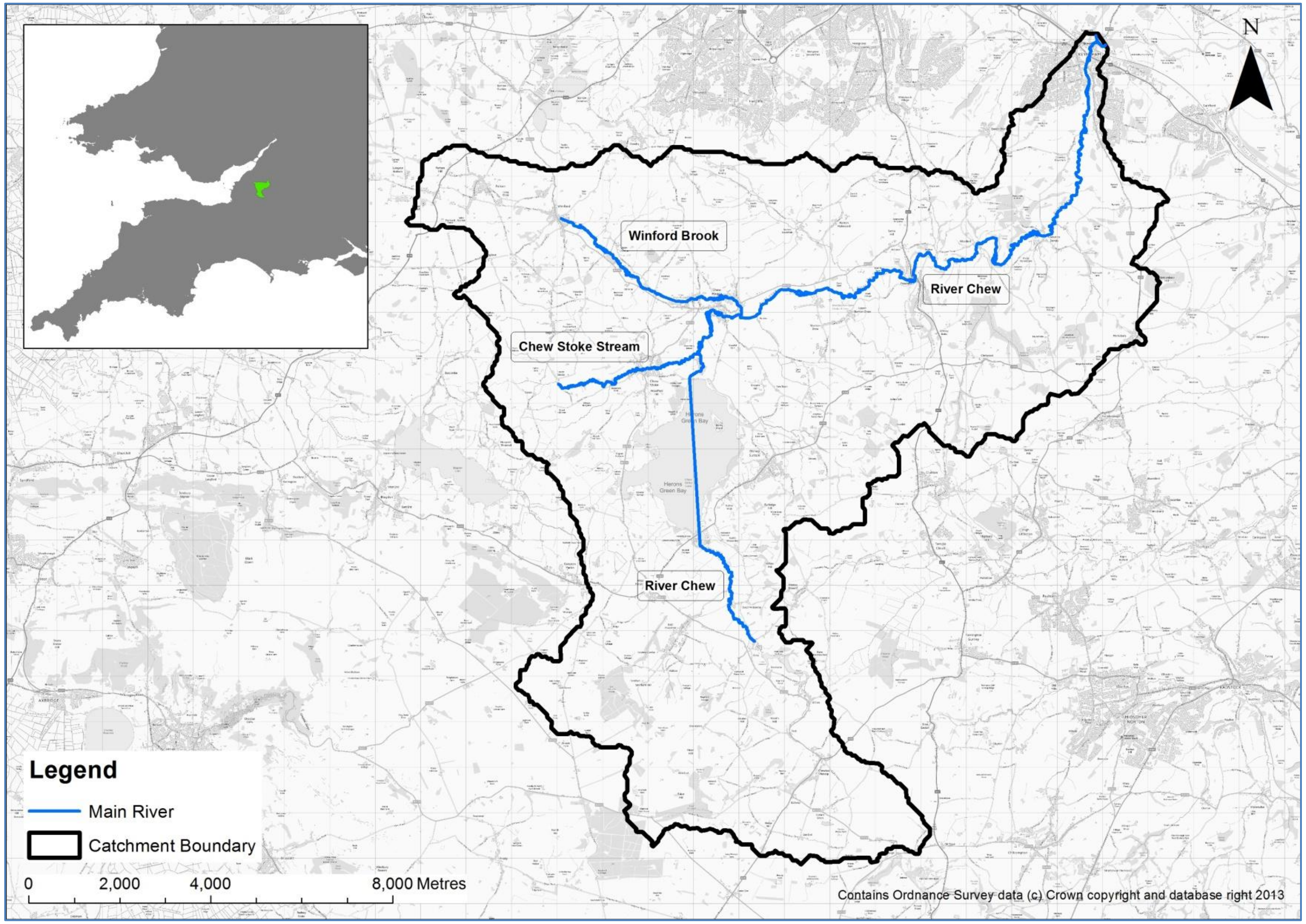

Figure 2-1 - River Chew catchment

2.2 Flood history

Information for a number of flood events was supplied by the Environment Agency from the Flood Reconnaissance Information System (FRIS) database. Information for the key events is summarised in Table 2-1. A full summary of recorded flood events is provided in the Calculation Record (Section 7.3).

The largest flood in the catchment during the Compton Dando gauging station period of record occurred in July 1968 and is detailed in a summary report of the event[2]. In 18 hours 175mm of rain fell in Chew Stoke, double the area's average rainfall for the whole of July. Soils in the area were already waterlogged after previous rainfall. Sheet runoff occurred on many fields with a depth of about 75-100mm which washed branches, debris and small trees into the river. This debris blocked bridges and weirs, raising water levels further. Along the valley almost 250 homes were flooded, some to a depth of 3m. In Chew Magna, 88 properties were flooded, many up to 2m deep. Houses in Tunbridge Road, Dumpers Lane, Silver Street and Lower Batch were particularly badly hit as well as the church hall, village school and fire station near Tun Bridge. Along Silver Street most buildings were flooded to depths of up to 2.5m. Although levels in Chew Valley Lake rose by 0.45m on 10th July it did not overflow. This rise seems small given the saturated soils and 175mm of rainfall. However this intense rainfall was recorded at Chew Stoke and it may be that across the wider catchment upstream of Chew Valley Lake the rainfall was not as intense. The summary report states it is estimated that the lake held back two-thirds as much water again as flowed down the valley from areas downstream of the lake. Chew Magna Reservoir was already near capacity before the storm.

More recently significant floods in the catchment occurred during September and November 2012 with a number of properties flooding during both events, particularly in Chew Magna and Chew Stoke. On the Winford Brook it was the September 2012 flood which was most significant in terms of flood depth and hazard, number of properties flooded and lasting impact. In both September and November 2012 the most significant flood depths, velocities and hazards to people from the Winford Brook were experienced at the junction of Streamside and Butham Lane, and continuing along The Batch. Photographic, wrack mark and anecdotal evidence suggests that the maximum flood depth on The Batch, adjacent to North Chew Terrace footbridge, was approximately 0.8m. The depth and velocity of the flow were demonstrated by a garden shed becoming lodged under the footbridge linking The Batch to North Chew Terrace.

On the River Chew, along Tunbridge Close and Dumpers Lane, the most significant flooding (in terms of depth and consequences) occurred from the November 2012 event. Along the River Chew flooding during both events was experienced in Dumpers Lane, Tunbridge Close, Madam's Paddock and also to the Fire Station at the junction of Tunbridge Close and Tunbridge Road. The greatest flood depths were along Dumpers Lane, specifically the low points north of Tunbridge Close where depths in excess of 1m were reported.

Table 2-1 - Summary of key flood events

Date	Location	Details	Source	Impact
10/07/1968	Chew Magna	Property flooded at The Orange Coach House, Chew Magna, during the severe flood of July 1968. Lower Batch, old peoples bungalows were reported as flooded. Fire Station at Tunbridge Road. Battle Bridge Lane was damaged with its parapets washed away together with the footbridge upstream. Port Bridge Mill reported as badly flooded and the parapet of port bridge was washed away. Silver Street, The Batch and Sprat's Bridge – new property on both banks downstream of Sprat's Bridge. All properties in the vicinity of Silver Street severely swamped and all bridges were flooded and damaged to varying degrees. Sprat's Bridge – parapets washed away and arch damaged. Dumpers House – flooded almost to first floor. Tunbridge Road – one property identified as	Fluvial	Buildings; land; transport route

[2] Environment Agency. June 2008. The Chew Valley floods of 1968.

Date	Location	Details	Source	Impact
		flooded up to five feet. Tunbridge Close – various homes flooded. B3130 road bridge was flooded and parapet walls demolished. New housing development south of Dumpers Lane had four feet of water in the houses. Dumpers Farm was severely flooded and stock drowned. Chew Magna Reservoir – some overtopping along its length and some properties downstream of reservoir flooded with damage to masonry walls. Environment Agency publication reported a total of 88 properties were known to have flooded. Flood level – old stone bridge downstream of Sprat's Bridge was 147.64mAOD and upstream at 160.76mAOD. Flood level – Plot 2 Streamleaze, flood depth level 141.00mAOD. Flood level – Silver Street, 144.50-145.11mAOD. Flood level – Red Gables, Silver Street 145.60mAOD, described as 'catastrophic'.		
10/07/1968	Chew Stoke	Several cottages in The Street were flooded. The Old Post Office, The Street flooded.	Fluvial	Buildings, transport route
10/07/1968	Pensford	Bye Mills – serious flooding of the disused mill. Church was flooded to a depth of eight feet and School three feet, both were damaged. The river knocked down outbuildings and seriously damaged houses nearby. The A37 main road bridge was washed away. Woodborough Mill House flooded to a depth of five feet.	Fluvial	Buildings; land; transport route
10/07/1968	Publow / Woollard	Ten houses and bungalow flooded. Woolard Bridge damaged beyond repair. One resident rescued by helicopter from a bungalow.	Fluvial	Buildings; land; transport route
10/07/1968	Winford	Property flooded – Snuff Mill Cottage, Kentshare Lane. Reported that the eight foot high wall at the bottom of the orchard was broken up and soil was washed away. Upper Littleton Mill, Powdermill Farm and Littleton Farm all flooded with varying degrees of damage. All bridges submerged and many houses and shops were affected. Walls on each side of the brook collapsed and some structural damage to buildings. Large quantities of hay and livestock lost. A car was thrust through the front wall of Waterloo Villa Cottage in the High Street and a hair salon was flooded to an estimated depth of four feet.	Fluvial	Buildings; land; transport route
24/09/2012	Chew Magna	Chew Magna Primary School flooded. Flooding of properties in Dumpers Lane, Tunbridge Close, The Batch, Silver Street, Stoneleigh, Streamleaze. Marbel House, The Batch reported as flooded through rear single skin defence. 3 Stoneleigh, property flooded up to outside doorstep but not inside property. 27 Tunbridge, garden flooded only	Surface water runoff; fluvial	Buildings; transport route
21/11/2012	Chew Magna	21-24 Tunbridge Close, 13-15 Tunbridge Close, Tunbridge Cottage, Applewood at Streamleaze, Willow Lodge, Bridge House, and The Mayals confirmed flooded. Other properties reported as flooded but not validated in Butham Lane, Silver Street, The Queens Arms, and The Batch. 17-20 Dumpers Lane – evidence of flooding around properties but unable to reach property due	Fluvial	Buildings

Date	Location	Details	Source	Impact
		to water depth; unconfirmed if flooding inside properties.		
21/11/2012	Chew Stoke	1, 3 and 4 The Street flooded. The Dene, Mill Lane garden flooded and garden indicated as flooded on 3 Sunny Patch, School Lane. Neither property flooded. The Old Post Office flooded. 4-8 Wallycourt Road and 1-6 Bilbie Close flooded due to surface water and properties in School Lane and The Street flooded from surface water, part combined with local watercourse.	Fluvial; surface water runoff	Buildings; transport route
23/11/2012	Pensford	Riverside, Pensford Hill reported flooded.	Unknown	Buildings

2.3 Previous studies

Mott MacDonald was commissioned in August 2010 to undertake a comprehensive review and update of the hydrology of the River Chew model[3] for the purposes of flood risk mapping. Additional 1D-2D modelling was also carried out. The existing Flood Map for the River Chew at the time that the Mott MacDonald study was undertaken was based on JFLOW; this needed to be updated and refined with more detailed modelling. An existing 1D ISIS model of the River Chew was available which was commissioned in 2004 to update the Flood Map and was produced by Capita Symonds. The River Chew model extent was from Chew Valley Lake to the confluence with the River Avon at Keynsham, and included the Regil Stream, Chew Stoke Stream and Winford Brook tributaries. There were significant uncertainties with the hydrology used within the 2004 model. The main aim of the Mott MacDonald study was to thoroughly review, investigate and update the hydrology for the River Chew and its tributaries as this was the largest area of uncertainty in the modelling.

Following the completion of the Mott MacDonald study two large flood events occurred within the River Chew catchment in September 2012 and November 2012, during which a number of properties were flooded. The outputs from the hydraulic model did not represent the flood extents and flooded properties seen during the floods, particularly for the Winford Brook in Chew Magna. The hydrology for the Upper River Chew catchment is very complicated and was the main uncertainty with the previous study. The 2012 floods indicated that the Winford Brook flows in the model are too low and this aspect of the assessment needs to be improved. New data and information has become available since the Mott MacDonald analysis which will be used within the current assessment. A review of the existing hydrological assessment was required prior to determining the most appropriate method(s) for improving the design peak flow estimates and hydrographs. The review is detailed in the document '2013s7440 – Technical Review Certificate_Hydrology_v1.0' which was submitted to the Environment Agency on 8th October 2013.

The final design peak flow estimates provided in the November 2012 report for key locations are detailed in Table 2-2. The report indicates that these locations correlate to:

- Chew Valley Lake – Inlet of Chew Valley Lake (357250, 157850)
- Chew Stoke – Chew Stoke gauging station (355950, 161800)
- Winford Brook – Confluence with the River Chew (358100, 163000)
- Compton Dando – Compton Dando gauging station (364850, 164750)

These flow estimates were derived using the FEH Rainfall Runoff method for Chew Valley Lake, Chew Stoke and Winford Brook. The Chew Stoke estimates were improved by using flow data from a modelled rating to generate model parameters from observed data. FEH Statistical method single-site analysis was used to derive the peak flow estimates for Compton Dando.

(Note: Flood frequency is provided as a return period (years) in this report when discussing results rather than annual exceedance probability (AEP). This is compatible with the FEH and makes discussion of the results easier. A conversion table is provided on Page viii of this report. The frequency is provided as an AEP in the Calculation Record summary information.

[3] Mott MacDonald. November 2012. River Chew – Flood Map Hydrology Update. Main Report.

Table 2-2 - Mott MacDonald (November 2012) final design peak flow estimates (m^3/s)

Return period (yrs)	Chew Valley Lake (4hr duration)	Chew Stoke (4hr duration)	Winford Brook (9hr duration)	Compton Dando (9hr duration)
5	11.4	19.6	6.9	30.2
10	14.4	24.1	8.7	39.0
50	17.8	29.8	10.8	49.3
50	22.8	37.8	13.8	66.3
75	25.0	41.3	15.2	75.6
100	26.9	44.2	16.2	82.6
200	31.9	51.9	19.2	102.8
1000	49.1	77.4	29.3	170.1

The Compton Dando design peak flow estimates were reviewed by Mott MacDonald in April 2013[4]. This analysis made use of an additional annual maximum (AMAX) value for water year 2011 and used a value of $180m^3/s$ for the July 1968 flood event (water year 1967) rather than the value of $100m^3/s$ quoted in HiFlows-UK. An updated single-site and pooled analysis was undertaken for the gauge location. The recommendation was to use the updated single-site analysis design peak flow estimates up to the 100 year return period event and ratios from the ReFH method, applied to the single-site 100 year event estimate, to derive the 200 and 1000 year return period event estimates. The revised peak flow estimates for Compton Dando are provided in Table 2-3. The ReFH ratios were not supplied in the report therefore only peak flow estimates up to the 100 year return period event are given in the table. The 100 year event estimate is just over 30% larger than the original estimate.

Table 2-3 - Mott MacDonald (April 2013) Compton Dando peak flow estimates

Return period (yrs)	Compton Dando (m^3/s)
5	31.3
10	42.4
20	56.6
50	82.4
75	Not provided
100	109.5
200	ReFH ratios not provided
1000	

2.4 Hydrometric data

2.4.1 Overview

There are a number of hydrometric gauges within and close to the River Chew catchment. The locations of the hydrometric gauges are shown in Figure 2-2. The Environment Agency gauge metadata and period of record of data supplied for this study is detailed in Table 2-4. The quality of the data was ascertained by spot checks on the quality flag provided with the data; this is summarised in the Calculation Record (Section 7.4).

[4] Mott MacDonald. April 2013. River Chew Model Improvements. Data Review and Scoping Report.

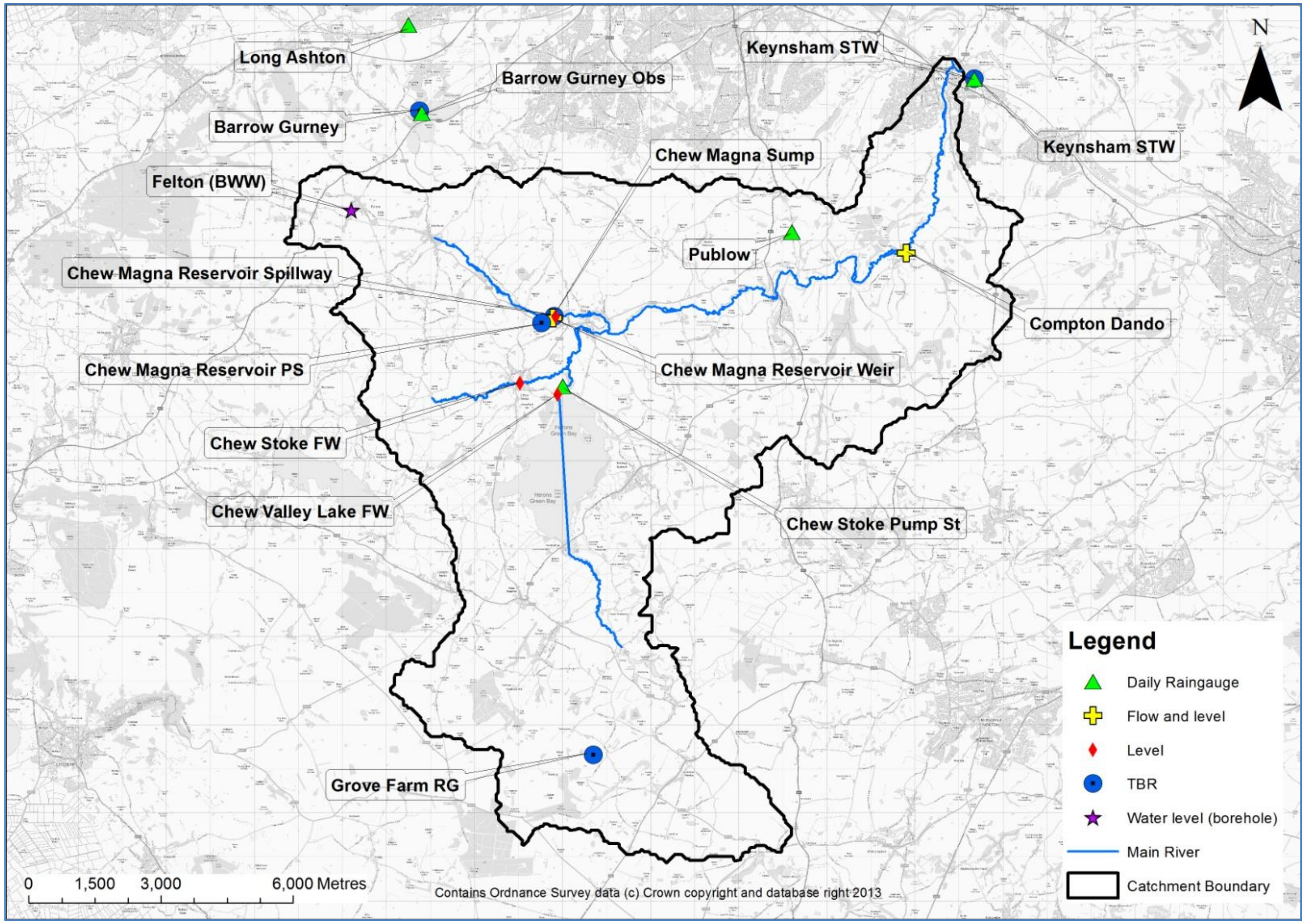

Figure 2-2 - Hydrometric gauges

Table 2-4 - Hydrometric gauge metadata

Gauge name	Gauge type	Grid reference	Period of record
Compton Dando (River Chew)	Stage / Flow	364803, 164722	10/12/1991-01/08/2013
Chew Magna Level (Winford Brook) – located at the stilling basin downstream of Chew Magna Reservoir – referred to as Chew Magna Reservoir Weir in this study	Stage / Flow	356841, 163250	01/12/2003-19/08/2013
Chew Magna Sump - Chew Magna Reservoir located in the pumping complex	Level	356780, 163230	01/12/2003-01/08/2013
Chew Valley Lake_FW	Level	356889, 161515	31/03/2005-01/08/2013
Chew Stoke FW	Level	356031, 161766	22/01/2004-21/08/2013
Felton (BWW)	Water level (15-minute continuous measurement)	352200, 165700	23/11/2009-13/03/2013
Felton (BWW)	Water level (manual dip – irregular interval)	352200, 165700	04/06/1982-11/12/2012
Barrow Gurney TBR	Recording raingauge	353750, 167960	01/10/1991-01/08/2013
Chew Magna PS TBR	Recording raingauge	356520, 163140	01/04/1992-08/03/2012
Chew Magna Spillway TBR	Recording raingauge	356803, 163285	13/04/2011-01/08/2013
Grove Farm RG	Recording raingauge	357691, 153319	01/01/2011-01/08/2013
Keynsham STW TBR	Recording raingauge	366346, 168683	01/10/1991-01/08/2013
Barrow Gurney Obs	Storage raingauge (daily)	353800, 167900	01/01/1925-30/04/2012
Chew Stoke Pump St	Storage raingauge (daily)	357000, 161700	01/01/1961-30/04/2012
Keynsham STW	Storage raingauge (daily)	366350, 168680	01/01/1975-31/03/2013
Publow	Storage raingauge (daily)	362200, 165200	01/01/1982-31/03/2013
Long Ashton	Storage raingauge (daily)	353500, 169900	01/01/1961-30/06/2002

2.4.2 Compton Dando gauge summary

Compton Dando gauging station opened in 1958 and consists of a trapezoidal critical depth flume in a deeply incised channel. The primary purpose of the gauge is for 'Water Resources Catchment Management'. It is a full range station but overestimates flow above 2m due to backing-up. The flow record is unreliable for a year after the July 1968 flood due to bank collapse and accumulated debris. The peak flow associated with the event is roughly estimated as 100m^3/s in the HiFlows-UK database; this was previously estimated as 226m^3/s. The estimate of flow for this event is very uncertain due to significant extrapolation and debris in the flume.

Chew Valley Lake provides significant storage in the headwaters and controls approximately 40% of the catchment area. The reservoir was constructed prior to the installation of the gauge and therefore the entire record incorporates the influence of the reservoir. Flows are substantially affected by compensation flows and surface water abstractions for public water supply and industry.

The rating for the gauge was reviewed in January 2001 following gaugings taken in 1999 and 2000. The new rating was applied from the start of the archived 15-minute record in December 1991. More high flow gaugings are stated to be required to confirm the rating. Spot gaugings were supplied by the Environment Agency for this commission; the largest of these is 4.24m (35.0m^3/s). The National River Flow Archive (NRFA) states that an ultrasonic gauge was installed in November 2007. The record of rating-derived flows has been continued in parallel to the ultrasonic flows.

Mott MacDonald derived a new high flow rating for the gauge as part of the 'River Chew – Flood Map Hydrology Update' project (November 2012). This maintains the existing Environment Agency rating for stage up to 3.5m; above this the new rating is applied. Figure 2-3 shows the existing Environment Agency rating ('HiFlows-UK') and the revised Mott MacDonald rating ('WISKI'). The supplied spot gaugings and bankfull stage are also plotted.

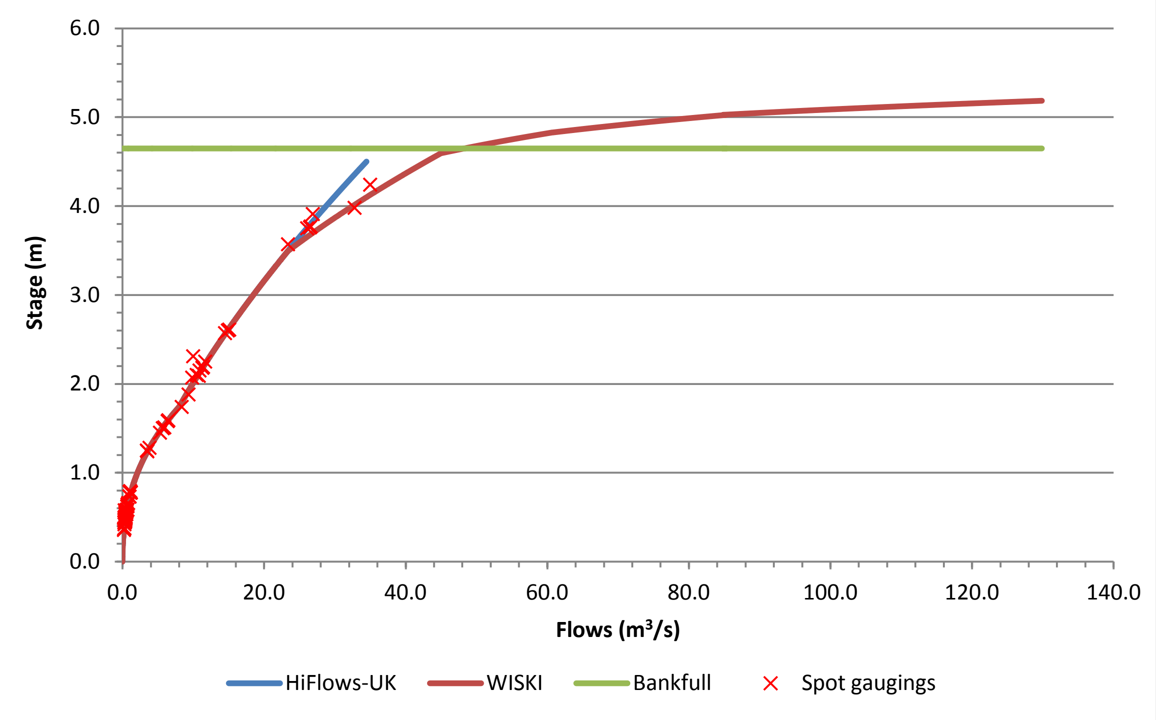

Figure 2-3 - Compton Dando ratings

The new rating was applied from water year 1979 onwards. It is not clear why the new rating was applied from this time. The previous revised rating was applied from water year 1992; the Hydrometry and Telemetry team has stated that it is not possible to re-rate the gauge prior to the start of the high resolution data in 1991. In HiFlows-UK no stage values are provided for water years 1958-1991, with the exception of 1967 and 1982-1987, which are stated to come from NRFA. The source of the flow data from 1958-1991 is attributed to 'EA AMAX only'. Correspondence from Peter Spencer (Environment Agency) to Ken Moss (Environment Agency) in May 2013 indicates that the WISKI DayMax stage data starts on 21st January 1980. As the first water year that Mott MacDonald applied the new rating is 1979 (1st April 1980) it may be that the new rating was applied from the start of the DayMax stage record. AMAX values derived by Mott MacDonald differ to those in HiFlows-UK between water years 1979 and 1992, even for those years which have a stage value in HiFlows-UK which is <3.5m. From water year 1992 onwards the HiFlows-UK and Mott MacDonald AMAX values only differ where stage is >3.5m.

Peter Spencer recommended that for 1982-1987 the HiFlows-UK stage data is used, with the Mott MacDonald rating applied for values >3.5m; for the remaining years (1979-1981 and 1989-1991) the 'EA AMAX only' values in HiFlows-UK should be used. As these comments were received following the completion of the Mott MacDonald hydrological assessment they were not incorporated into the analysis.

The Environment Agency supplied flow data for the Compton Dando gauge from water year 1991 for this commission; this data uses Mott MacDonald's new high flow rating for stage >3.5m. Peter Spencer's recommendations were incorporated in the assessment of the Compton Dando data for the current commission to generate an AMAX series from 1958-2012. The data for water years 2000, 2001 and 2012 is incomplete. The AMAX date for water year 2000 is 30th October 2000. This is a known significant flood event and the value was retained within the analysis. The AMAX date for water year 2001 was compared to other local gauges. This was an AMAX event at Station 53005 Mitford Brook at Mitford and Station 53009 Wellow Brook at

Wellow and has a similar rank to Compton Dando at these stations. Therefore this value was also retained in the analysis. Water year 2012 is 92% complete. The AMAX value occurred on 23rd November 2012 and is the highest flow in the record after July 1968. This is a known significant large flood event in the catchment and no larger events are known to have occurred in the latter part of the water year therefore the value was used in the analysis. This gives 54 water years of record to use in the assessment; water year 1970 is missing. The AMAX series is shown in Figure 2-4; the yellow columns indicate years for which the data for the water year is incomplete. This AMAX series is plotted with the July 1968 (water year 1967) event taking a value of 100m^3/s. The QMED value derived using the entire period of record (1958-2012) is 18.9m^3/s.

