School of
Civil Engineering and Geosciences

Newcastle University

Quantifying and simulating the impact of flood mitigation features in a small rural catchment

By

Alexander Robert Nicholson MEng

School of Civil Engineering and Geosciences,
Newcastle University,
Newcastle upon Tyne,
NE1 7RU,
UK.

A thesis submitted for the degree of Doctor of Philosophy (Ph. D) at Newcastle University

August 2013

Declaration

I certify that no part of the material offered in this thesis has been previously submitted by me for a degree or other qualification in this or any other university.

Signed

A Nicholson

Alexander Robert Nicholson

Abstract

The management of fluvial flood risk in the UK is undergoing a paradigm shift, with a change in emphasis from structural defences to working with natural processes where possible. Natural Flood Management has been advocated by several interest groups as a potential option for providing a low cost, sustainable solution to catchment flooding. An integrated monitoring, field experimentation and modelling campaign has been undertaken to assess the potential of Natural Flood Management (NFM) to reduce flood risk in the rural Belford Burn catchment, Northumberland ($5.7km^2$). The village of Belford failed to satisfy a risk-based cost-benefit criterion for structural defences, despite a number of floods occurring in recent years. The alternative low cost NFM mitigation approach taken in Belford involves the use of soft-engineered Runoff Attenuation Features (RAFs) that intercept or modify hydrological flow pathways.

Within the Belford catchment 35 RAFs have been installed to date, including interception bunds, permeable timber barriers, large woody debris and offline storage ponds. The performance of a number of RAFs has been rigorously assessed using a combination of analyses of in situ observed data and modelling techniques. An innovative 'Pond' Model has been developed, which uses in situ observational data and physically-based methods, for evaluating the operational performance of the RAFs and assessing their impact on a number of historical flood events and design storms. In addition, the physical functioning and methodological approach of the Pond Model has been evaluated against a peer-reviewed hydraulic model. Also a hydrological modelling package was modified to also demonstrate the impact of RAF attenuation at the catchment scale, with the aim of creating a methodology for transferring the knowledge gained at Belford to other small catchments.

This research has quantified the impacts of individual RAFs in the Belford catchment. From analyses of historical events, the Pond Model reveals that a network of attenuating features has the potential to significantly reduce peak flow (by up to 30%). However, for larger return interval design storms (for example 1:100 year return interval 24 hour duration) it is demonstrated that a certain/threshold of RAF attenuation features are required before the aggregate effects cause reduction in peak flow. The potential transferability of the approach and the methods used could have benefits for other similar small catchments ($<10km^2$). An assessment of cost effectiveness is made that includes the comparison between the original

cost of the proposed Belford flood alleviation scheme using a traditional structural methods and the RAF based scheme.

Acknowledgements

I would like to thank my first supervisor Dr Paul Quinn for his many hours of input and advice towards this PhD work. His enthusiastic outlook has fuelled my own motivation and thirst for continued knowledge in the field of hydrology and catchment sciences. I can only hope that I can learn how to engage others as he has engaged me.

My second supervisor, Dr Greg O'Donnell, has spent countless hours with me; helping me learn Python programming, reading and correcting pieces of work and encouraging my further development and understanding of complex procedures in hydrological modelling. For his continued support throughout my PhD, there really are no words. I am extremely grateful for everything he has done for me.

I would like to add thanks to Dr Qiuhua Liang for providing me with the tools to complete the hydraulic modelling section of this PhD project. It has been extremely valuable to this research to have been able to approach the situation from the 'hydraulics angle'. I am extremely lucky to have had his support since my Masters dissertation to the end of my PhD, and beyond.

When the Belford project began, in late 2007, Dr Mark Wilkinson was the project manager. When I started in September 2009 he taught me many things; from flow gauging and installing/downloading hydrometry to working with large datasets and giving practical advice on papers, posters and presentations. Thank you for your work at the start of the Belford project, as it inevitably saved me time, and for your invaluable help along the way.

Additional thanks go to Dr Mike Palmer, Mr Nick Barber, Mr Tom Bramald, Mr Martin Robertson, Mr Alistair Ford and Mr Gareth Owen for their help with field work, surveying and software. Without these people I would have found the task exponentially more difficult. I would like to add that it has been a pleasure to have worked with all the people I have mentioned, and it makes me very proud to say that I have done so.

This project has been 80% funded by the Engineering and Physical Sciences Research Council (EPSRC) through the Flood Risk Management Research Consortium (FRMRC2) and 20% funded by the Environment Agency (EA) through the North-East regional flood levy. I would like to thank Mr Peter Kerr and Mr Phil Welton, from the EA, who started and funded the work on the entire Belford project. I would also like to thank Mr Johnny Riddle for allowing so much access to his farm and the permission to construct new mitigation features.

I would like to thank my mother, father and brother for their emotional and financial support over my entire life. I would not be where I am today without their belief that I could make something of myself. They are the best family a person could wish for. My wife, Anna's, family have welcomed me into their lives – I have gained two more brothers and an extremely supportive mother-in-law. My friends at university have allowed me to escape the grind of the working regime – and for that, and many other aspects of our friendship, I am extremely grateful. Anna has had to put up with a great many tales of 'broken code' or 'unresponsive GIS-software'. Thank you for pretending to be interested in these pointless stories. Thank you for helping me develop into the person I am today and for sharing the ups and downs of life with me. I could not have done any of this without you.

Table of Contents

Table of Figures

Table of Tables

1. Introduction

1.1 Flooding and its wider context

A report published by the Environment Agency (2007) stated that 10% of the UK population lives on natural floodplains. The report also highlighted that an estimated 1.8 million homes, 130,000 commercial properties and 14,000km^2 of agricultural land (12% of the UK total) are at risk from flooding. Population growth and an increase in urbanization in flood prone areas have led to greater risk of impact on human life as a direct result of floods than ever before (Jonkman, 2005).

Flooding is already the most costly natural hazard in Europe and South Asia, but future risk projections are much less certain than for drought and heat wave (Dankers & Feyen, 2008). In 2007 alone there were 200 major flood events worldwide, affecting 180 million people, killing 8,000, and causing £40 billion worth of damage (Pitt, 2007). The floods that devastated areas of England in summer 2007 were ranked as the most financially costly in the world for that year.

The flooding in England in summer 2007 is an excellent case study to consider. After an intense period of extreme rainfall some 55,000 properties were flooded rendering many businesses out of action for several months. The flooding was characterised as both fluvial and pluvial flooding; rivers flooded surrounding land and, following exceptionally high rainfall, there was direct flooding of areas with insufficient drainage capacity (Pitt, 2007). The emergency services rescued 7,000 people, although 13 people died. England saw its largest loss of essential services since the Second World War. Almost half a million people were without mains water or electricity. Transport networks failed, leaving 10,000 people stranded on the M5 (overnight) and many others stranded on rail networks (Pitt, 2007).

The majority of large cities are situated in downstream river plains and are inherently vulnerable to flood damage (Kaneki, 2001). To combat this, many rivers, running through large towns and cities, have been subject to flood prevention schemes. Smaller towns and villages, however, can be frequently affected by severe local rainstorms, which can bring increased inundation of inland waters and eventually lead to the overflow of rivers (Kaneki, 2001). Flooding in rural areas has been a problem in the UK for many years; a problem which planners and engineers have attempted to control or mitigate by the application of computer modelling combined with engineered defences or improved land management. Recent floods in areas of North East England like Morpeth in 2008 have shown the relevance of continued work in this field (Environment Agency, 2009). The EA estimated that the flood that struck in Morpeth, in 2008, had a