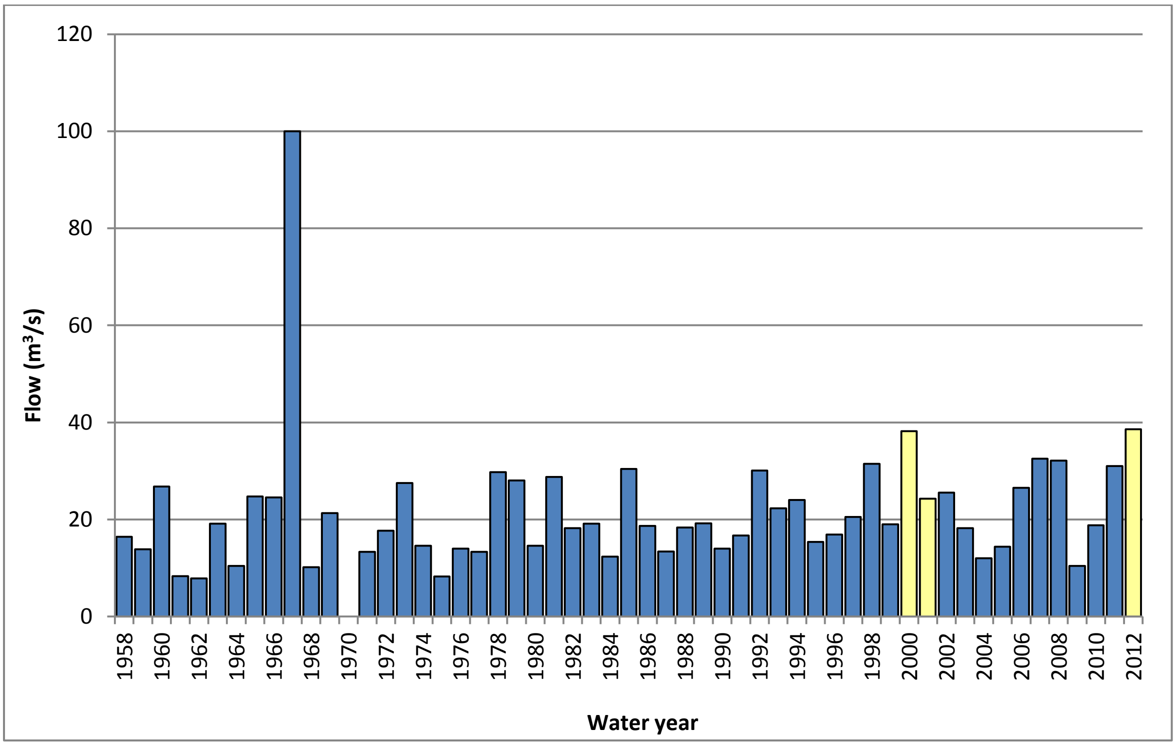

Figure 2-4 - Compton Dando AMAX series

JBA's Hydrometric Database (HyDB) was used to examine the Compton Dando flow data using the digital data from 1992 supplied by the Environment Agency. The data shows that there is generally very low baseflow over the summer months (about 0.3-0.4m^3/s). Baseflow is usually maintained at a higher level for several days and sometimes weeks after a peak flow event, even when there is no additional rainfall. The hydrographs tend to have a steep rising limb and slightly less steep receding limb. There is a period of missing data between February 2001 and January 2002 and for other short periods during the record. The data shows some anomalous values throughout the record, as shown in Figure 2-5; this does not tend to affect peak flow values. Figure 2-6 shows water year 2012 which includes the November 2012 flood event and highlights the number of high peak flows which occurred during late 2012 and early 2013. Further plots from the HyDB are provided in the Calculation Record (Section 7.4.2).

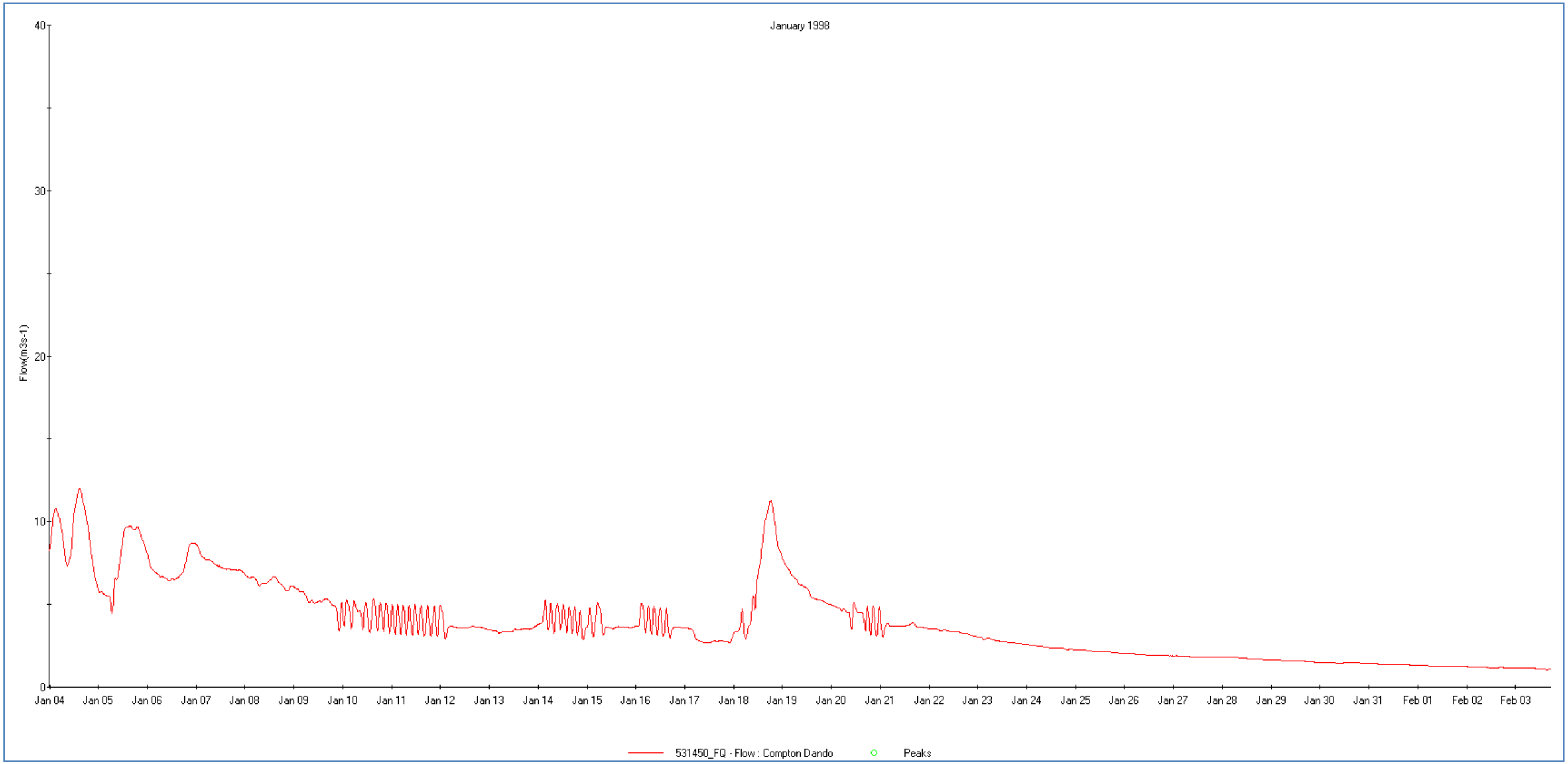

Figure 2-5 - Compton Dando anomalous data example

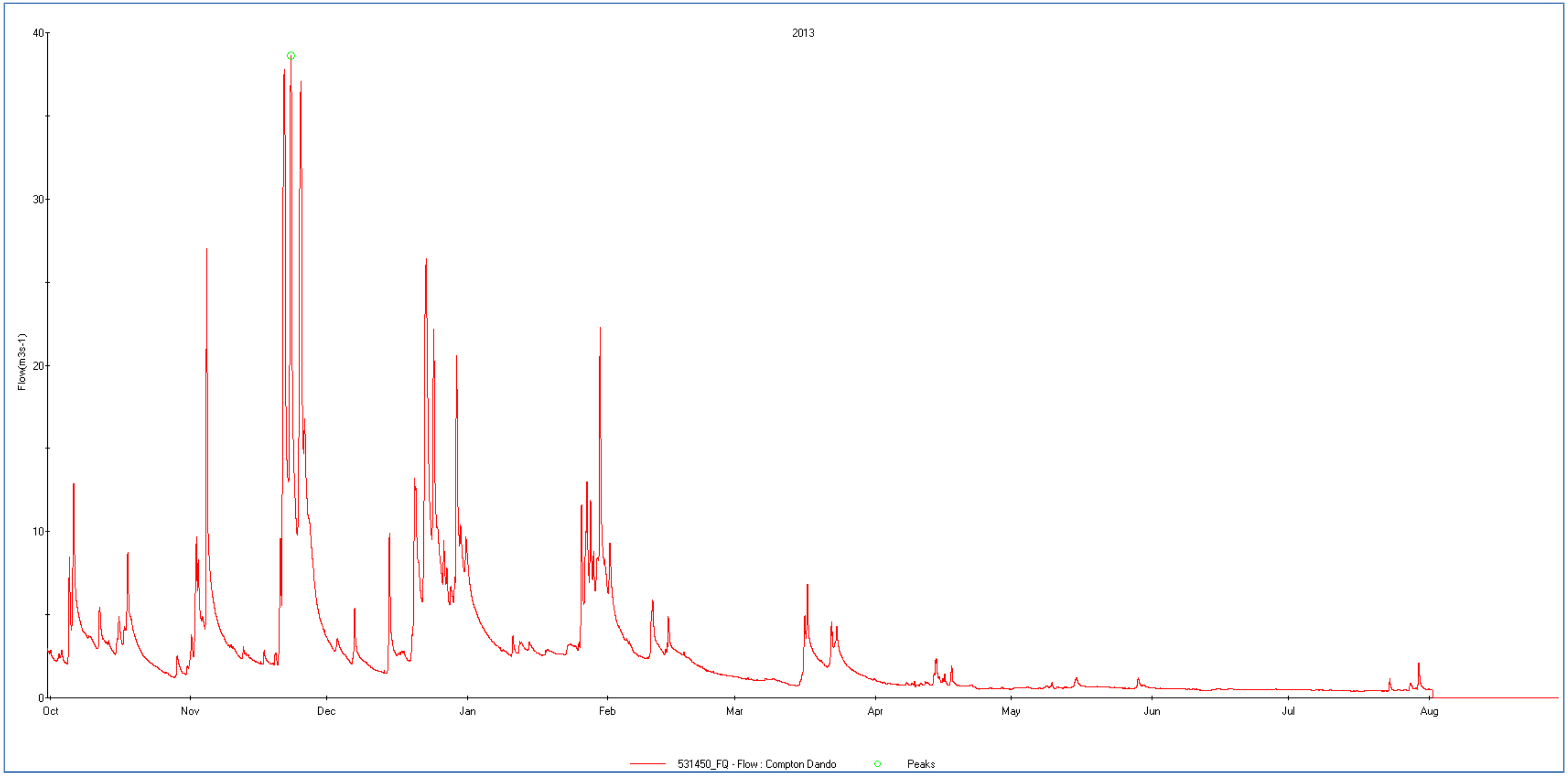

Figure 2-6 - Compton Dando water year 2012

2.4.3 Chew Magna Reservoir Weir gauge summary

Flow data is available for the stilling basin just downstream of the outfall from Chew Magna Reservoir. Flow out of the stilling basin is controlled by a two-stage weir with a rectangular notch. This gauge is not contained in the HiFlows-UK database or the NRFA database. Data is available for approximately a ten year period between December 2003 and August 2013.

The Environment Agency and Bristol Water use different rating curves to determine flow passing over the weir; this leads to significant differences in the flow series. Halcrow was commissioned by Bristol Water in June 2013 to develop a new rating for the weir[5] due to uncertainty regarding the accuracy of the existing weir rating and the ability to measure flood flows over the weir. A new rating was developed for the weir using the Francis formula. The rating curve was calculated up to the top of the sides of the weir plate (810mm). Above this value the weir can no longer be treated as sharp-crested and at this point some flow bypasses the weir on the right bank. During flood conditions the weir is likely to drown out due to the relatively small conveyance capacity of the downstream river channel. Sensitivity analysis showed that the weir is likely to drown at flows between 6.4m^3/s and 8.6m^3/s. As the weir is surcharged at less than 5.0m^3/s it is not possible to take accurate flow readings for the condition at which the weir drowns. An alternative to using the weir for flow measurement is to use the reservoir spillway weirs (located just upstream of the two-stage weir) during flood flows. However when the level reaches 400mm above top water level the right side wall of the auxiliary spillway channel is overtopped causing the rating curve to underestimate the flow over the spillways.

Flow over the spillways and the weir was calculated for three flood flows (January, September and November 2012) using observed records of reservoir water level, level at the weir and the rating curves calculated for the study. The calculated flows for the weir were noticeably less than those over the spillways. The difference between the two flow series increases substantially above 4.0m^3/s. This is likely to be due to part of the spillway flow bypassing the weir, which is not accounted for in the rating, as well as non-uniform conditions at the weir at high flows. The Halcrow study concluded that the new rating for the weir should be accurate for flow measurement up to around 4.5m^3/s (810mm water level at the gauge). At higher flows the main and auxiliary spillways can be used for flow measurement up to around 20m^3/s.

The existing Environment Agency rating and Halcrow rating for the weir are plotted in Figure 2-7. Spot gaugings taken at the gauge location are also plotted although there are only four gaugings taken between January 2007 and January 2011. The largest spot gauging is 1.6m^3/s; the spot gaugings indicate that both rating curves may be underestimating flow.

The Environment Agency records for water years 2003, 2006, 2008 and 2012 are incomplete, although the latter three years have over 90% of the record available. The Compton Dando gauge record was investigated to determine if the Chew Magna Reservoir Weir AMAX dates were also AMAX events at Compton Dando. This was confirmed for 2003, 2006 and 2008. The AMAX date for 2012 at Compton Dando was 23rd November 2012 compared to 21st November 2012 at Chew Magna Reservoir Weir. There were three peak flow events at both gauges between 21st and 25th November and it is conceivable that the tributary catchment will respond differently to the River Chew catchment. Therefore the AMAX values for all water years were used within the analysis.

The AMAX series derived using the existing Environment Agency rating and the AMAX series derived from the 2013 Bristol Water ratings are provided in Figure 2-8. The incomplete years are shown in pale yellow for the Environment Agency rating and light green for the Bristol Water 2013 ratings. It can be seen that the Bristol Water ratings generate higher peak flows than the Environment Agency rating. This is only significant for the highest values in the record – 2007, 2011 and 2012. The Bristol Water ratings flow estimate for the September 2012 event, the largest in the record, is more than twice that generated by the Environment Agency rating. QMED using the Environment Agency rating is 3.3m^3/s for the period of record (2003-2012); using the Bristol Water ratings produces a QMED of 3.9m^3/s.

[5] Halcrow Group Limited. June 2013. Performance review of the compensation weir. Chew Magna Reservoir compensation study.

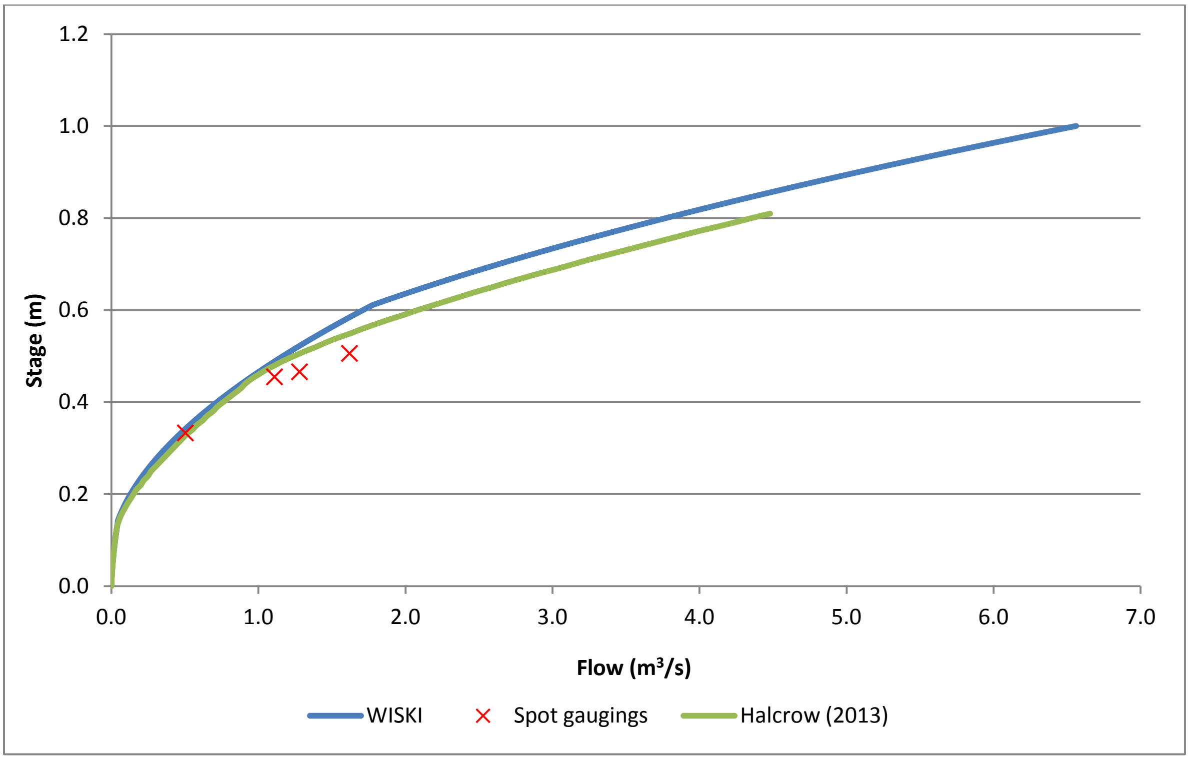

Figure 2-7 - Chew Magna Reservoir Weir ratings

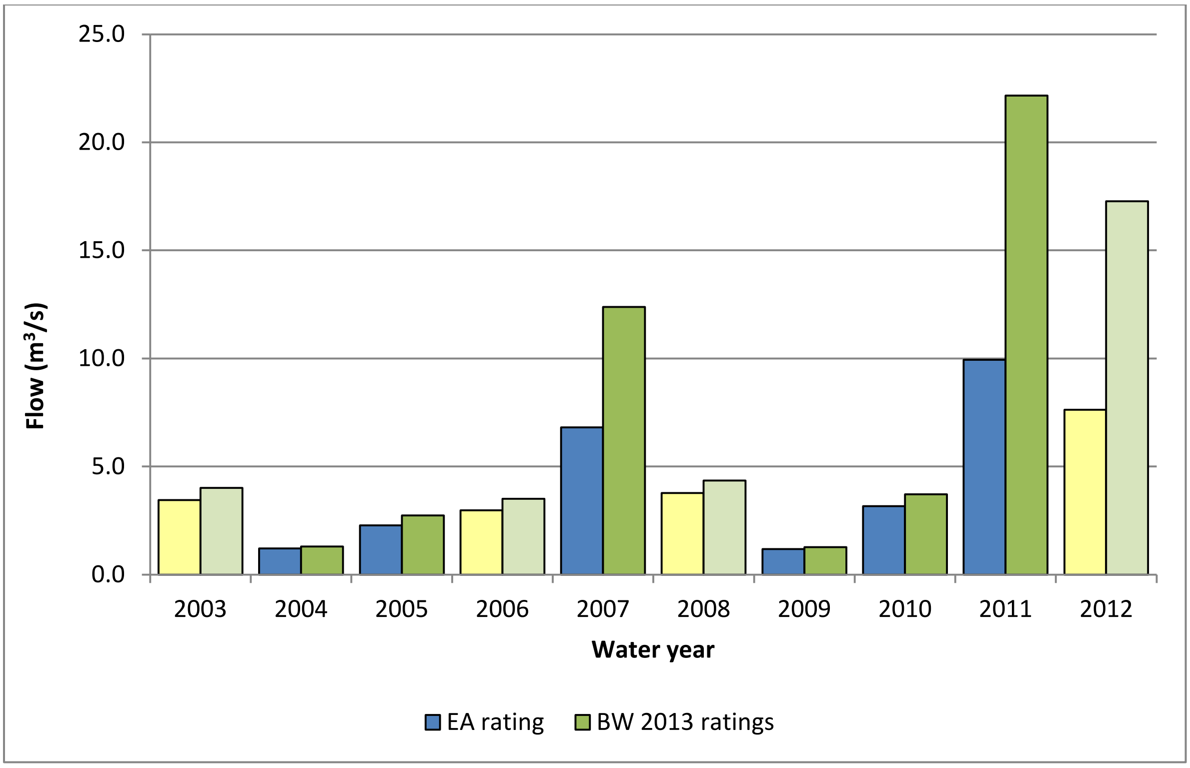

Figure 2-8 - Chew Magna Reservoir Weir AMAX series

The HyDB was used to examine the Chew Magna Reservoir Weir flow data and to assess the catchment response; this assessment was based on flow data derived from the existing Environment Agency rating. The hydrographs tend to have a steep rising limb and slightly less steep receding limb; baseflow generally returns to approximately the pre-event value about two to three days after the peak flow event. Baseflow appears to be very low even during the winter months, in fact, winter baseflow often seems to be slightly lower than the summer baseflow. This may be a result of the compensation flow from the reservoir which is $0.05m^3/s$ between May and September and $0.03m^3/s$ between October and April.

The data generally appears to be very ‘blocky’ with peaks truncated in many cases, although this is less noticeable in the data after May 2012. In January 2011 it looks as though there may have been a problem with the gauge as the flow becomes quite uniform for four to five days and then drops very rapidly back to baseflow (see Figure 2-9). There could potentially have been debris on the weir which was subsequently cleared but the actual reason for this anomaly in the data is unknown. The largest flow recorded at the gauge was on 24th September 2012, when significant flooding occurred in Chew Magna. Figure 2-10 shows water year 2011 with the September 2012 event at the right edge of the plot. Further plots from the HyDB are provided in the Calculation Record (Section 7.4.3).

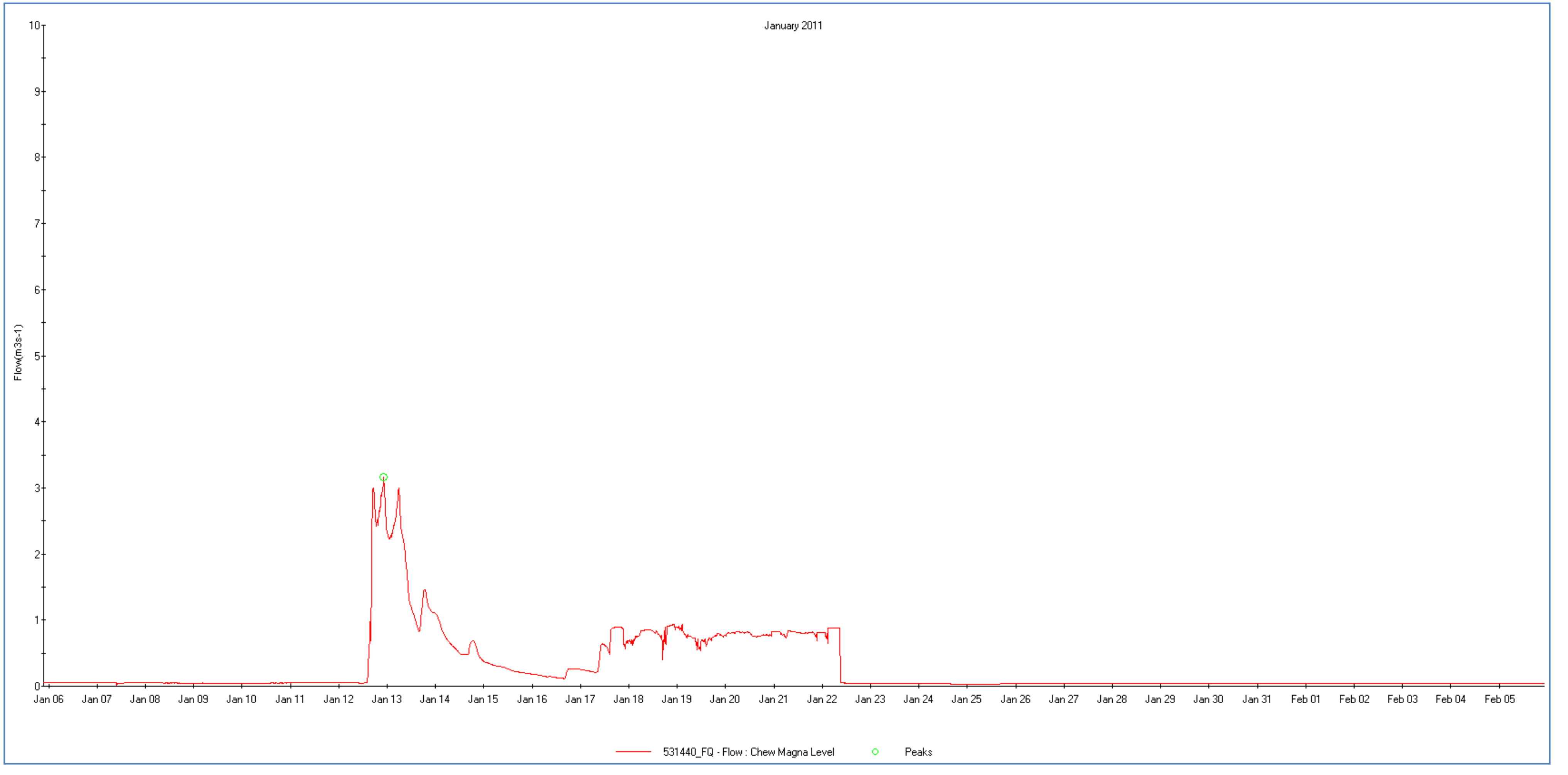

Figure 2-9 - Chew Magna Reservoir Weir data anomaly

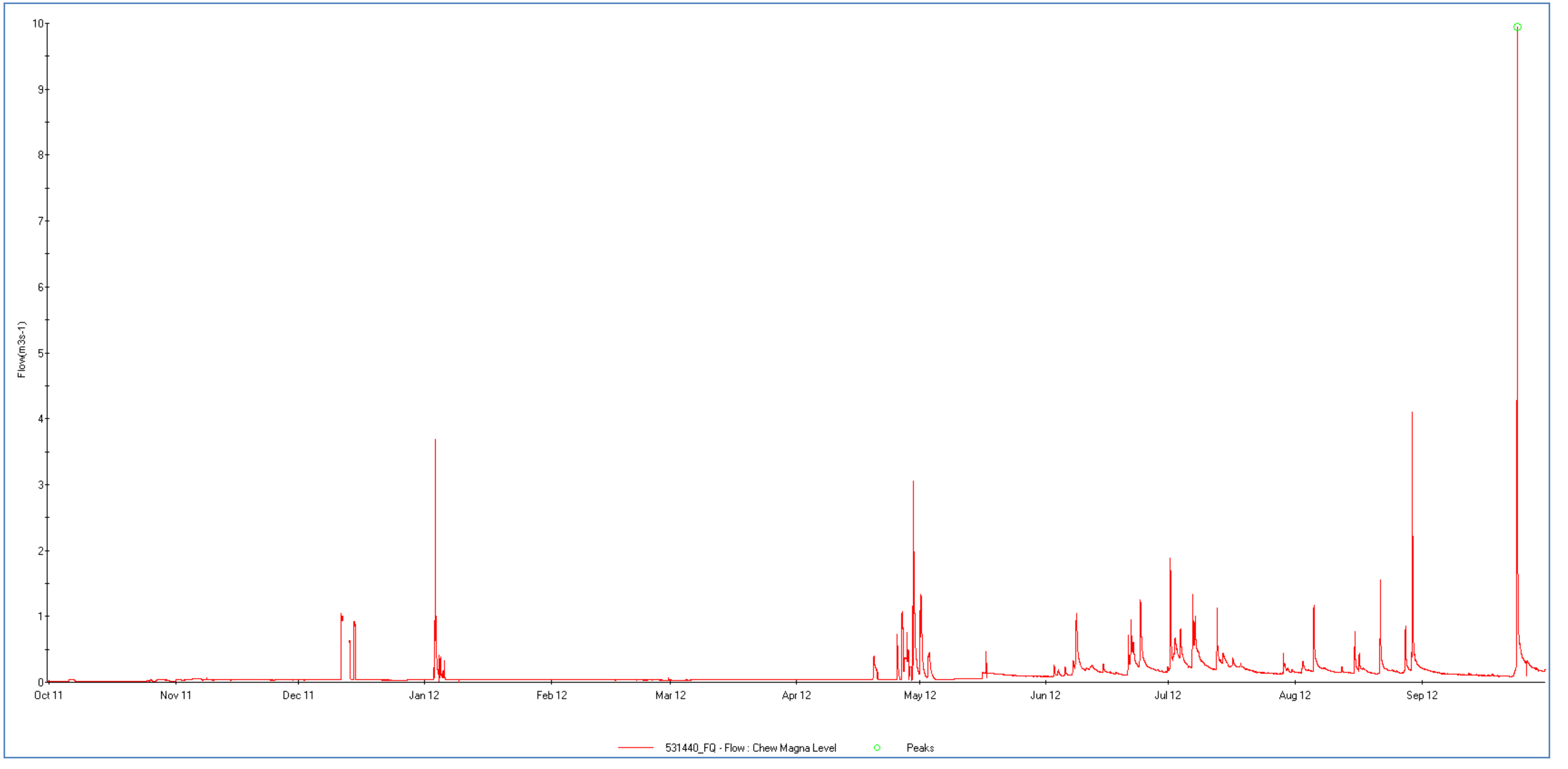

Figure 2-10 - Chew Magna Reservoir Weir water year 2011

2.4.4 Chew Magna Reservoir level gauge summary

The Environment Agency and Bristol Water both monitor water level within Chew Magna Reservoir. The Environment Agency previously monitored water level using a gauge located within the Bristol Water pumping complex (Chew Magna Sump), approximately 250m west of the reservoir outfall. Water levels recorded at this gauge did not necessarily reflect the water levels in the reservoir as sluice gates can be operated by Bristol Water to isolate the pump sump from the reservoir. The Environment Agency gauge has now been moved (August 2013) to a fishing pontoon towards the north-east end of the reservoir and consists of a pair of pressure transducers.

Bristol Water records water levels using a transducer sited on the reservoir side of the sluice gate and therefore reflects the reservoir level better than the previous Environment Agency gauge. This transducer was not calibrated more accurately than ±50-70mm hence there is some uncertainty in the Bristol Water level record.

Mott MacDonald undertook a comparison of the Environment Agency (Chew Magna Sump) and Bristol Water datasets as part of a previous study[4]. This showed a consistent difference between the two series for the September 2012 and November 2012 flood events. The Bristol Water levels were found to be approximately 150mm higher than the Environment Agency levels. Larger differences (approaching 1m) were found for the January 2012 event; this may have been caused by periodic operation of the pump by Bristol Water. No pumping took place during the September and November events.

Figure 2-11 shows the full period of record for the Environment Agency 'Chew Magna Sump' gauge. An approximate threshold line has been included to show the level above which overflow from the reservoir will occur. It can be seen that from about May 2012 to April 2013 the reservoir was continuously overflowing. Other significant overflow periods can be seen to occur in September to November 2006, April 2007, May to June 2007 and January 2008. There also appears to be a period of high water level / overflow between May 2013 and July 2013. This does not correlate with increased stage at the compensation weir gauge.

The full period of record for the Bristol Water level gauge is shown in Figure 2-12. The different line colours indicate a change in instrumentation. The data shows the same trend as the Environment Agency data as would be expected. The Bristol Water data is provided as a value above or below the top water level of the reservoir. Therefore the 'zero' line reflects the top water level and anything above this is spilling from the reservoir. Further plots from the HyDB are provided in the Calculation Record (Section 7.4.4).

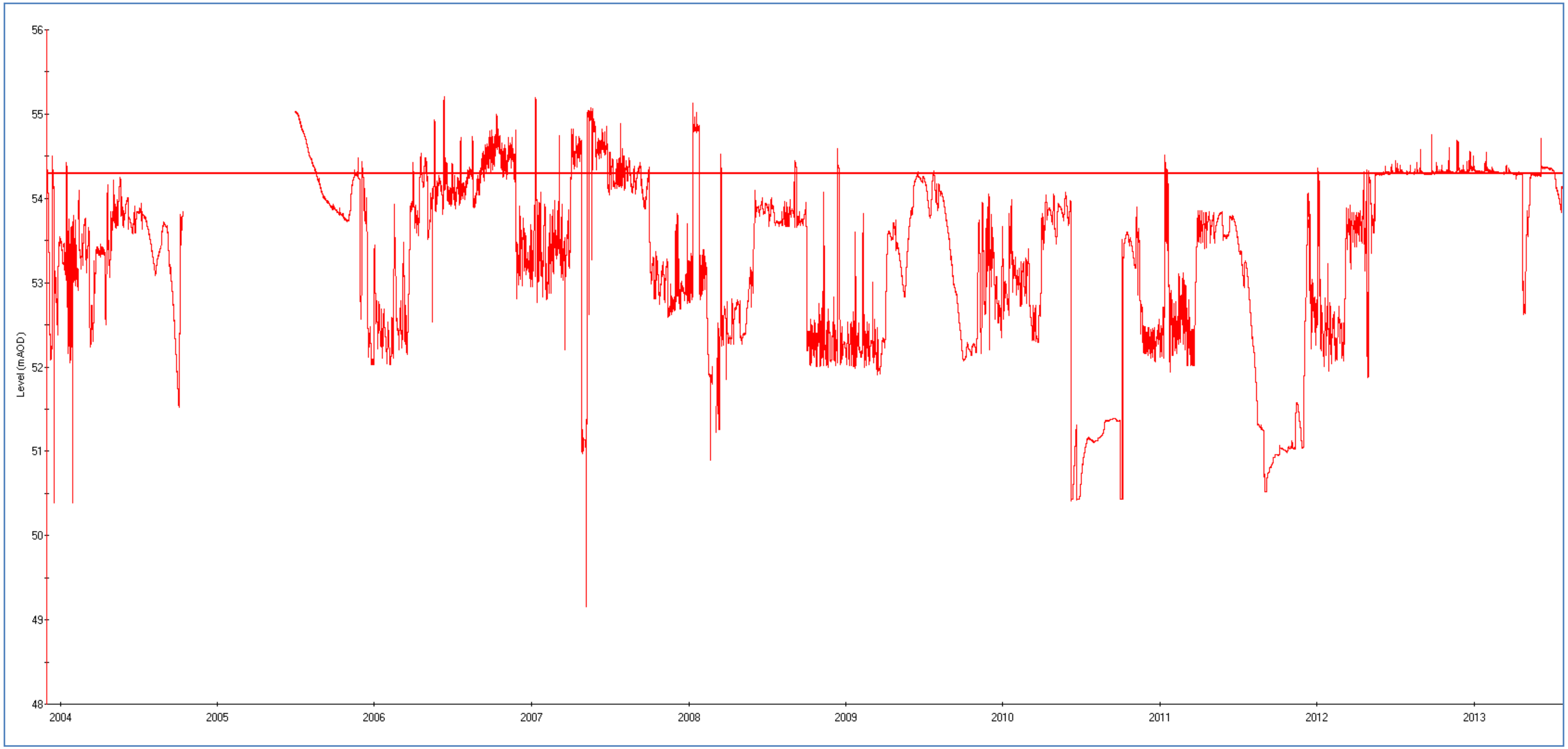

Figure 2-11 - Chew Magna Sump full period of record

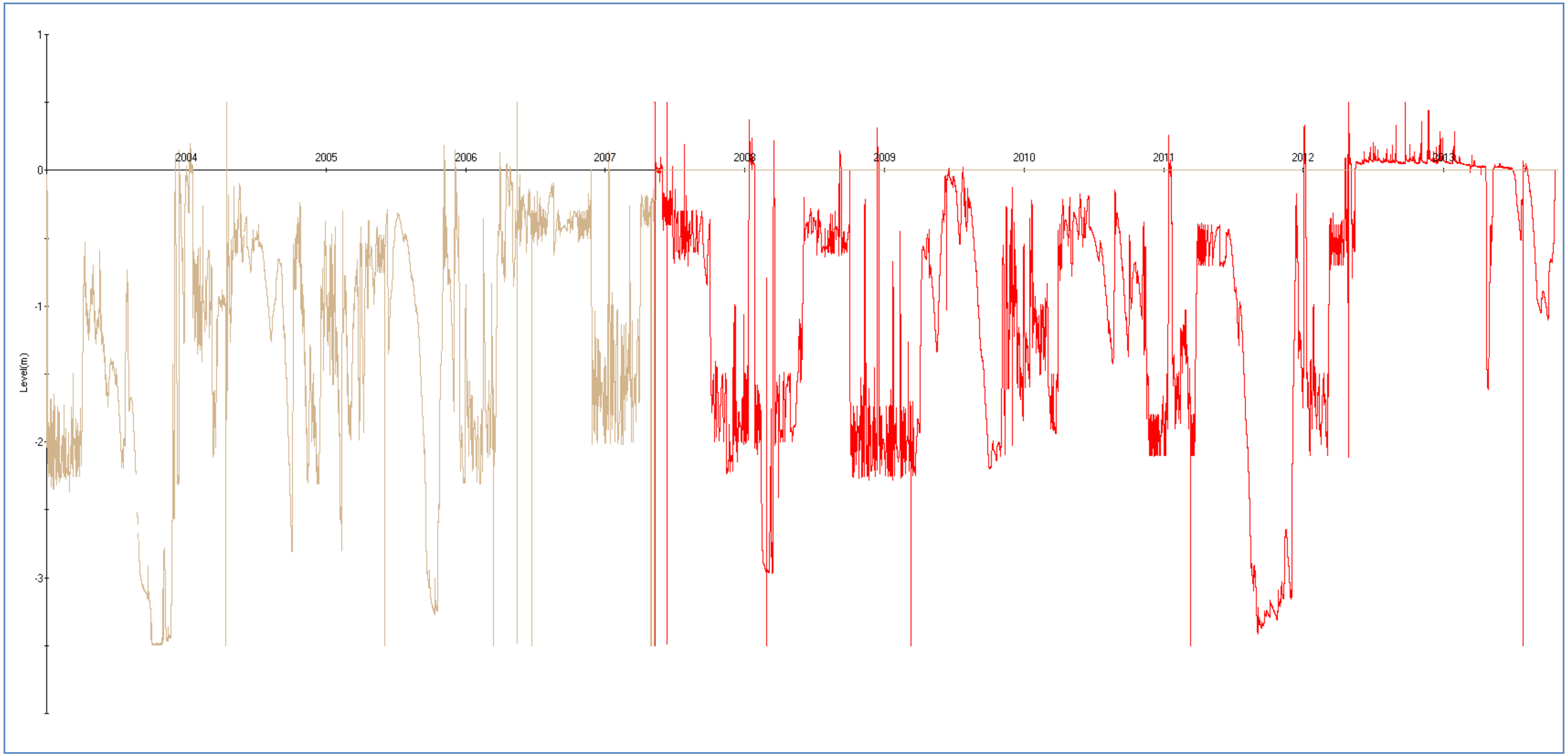

Figure 2-12 - Chew Magna Reservoir Bristol Water level gauge full period of record

2.4.5 Chew Valley Lake gauge summary

The Environment Agency does not monitor level at Chew Valley Lake via its own gauge and instead takes a feed from a Bristol Water gauge. Bristol Water records water levels in Chew Valley Lake using two separate instruments: (i) transducer since December 2001 and (ii) radar mounted on the reservoir tower since November 2008. The radar instrument is prone to spiking. It is believed that the Environment Agency feed is from the transducer instrument.

Figure 2-13 shows the full period of record for the Chew Valley Lake_FW gauge, supplied by the Environment Agency. The threshold applied is the spillway level of the reservoir (56.38mAOD). Chew Valley Lake was spilling almost continuously between June 2012 and February 2013. The lake level almost reached the spillway level in May 2012 but did not cross the threshold. This is the most significant period of overflow from the lake during the period of record supplied. Shorter overflow periods occurred in January 2007, February to March 2007, January 2008, March 2008 and February 2009. Figure 2-14 shows the water level record for water year 2012.

Flow is monitored by Bristol Water on the River Chew at a flume structure just upstream of the Chew Stoke Stream confluence; this captures compensation flows and overflows from Chew Valley Lake. The flow values are derived from a weir rating equation. Bristol Water has stated that this is accurate up to approximately 1.5m (2.2m^3/s). Although the weir rating provides flows for stage values up to 3m, the largest recorded value in a dataset supplied by Bristol Water (25/01/2007-10/03/2013) is 2.1m (4.2m^3/s). This generates truncated hydrographs where the stage / flow exceeds this value, for example, the November 2012 event. Further plots from the HyDB are provided in the Calculation Record (Section 7.4.5).

Figure 2-13 - Chew Valley Lake_FW full period of record

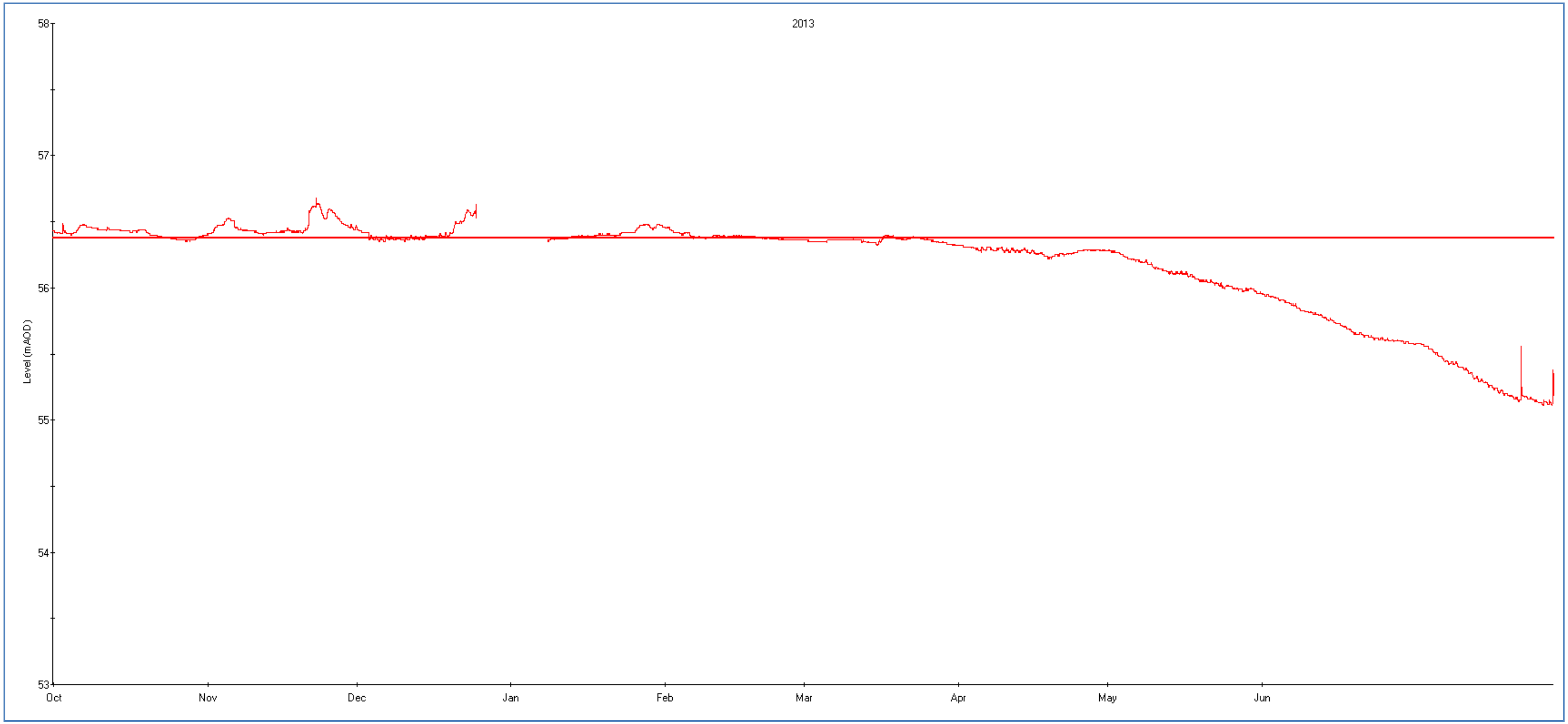

Figure 2-14 - Chew Valley Lake_FW water year 2012

2.4.6 Chew Stoke Stream gauge summary

The gauge on the Chew Stoke Stream in Chew Stoke consists of a pressure transducer with a stilling tube and stage board. This is a level-only site and was not designed for gauging flow. The gauge is positioned on the downstream face of the B3114 Bristol Road bridge. Mott MacDonald's 2013 report[4] states that it is possible that the channel bank may cause significant constriction to the channel just upstream of the gauge. Approximately 30m downstream of the gauge there is an in-channel island and tree which may impact gauge readings during flood events. The site was resurveyed in mid-2009 when dual sensors were installed. Gauged water levels are thought to be more reliable from this time.

Mott MacDonald derived a modelled rating curve for the site as part of the 2012 study[3] but there is significant uncertainty in the stage-discharge relationship. A rating equation was not developed for the site and therefore it is not possible to use the HyDB to generate a flow series from the level data. The level AMAX series is shown in Figure 2-15. Water years 2003, 2004 and 2012 are incomplete although the record for 2012 is >90% complete. The AMAX dates for water years 2004 and 2012 match those of Chew Magna Reservoir Weir and Compton Dando for 2004. The date for water year 2003 does not match the other two gauges and as this is the lowest ranking event it is thought that this was probably not the AMAX event for this water year. At the other gauges the AMAX date is prior to the installation of the Chew Stoke gauge. The median of the AMAX values for 2004-2012 was generated producing a HMED (median level) value of 49.00mAOD.

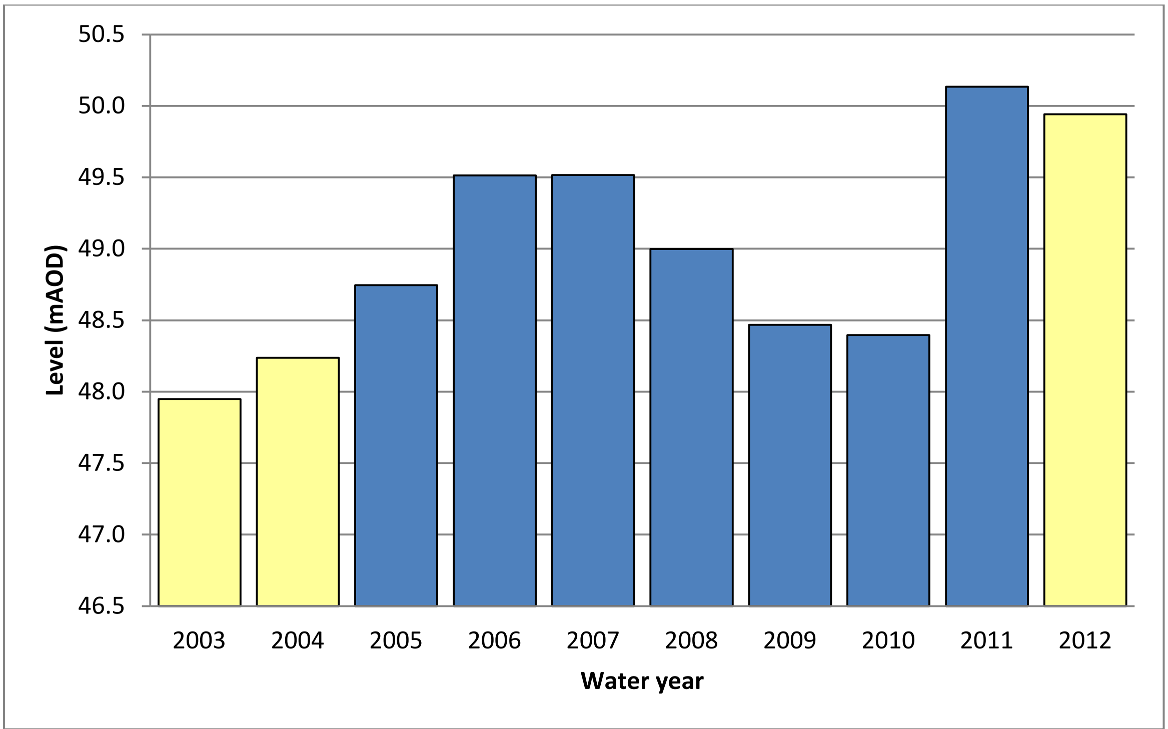

Figure 2-15 - Chew Stoke AMAX series

The HyDB was used to examine the Chew Stoke FW level data and to assess the catchment response. Figure 2-16 shows the full period of record and the change due to the resurvey and installation of dual sensors in mid-2009 can clearly be seen in the data. The hydrographs tend to have a steep rising limb with a more gentle recession back to base level. The hydrographs tend to return to just above the pre-event base level very quickly after the peak. Figure 2-17 shows water year 2011 with the September 2012 event at the right edge of the plot. Further plots from the HyDB are provided in the Calculation Record (Section 7.4.6).

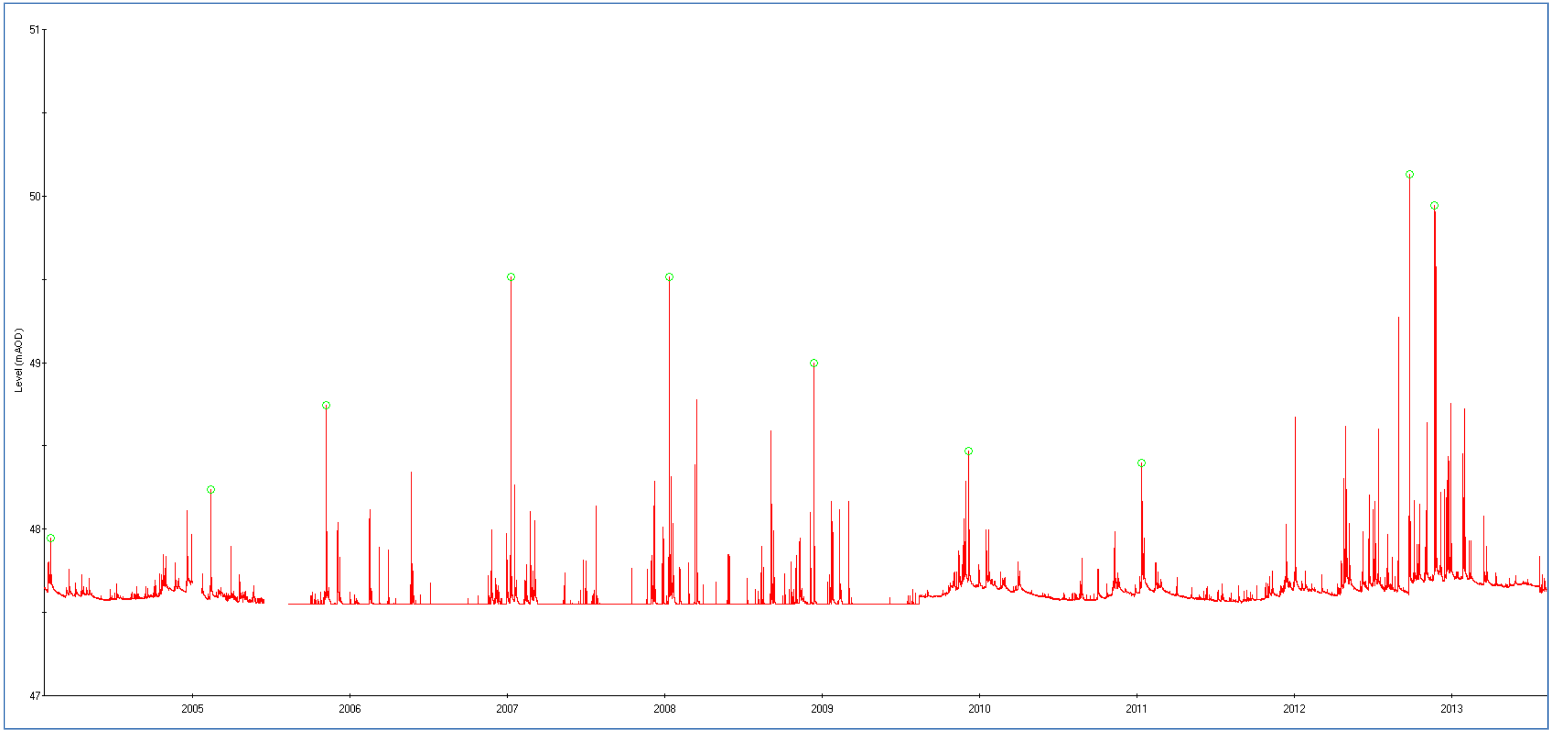

Figure 2-16 - Chew Stoke full period of record and AMAX events

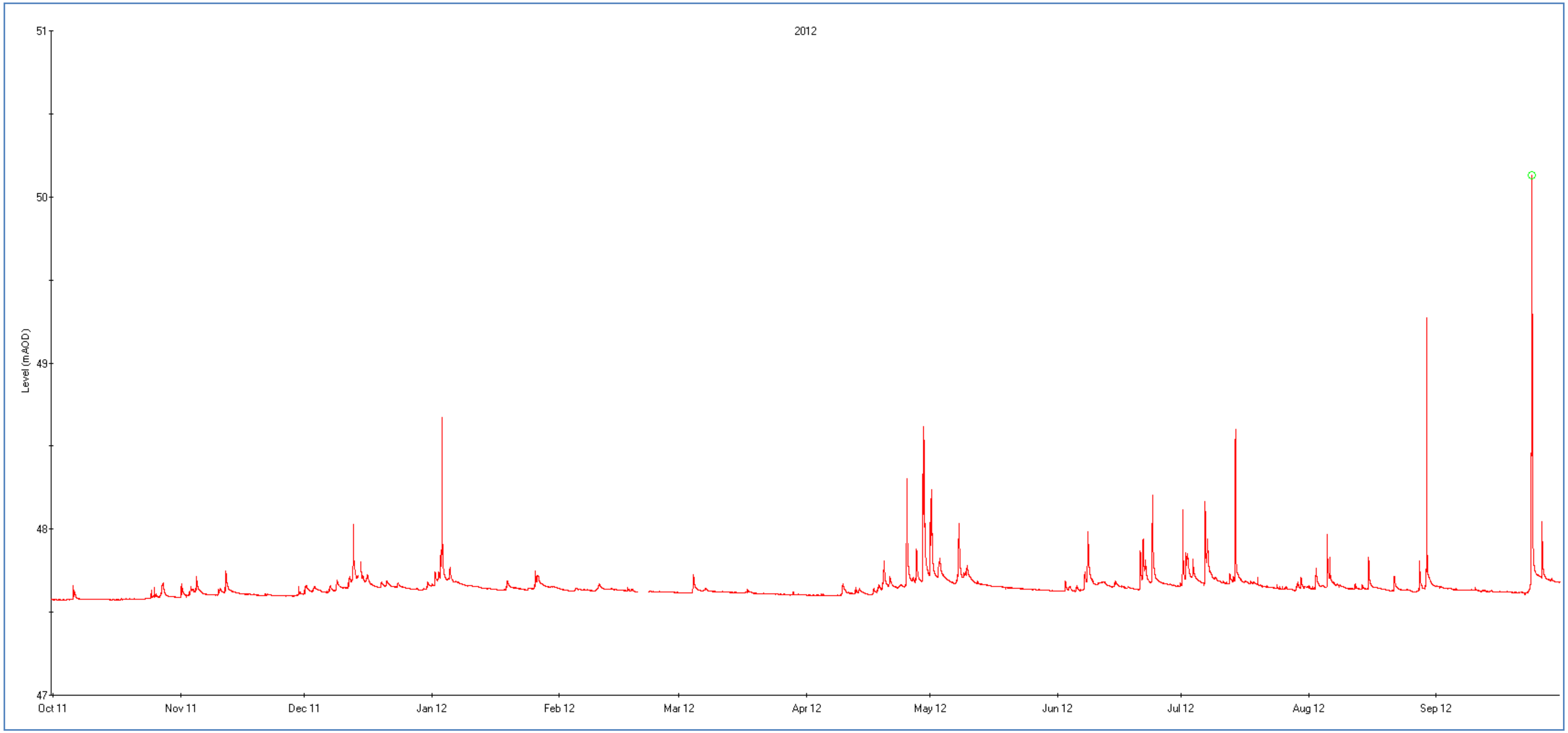

Figure 2-17 - Chew Stoke FW water year 2011

2.4.7 Felton (BWW) borehole gauge summary

Data was provided by the Environment Agency for the Felton (BWW) borehole gauge. This records the water level below the ground surface, with smaller values indicating an increase in the water level (i.e. closer to the ground surface). There is a short continuous measurement 15-minute record from November 2009 and a longer manual dip record (from June 1982) which is taken at irregular intervals.

The gauge is located at the top of the Winford Brook catchment in an area of limestone, which is likely to be of reasonably low storage potential (Secondary A aquifer designation). As the rest of the catchment is predominantly comprised of mudstone, the data from this gauge will not be representative of the majority of the catchment. However, the data does provide an indication of recharge in the local area at the top of the catchment.

Figure 2-18 shows how the groundwater level responded to rainfall during the largest recent floods in the catchment (September and November 2012). Particularly for the November 2012 flood it is clear that the water level rose considerably. The figure compares the September to December 2012 period to the same period during 2010. This shows how high the groundwater level was in 2012 compared to the same period in 2010. Figure 2-19 shows the full period of record for the 15-minute data. Although the record is short and with a long period of missing data the general trend of higher water levels in winter and lower water levels in the summer can clearly be seen as can the particularly high water levels at the end of 2012.

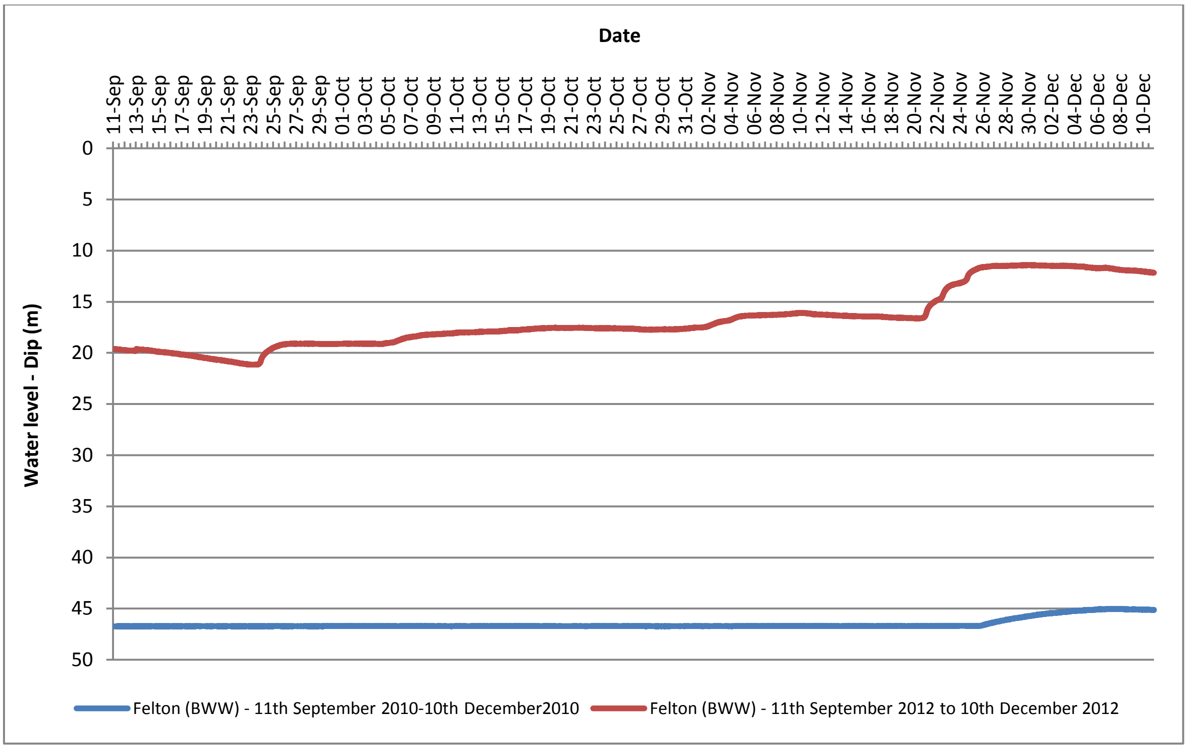

Figure 2-18 - Felton (BWW) borehole 2010 and 2012 data comparison

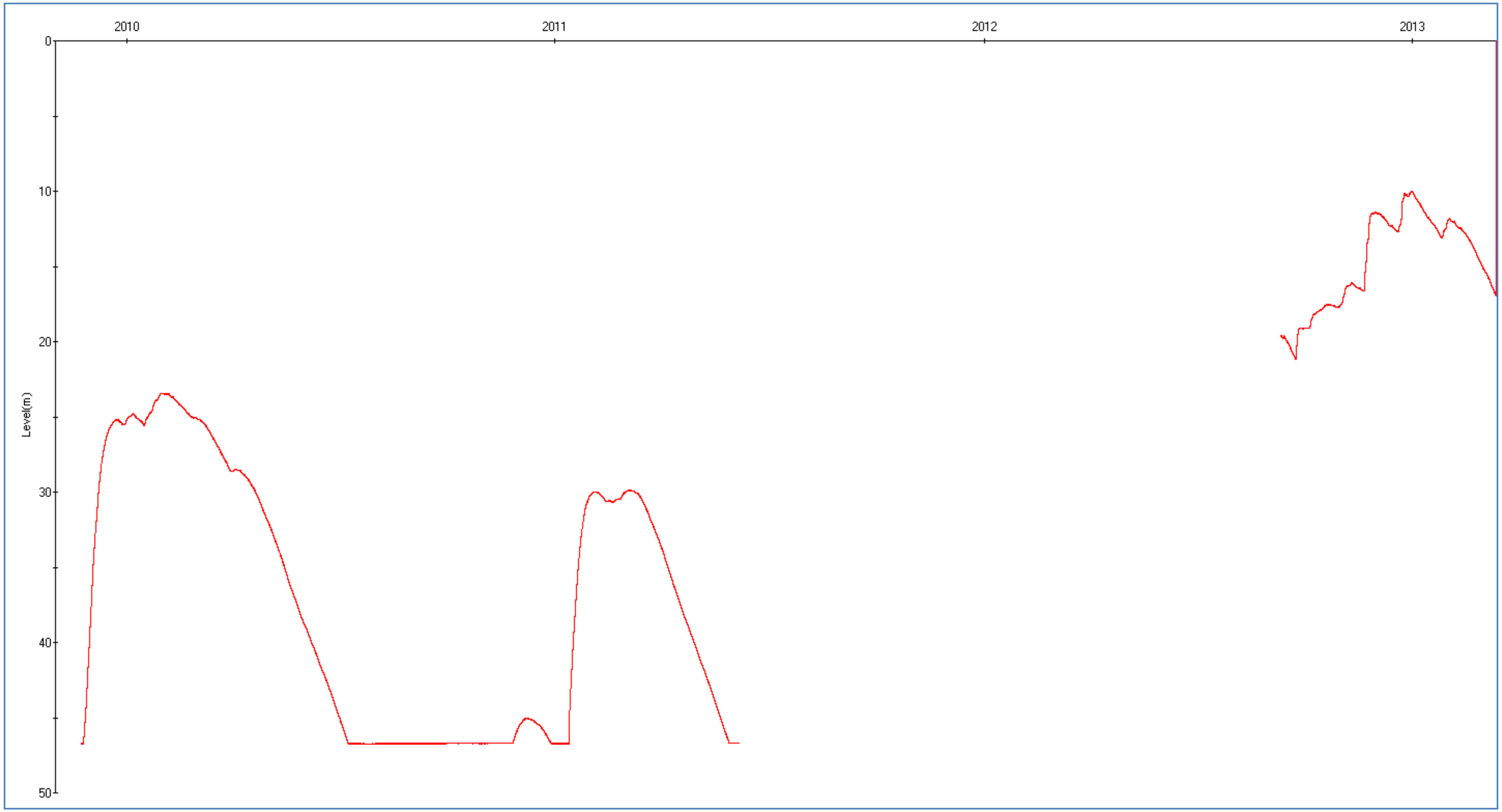

Figure 2-19 - Felton (BWW) borehole water level full period of record

3 Methodology Overview

3.1 Introduction

The River Chew catchment is complex, non-standard, and rapid response. Standard flow estimation methods may not perform well for this type of catchment and, in conjunction with the limitations of the available data, may produce flow estimates which are not representative of known flood events. The influence of additional flood sources, such as surface water and groundwater, may also play a role in the mismatch between modelled and observed flooding. A continuous simulation approach may perform better for this catchment as it does not use the design event method which has problems for complicated catchments. This approach was not taken forward by the Environment Agency at this stage and, instead, the suitability of standard FEH methods for application to the catchment was considered.

3.2 Methods

The FEH Statistical and ReFH methods are both appropriate for application to the River Chew catchment, as detailed in Section 1.8 of the Calculation Record. BFIHOST values upstream of Chew Valley Lake and for the Winford Brook exceed 0.65, indicating that the ReFH method may not perform well for these locations. However the values are not exceptionally high, as are found for chalk catchments, in the key areas of interest and the use of the ReFH method may be reasonable.

FARL values for the River Chew are <0.9 even at the downstream extent of the watercourse at Keynsham. The FEH Statistical method QMED (median flood) equation may not be suitable for use in reservoired catchments. If flood peak data is available downstream of the reservoir and close to the site of interest, QMED can be estimated directly and the effects of the reservoir accounted for implicitly. The Compton Dando gauge is approximately 12km downstream of Chew Valley Lake. Winford Brook, Chew Stoke Stream and a number of other relatively large tributaries enter the River Chew between the reservoir and the gauge. The key site of interest in terms of the impact of the River Chew (Chew Magna) is located about 3km downstream of the reservoir. The catchment area at Compton Dando is 129km^2 compared to 70km^2 just upstream of the Winford Brook confluence; the FARL values are 0.842 and 0.739, respectively. Therefore the flood peak data from Compton Dando may not provide a suitable representation of the catchment response at Chew Magna and hence the FEH Statistical method may not be the most appropriate approach. Instead the ReFH method may be used, along with a flood routing calculation, to determine the outflow from the reservoir.

It was decided to undertake a preliminary assessment using a small number of key locations for flow estimation to determine which methods generate the most representative estimates based on recent and historical flood events. The selected methods were taken forward and applied at other locations where inflows to the model are required as discussed in Section 7.

A number of analyses were undertaken at the preliminary stage to utilise the new hydrometric data and information available for the catchment gauges and to assess the influence of the two reservoirs on downstream flow. The FEH Statistical method was used to undertake analysis using the available flood peak data from the catchment at the key gauge locations. This utilised the new rating for the Chew Magna Reservoir Weir gauge and recent data for the Compton Dando gauge. An assessment of the most appropriate flow at Compton Dando for the July 1968 flood was also carried out. An alternative approach to deriving QMED and the flood frequency curve was applied to the Chew Magna Reservoir Weir gauge data. This approach is based on peaks over threshold (POT) data and makes best use of the numerous peak flow events recorded in 2012. The results of this assessment were compared to those from the conventional methods. Pooling group composition was investigated and the merits of using a 'permeable' pooling group assessed.

In order to determine the outflow hydrograph from Chew Valley Lake a routing calculation was performed using a rainfall runoff approach. This was undertaken using the ReFH method and a small hydraulic model to perform the routing calculation. The aim of this assessment was to provide the inflow to the top of the hydraulic model and to determine reservoir lag and the critical storm duration for the reservoir. A similar assessment was undertaken for Chew Magna Reservoir to assess the reservoir lag and critical storm duration. Analysis of the Compton Dando gauge data was undertaken to determine ReFH model parameters from observed data to improve the hydrograph shape and peak flow estimates. Lag analysis was carried out on the

Chew Magna Reservoir Weir and Chew Stoke Flood Warning gauge level data to improve the estimate of Tp(0) (instantaneous time to peak) and hence hydrograph shape.

All of these analyses are discussed in detail in Sections 4 and 6.

3.3 Initial flow estimation points

Six flow estimation points were selected as initial locations for flow estimation, as shown in Figure 3-1. These were used to route hydrographs through Chew Valley Lake and Chew Magna Reservoir, to determine the influence of the reservoirs on flooding downstream and to determine the most appropriate method of flow estimation to take forward and apply at other locations where inflows to the hydraulic model were required. Details of the initial flow estimation points are provided in Table 3-1 and the key catchment descriptors are given in Table 3-2.

Table 3-1 - Initial flow estimation points

Site code	Watercourse	Location	Grid reference	Area (km^2)
COMP	River Chew	Compton Dando gauge	364850, 164750	128.9
WINF_A	Winford Brook	Confluence with River Chew	358100, 163000	19.6
CMRW	Winford Brook	Chew Magna Reservoir Weir gauge	356800, 163250	17.3
CHST_A	Chew Stoke Stream	Confluence with River Chew	357100, 162100	10.4
CSFW	Chew Stoke Stream	Chew Stoke_FW gauge	355950, 161800	9.1
CVL	River Chew	Chew Valley Lake spillway	356950, 161550	57.9

Table 3-2 - Initial flow estimation point catchment descriptors

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	$URBEXT_{1990}$	$URBEXT_{2000}$
COMP	0.842	0.35	0.591	15.74	70.9	987	0.0094	0.0096
WINF_A	0.961	0.35	0.692	5.26	73.5	955	0.0153	0.0188
CMRW	0.956	0.35	0.702	4.13	70.6	962	0.0138	0.0170
CHST_A	0.997	0.35	0.553	3.75	71.1	941	0.0124	0.0102
CSFW	0.996	0.35	0.557	2.76	74.0	945	0.0078	0.0060
CVL	0.692	0.36	0.608	7.26	61.8	1069	0.0062	0.0080

A number of catchment descriptors - AREA, BFIHOST, FARL, URBEXT - were checked, as detailed in Section 2.3 and Section 7.2 of the Calculation Record. No changes were made to the catchment descriptors on the basis of these checks. $URBEXT_{1990}$ and $URBEXT_{2000}$ were updated to 2013 (when the initial analysis was undertaken), to account for any small changes in urbanisation within the catchment.

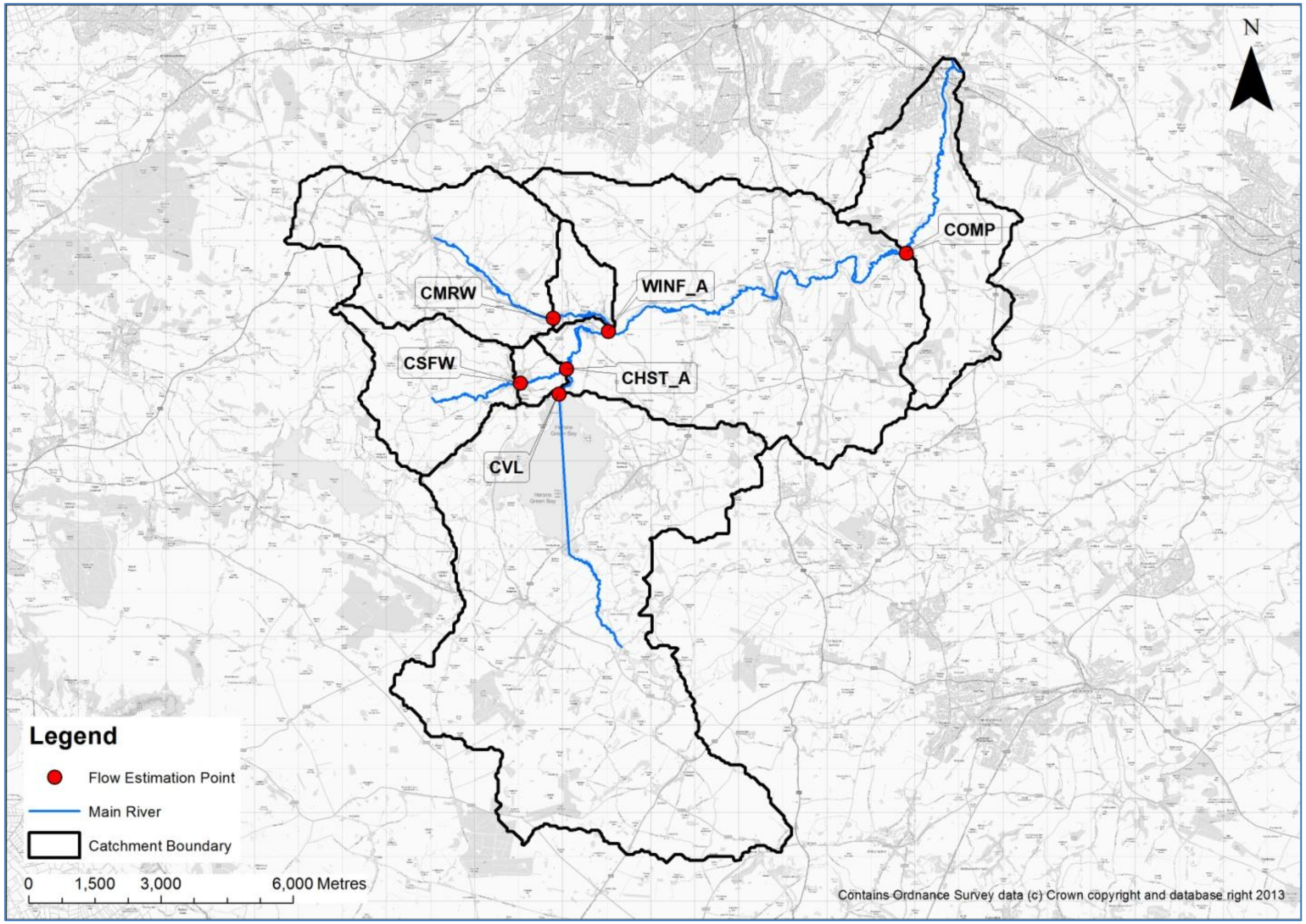

Figure 3-1 - Initial flow estimation points

4 ReFH Method & Analyses

4.1 Reservoir routing

4.1.1 Chew Valley Lake

The 2012 study report indicates that reservoir routing analysis was undertaken to derive the outflow hydrograph from the reservoir for the design events. Calibration and verification of the hydraulic model used observed flows rather than the output from the routing model. The report states that the reservoir is very effective in attenuating runoff from the upstream catchment, reducing the peak flow to a small fraction of that entering the lake, and delaying the peak for many hours after the peak flow from the tributaries and catchment downstream. The flow from Chew Valley Lake was therefore regarded as of low significance to the flow at Compton Dando, especially during the calibration and verification process where the largest events had a return period of ten years or less. An example provided in the report is that during the ten year return period event the flow at the confluence of the Chew Stoke Stream and the River Chew peaks at $13.5m^3/s$ approximately four hours after the start of the event. The contribution from Chew Valley Lake at this time is $0.08m^3/s$, less than 1% of the total flow, despite the reservoir water level being set to overflow crest level at the start of the event.

There are a number of areas of this analysis where it is not clear exactly what has been done. The main areas of uncertainty are:

- If only the inflow at the upstream extent of Chew Valley Lake was routed or if runoff from the contributing area between this point and the lake outlet (and rainfall onto the surface of the lake itself) was accounted for. The routing model, supplied by the Environment Agency for the current commission with the other model files, uses a ReFH boundary unit (REFHBDY) containing catchment descriptors which appear to correspond to the catchment upstream of the reservoir inlet. The report indicates that catchment descriptors were calculated for the area between the reservoir inlet and outlet. It is not clear how, or if, this was incorporated within the routing model.
- If a critical storm duration incorporating reservoir lag was determined for Chew Valley Lake. The REFHBDY applied within the routing model used a storm duration of 8.8hr; this would not be the critical storm duration for Chew Valley Lake.
- The report discusses the minimal influence of the reservoir on flood flows downstream for return period events of ten years or less but does not provide any information on the impact of the reservoir for larger magnitude events. Inflow and outflow hydrographs are provided for the 100 year return period event in Figure 3.4 of the report but no reference is made to this.
- The report states that observed flows were used for model calibration and verification rather than output from the Chew Valley Lake model. Bristol Water has developed a rating for flow over the spillway and it is assumed that this rating was used to generate the 'observed flows' used for the model calibration / verification. There is also a flume just upstream of the Chew Stoke Stream confluence which captures compensation flows and overflow from Chew Valley Lake. The rating for this structure is only accurate to approximately $2.2m^3/s$ and the stage data also appears suspect. Therefore it is thought unlikely that this gauge was used to provide the 'observed flows' for the calibration / verification events.

Due to the uncertainties regarding how the outflow hydrograph from Chew Valley Lake had been derived it was decided to undertake new analyses for the current study. The original routing model was refined for the purpose of this. A new REFHBDY was applied to supply the inflow hydrograph. This took the catchment descriptors for the catchment to the outlet of the reservoir in order to account for all of the catchment area draining to the reservoir. As the surface area of the reservoir is more than 5% of the total contributing catchment direct rainfall on to the reservoir was accounted for separately as there will be no losses from this input. This area was subtracted from the total catchment area and the reduced area was used in the REFHBDY (no other catchment descriptors were amended). The rainfall over the reservoir was applied using a Rainfall / Evaporation boundary unit (REBDY). The initial water level in the reservoir was set to the lowest crest level of the spillway, i.e. the reservoir was assumed to be full at the start of the storm.

The outflow hydrograph derived using the routing model only represents overflow from the reservoir. Compensation flow will be represented using an abstraction unit in the hydraulic model to remove water from the reservoir and feed it into the downstream channel. It was not believed to be necessary to include this for the purpose of the reservoir routing analysis.

An initial storm duration of 10.75hr was applied within the REFHBDY and to generate the rainfall hyetograph. This was based on the recommended storm duration for a location just downstream of the Winford Brook confluence as this approximates the key area of interest in terms of flood risk from the River Chew (Chew Magna). The areal reduction factor (ARF) for this location was also applied. The routing model was run to generate the outflow hydrograph and the lag time between the inflow and outflow hydrograph peak determined. This reservoir lag was incorporated within the storm duration equation (D=(Tp+RLAG)(1+SAAR/1000)) to generate the next storm duration to be tested; the ARF was updated each time the storm duration was changed. This iterative process continued until the lag time became constant; this was found to be approximately 19.5hr for a duration of approximately 50hr. As the ReFH model requires an odd number of integers when the duration is divided by the data interval, and the data interval was 1hr, the critical storm duration applied for Chew Valley Lake was 49hr.

Figure 4-1 shows a plot of the Chew Valley Lake inflow and outflow hydrographs from ISIS for the 100 year return period event using a storm duration of 49hr. The extent of attenuation and delay of the hydrograph peak at the outflow can clearly be seen. Note: this plot only shows the inflow hydrograph and not the hyetograph which is also contributing to the outflow hydrograph.

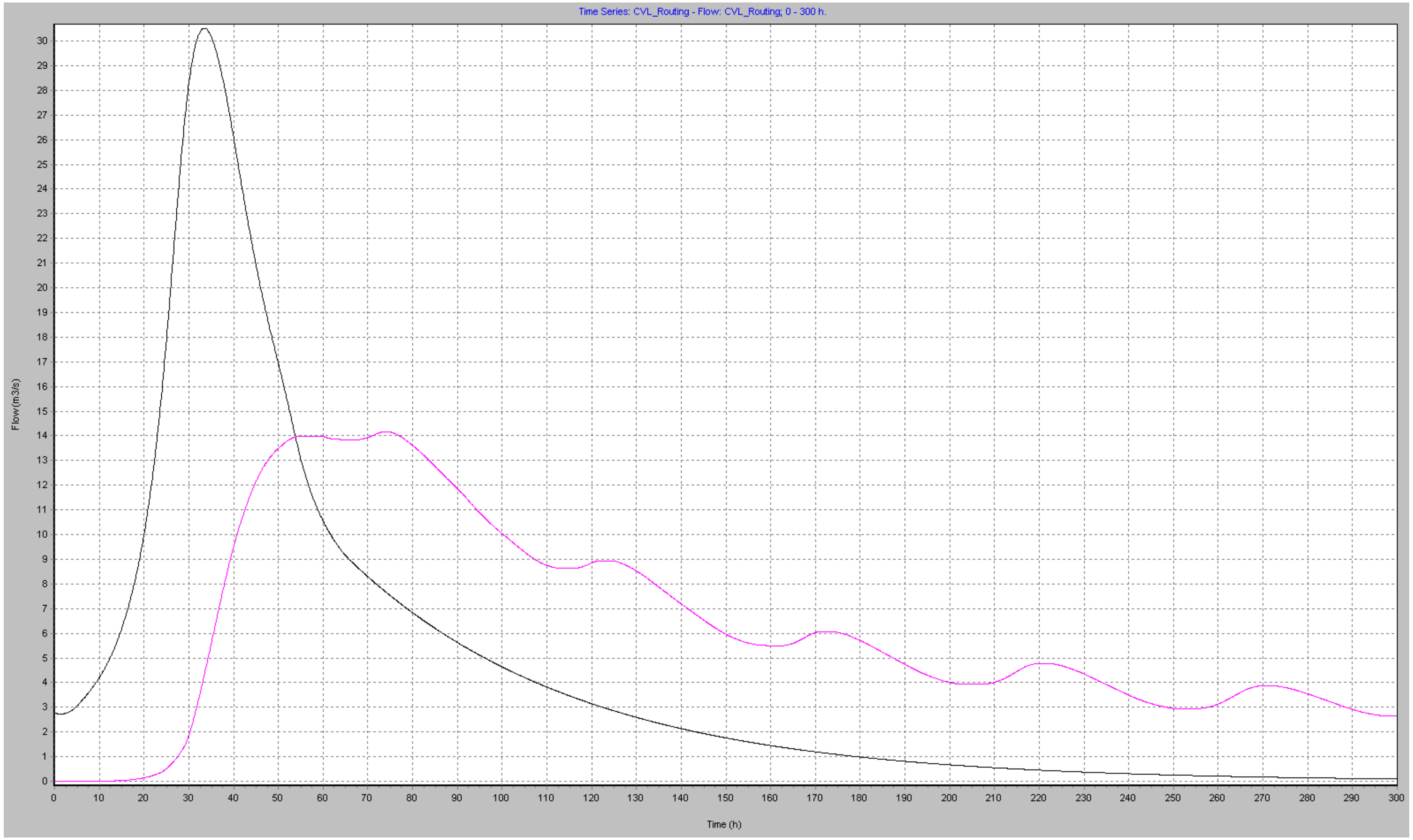

Figure 4-1 - Chew Valley Lake ISIS routing inflow (black) and outflow (pink) hydrographs (100 year return period)

4.1.2 Chew Magna Reservoir

The previous study report states that there were uncertainties in the information available for Chew Magna Reservoir including missing information on the regulation of outflows and uncertainties / inconsistencies in the gauged reservoir levels. For the purpose of model calibration the outflow record was used for the model downstream of Chew Magna Reservoir, making the assumption that the respective stage record and rating curve at the weir were correct. This was stated to have been done to avoid error accumulation due to insufficient information. The model used for the design event runs did not include the reservoir. This was stated to remove an unwarranted degree of implied accuracy and ensure a conservative estimate of the flood extent through Chew Magna, representing the worst case scenario assuming no attenuation. It was recommended in the report that future investigations could use a standalone model to represent the reservoir and incorporate data records from the newly installed gauges when they are of a reasonable length for the purpose of modelling and operation assessments.

The current study requires scenario testing of the reservoir and therefore the reservoir needed to be represented within the hydraulic model. It was also of interest to determine the reservoir lag and critical storm duration for Chew Magna Reservoir. A reservoir routing model was constructed for Chew Magna Reservoir. As the surface area of the reservoir is less than 5% of the contributing catchment direct rainfall did not need to be applied in this case. Catchment descriptors for the catchment to the reservoir outlet were applied within a REFHBDY to supply the inflow hydrograph. The dimensions of the reservoir (level-volume relationship) were obtained from information provided by the Environment Agency. Siltation of 35% was assumed from this information. The initial water level in the reservoir was set to the lowest level of the main spillway and the auxiliary spillway was also represented. A scour valve was used by Bristol Water in the past to discharge water from the reservoir into the stilling basin to manage reservoir levels. Bristol Water has indicated that the scour valve is no longer operated for this purpose as the storage provided is insignificant relative to a flood inflow. Therefore this was not represented within the routing model. Compensation flow will be represented by an abstraction unit when the routing model is incorporated into the main hydraulic model.

An initial storm duration of 7.25hr was applied within the REFHBDY. This was based on the recommended storm duration at the downstream extent of the Winford Brook catchment to represent the key area of flood risk. The ARF for this location was also applied. The routing model was run as described for Chew Valley Lake. At the second iteration the change in the reservoir lag time was less than the data interval being used to represent the inflow hydrograph (0.25hr). The storm duration, including reservoir lag, was 8.65hr. A duration of 8.75hr is the closest value to this which can be applied to give an odd integer when divided by the data interval. This was the duration applied for the second iteration therefore no further iterations were necessary. This indicated a lag time of 1.1hr and a critical storm duration of 8.75hr. Figure 4-2 shows a plot of the inflow hydrograph and the outflow hydrographs from ISIS for the main and auxiliary spillways for the 100 year return period event and a storm duration of 8.75hr. Due to the outflow hydrograph being split between the two spillways the influence of Chew Magna Reservoir on the outflow is not as clear as for Chew Valley Lake. However it can be deduced that there is limited delay and attenuation of the outflow hydrograph peak by the reservoir.

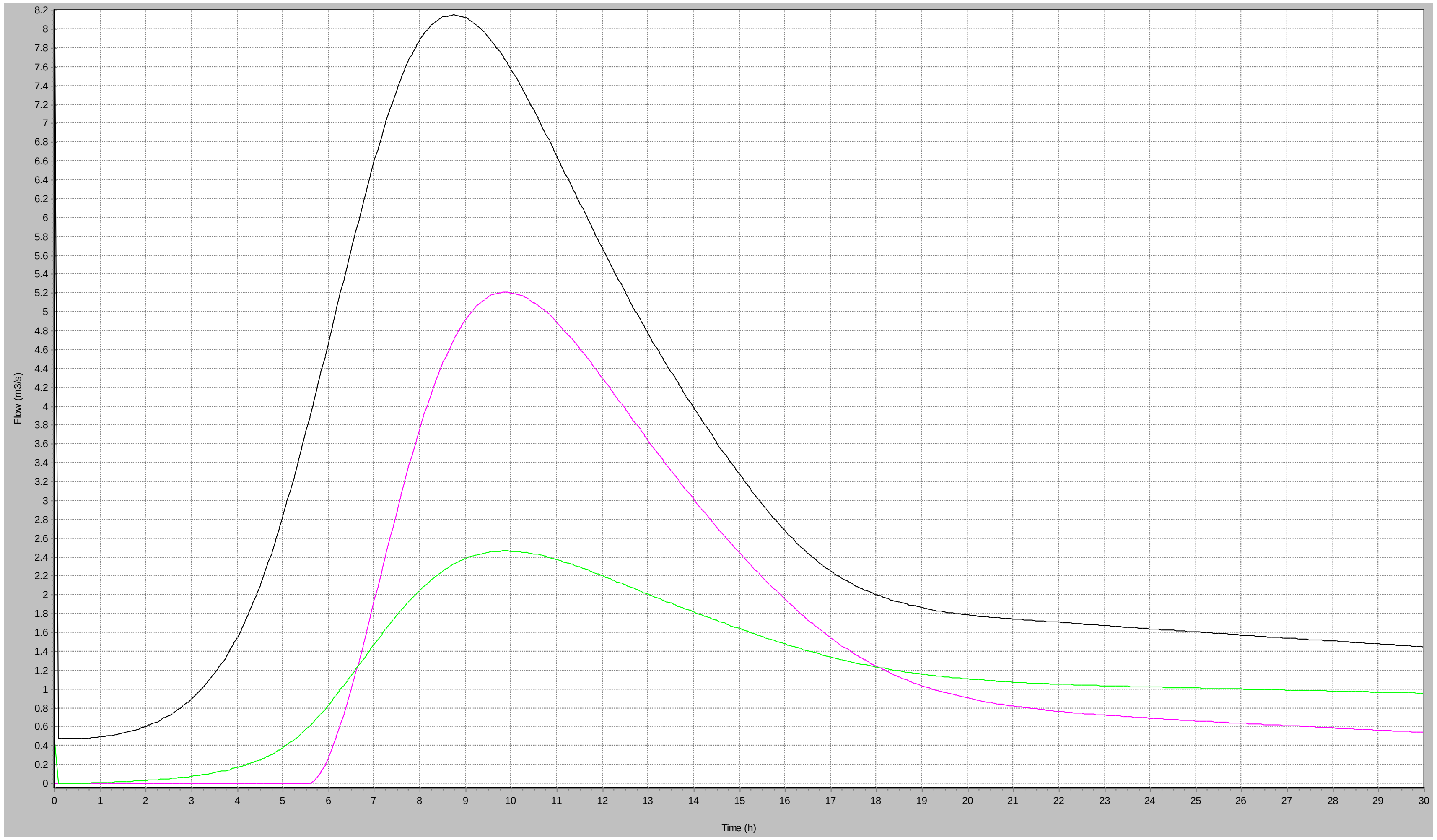

Figure 4-2 - Chew Magna Reservoir ISIS routing inflow (black) and outflow (primary spillway green and auxiliary spillway pink) hydrographs (100 year return period)

4.2 ReFH model parameter derivation

The available catchment gauge data was used to try to improve the ReFH method design peak flow estimates and hydrograph shape at the initial flow estimation locations. Analyses were undertaken for the Chew Magna Reservoir Weir, Chew Stoke Flood Warning and Compton Dando gauging stations, as detailed in the following sections.

4.2.1 Chew Magna Reservoir Weir gauge lag analysis

It is possible to derive the ReFH model parameters (Tp(0), C_{max}, BL and BR) using the ReFH modelling software where event flow and rainfall data are available. Although ratings have been provided to estimate flow from the Chew Magna Reservoir Weir gauge stage data and Chew Magna Reservoir gauge level data it is not possible to apply these directly within the HyDB. Therefore it would be time-consuming to generate the 15-minute event flow data for use in the ReFH modelling software. Additionally, due to the permeable nature of the catchment (BFIHOST 0.702), the ReFH software may not generate sensible results.

ReFH does not include the provision to use river stage or level data for deriving Tp(0) as the FEH Rainfall Runoff method does. However this data may guide the selection of parameters for the ReFH method. For example, it may be assumed that Tp(0) in ReFH can be adjusted using a factor derived from comparing the catchment descriptor estimate of the FEH Rainfall Runoff Tp(0) with that derived from lag analysis. This analysis was undertaken using the Chew Magna Reservoir Weir gauge stage data.

The analysis also requires event rainfall data. Thiessen polygons were constructed to determine proportions to apply the catchment and local TBR data in order to generate a catchment average rainfall dataset. This found that only the Barrow Gurney and Chew Magna PS / Chew Magna Spillway TBRs were applicable for the Chew Magna Reservoir Weir gauge catchment. The weightings applied are shown in Table 4-1.

Table 4-1 - Chew Magna Reservoir Weir gauge catchment TBR weightings

TBR	Weighting
Barrow Gurney	0.462
Chew Magna PS / Chew Magna Spillway	0.538

The gauge stage data was plotted in the HyDB and twelve peak stage events selected for analysis. This selection included the AMAX1 to AMAX5 events as detailed in Table 4-2.

Table 4-2 - Chew Magna Reservoir Weir gauge selected events

Date	Stage (m)	AMAX rank
13/12/2003	0.678	
12/01/2004	0.773	5
26/07/2007	0.713	
11/01/2008	1.016	3
13/12/2008	0.800	4
29/08/2012	0.826	
24/09/2012	1.196	1
04/11/2012	0.909	
21/11/2012	1.066	2
22/11/2012	1.052	
25/11/2012	1.036	
30/01/2013	0.787	

The maximum, minimum and geometric mean Tp(0) values were derived using all of the selected events and subsets of winter (October to March) and summer (April to September) events. These values are provided in Table 4-3. Three of the events occur in the summer months and nine in the winter months. The largest event occurs in the summer (24th September 2012). As the majority of peak stage events occur in the winter but summer events also appear to be key a decision was required as to whether or not to look at a seasonal aspect to the events. Generally the largest events are likely to occur in winter with the September 2012 event being unusual in its size for a summer event. It was decided that at this stage it was not necessary to consider summer and winter events separately and that the most appropriate option was to apply the result using all events.

Table 4-3 - Chew Magna Reservoir Weir gauge lag analysis Tp(0) results

Parameter	All events	Winter	Summer
MAX (hr)	5.17	4.60	5.17
MIN (hr)	0.37	0.37	2.02
GEOMEAN (hr)	2.39	2.17	3.16

The geometric mean Tp(0) value was compared to the FEH Rainfall Runoff catchment descriptor Tp(0) value and a factor derived. This factor was then applied to the ReFH catchment descriptor Tp(0) value to generate the adjusted ReFH Tp(0) estimate, as shown in Table 4-4. It can be seen that the Tp(0) value from observed data is nearly half that from catchment descriptors. This will result in the hydrograph peak being reached more quickly and the peak flow being larger than that estimated from catchment descriptors.

Table 4-4 - Chew Magna Reservoir Weir gauge Tp(0) calculation

Parameter	Tp(0) (hr)	Factor
$Tp(0)_{CD}$ (FEHRR)	4.43	
$Tp(0)_{Ob}$ (LAG)	2.39	
Ratio		0.54
$Tp(0)_{CD}$ (ReFH)	3.33	
$Tp(0)_{ADJ}$ (ReFH)	1.80	

4.2.2 Chew Stoke Flow Warning gauge lag analysis

The analysis described in Section 4.2.1 was also undertaken for the Chew Stoke Flood Warning gauge. Thiessen polygons showed that only the Barrow Gurney and Chew Magna PS / Chew Magna Spillway TBRs were applicable for the Chew Stoke Flood Warning gauge catchment. The weightings are shown in Table 4-5. As the weighting for the Barrow Gurney TBR was so small only the Chew Magna PS / Chew Magna Spillway data was used to generate the event rainfall data.

Table 4-5 - Chew Stoke Flood Warning gauge catchment TBR weightings

TBR	Weighting
Barrow Gurney	0.004
Chew Magna PS / Chew Magna Spillway	0.996

The gauge stage data was plotted in the HyDB and twelve peak stage events selected for analysis. This selection included the AMAX1 to AMAX6 events, as detailed in Table 4-6.

Table 4-6 - Chew Stoke Flood Warning gauge selected events

Date	Stage (m)	AMAX rank
06/11/2005	48.745	6
10/01/2007	49.513	4
11/01/2008	49.516	3
16/03/2008	48.778	
13/12/2008	48.998	5
29/08/2012	49.274	
24/09/2012	50.133	1
21/11/2012	49.941	2
22/11/2012	49.907	
25/11/2012	49.574	
29/12/2012	48.755	
30/01/2013	48.721	

The maximum, minimum and geometric mean Tp(0) values were derived as described for the Chew Magna Reservoir Weir gauge and the results are provided in Table 4-7. Two of the twelve events occur in the summer months and the largest event (24th September 2012) also occurs in the summer. As discussed for the Chew Magna Reservoir Weir gauge analysis the September 2012 is unusual in its size for a summer event and the largest events will generally occur in the winter. Therefore the result derived from using all of the selected events was used in the analysis.

Table 4-7 - Chew Stoke Flood Warning gauge lag analysis Tp(0) results

Parameter	All events	Winter	Summer
MAX (hr)	5.93	4.10	5.93
MIN (hr)	0.37	0.37	2.78
GEOMEAN (hr)	1.92	1.65	4.06

The calculations to derive the adjusted ReFH Tp(0) value are provided in Table 4-8. The Tp(0) value from observed data is nearly half that from catchment descriptors and gives a virtually identical factor to that generated for the Chew Magna Reservoir Weir gauge. This gives more confidence in the results for both gauges as the catchments would be expected to respond in a similar manner despite the Chew Stoke Flood Warning gauge catchment not containing any substantial online water bodies. Although Chew Magna Reservoir is just upstream of the Chew Magna Reservoir Weir gauge, due to its size it is not expected to have a significant attenuating influence on flood flows.

Table 4-8 - Chew Stoke Flood Warning gauge Tp(0) calculation

Parameter	Tp(0) (hr)	Factor
$Tp(0)_{CD}$ (FEHRR)	3.63	
$Tp(0)_{Ob}$ (LAG)	1.92	
Ratio		0.53
$Tp(0)_{CD}$ (ReFH)	2.64	
$Tp(0)_{ADJ}$ (ReFH)	1.39	

4.2.3 Compton Dando ReFH model parameter analysis

As event flow and rainfall data were available for the Compton Dando gauge a full analysis was undertaken to derive the model parameters using the ReFH modelling software. The data was plotted in the HyDB and 19 peak flow events between January 1999 and January 2013 were selected and analysed. This included several AMAX events, as detailed in Table 4-9. Only one of these events occurs in the summer months, 24th September 2012, therefore a seasonal aspect to the analysis was not considered.

Table 4-9 - Compton Dando gauge selected events

Date	Flow (m^3/s)	AMAX rank
16/01/1999	22.7	
30/10/2000	38.2	3
05/11/2000	23.0	
08/12/2000	28.1	
26/01/2002	24.3	19
14/11/2002	21.5	
30/12/2002	22.6	
01/01/2003	25.5	16
10/01/2007	26.5	15
11/01/2008	32.4	4
13/12/2008	32.1	5
24/09/2012	31.0	7
04/11/2012	27.0	
21/11/2012	37.8	
23/11/2012	38.6	2
25/11/2012	37.1	
22/12/2012	26.4	
24/12/2012	22.2	
30/01/2013	22.3	

Rainfall data was available for different combinations of TBRs for these events; Thiessen polygons were generated to provide gauge weightings for these combinations, as detailed in Table 4-10.

Table 4-10 - Compton Dando gauge catchment TBR weightings

TBR	Weighting	TBR	Weighting	TBR	Weighting	TBR	Weighting
Barrow Gurney	0.063	Barrow Gurney	0.060	Barrow Gurney	0.063	Barrow Gurney	0.788
Chew Magna Spillway	0.569	Chew Magna PS	0.864	Chew Magna Spillway	0.868		
Grove Farm	0.300						
Keynsham	0.068	Keynsham	0.075	Keynsham	0.068	Keynsham	0.212

Initial results from the analysis showed that modelled peak flows were generally lower than observed peak flows. The timing of the peaks and gradient of the rising and receding limbs were a reasonable match for most events. The modelled hydrographs for two events showed a particularly poor fit to the observed data and also had large event mean square error values - 20th January 1999 and 24th September 2012. The January 1999 event peak flow was substantially underestimated with the modelled peak occurring earlier than observed. The September 2012 event peak flow was substantially overestimated with the modelled hydrograph responding more quickly and significantly than the observed hydrograph; peak timing was a good match. The January 1999 event was removed from the analysis and the model re-run to determine if the model results could be improved. The September 2012 event was retained as this is a key event in the catchment.

The results from the second model run were similar to those from the initial run. The C_{max} value increased by approximately 2% compared to the initial run and the Tp value decreased by just over 1%. It was decided that a better match between modelled and observed hydrographs was unlikely to be obtained by undertaking further runs with more event hydrographs removed. The model parameter values derived from the second run were taken forward. These values are provided in Table 4-11, along with the catchment descriptor values and the derived donor correction factors. The parameter which will have the greatest impact on peak flows is Tp. It can be seen that the value derived from observed data is smaller than that from catchment descriptors; this is the same trend that was seen for the Chew Magna Reservoir Weir and Chew Stoke Flood Warning gauges. This will generate larger peak flows than from catchment descriptors. Although the BL and BR donor correction factors are large these parameters will have less of an impact on the results.

Table 4-11 - Compton Dando gauge ReFH model parameters

	Tp (hrs)	C_{max} (mm)	BL	BR
Catchment descriptors	7.53	466.02	60.07	1.46
Observed data	5.51	492.95	134.17	2.18
Donor correction factor	0.73	1.06	2.23	1.50

4.2.4 Data transfer

The results of the analyses described in Sections 4.2.1 to 4.2.3 were used to improve the ReFH method design peak flow estimates and the shape of the derived design hydrograph. The adjustments applied to the six initial flow estimation locations are detailed in Table 4-12. The model parameter values used in the ReFH analysis are provided in Section 4 of the Calculation Record. The model parameters for CVL were not adjusted. Although the location of this flow estimation point is at the reservoir spillway, the hydrograph will be used to provide an inflow to the reservoir to represent the catchment draining into it and will be routed through the reservoir. The Compton Dando gauge record will account for the influence of Chew Valley Lake and this will not be representative of the inflow hydrograph to the reservoir. It is also unlikely that the Compton Dando gauge will provide a reasonable representation of the hydrograph just downstream of the reservoir, for the purposes of estimating the lumped flow at this location, as the reservoir influence will be significantly less at Compton Dando.

Table 4-12 - Donor correction factors for data transfer

Site code	Donor gauge	Donor correction factors			
		Tp(0)	Cmax	BL	BR
COMP	Compton Dando	0.73	1.06	2.23	1.50
WINF_A	Chew Magna Reservoir Weir	0.54			
CMRW	Chew Magna Reservoir Weir	0.54			
CHST_A	Chew Stoke Flood Warning	0.53			
CSFW	Chew Stoke Flood Warning	0.53			
CVL	N/A				

4.3 Results

4.3.1 Design peak flow estimates

Peak flow estimates for the ReFH method were generated using the recommended storm duration (FEH equation) for each location. This was to enable the estimates to be compared with those derived using the Statistical approaches discussed in Section 5. These estimates were derived using the adjustments to the model parameters detailed in Section 4.2.4 and are provided in Table 4-13. The storm durations used and the catchment descriptor peak flow estimates are provided in the Calculation Record. Peak flow estimates have not been provided for CVL as these will be derived using the reservoir routing model. Although CMRW is downstream of Chew Magna Reservoir the influence of the reservoir is likely to be small and the peak flow estimates have been adjusted using data from the gauge at this location. Therefore the ReFH method estimates should provide a reasonable indication of flow. In the hydraulic model the hydrographs (and hence flow estimates) will be generated by routing the modelled upstream flow hydrograph through the reservoir.

Table 4-13 - ReFH method design peak flow estimates

Return period (yrs)	COMP (m^3/s)	CMRW (m^3/s)	WINF_A (m^3/s)	CSFW (m^3/s)	CHST_A (m^3/s)	CVL (m^3/s)
2	29.1	4.3	3.8	4.3	4.0	
5	36.7	5.7	5.0	5.7	5.4	
10	42.7	6.8	6.0	6.8	6.4	
20	48.5	8.0	7.1	7.9	7.5	
30	52.4	8.7	7.8	8.7	8.2	
50	57.9	9.8	8.8	9.7	9.3	
75	62.9	10.9	9.7	10.7	10.2	
100	66.8	11.7	10.5	11.4	10.9	
200	77.8	14.0	12.6	13.6	12.9	
1000	115	22.3	20.2	20.9	20.1	

4.3.2 Storm duration testing

Identification of the critical storm duration for Chew Valley Lake and Chew Magna Reservoir is discussed in Sections 4.1.1 and 4.1.2. Durations of 49hr and 8.75hr were determined, respectively, for the reservoirs. The hydraulic model will be run with all point and lateral inflow hydrographs set to these durations. The ARF for the catchment just downstream of the confluence of the River Chew and Winford Brook will be applied for the 49hr duration. For the 8.75hr duration the ARF for the Winford Brook catchment to the River Chew confluence will be applied. These ARF values have been selected based on the key flood risk area (Chew Magna) for these watercourses in the Upper Chew catchment.

The Chew Stoke Stream may be sensitive to a shorter storm duration. Therefore a model run will be undertaken using hydrographs with a 2.75hr duration and ARF set to the catchment to the Chew Stoke Flood Warning gauge. This is located in the key flood risk area (Chew Stoke) for the Chew Stoke Stream.

An exceptionally long storm may have implications for the storage capacity and attenuation within Chew Valley Lake. Although the reservoir routing analysis showed the 49hr duration to be critical it is thought prudent to undertake a model run using a longer storm event. A duration of 97hr was selected for this test and will use the same ARF as the 49hr storm.

5 Statistical Methods & Analyses

5.1 July 1968 event - Compton Dando

The project SoW requested a review of the July 1968 flood flow estimate at the Compton Dando gauging station. HiFlows-UK states that the July 1968 event was exceptional. The level at the gauging station was approximately 0.9m above the top of the flume and water was out of bank; the flume was partially filled with debris and stones. Therefore the peak flow assigned to this event is very uncertain. The value used in the HiFlows-UK AMAX dataset is $100m^3/s$; an earlier estimate assigned the event a peak flow of $226m^3/s$.

Correspondence from Peter Spencer indicates that $100m^3/s$ was a very approximate rounded value to supersede $226m^3/s$ which was thought to be too high. He suggested that $100m^3/s$ might be regarded as a minimum estimate based on extrapolating the hydrograph and with some reduction for sediment. It was suggested that rounded estimates (for example, $150m^3/s$) should be used due to the uncertainty and that flood frequency analyses could be undertaken using a range of likely flows for the event.

The 2012 study based the design peak flow estimates at Compton Dando on the FEH Statistical single-site method. The AMAX series updated using the new rating derived as part of the study (applied from 1979 onwards) and an AMAX value of $100m^3/s$ for the July 1968 event was used in this analysis. Further analysis was undertaken in April 2013 which used a value of $180m^3/s$ for the event, as discussed in Section 2.3.

A scan of the July 1968 event hydrograph was provided by the Environment Agency for the current study, as shown in Figure 5-1; this shows the truncation of the hydrograph peak. The ordinates of the hydrograph were approximated from the chart and entered into an Excel spreadsheet to generate a digitised version of the hydrograph. This was extended past the truncation by roughly sketching the top section of the hydrograph for a number of different peak flow values - $150m^3/s$, $175m^3/s$, $200m^3/s$, $250m^3/s$ and $300m^3/s$.

Rainfall data for the event was used to estimate the percentage runoff (PR) to identify which peak flow value might be the most realistic. Daily rainfall data is available for the event from the Barrow Gurneys Obs, Chew Stoke Pump St, and Long Ashton gauges; no TBR (hourly or 15-minute) data is available. Rainfall prior to that recorded for 10th July 1968 was minimal and was not included in the calculations. The gauge rainfall totals for the 10th were 140.7mm, 143.5mm and 129.5mm, respectively. An average of these totals was assumed for the calculations. Thiessen polygons were not generated to determine gauge proportions due to the inherent uncertainty in the overall assessment and the potential to imply an unrealistic level of confidence in the results by doing this.

The volume of rainfall across the catchment was determined. The Environment Agency report on the 1968 event, discussed in Section 2.2, indicates that Chew Valley Lake did not spill during this event. Only the compensation flow would have been entering the River Chew from the upstream catchment; this would be insignificant compared to the runoff contributing to the river downstream of the lake. Therefore only the rainfall across the catchment downstream of Chew Valley Lake (catchment area $71km^2$) was used in the rainfall volume calculation. This gave a total catchment average rainfall volume of just under 10 million cubic metres (10^6m^3). The flow volume for each hydrograph peak was roughly estimated. The flow volume divided by the rainfall volume gives an estimate of the PR for each peak flow estimate assessed. The results of this analysis are provided in Table 5-1. The peak flow of $74m^3/s$ relates to the peak value recorded for the scanned truncated hydrograph.

Table 5-1 - Percentage runoff for various peak flows

Peak flow (m^3/s)	PR (%)
74	46
150	65
175	73
200	80
250	93
300	99

The Environment Agency report suggests that when the event occurred the soils in the catchment were saturated following previous rainfall and could not absorb any more water. Sheet runoff is also stated to have occurred on many fields, with depths of 75-100mm. This indicates that the PR for the event is likely to have been very high. Due to the uncertainty in the values determined from this analysis it was believed to be unrealistic to take the highest peak flows as the assigned value for the event despite the suggestion of high saturation of the ground from the PR values. It is therefore recommended that a value of 175m^3/s is considered to be the most suitable estimate for the July 1968 event peak flow. This implies a high level of ground saturation whilst also taking into consideration the assumptions made and limitations of the analysis. The hydrograph generated for the 175m^3/s peak flow is shown in Figure 5-2.

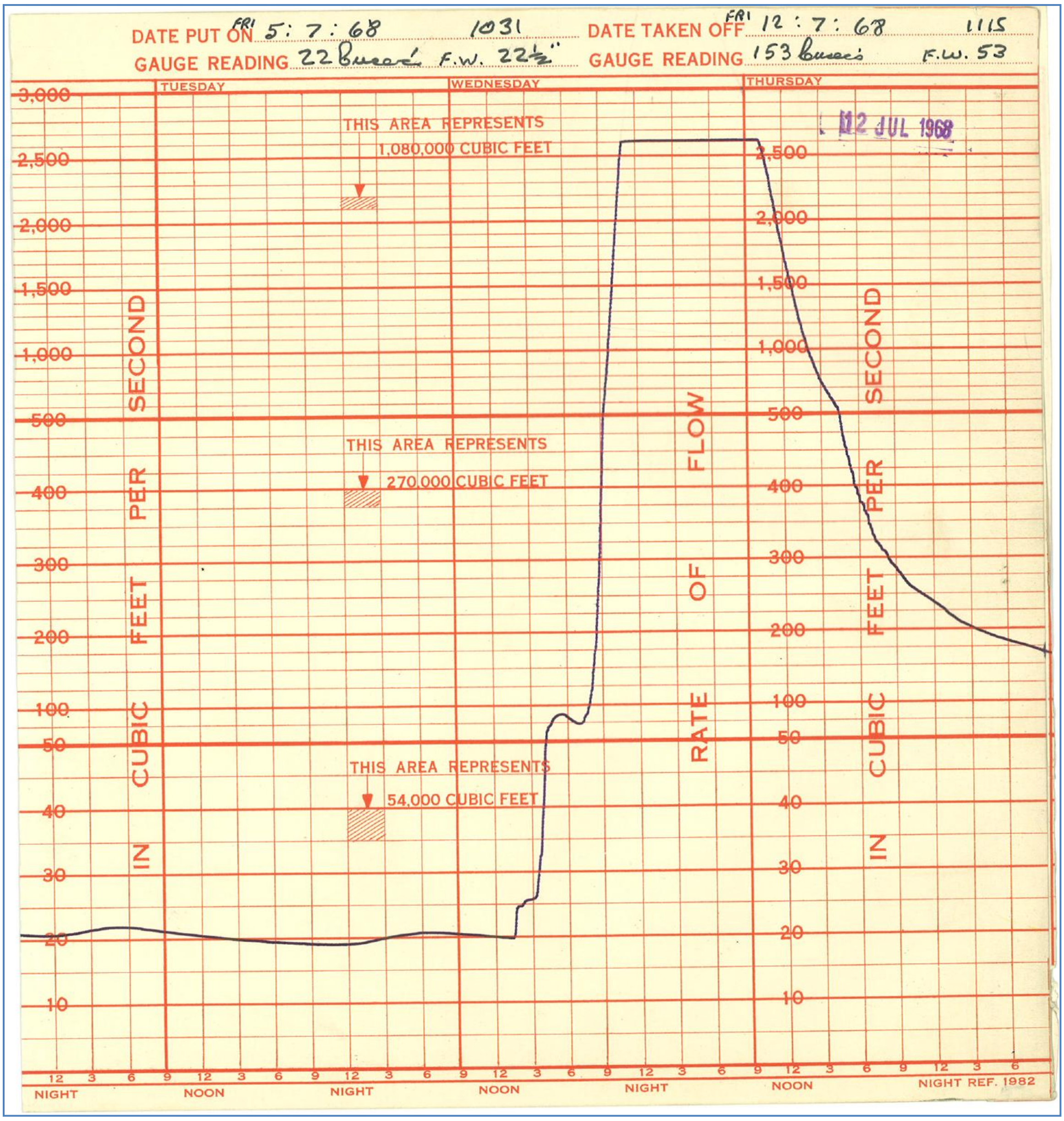

Figure 5-1 - July 1968 event hydrograph

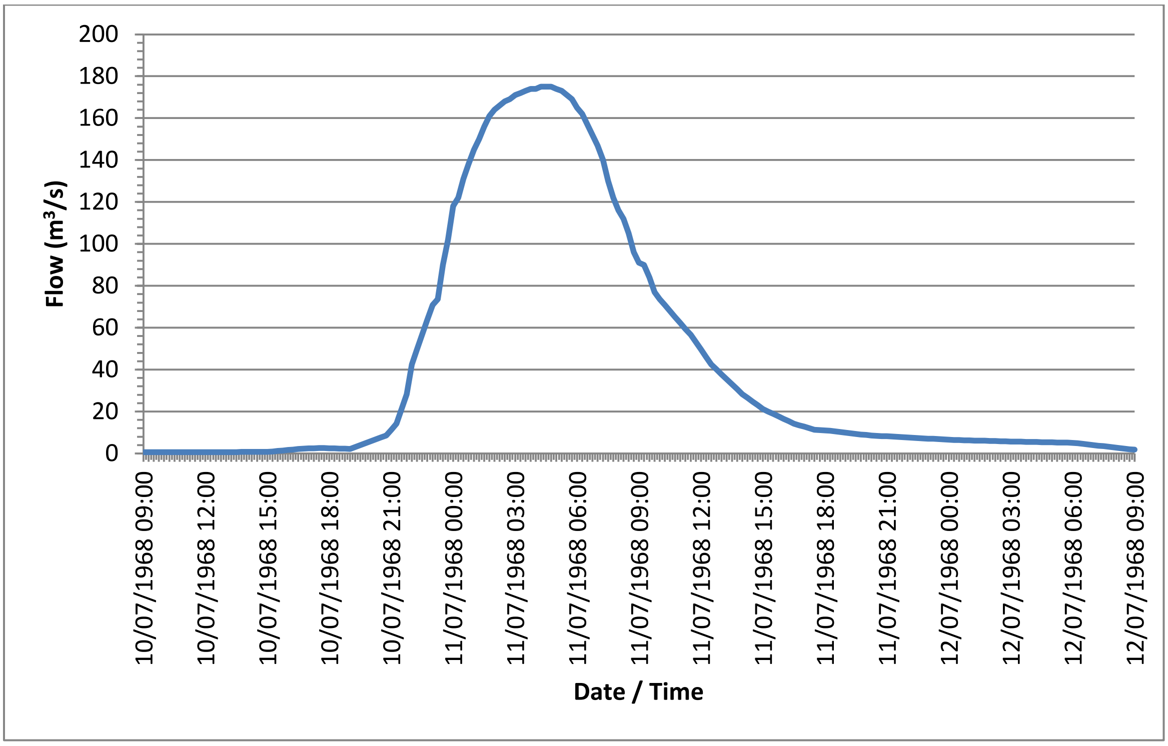

Figure 5-2 - July 1968 event hydrograph with 175m^3/s peak

5.2 Single-site analysis

Statistical method single-site analysis was undertaken for the two gauges for which flow data is available - Chew Magna Reservoir Weir and Compton Dando. Two single-site analysis methods were used for Chew Magna Reservoir Weir: (i) the Flood Studies Report (FSR) peaks-over-threshold (POT) method, and (ii) the FEH AMAX method. Only the FEH AMAX method was applied for the Compton Dando gauge. These analyses and the results are discussed in the following sections.

5.2.1 Chew Magna Reservoir Weir FSR POT analysis

One of the key issues found with the 2012 modelling study was the fact that the model outputs did not match the flood extents and flooded properties seen during the September and November 2012 floods. This was particularly significant for the Winford Brook through Chew Magna where flows derived for the 2012 study were believed to be too low.

The 2012 study estimated design peak flows for the Winford Brook using the FEH Rainfall Runoff method. These peaks were fitted to ReFH method hydrographs as previous analyses had generated a good match between the modelled ReFH hydrographs and observed event hydrographs. The FEH Statistical method was originally used to derive design peak flow estimates but it was subsequently decided that this method was underestimating flows.

As discussed in Section 2.4, a new rating is now available for the Chew Magna Reservoir Weir gauge. Although there is still uncertainty associated with this rating for larger flows, it should be an improvement on the existing rating for the gauge, used by the Environment Agency. The number of peak flow events which occurred in 2012, and the relatively short record, suggests that POT analysis may be preferable to AMAX analysis for the Chew Magna Reservoir Weir gauge. Rather than using the method described in the FEH, which only estimates QMED, it was decided to apply the methodology detailed in the FSR, which estimates the full flood frequency curve.

The FSR method provides equations for estimating the mean annual flood (QBAR) and the flow for any return period $Q_{(T)}$, where T is the return period in years. The method requires a threshold to be set such that, on average, three to five peaks per year exceed this threshold. There are eight complete water years of record for the Chew Magna Reservoir Weir gauge therefore between 24 and 40 events are required for the analysis. The peaks analysed need to be independent of each other, using a rule for independence that peaks should be separated in time by three times the time-to-peak (Tp) and that flow should decrease between peaks to two-thirds of the first peak. The HyDB was used to apply and identify all peaks above the selected threshold and to determine the independence of events using the stage record for the Chew Magna Reservoir Weir gauge.

An initial threshold was selected by assessing the AMAX data for the gauge; a value of about $3m^3/s$ appeared to be reasonable. The rating table showed that $3.03m^3/s$ is equivalent to a stage of 0.69m and this stage value was used as the threshold in the HyDB. This generated 21 peaks using the full period of record and a QBAR estimate of $7.6m^3/s$. As this threshold did not generate sufficient peaks to give, on average, three to five peaks per year, the threshold was lowered to 0.59m, equivalent to about $2m^3/s$. This value generated 31 independent peaks using the full period of record and a QBAR estimate of $6.8m^3/s$. The equation provided in the FSR[6] (Pages 9-10) was used to produce the flood frequency curve for the range of design events required by the study, as shown in Table 5-2. The growth factor for the 100 year return period event is 3.4; this is within the guideline typical range of 2.1 to 4.0.

Table 5-2 - Chew Magna Reservoir Weir gauge single-site (FSR POT) design peak flow estimates

Return period (yrs)	Flow (m^3/s)
2	7.2
5	10.9
10	13.6
20	16.4
30	18.0
50	20.0
75	21.7
100	22.8
200	25.6
1000	32.0

If the return period is derived from the AMAX series (as is widely used by the FEH) then QMED is equivalent to the two year return period (50% AEP) event and QBAR is equivalent to the 2.33 year return period (43% AEP) event, hence QBAR will be larger than QMED. In this case the QBAR estimate is less than the 50% AEP event estimate. This may be due to the different definitions of return period used in AMAX and POT analysis[7]. An additional reason may be because the gauge data does not follow the Gumbel distribution on which the model is based.

5.2.2 Chew Magna Weir FEH AMAX analysis

The single-site flood frequency curve for the Chew Magna Reservoir Weir gauge was also estimated using the AMAX series and the procedures outlined in the FEH. The AMAX series derived using the new rating was applied within WINFAP-FEH and single-site analysis undertaken on this data to generate the flood frequency curves. The Generalised Logistic (GL distribution was selected to fit the data. A permeable adjustment was applied as the SPRHOST value is on the threshold of the FEH Statistical method permeable catchment definition (SPRHOST <20%). The unadjusted and adjusted flood frequency curves are shown in Table 5-3; the difference between the curves is small for the lower return periods but increases for the higher return periods. It must be remembered that the flow estimates derived from the rating for the largest events will have a high degree of uncertainty associated with them. Additionally confidence in the results of the single-site analysis from AMAX data for this gauge will be low for design peak flow estimates larger than the ten year return period event due to the short gauge record.

The 100 year return period event growth factor is 10.3 for the unadjusted curve and 9.0 for the adjusted curve. These growth factors are exceptionally high even taking into consideration the steep nature of the catchment and the potential for the small amount of attenuation provided by Chew Magna Reservoir to have some impact on the smaller magnitude events.

[6] Institute of Hydrology. March 1978. Report No. 49. Methods of Flood Estimation: A Guide to the Flood Studies Report.

[7] The AMAX return period is defined as "the average interval between years containing one or more floods exceeding a flow, Q". The POT return period is defined as "the average interval between floods of that magnitude or greater". The two types of return period are related using Langbein's formula. The difference is only important as short return periods under 20 years.

Table 5-3 - Chew Magna Reservoir Weir gauge single-site (FEH AMAX) design peak flow estimates

Return period (yrs)	Unadjusted	Permeable adjustment
2	3.9	3.9
5	8.2	7.9
10	12.4	11.7
20	17.9	16.6
30	21.9	20.1
50	28.2	25.4
75	34.4	30.6
100	39.5	34.8
200	55.0	47.3
1000	117	95.6

5.2.3 Compton Dando single-site analysis

The single-site flood frequency curve for the Compton Dando gauge was estimated using the updated AMAX series, as detailed in Section 2.4.2. The AMAX series was applied within WINFAP-FEH and the flood frequency curves generated using the GL, GEV and P3 distributions. These design event estimates were generated using values of 100m^3/s and 175m^3/s for water year 1967. The flood frequency curves for 1967=100m^3/s and 1967=175m^3/s are shown in Figure 5-3 and Figure 5-4, respectively, for comparison.

None of the distributions appear to fit particularly well to the data for the larger AMAX events due to the fact that the largest event (1967) is significantly larger than all others in the dataset. The GL and GEV distributions give similar results to each other up to about the 200 year return period event. The P3 distribution has the worst fit and suggests a smaller return period (between five and ten years but closer to the former) for the November 2012 flood than the other two distributions. GL suggests just over a ten year return period for the event and GEV just under the ten year return period. Even the larger magnitude AEP estimates are thought to be low given the flooding that is known to have occurred during this event therefore the P3 distribution was discounted. It should be noted that the data shows generally small differences between the values of the largest AMAX events (with the exception of 1967). AMAX2 and AMAX3 are just above the highest gaugings which verify the rating and just below the bankfull level / flow. As a result there may be some uncertainty in these AMAX values and this could have an influence on the single-site flood frequency curve. More gaugings are needed to verify the high flow section of the rating.

The 100 year return period event growth factor is much larger when using the 175m^3/s value for water year 1967. The growth factor falls within the typical range (2.1-4.0) for all distributions using 100m^3/s but is outside the range for all distributions using 175m^3/s. For example, the GL distribution gives a value of 3.55 when using a value of 100m^3/s and 5.05 when using a value of 175m^3/s. The July 1968 flood was an extreme event which caused extensive flooding and significant damage across the River Chew catchment and the larger growth factor for the 175m^3/s scenario could be representative of the catchment response. However there is significant uncertainty in all of the design flow estimates above the 50 year return period event due to the length of the gauge record.

There is only a small difference between the results using the GL and GEV distributions and GL is the recommended distribution for UK catchments. The results using the 175m^3/s value for water year 1967 and the GL distribution were selected as the most appropriate from the single-site analysis, as shown in Table 5-4.

Table 5-4 - Compton Dando gauge single-site peak flow estimates

Return period (yrs)	1967 water year 175m^3/s GL - LMED
2	18.9
5	28.9
10	38.2
20	50.2
30	58.9
50	72.2
75	84.9
100	95.4
200	127
1000	248

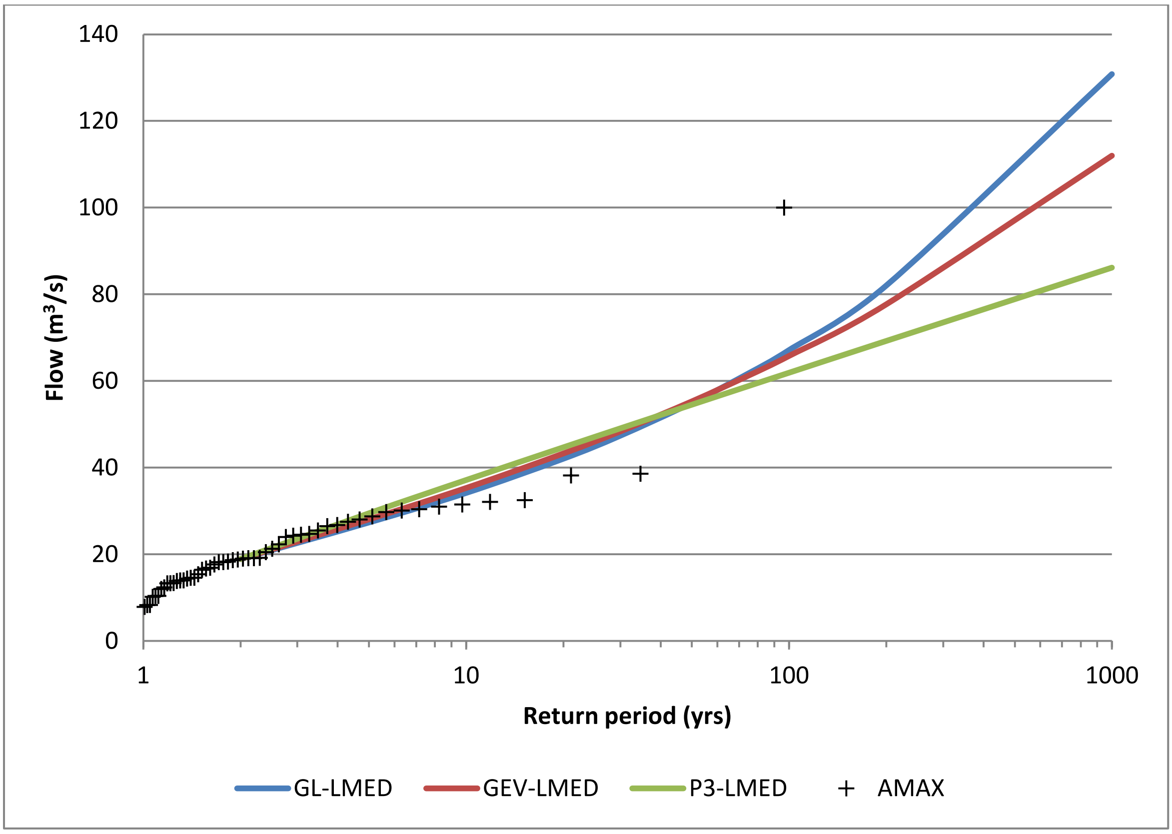

Figure 5-3 - Compton Dando gauge single-site flood frequency curves (1967=100m^3/s)

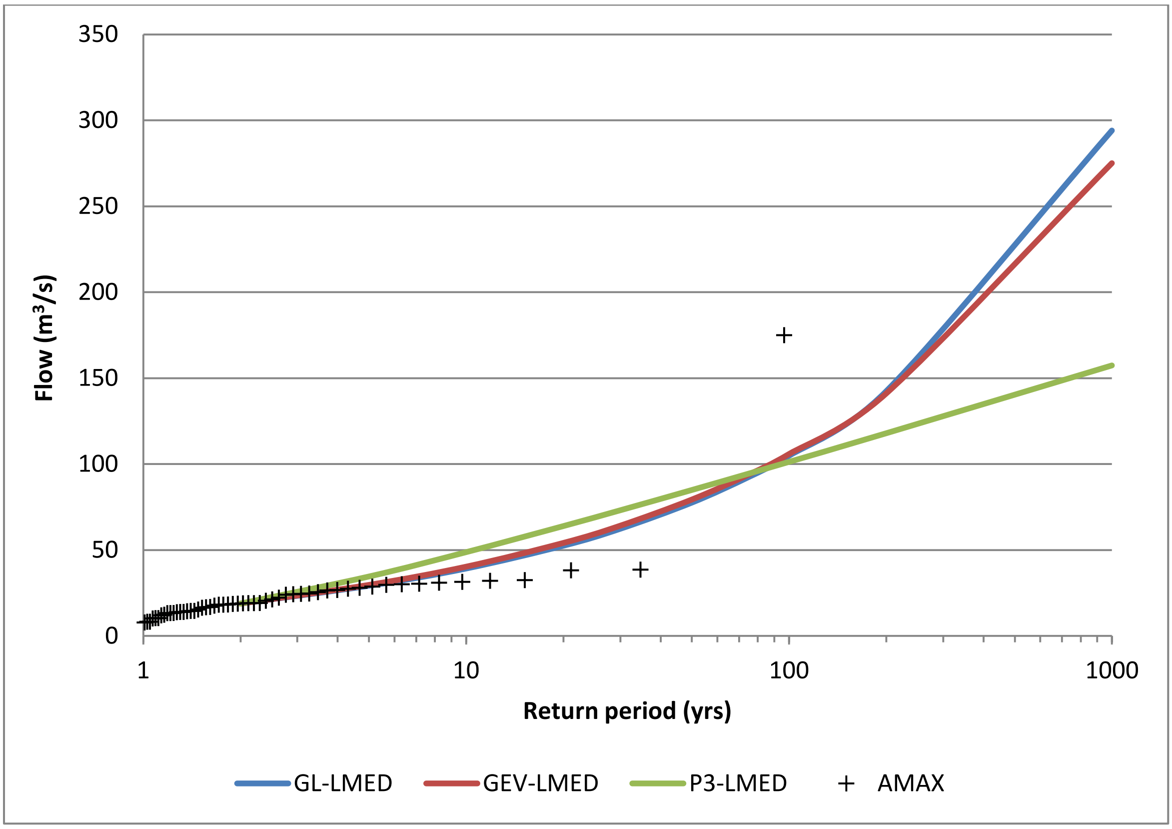

Figure 5-4 - Compton Dando gauge single-site flood frequency curves (1967=175m^3/s)

5.3 Data transfer for QMED

5.3.1 COMP

The Compton Dando gauge AMAX data was used to provide the QMED estimate for the COMP flow estimation point. This is located at the gauge and there is confidence in the data for estimating QMED. The factor between the AMAX QMED estimate and that from catchment descriptors is 1.288 indicating that the catchment descriptor equation is underestimating QMED.

5.3.2 CMRW and WINF_A

The Chew Magna Reservoir Weir gauge would seem to be the obvious choice to provide the QMED estimate for CMRW, where the gauge is located, and for data transfer to WINF_A. Three QMED estimates were derived from the Chew Magna Reservoir gauge data using various Statistical approaches. These are detailed in Table 5-5 along with the catchment descriptor estimate for the site (including urban adjustment).

Table 5-5 - Chew Magna Reservoir Weir gauge QMED estimates (m^3/s)

Catchment descriptors	FSR POT	FEH AMAX	FEH POT
2.6	7.2	3.9	6.3

All of the QMED estimates using the gauge data are larger than that from catchment descriptors, suggesting that the catchment descriptor equation is underestimating QMED. The QMED estimates from POT data are substantially larger than that from AMAX data. Due to the relatively short record length (about ten years) the POT estimates should be preferred over the AMAX estimate. The POT analysis takes into account the significant number of peak flow events which occurred in 2012 and should be more representative than the AMAX data.

The FEH POT analysis employs a simplified method of estimating QMED whilst the FSR POT method utilises more information from the data in the estimation of QBAR. Hence it may be considered that there is more confidence in the FSR POT analysis results. QMED derived using the equation to generate the FSR POT flood frequency curve is larger than QBAR, as discussed in Section 5.2.1.

There is some uncertainty in the gauge data even for the smaller events therefore consideration was given to identifying an alternative donor from a local gauged catchment. Gauging stations within approximately a 50km radius were investigated which had similar catchment descriptors to the study catchments. Twelve stations were identified as potential donors and this was reduced to five following a more detailed review of station and catchment information. Details of these gauges are provided in Table 5-6.

Table 5-6 - Potential CMRW and WINF_A donor stations

Station number	Station name	Distance between gauge and study catchment centroids (km)	Gauge adjustment factor ($QMED_{Obs}/QMED_{CD}$)
51001	Doniford at Swill Bridge	52	0.994
51003	Washford at Beggearn Huish	61	0.538
52014	Tone at Greenham	62	0.894
52015	Land Yeo at Wraxall	5	1.118
53023	Sherston Avon at Fosseway	34	1.023

Stations 51001 and 53023 were rejected as there was no adjustment to the study catchment QMED value following application of the distance factor. Therefore, in the case of these two stations, there would be no benefit from selecting a donor.

Station 51003 was rejected as the relationship between the observed and catchment descriptor QMED estimates is different to that found for the Chew Magna Reservoir Weir gauge. For this station the observed value is approximately 50% less than the catchment descriptor value. The distance factor reduces the adjustment to the study catchment QMED estimate considerably (0.919). However it was decided that it was not appropriate to use this station as a donor as it is unlikely to be representative of the study catchment and would reduce the QMED estimate when observed data for the catchment suggests QMED should increase.

The adjustment factor from Station 52014 is 0.985 when the distance factor is applied. This is the opposite relationship to that found for the Chew Magna Reservoir Weir gauge and also makes minimal difference to the study catchment QMED estimate.

Station 52015 was believed to be the best candidate for data transfer to the study catchments. The relationship between the observed and catchment descriptor estimates is the same as that for the Chew Magna Reservoir Weir gauge, although the difference between the two estimates is not as large. When the distance factor is applied the adjustment to the QMED estimate is 1.052. This does not result in a substantial increase in the QMED estimate. Design flows for the Winford Brook were believed to be too low for the previous study. Although the QMEDflow is not tabulated in the 2012 report it is believed to be in the region of $4.5m^3/s$. Use of Station 52015 for data transfer will result in a QMED estimate which is approximately $2m^3/s$ lower than this value.

At this stage there is most confidence in the QMED estimates from the Chew Magna Reservoir Weir gauge POT analyses (FSR and FEH). The FSR POT QMED estimate for the Chew Magna Reservoir Weir gauge was area-weighted to obtain an estimate for WINF_A. The FEH data transfer procedure was applied to obtain the QMED estimate for WINF_A from the FEH POT estimate for Chew Magna Reservoir Weir.

5.3.3 CSFW and CHST_A

Although a modelled rating was derived for the Chew Stoke Flood Warning gauge in the 2012 study, a rating equation was not developed and a flow series could not easily be derived from the stage data. The modelled rating could potentially have been used to determine the AMAX or POT series however confidence in the flow data will be limited and therefore this analysis was not undertaken. Therefore local gauged catchments were considered for data transfer as described in Section 5.3.2. Seven stations were identified as potential donors; this was reduced to two following a more detailed review of station and catchment information. Details of these gauges are provided in Table 5-7.

Table 5-7 - Potential CSFW and CHST_A donor stations

Station number	Station name	Distance between gauge and study catchment centroids (km)	Gauge adjustment factor ($QMED_{Obs}/QMED_{CD}$)
51003	Washford at Beggearn Huish	59	0.538
53017	Boyd at Bitton	22	1.275

Station 51003 was rejected for the same reasons given in Section 5.3.2. The results of the Chew Stoke Flood Warning gauge rating do not appear to have been applied to the Chew Stoke Stream FEH Statistical method peak flow estimates in the 2012 study to determine a QMED estimate from POT data. However the derived flow data was used to adjust the FEH Rainfall Runoff parameters. This appears to have increased the FEH Rainfall Runoff 100 year return period event peak flow by a factor of more than three compared to the catchment descriptor estimate. This suggests that catchment descriptors are likely to underestimate flow. As this was also found for the Chew Magna Reservoir Weir gauge it was decided that Station 51003 was unlikely to be representative of the Chew Stoke Stream catchment.

Using Station 53017 as a donor leads to an increase in the study catchment QMED of about 7.5%. This trend is believed to be representative based on the Chew Magna Reservoir Weir gauge results and those for the Chew Stoke Flood Warning gauge from the 2012 study. This station was therefore used to adjust the QMED estimates for CSFW and CHST_A.

Consideration was given to using the Chew Magna Reservoir Weir gauge as a donor. The FARL and BFIHOST catchment descriptor values are somewhat different between the two catchments and it was decided at this stage that Station 53017 was the most appropriate selection. However, it is worth noting that if the Chew Magna Reservoir Weir gauge was used as a donor, using the QMED estimate from the FSR POT analysis, then the adjustment factor for the Chew Stoke Stream locations would be about 1.8. This would lead to more consistency between the design peak flow estimates for the tributaries which might be anticipated despite the differences in FARL and BFIHOST. The decision regarding the most appropriate estimates for this, and the other watercourses, will be revisited following initial model runs. If the model results indicate less flooding than would be anticipated from flood history and local knowledge then consideration will be given to using the Chew Magna Reservoir Weir gauge as a donor for Chew Stoke Stream.

5.3.4 CVL

It is believed that the most appropriate method to derive the outflow hydrograph from Chew Valley Lake is by routing a ReFH hydrograph through the reservoir using a routing model. However, consideration was given to generating design peak flow estimates at the spillway using the FEH Statistical method as a comparison to the ReFH routing estimates.

There is no suitable donor for CVL due to the significant influence of the reservoir. For this comparative assessment the Compton Dando gauge was used to adjust the QMED estimate. The factor applied to the CVL catchment descriptor QMED estimate is 1.136 once the distance factor has been accounted for.

5.4 Pooled analysis

Pooled growth curves were derived for all flow estimation points. Although the Compton Dando gauge record is long it does not provide sufficient data for estimating design peak flows for return period events larger than 50 years, according to FEH recommendations.

Pooling groups were generated for each of the flow estimation points and assessed to determine if generic groups could be used to represent more than one location:

- Winford Brook - The CMRW and WINF_A pooling groups were very similar and one pooling group was selected to represent both locations. The WINF_A pooling group was chosen as this is closest to the flood risk area of Chew Magna.
- Chew Stoke Stream - The CSFW and CHST_A pooling groups were not similar enough to the Winford Brook flow estimation point groups to utilise one group to represent all four locations. However the two Chew Stoke Stream pooling groups were similar and one group was chosen to represent both locations. CFSW was selected as this is within the flood risk area of Chew Stoke.

- River Chew - The COMP and CVL pooling groups were very different and the individual pooling groups were used for the two locations. It was not possible to undertake an enhanced single-site analysis for COMP as the Compton Dando gauge is not suitable for pooling.

5.4.1 Pooling group review

The selected pooling groups were reviewed to determine if any stations should be removed. The groups were reviewed with regard to similarity in catchment descriptors and characteristics to the study catchment, similarity to the rest of the stations in the pooling group, and data quality. Due to the relatively small size of the Winford Brook and Chew Stoke Stream catchments it is unlikely that entirely representative catchments will be found for inclusion within the pooling groups. Additionally, the selection of stations for inclusion in the pooling groups no longer uses a soil term (BFIHOST or SPRHOST) and it is common to find permeable catchments included in pooling groups for impermeable catchments and vice versa. Therefore the review was focused on stations at the top of the pooling groups, which will have the most influence on the growth curve, and the data quality and characteristics of the catchment (for example, land use and land use change) compared to the study catchment. Significant research has gone into identifying the most important catchment descriptors for selection of stations to include in the pooling groups hence catchment descriptors were only considered briefly in the review. Further details and final pooling groups are provided in the Calculation Record.

COMP

- Many of the stations in the pooling group are located in Scotland and the catchment characteristics (for example, geology, altitude and land use) are different to the study catchment.
- Several stations have AMAX series which appear to show a change in trend in the later record but it is not clear why this is; there are no comments in HiFlows-UK to suggest reasons for this.
- Due to the reservoir influence from Chew Valley Lake none of the catchments in the pooling group are likely to be particularly representative of the River Chew even if they also have a reservoir influence.
- All stations were retained with the exception of Station 39035 Churn at Cerney Wick. This station was discordant and had a very shallow growth curve as a result of truncated high flows which are probably due to bypassing of the gauge.
- No additional stations were added as the total of 484 years of data was deemed to be acceptable.

WINF_A

- A number of stations in the pooling group are small chalk catchments. The study catchment is permeable in the upper reaches but this is due to limestone geology.
- Many of the stations have impermeable geology. The pooling group is unlikely to be particularly representative due to the relatively small catchment size, permeable upper section and the influence of Chew Magna Reservoir.
- Stations 44809 Piddle at Little Puddle, 50009 Lew at Norley Bridge and 73015 Keer at High Keer Weir were removed from the pooling group.
- Station 72014 Conder at Galgate was added to bring the years of data to 490. This was deemed to be acceptable rather than adding further stations with a larger similarity distance measure.

CSFW

- Similar to WINF_A in terms of the catchment types included. Chalk catchments are even less representative for this catchment as it does not have a permeable component.
- Stations 32029 Flore at Experimental Catchment and 5009 Lew at Norley Bridge were removed from the pooling group.
- Station 49003 De Lank at De Lank was added to bring the years of data to 521.

CVL

- Similar to COMP in terms of the catchment types included. Most catchments are located in Scotland or northern England. No pooling group is likely to provide a representative growth curve for this location.
- No stations were removed.

5.4.2 'Permeable' pooling group

The Winford Brook catchment has permeable geology in its upper catchment which leads to SPRHOST values of just over 20% at the CMRW and WINF_A flow estimation points. Due to the absence of a soil parameter in the selection of pooling groups by WINFAP-FEH the permeability of the study catchment may not be adequately represented in the results. An investigation was undertaken to determine what impact on design peak flows the use of a 'permeable' pooling group might have.

The 'permeable' pooling group was generated by manual editing of the WINFAP_FEH default pooling group. Only stations with BFIHOST >0.650 were included within the pooling group; all but two of these stations also had SPRHOST <20%. The stations were selected on the basis of the similarity distance measure. Permeable stations with the smallest similarity distance measure were selected first with the measure value increasing as more permeable stations were sought to provide approximately 500 years of data. Fifteen stations were selected, providing 494 years of data. The stations at the bottom of the pooling group had similarity distance measure values of about 2.0 compared to those at the bottom of the default pooling group which had values of just over 1.0. The derived pooling group was not reviewed. If stations were removed from the 'permeable' pooling group others with higher similarity distance measures would have to be added or a smaller number of stations years accepted. In this case, it was deemed appropriate to retain all stations classified by WINFAP-FEH as suitable for pooling without further review.

The 100 year return period event growth factor for the reviewed default pooling group for WINF_A was 3.00; the growth factor for the 'permeable' pooling group was 2.61. A permeable adjustment was applied to the 'permeable' pooling group, as described in the FEH, which produced a 100 year return period growth factor of 3.19. This results in the 'permeable' pooling group generating higher design peak flow estimates than the reviewed default pooling group, although the difference is not substantial.

5.5 Results

A number of analyses were undertaken for the key flow estimation locations to generate Statistical method design peak flow estimates, as detailed in the preceding sections. Most consideration needs to be given to COMP (Compton Dando gauge) and CMRW (Chew Magna Reservoir Weir gauge) where a number of approaches were applied to generate design peak flow estimates.

5.5.1 COMP

Due to the long record of peak flow data at this location the best estimate of QMED is likely to be derived from the AMAX series rather than from catchment descriptors. Therefore the key decision to be made regarding the most appropriate design peak flow estimates is in terms of the derivation of the flood frequency curve.

Single-site analysis to derive the flood frequency curve considered two values for water year 1967 ($100m^3/s$ and $175m^3/s$) and three distributions to fit the data (GL, GEV and P3). The $100m^3/s$ estimate for the July 1968 flood is believed by the Environment Agency to be too low. The estimate of the flood return period is not substantially different for the results using the $100m^3/s$ and the $175m^3/s$ value. The P3 distribution appears to give the worst fit to the data and suggests a smaller return period for the November 2012 flood than the other two distributions. GL suggests just over the ten year return period, GEV just under the ten year return period and P3 between the five and ten year return period but slightly closer to the former. Even the larger return period estimates are thought to be low given the flooding that is known to have occurred during this event therefore the P3 distribution was discounted. There is only a small difference between the results using the GL and GEV distributions and GL is the recommended distribution for UK catchments. Therefore the results using the $175m^3/s$ value for water year 1967 and the GL distribution were selected as the most appropriate from the single-site analysis.

Design peak flow estimates were also generated using a pooled analysis. These results were based on QMED derived from the gauge AMAX data, consistent with the single-site analysis.

The results of the single-site and pooled analysis are provided in Table 5-8. This shows that a flow of 175m^3/s would have a return period of 200-1000 years using the single-site results and significantly greater than the 1000 year return period using the pooled results. Given the known impacts of the July 1968 flood the estimate from the single-site results would appear reasonable. The pooled analysis estimate appears to be excessive. The November 2012 flood, during which properties in Chew Magna were flooded by the River Chew, has an estimated flow at Compton Dando of 38.6m^3/s. This would give a return period of just over ten years using the single-site results and just over 20 years using the pooled results. Both of these estimates seem a little low. AMAX3 is 38.2m^3/s (October 2000), the ten year return period value from the single-site analysis, with a period of 12 years between the two events (October 2000 and November 2012). Therefore the single-site results would appear to be sensible and there should be confidence in the results for design events of this magnitude given the length of the record.

Table 5-8 - COMP single-site and pooled analysis results

Return period (yrs)	Single-site (GL & WY1967 175m^3/s) (m^3/s)	Pooled (GL) (m^3/s)
2	18.9	18.9
5	28.9	26.3
10	38.2	31.9
20	50.2	37.9
30	58.9	41.7
50	72.2	47.1
75	84.9	51.7
100	95.4	55.2
200	127	64.7
1000	248	92.8

It was concluded that for the COMP flow estimation point the single-site analysis results (using 175m^3/s for water year 1967) gave the most appropriate design peak flow estimates.

5.5.2 CMRW (and WINF_A)

As detailed in Section 5.3.2, confidence in the QMED estimate from the POT analysis (FSR and FEH) was highest due to the inclusion of more peak events from a short period of record than the AMAX QMED estimate. The FEH POT analysis employs a simplified method to estimate QMED whereas the FSR method utilises more information from the data in the estimation of QBAR. Therefore greater confidence may be placed in the FSR POT estimate of QMED. However, consideration must be given to the independence of some of the peak events in the POT record. For example, although the three peaks in November 2012 (21st, 22nd and 25th) meet the FSR independence criteria it could possibly be argued that the rainfall events themselves are not independent.

Four approaches were considered to generate the flood frequency curve: the FSR POT approach, FEH AMAX single-site analysis, FEH pooled analysis and FEH pooled analysis using a 'permeable' pooling group. The GL distribution was used for the pooled analysis and the FEH AMAX single-site analysis. The 100 year return period event growth factor for the pooled analyses and the FSR POT analyses is reasonably similar - 3.00 for pooled, 3.19 for pooled 'permeable' and 3.15 for FSR POT. However the growth factor for the FEH AMAX single-site analysis is 10.26 without permeable adjustment and 9.0 with permeable adjustment. This is significantly higher than the typical range of 2.1-4.0.

The results from each of these approaches are provided in Table 5-9. Only the pooled 'permeable' results are provided as this approach was believed to generate better results than the pooled growth curve without permeable adjustment. The pooled (permeable) results using both the FEH AMAX and FEH POT QMED estimates are detailed given the potential uncertainty in QMED derived from both approaches (and from the FSR POT approach).

Table 5-9 - CMRW results from various Statistical approaches

Return period (yrs)	Single-site		Pooled ('permeable')	
	FSR POT (m^3/s)	FEH AMAX - permeable adjustment (m^3/s)	$QMED_{AMAX}$ (m^3/s)	$QMED_{POT}$ (m^3/s)
2	7.2	3.9	3.9	6.3
5	10.9	7.9	5.4	8.9
10	13.6	11.7	6.6	10.9
20	16.4	16.6	8.0	13.2
30	18.0	20.1	9.0	14.7
50	20.0	25.4	10.3	16.9
75	21.7	30.6	11.4	18.7
100	22.8	34.8	12.3	20.2
200	25.6	47.3	14.7	24.2
1000	32.0	95.6	22.5	37.0

The September 2012 flood has an estimated flow of 22.2m^3/s using the new rating for the Chew Magna Reservoir Weir gauge. It should be remembered that the rating review report indicated that this could be an underestimate due to flow bypassing the spillway and the gauge. The FSR POT results suggest that this has a return period of 75-100 years. The FEH AMAX single-site results indicate that it has a return period of 30-50 years. The pooled analysis using the AMAX QMED estimate suggests a return period of just under 1000 years and the pooled analysis using the POT QMED estimate suggests a return period of 100-200 years.

Therefore the pooled analyses would appear to be generating design flow estimates which are too low, if the gauge flow estimate is to be believed, and the single-site analyses generating design flow estimates which are more sensible. The November 2012 flood peak flow was 17.3m^3/s which would be a return period of 20-30 years using the FSR POT results and just over the twenty year return period using the FEH AMAX results. Although the FEH AMAX single-site results suggest a more frequent occurrence for these floods, the FSR POT results are also plausible and there is more confidence in the method applied. It is recommended that the FSR POT results are selected in preference to the FEH AMAX single-site results at this stage.

Design peak flow estimates for WINF_A were generated using the preferred estimates for CMRW. These were derived by applying an area-weighting to the CMRW estimates to produce a proportionally larger estimate for WINF_A. This is believed to be a reasonable approach given the similarity in catchment descriptors at the two locations.

5.5.3 CSFW (and CHST_A)

The only decision to be made regarding the Statistical approach for CSFW and CHST_A was whether to use a catchment descriptor estimate of QMED or one based on data transfer from a donor. The QMED estimate was adjusted using Station 53017 Boyd at Bitton, as detailed in Section 5.3.3. The catchment descriptor and adjusted QMED estimates results are provided for both locations in Table 5-10.

Table 5-10 - CSFW and CHST_A catchment descriptor and adjusted QMED design peak flow estimates

Return period (yrs)	$QMED_{CD}$ (m^3/s)	$QMED_{Adj}$ (m^3/s)
2	3.3	3.5
5	4.7	5.0
10	5.7	6.2
20	7.0	7.5
30	7.8	8.4
50	9.0	9.6
75	10.0	10.7
100	10.8	11.6
200	13.0	14.0
1000	20.0	21.4

The adjusted QMED results are slightly larger than the catchment descriptor estimates. The donor station is believed to be reasonable for data transfer to this catchment therefore the adjusted results are recommended.

5.5.4 CVL

As stated in Section 5.3.4, it is believed that the most appropriate method to derive the outflow hydrograph from Chew Valley Lake is by routing a ReFH hydrograph through the reservoir using a routing model. Design peak flow estimates from the Statistical method were generated using a catchment descriptor estimate of QMED and an estimate adjusted using the Compton Dando gauge as a donor. The results of both assessments are provided in Table 5-11. As the adjusted QMED estimate uses data from the study watercourse this may provide the results in which there is most confidence. However it is likely that neither approach generates representative results due to the substantial influence of Chew Valley Lake.

Table 5-11 - CVL catchment descriptor and adjusted QMED design peak flow estimates

Return period (yrs)	$QMED_{CD}$ (m^3/s)	$QMED_{Adj}$ (m^3/s)
2	4.2	4.7
5	5.8	6.6
10	7.0	7.9
20	8.3	9.5
30	9.2	10.5
50	10.4	11.8
75	11.5	13.1
100	12.3	14.0
200	14.5	16.5
1000	21.3	24.2

6 Comparison of Results

Sections 4 and 5 describe the analyses undertaken using the ReFH method and various Statistical method approaches. Recommendations were provided regarding the most appropriate design peak flow estimates for each of these methods. This section compares the selected ReFH and Statistical method results and provides recommendations for the preferred estimates at this stage of the study. These will be sensibility-checked using the hydraulic model and by comparing the model results to historical flood events. If necessary, the hydrological assessment will be revisited to determine if an alternative suite of design flow estimates are more appropriate.

6.1 Compton Dando gauging station (COMP)

ReFH method design peak flow estimates were derived using model parameters adjusted using data from the Compton Dando gauge and a storm duration of 10.75hr. Statistical method design peak flow estimates were derived using the FEH single-site approach, a value of 175m^3/s for water year 1967 and the GL distribution. The two sets of results are detailed in Table 6-1.

Although the ReFH method has larger flows for the smaller events than the Statistical method, this reverses after the ten year return period event. This is because the Statistical method growth curve is much steeper than that of the ReFH method. The July 1968 flood would take a return period greater than 1000 years on the basis of the ReFH method results. Therefore the FEH Statistical method single-site analysis results are the preferred estimates.

Table 6-1 - COMP ReFH and Statistical method results

Return period (yrs)	$ReFH_{Adj}$ (m^3/s)	FEH Statistical single-site (m^3/s)
2	29.1	18.9
5	36.7	28.9
10	42.7	38.2
20	48.5	50.2
30	52.4	58.9
50	57.9	72.2
75	62.9	84.9
100	66.8	95.4
200	77.8	127
1000	115	248

6.2 Chew Magna Reservoir Weir gauging station (CMRW) & WINF_A

The Chew Magna Reservoir Weir gauge data was used to adjust the ReFH method Tp(0) and design peak flow estimates were based on a storm duration of 3.75hr for CMRW. The FSR POT single-site approach was used to generate the flood frequency curve for the Statistical method. A threshold of 0.59m (1.99m^3/s) was selected for the analysis. Table 6-2 details the two sets of results.

The ReFH method has substantially smaller flows than the Statistical method for all design events. The September 2012 flood would take a return period of greater than 1000 years using the ReFH results and the November 2012 flood would have a return period of 200-1000 years. This is not believed to be sensible but it must be borne in mind that there is uncertainty in the gauge data. Therefore the FSR POT single-site analysis results are the preferred estimates.

WINF_A ReFH method design peak flow estimates were generated using the Tp(0) donor correction factor from the Chew Magna Reservoir Weir gauge and a storm duration of 4.25hr. The Statistical method design peak flow estimates were derived by applying an area-weighting to the CMRW results (1.14). For consistency, the area-weighted FSR POT single-site analysis results were taken forward at this stage as the preferred estimates. The results are provided in Table 6-2.

Table 6-2 - CMRW and WINF_A ReFH and Statistical method results

Return period (yrs)	CMRW		WINF_A	
	$ReFH_{Adj}$ (m^3/s)	FSR Statistical POT single-site (m^3/s)	$ReFH_{Adj}$ (m^3/s)	Area-weighted CMRW FSR Statistical POT single-site (m^3/s)
2	3.8	7.2	4.3	8.2
5	5.0	10.9	5.7	12.4
10	6.0	13.6	6.8	15.5
20	7.1	16.4	8.0	18.6
30	7.8	18.0	8.7	20.5
50	8.8	20.0	9.8	22.8
75	9.7	21.7	10.9	24.6
100	10.5	22.8	11.7	25.9
200	12.6	25.6	14.0	29.0
1000	20.2	32.0	22.3	36.3

6.3 Chew Stoke Flood Warning gauging station (CSFW) & CHST_A

The Chew Stoke Flood Warning gauge data was used to adjust the ReFH method Tp(0) and design peak flow estimates were based on a storm duration of 2.75hr for CSFW. The FEH pooled approach was used to generate the flood frequency curve for the Statistical method. QMED was adjusted using data transfer from Station 53017 Boyd at Bitton. The results from the two sets of analyses are provided in Table 6-3.

The ReFH method design peak flow estimates are larger than the FEH Statistical pooled method estimates for all events. The difference between the results from the two methods becomes smaller for the larger magnitude events as the ReFH method growth curve is less steep than the Statistical method growth curve. There is no gauge flow data against which the design estimates can be verified. The ReFH method uses catchment-specific data whereas the Statistical method relies on the use of data from a local donor catchment. As this stage of the study it is recommended that the ReFH method results are the preferred estimates.

The CHST_A ReFH method design peak flow estimates were generated using the Tp(0) donor correction factor from the Chew Stoke Flood Warning gauge and a storm duration of 3.25hr. The FEH Statistical method pooled analysis used the same donor as CSFW. Table 6-3 shows that for CHST_A the ReFH method results are only higher than the Statistical method results up to the 75 year return period event. After this the trend is reversed, although there is only a small difference between the estimates. The ReFH method results are recommended as the preferred estimates for CHST_A as catchment-specific data was used to improve the estimates.

Table 6-3 - CSFW and CHST_A ReFH and Statistical method results

Return period (yrs)	CSFW		CHST_A	
	$ReFH_{Adj}$ (m^3/s)	FEH Statistical pooled (m^3/s)	$ReFH_{Adj}$ (m^3/s)	FEH Statistical pooled (m^3/s)
2	4.0	3.1	4.3	3.5
5	5.4	4.4	5.7	5.0
10	6.4	5.4	6.8	6.2
20	7.5	6.6	7.9	7.5
30	8.2	7.4	8.7	8.4
50	9.3	8.5	9.7	9.6
75	10.2	9.4	10.7	10.7
100	10.9	10.2	11.4	11.6
200	12.9	12.3	13.6	14.0
1000	20.1	18.8	20.9	21.4

6.4 CVL

The most appropriate method to derive the outflow hydrograph from Chew Valley Lake is by routing a ReFH hydrograph through the reservoir using a routing model. As there are no river gauges in the catchment draining to the reservoir it is not possible to adjust the ReFH model parameters. The hydraulic model will be used to ensure that the preferred peak flow estimates for the River Chew at Compton Dando are approximated and that the model is adequately representing recent flood event extents (September and November 2012). Long duration storm events will be run through the routing model to test the capacity of the reservoir for attenuating large volumes of runoff.

6.5 Preliminary choice of method summary

At this stage the recommended preferred peak flow estimates for the key flow estimation locations are:

- COMP - FEH Statistical single-site analysis using a value of 175m^3/s for water year 1967 and the GL distribution to fit the data.
- CMRW - FSR POT single-site Statistical analysis using a threshold of 0.59m (1.99m^3/s) to derive the POT series.
- WINF_A - Area-weighted CMRW FSR POT single-site Statistical analysis.
- CSFW - ReFH method with Tp(0) adjusted using Chew Stoke Flood Warning gauge data.
- CHST_A - ReFH method with Tp(0) adjusted using Chew Stoke Flood Warning gauge donor correction factor.
- CVL - ReFH hydrograph routed through the reservoir using a routing model.

A summary of the preferred design peak flow estimates is provided in Table 6-4.

Table 6-4 - Preferred design peak flow estimates for key flow estimation locations

Return period (yrs)	COMP (m^3/s)	WINF_A (m^3/s)	CMRW (m^3/s)	CHST_A (m^3/s)	CSFW (m^3/s)	CVL (m^3/s)
2	18.9	8.2	7.2	4.3	4.0	
5	28.9	12.4	10.9	5.7	5.4	
10	38.2	15.5	13.6	6.8	6.4	
20	50.2	18.6	16.4	7.9	7.5	
30	58.9	20.5	18.0	8.7	8.2	
50	72.2	22.8	20.0	9.7	9.3	
75	84.9	24.6	21.7	10.7	10.2	
100	95.4	25.9	22.8	11.4	10.9	
200	127	29.0	25.6	13.6	12.9	
1000	248	36.3	32.0	20.9	20.1	

6.6 Comparison with previous study

There are three locations where the preferred design peak flow estimates from the current study can be compared with those from the 2012 study, as detailed in Table 6-5. The results from the two studies are detailed in Table 6-6. Values in brackets for the River Chew at Compton Dando are the updated values from the Mott MacDonald 2013 study, as detailed in Section 2.3.

Table 6-5 - 2012 and 2014 study comparable flow estimation locations

Study	River Chew location reference	Winford Brook location reference	Chew Stoke Stream location reference
November 2012 - Mott MacDonald	Compton Dando	Winford Brook	Chew Stoke
January 2014 - JBA Consulting	COMP	WINF_A	CSFW

Table 6-6 - 2012 and 2014 study design peak flow estimates

Return period (yrs)	COMP		WINF_A		CSFW	
	2012 (2013) study	2014 study	2012 study	2014 study	2012 study	2014 study
2	-	18.9	-	8.2	-	4.0
5	30.2 (31.3)	28.9	6.9	12.4	19.6	5.4
10	39.0 (42.4)	38.2	8.7	15.5	24.1	6.4
20	49.3 (56.6)	50.2	10.8	18.6	29.8	7.5
30	-	58.9	-	20.5	-	8.2
50	66.3 (82.4)	72.2	13.8	22.8	37.8	9.3
75	75.6 (N/A)	84.9	15.2	24.6	41.3	10.2
100	82.6 (110)	95.4	16.2	25.9	44.2	10.9
200	103 (N/A)	127	19.2	29.0	51.9	12.9
1000	170 (N/A)	248	29.3	36.3	77.4	20.1

6.6.1 River Chew

The current study design peak flow estimates at the Compton Dando gauge are approximately 10% lower than those from the 2013 study (updated from the 2012 study). The latest data and recommendations regarding the construction of the AMAX series (see Section 2.4.2) were used in the current study. Therefore it is believed that the greatest confidence can be placed in the preferred flow estimates from the current study.

6.6.2 Winford Brook

The 2012 study peak flow estimates for the Winford Brook were derived using the FEH Rainfall Runoff method based on catchment descriptors. The current study preferred flow estimates at this location are based on area-weighted results from the Chew Magna Reservoir Weir gauge location. These results were derived using catchment-specific data and an approach which has accounted for the large number of peak events which occurred in 2012 and resulted in flooding of Chew Magna.

The current study peak flow estimates are larger than the 2012 study estimates, by a factor of 1.72 for the 20 year return period event, 1.60 for the 100 year return period event and 1.24 for the 1000 year return period event. There was concern that the Winford Brook flows derived for the previous study were too small. The current study appears to have addressed this and there is greater confidence in these estimates at this stage of the study.

6.6.3 Chew Stoke Stream

The preferred peak flow estimates derived for the current study are significantly lower than those from the 2012 study (by a factor of about 0.25). The 2012 study peak flow estimates were generated using the FEH Rainfall Runoff method with model parameters adjusted using observed data. The flow data used to adjust the model parameters was derived using a modelled rating for the Chew Stoke Flood Warning gauge. This gauge was not designed to gauge flow and there is significant uncertainty in the stage-discharge relationship. The adjustments to the model parameters were derived on the basis of analysis of only four events. These all occurred prior to resurvey of the gauge in mid-2009; there is less certainty in the data before this time. The specific discharge for the 100 year return period event equates to $4.9m^3/s/km^2$; this seems excessive for a catchment of this type.

Although there is some uncertainty in the design peak flow estimates derived for the current study, due to the lack of catchment flow data, it is believed that the magnitude of the values is more sensible than the 2012 study values. At this stage there is more confidence in the current study estimates; this will be sensibility-checked using the hydraulic model, as discussed in Section 5.3.3.

7 Catchment-wide Application (Upper Chew)

Following the selection of the preferred design peak flow estimates at the key flow estimation locations it was necessary to apply inflows to the hydraulic model to sensibility-check the estimates. At this stage the catchment was split into sub-catchments based on the required model inflow locations. Intervening areas between flow estimation locations were used to generate lateral inflows to be input to the hydraulic model, where required. The following sections detail where flow estimates and lateral inflows were required, and initial hydraulic model tests to be undertaken. At this stage the focus of the assessment was on the Upper Chew catchment which is modelled to just downstream of the Winford Brook confluence. At a later stage flow estimates and lateral inflows will be required for the catchment downstream to Keynsham, to be input to the broadscale River Chew catchment model.

7.1 Flow estimation points

A number of additional flow estimation points were required to provide inflows to the top of the modelled watercourse reaches, to derive lateral inflows to the model and to act as check nodes for the modelled flows. The naming convention of the estimation locations was adjusted from the initial analysis to aid consistency when the assessment is extended to include the Lower Chew catchment. The Upper Chew model flow estimation points are detailed in Table 7-1 and shown in Figure 7-1. Of these twelve locations only five provide point inflows to the model - TRIB_F, WINF_C, REGI_A, CHST_C and CHEW_G. The rest were selected to check modelled flows and to assist in applying lateral inflows to the model.

Table 7-1 - Upper Chew model flow estimation points

Site code	Previous site code	Watercourse	Location	Grid reference	Area (km^2)
TRIB_F	N/A	Unnamed right bank tributary	Confluence with River Chew downstream of Winford Brook confluence	358450, 162900	3.9
CHEW_D	N/A	River Chew	Downstream of Winford Brook confluence	358100, 162950	90.1
WINF_A	WINF_A	Winford Brook	Confluence with River Chew	358100, 163000	19.6
WINF_B	CMRW	Winford Brook	Chew Magna Reservoir Weir gauge	356800, 163250	17.3
WINF_C	N/A	Winford Brook	Upstream model extent at Winford	354050, 165050	6.3
CHEW_E	N/A	River Chew	Upstream of Winford Brook confluence	357950, 162950	70.5
CHEW_F	N/A	River Chew	Downstream of Chew Stoke Stream confluence	357100, 162150	68.6
CHST_A	CHST_A	Chew Stoke Stream	Confluence with River Chew	357100, 162100	10.4
CHST_B	CSFW	Chew Stoke Stream	Chew Stoke_FW gauge	355950, 161800	9.1
REGI_A	N/A	Regil Stream	Confluence with Chew Stoke Stream	355800, 161850	2.2
CHST_C	N/A	Chew Stoke Stream	Upstream model extent at Lower Strode	354050, 161450	3.6
CHEW_G	CVL	River Chew	Chew Valley Lake spillway	356950, 161550	57.9

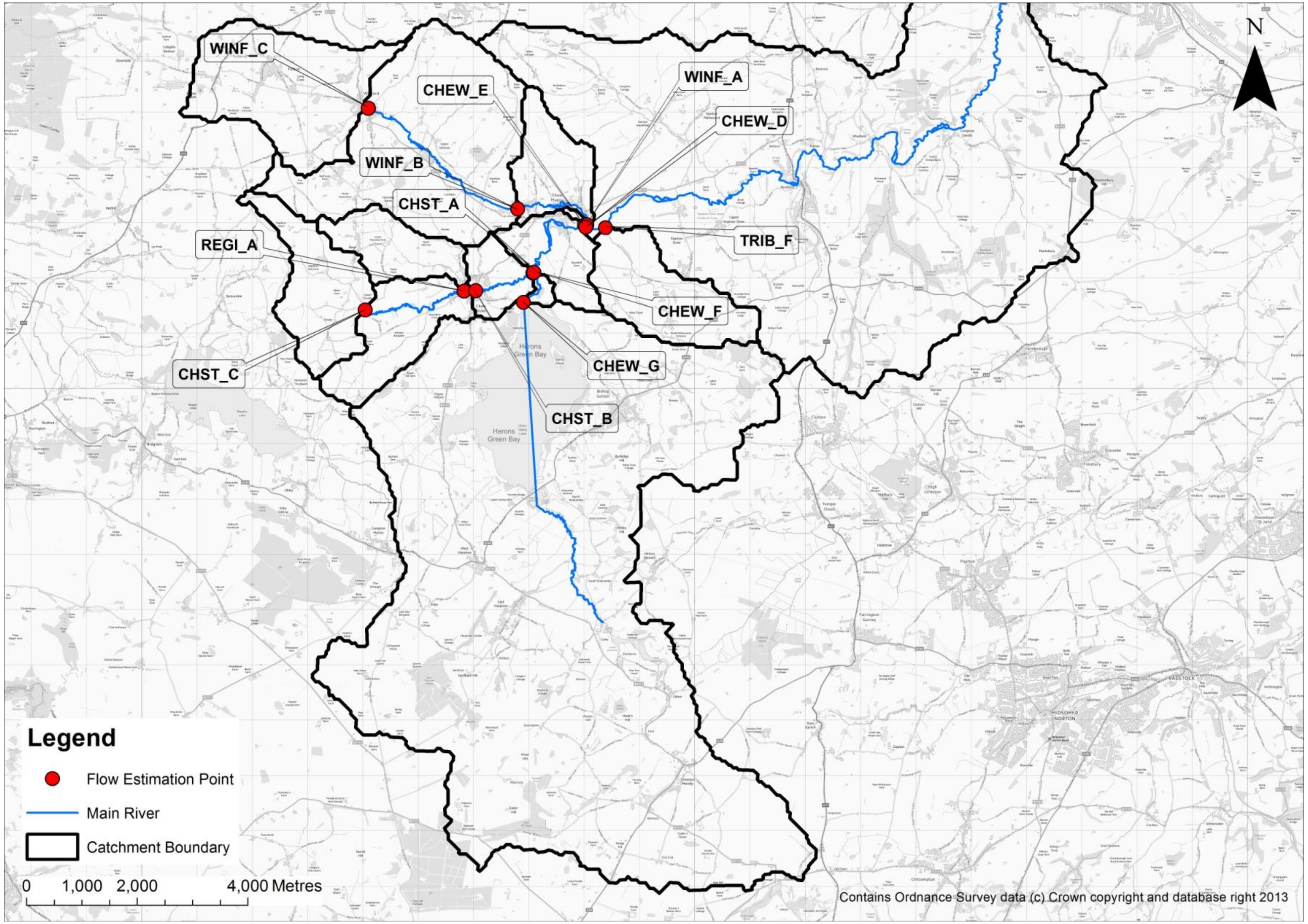

Figure 7-1 - Upper River Chew flow estimation locations

The key catchment descriptors for the new sites are provided in Table 7-2. The catchment boundaries were checked and no changes made. No further checks were made on the catchment descriptors except for a brief visual check on the FARL value. $URBEXT_{1990}$ and $URBEXT_{2000}$ were updated to 2014, to account for any small changes in urbanisation within the catchment.

Table 7-2 - Initial flow estimation point catchment descriptors

Site code	FARL	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	URBEXT 1990	URBEXT 2000
TRIB_F	1.000	0.35	0.558	2.07	58.8	915	0.0000	0.0000
CHEW_D	0.782	0.35	0.620	8.42	65.1	1025	0.0095	0.0112
WINF_C	1.000	0.35	0.776	2.09	54.1	983	0.0271	0.0344
CHEW_E	0.739	0.35	0.599	9.08	62.8	1045	0.0077	0.0091
CHEW_F	0.733	0.35	0.599	7.42	63.1	1049	0.0071	0.0082
REGI_A	1.000	0.35	0.542	1.80	59.2	938	0.0048	0.0093
CHST_C	0.991	0.35	0.602	1.55	76.7	956	0.0026	0.0000

7.2 Design peak flow estimates

The design peak flow estimates for the additional flow estimation locations are provided in Table 7-3. These were determined based on the recommendations made for the initial flow estimation locations:

- TRIB_F - Catchment descriptors are similar to those for CHST_B (Chew Stoke Flood Warning gauge - CSFW). Therefore the ReFH method with Tp(0) adjusted using the Chew Stoke Flood Warning gauge donor correction factor was used. The flows in Table 7-3 use the TRIB_F individual storm duration of 2.25hr.
- CHEW_D, CHEW_E and CHEW_F - These are located on the River Chew therefore it was assumed that Compton Dando was a reasonable donor. In order to maintain consistency with the estimate at Compton Dando but also to account for the decreasing influence of the gauge with distance, QMED was adjusted using the data transfer procedure. The flood frequency curve was generated using the FEH Statistical method single-site growth factors.
- WINF_C - Although this location is upstream of Chew Magna Reservoir it was decided that the influence of the reservoir was small enough to allow the Chew Magna Reservoir Weir gauge FSR POT analysis results to be area-weighted for this site. It is unlikely that a more suitable donor will be found for WINF_C given the small and permeable nature of the catchment.
- REGI_A - This is a tributary of the Chew Stoke Stream and the catchment descriptors are similar enough to the Chew Stoke Flood Warning gauge catchment to apply the method selected for CSFW. Therefore the ReFH method with Tp(0) adjusted using the Chew Stoke Flood Warning gauge donor correction factor was used. The flows in Table 7-3 use the REGI_A individual storm duration of 2.25hr.
- CHST_C - This is located on the Chew Stoke Stream and the catchment descriptors are similar enough to apply the method selected for CSFW. The ReFH method with Tp(0) adjusted using the Chew Stoke Flood Warning gauge donor correction factor was used. The flows in Table 7-3 use the CHST_C individual storm duration of 1.75hr.

Table 7-3 - Preferred design peak flow estimates for additional flow estimation locations

Return period (yrs)	TRIB_F (m^3/s)	CHEW_D (m^3/s)	WINF_C (m^3/s)	CHEW_E (m^3/s)	CHEW_F (m^3/s)	REGI_A (m^3/s)	CHST_C (m^3/s)
2	1.7	9.7	2.7	7.1	6.7	1.1	1.6
5	2.3	14.9	4.0	10.8	10.3	1.5	2.1
10	2.8	19.7	5.0	14.3	13.6	1.8	2.6
20	3.2	25.9	6.0	18.8	17.9	2.1	3.0
30	3.5	30.4	6.6	22.0	21.0	2.3	3.3
50	4.0	37.2	7.3	27.0	25.7	2.6	3.8
75	4.4	43.8	7.9	31.7	30.3	2.9	4.2
100	4.7	49.2	8.4	35.7	34.0	3.1	4.5
200	5.6	65.3	9.4	47.4	45.1	3.7	5.3
1000	8.8	128	11.7	92.6	88.3	5.8	8.4

7.3 Initial hydraulic model testing

Hydrographs need to be routed through Chew Valley Lake and Chew Magna Reservoir in order to derive outflow hydrographs which account for attenuation and reduction of the hydrograph peak. Therefore all initial model testing will be undertaken using ReFH method hydrographs. Two verification events will be run though the model, as described in Section 7.3.2. The results will be used to determine how the ReFH hydrographs and the hydraulic model may be adjusted to better approximate the known flood extents and depths for these events.

Following this the ReFH hydrographs for the selected storm durations detailed in Section 7.3.3 will be run through the model. The results will be sensibility-checked and the modelled flows compared to the preferred flow estimates detailed in Sections 6.5 and 7.2 of this report. This will be an iterative process to determine the best design peak flow estimates based on the work undertaken during the hydrological assessment and the results generated by the hydraulic model.

7.3.1 Lateral inflows

Lateral inflow hydrographs are required to provide inputs to the hydraulic model for the intervening catchment area between upstream and downstream flow estimation locations. As stated in Section 7.1 only five of the flow estimation locations are point inflows. Consideration was given as to whether or not lateral inflows were required for the other locations.

Intervening area catchment descriptors were derived using a back-calculation of the area-weighting procedure described in Volume 5 of the FEH. Values for AREA, BFIHOST, DPLBAR, DPSBAR, PROPWET, SAAR, $URBEXT_{1990}$ and the depth-duration-frequency (DDF) parameters were derived as these are the descriptors that the ReFH method utilises.

The intervening area catchment descriptors are provided in Table 7-4. Lateral inflows were not derived for CHEW_D and CHEW_F. The intervening catchment area for these locations is so small compared to the drainage catchment as a whole that the resulting hydrograph is insignificant.

Table 7-4 - Intervening area catchment descriptors

Site code	AREA (km^2)	PROPWET	BFIHOST	DPLBAR (km)	DPSBAR (m/km)	SAAR (mm)	$URBEXT_{1990}$
WINF_A_IA	2.35	0.35	0.618	1.60	94.8	904	0.0246
WINF_B_IA	10.95	0.35	0.659	3.71	80.1	950	0.0058
CHEW_E_IA	1.88	0.35	0.599	1.41	51.9	899	0.0291
CHST_A_IA	1.29	0.35	0.525	1.15	50.7	913	0.0418
CHST_B_IA	3.21	0.35	0.517	1.89	81.2	937	0.0148

7.3.2 Verification events

Two flood events were selected for the purpose of verifying the hydraulic model - 24th September 2012 and 21st-25th November 2012. Both of these events led to significant flooding within Chew Magna.

ISIS REFHBDY's were used to apply catchment average event rainfall to the ReFH model to generate hydrographs to be applied to the hydraulic model for the point and lateral inflows. At this stage the donor correction factors applied to generate the design hydrographs have been retained. Consideration may be given to using event-specific donor correction factors following assessment of the hydraulic model results.

The TBR proportions to be applied to generate catchment average rainfall were already available for the Chew Magna Reservoir Weir (WINF_B) and Chew Stoke Flood Warning (CHST_B) gauge locations (see Section 4.2). This assessment was also undertaken for the Chew Valley Lake spillway (CHEW_G) location, as shown in Table 7-5.

Table 7-5 - CHEW_G catchment TBR weightings

TBR	Weighting
Grove Farm	0.668
Chew Magna Spillway	0.332

Thiessen polygons were not generated to derive TBR proportions for all inflow locations, in order to avoid implying greater confidence in the resultant hydrographs than is actually the case. In addition, many of the inflows are lateral inflows which represent a disconnected portion of the catchment rather than a real drainage catchment. Instead it was decided to apply the Chew Magna Reservoir Weir gauge, Chew Stoke Flood Warning gauge and Chew Valley Lake spillway proportions based on location / watercourse, as detailed in Table 7-6.

Table 7-6 - TBR proportions for verification event REFHBDYs

Inflow	TBR proportion location
TRIB_F	CMRW
WINF_A_IA	CMRW
WINF_B_IA	CMRW
WINF_C	CMRW
CHEW_E_IA	CVL
CHST_A_IA	CSFW
CHST_B_IA	CSFW
REGI_A	CSFW
CHST_C	CSFW
CHEW_G	CVL

The ReFH modelled hydrographs for the catchment (full not intervening area) to WINF_B (Chew Magna Reservoir Weir gauge) and CHST_B (Chew Stoke Flood Warning gauge) were generated to compare the hydrograph shape with the observed level hydrograph shape for both events.

24th September 2012

The rainfall used to represent the 24th September 2012 event is detailed in Table 7-7.

Table 7-7 - 24th September 2012 rainfall event information

TBR proportion location	Rainfall start	Rainfall end	Duration (hr)	Depth (mm)
CMRW	23/09/2012 08:15	24/09/2012 07:45	23.50	67.0
CSFW	23/09/2012 08:15	24/09/2012 07:45	23.50	65.0
CVL	23/09/2012 07:30	24/09/2012 07:45	24.25	64.5

The ReFH modelled flow hydrographs and observed stage / level hydrographs for WINF_B and CHST_B are shown in Figure 7-2 and Figure 7-3, respectively. It can be seen that the shape of the ReFH modelled flow hydrographs matches the observed stage / level hydrographs well. Timing of the peaks is reasonable; the observed peaks occur 30 minutes prior to the modelled peaks at both locations.

The ReFH modelled peak flow at WINF_B is 8.6m^3/s compared to the peak of 22.2m^3/s derived from the rating for the Chew Magna Reservoir Weir gauge. Therefore the rating is predicting a flow approximately 2.6 times larger than the ReFH model predicts from the rainfall data. The hydraulic model will generate flood extents and depths using the modelled ReFH hydrographs. These will be compared to the flood reconnaissance for the event to determine if the modelled hydrographs are providing a good representation of the event and the gauge rating is overestimating the flow or if the modelled flows need to be increased.

The ReFH modelled peak flow at CHST_B is 7.1m^3/s. This would be equivalent to a return period of 10-20 years based on the preferred flow estimates for this location.

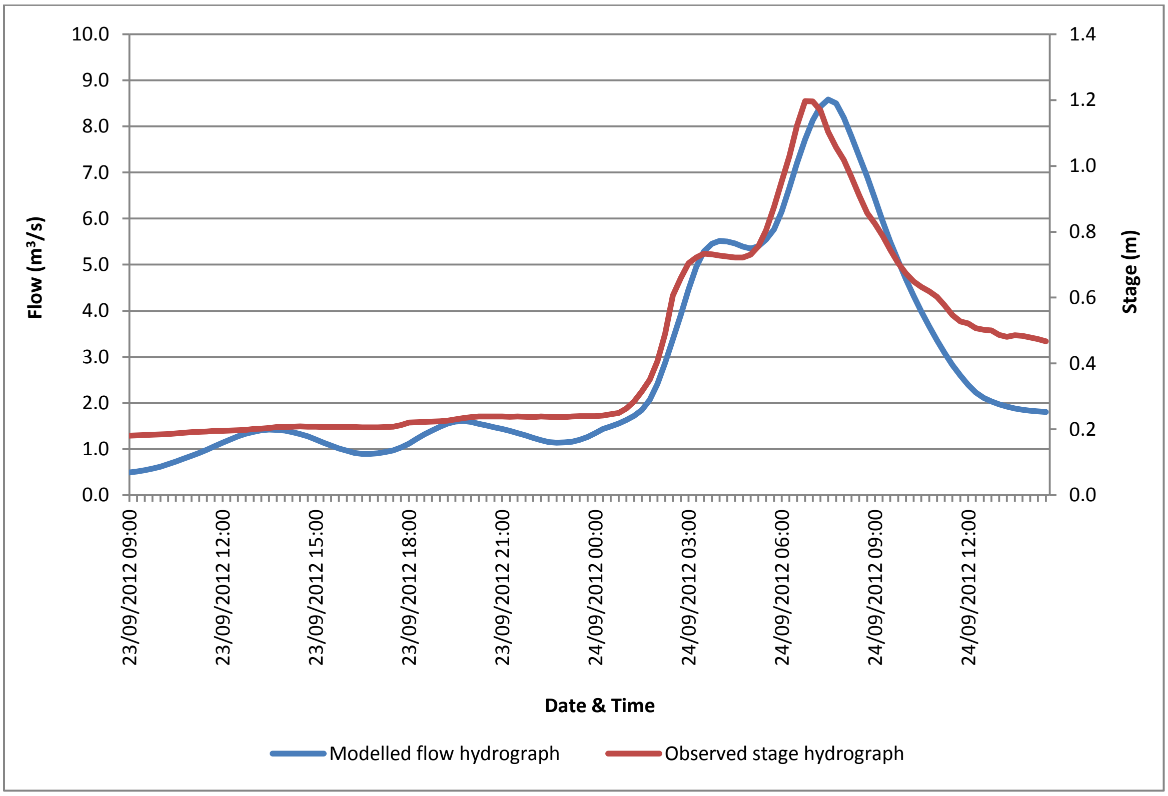

Figure 7-2 - WINF_B 24th September 2012 modelled flow and observed stage hydrographs

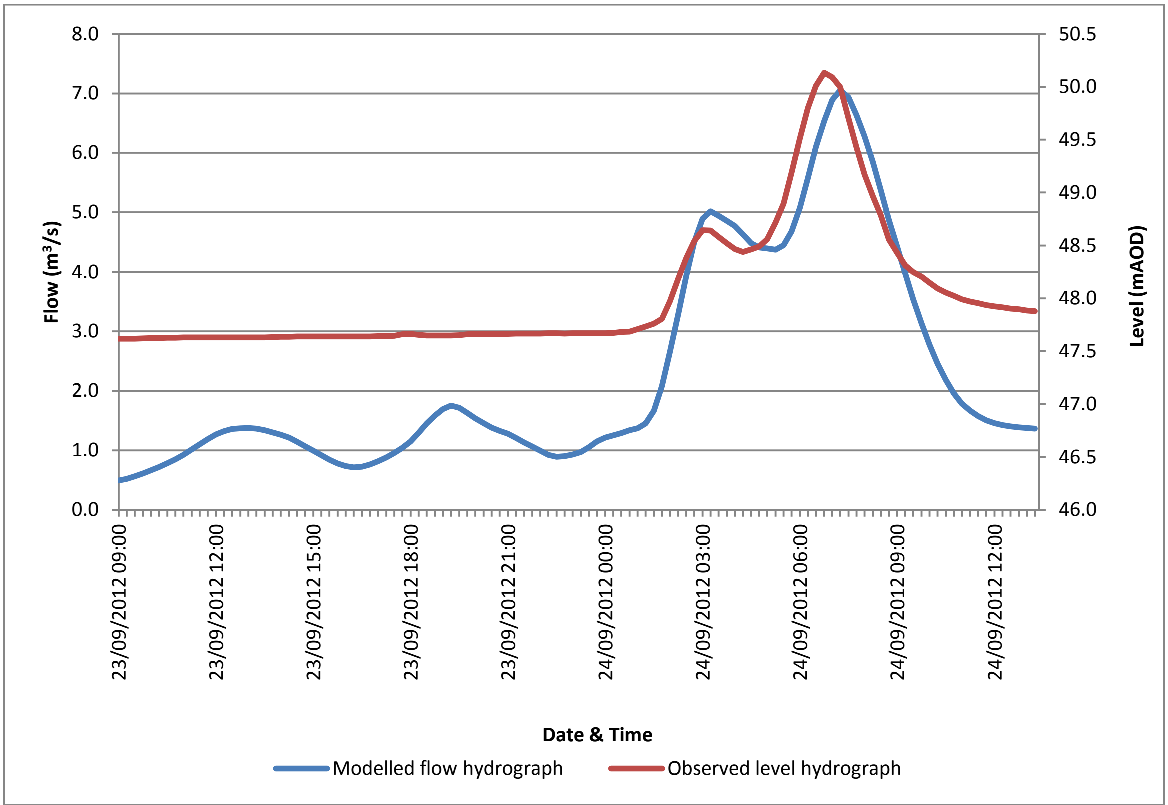

Figure 7-3 - CHST_B 24th September 2012 modelled flow and observed level hydrographs

21st-25th November 2012

The rainfall used to represent the 21st-25th November 2012 event is detailed in Table 7-8. Although the three peak events in November 2012 were deemed to be independent by the rules provided in the FSR it was thought that the best representation of the peaks in the hydraulic model would be obtained by considering the rainfall for the three events as a whole.

Table 7-8 - 21st-25th November 2012 rainfall event information

TBR proportion location	Rainfall start	Rainfall end	Duration (hr)	Depth (mm)
CMRW	20/11/2012 03:15	25/11/2012 02:00	118.75	113.3
CSFW	20/11/2012 03:15	25/11/2012 02:00	118.75	118.8
CVL	20/11/2012 02:45	25/11/2012 02:00	119.25	117.3

The ReFH modelled flow hydrographs and observed stage / level hydrographs for WINF_B and CHST_B are shown in Figure 7-4 and Figure 7-5, respectively. It can be seen that the shape of the ReFH modelled flow hydrographs matches the observed stage / level hydrographs well. Timing of the peaks is reasonable; the observed peaks occur 30 minutes prior to the modelled peaks at the CHST_B location. At WINF_B the observed peaks for the 21st and 25th occur 45 minutes prior to the modelled peaks; for the 22nd the observed peak occurs 1 hour and 15 minutes earlier than the modelled peak.

The main difference between the ReFH modelled and observed hydrographs is the event which is predicted to have the largest peak. At both gauge locations the stage / level data shows the 21st November peak to be the largest. The difference between the observed peak stage for the three events at WINF_B is small - 1.07m for the 21st, 1.05m for the 22nd and 1.04m for the 25th. There is only a small difference between the 21st and 22nd peak level at CHST_B (49.94mAOD and 49.91mAOD, respectively). There is a larger difference for the 25th peak (49.57mAOD).

The ReFH modelled hydrographs for WINF_B show an increase in peak flow between the 21st and 25th peaks (21st - 5.1m^3/s, 22nd - 6.5m^3/s and 25th - 7.8m^3/s). For CHST_B the ReFH modelled hydrograph for the 21st has the smallest flow and the 22nd has the largest flow (21st - 4.9m^3/s, 22nd - 6.4m^3/s and 25th - 6.2m^3/s). It is unlikely to be possible to replicate the trend in peak flows using this approach.

The largest ReFH modelled peak flow for the November events at WINF_B is 7.8m^3/s compared to the largest peak of 17.3m^3/s derived from the rating for the Chew Magna Reservoir Weir gauge. Therefore the rating is predicting a flow approximately 2.2 times larger than the ReFH model predicts from the rainfall data. The largest ReFH modelled peak flow at CHST_B is 6.4m^3/s. This would be equivalent to a return period of 10 years based on the preferred flow estimates for this location.

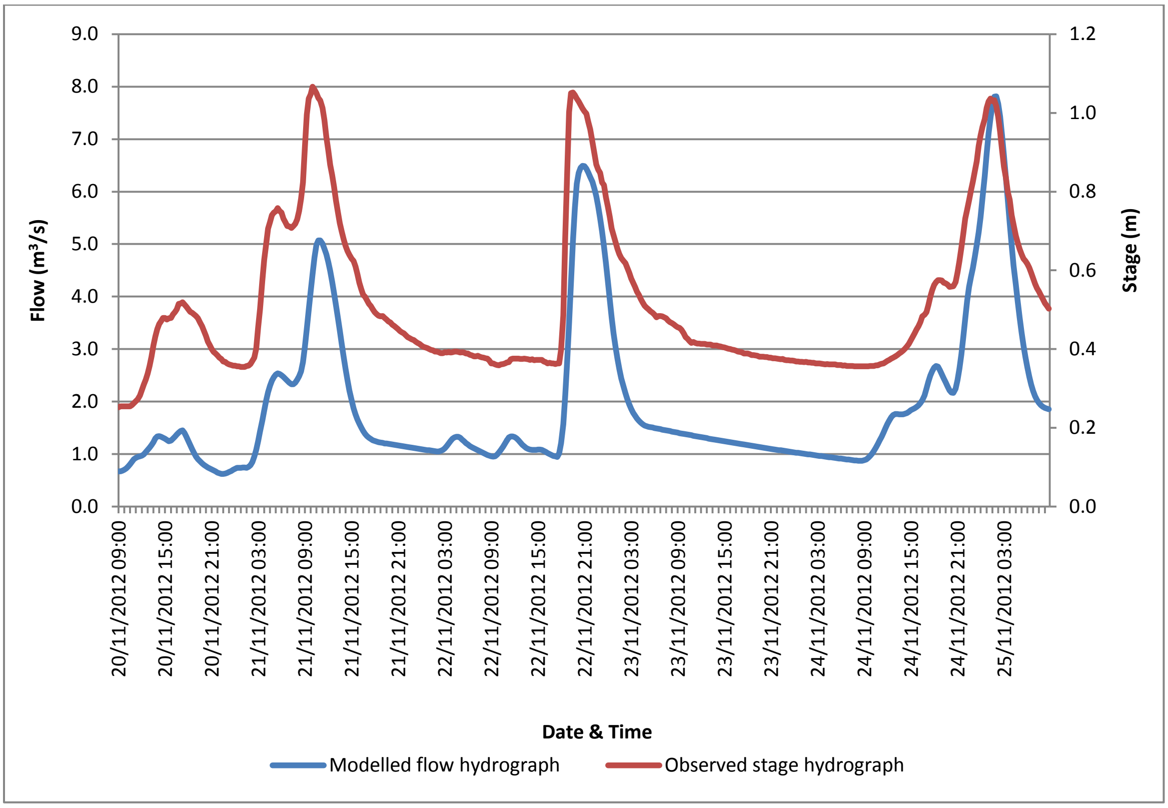

Figure 7-4 - WINF_A 21st-25th November 2012 modelled flow and observed stage hydrographs

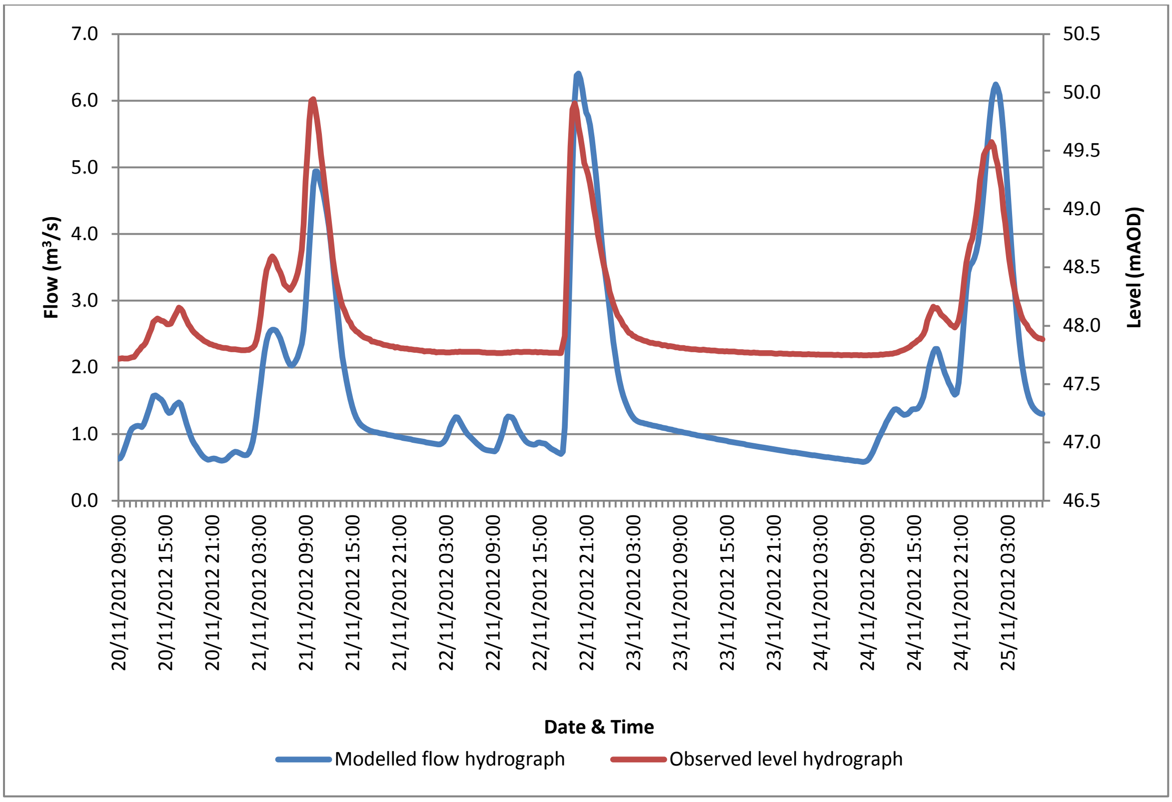

Figure 7-5 - CHST_B 21st-25th November 2012 modelled flow and observed level hydrographs

7.3.3 Selected storm durations for testing

Section 4.3.2 details the four storm durations selected for testing within the hydraulic model - 2.75hr, 8.75hr, 49hr and 97hr. Initial model runs were undertaken using a cut-back model, using only three inflows, to aid in the understanding of the impact of the reservoirs on downstream flows, particularly Chew Valley Lake. This was done for the 8.75hr, 49hr and 97hr storm durations with the reservoir water levels set to the level of the spillway. There was little difference in the flood extents for the River Chew for the three durations. This suggests that Chew Valley Lake may act to attenuate runoff even for long duration storm events.

However, these results are from a basic model and the full Upper Chew model, containing all model point inflows and lateral inflows may show different results. The full Upper Chew model will be run for the 2.75hr, 8.75hr and 49hr storm durations. Once the results of these runs have been assessed it will be decided if the 97hr storm duration should also be run through this model. This model will take an extensive period of time to run. The results from the verification events will be assessed before carrying out the duration testing runs to ensure that the most appropriate adjustments to the ReFH hydrographs are made.

8 Summary & Further Work

8.1 Summary of results

A number of analyses were undertaken at the preliminary stage of the hydrological assessment to utilise the new hydrometric data and information available for the catchment gauges and to assess the influence of the two reservoirs on downstream flow:

- Routing calculations were performed using a rainfall runoff approach to determine the outflow hydrographs from Chew Valley Lake and Chew Magna Reservoir.
- Analysis of the Compton Dando gauge data was undertaken to determine ReFH model parameters from observed data to improve the hydrograph shape and peak flow estimates.
- Lag analysis was carried out on the Chew Magna Reservoir Weir and Chew Stoke Flood Warning gauge level data to improve the estimate of Tp(0) (instantaneous time to peak) and hence hydrograph shape for the ReFH method.
- Assessment of the most appropriate flow at Compton Dando for the July 1968 flood was undertaken.
- An alternative approach to deriving QMED and the flood frequency curve was applied to the Chew Magna Reservoir Weir gauge data. This is the FSR POT approach which makes best use of the numerous peak flow events recorded in 2012.
- FEH Statistical method analyses using the available flood peak data from the catchment at the key gauge locations. This utilised the new rating for the Chew Magna Reservoir Weir gauge and recent data for the Compton Dando gauge to determine QMED and to undertake single-site analyses.
- Pooling group composition was investigated and the merits of using a 'permeable' pooling group assessed.

The results generated by the application of these methods and approaches to six initial key flow estimation locations were assessed and the preferred design peak flow estimates selected for each location. These preferred design peak flow estimates are provided in Table 8-1.

Table 8-1 - Preferred design peak flow estimates for key flow estimation locations

Return period (yrs)	COMP (m^3/s)	WINF_A (m^3/s)	CMRW (m^3/s)	CHST_A (m^3/s)	CSFW (m^3/s)	CVL (m^3/s)
2	18.9	8.2	7.2	4.3	4.0	
5	28.9	12.4	10.9	5.7	5.4	
10	38.2	15.5	13.6	6.8	6.4	
20	50.2	18.6	16.4	7.9	7.5	
30	58.9	20.5	18.0	8.7	8.2	
50	72.2	22.8	20.0	9.7	9.3	
75	84.9	24.6	21.7	10.7	10.2	
100	95.4	25.9	22.8	11.4	10.9	
200	127	29.0	25.6	13.6	12.9	
1000	248	36.3	32.0	20.9	20.1	

Additional flow estimation locations were selected to provide point inflows to the upstream extents of the Upper River Chew modelled watercourses, to provide lateral inflows to the model and to act as check nodes to ensure that modelled flows approximate preferred flow estimates. The design peak flow estimates for these locations were determined based on the decisions made for the key flow estimation locations.

8.2 Further work

- Hydraulic model runs will be carried out to determine if the preferred design peak flow estimates are sensible based on design event flood extents and depths and comparison to flood history.
- Further flow estimation locations will need to be considered for the River Chew catchment model which extends to the confluence with the River Avon at Keynsham.

The design peak flow estimates will be generated using a broadscale assessment and will utilise the work undertaken in this detailed hydrological assessment where possible.

Appendices

A Flood Estimation Calculation Record

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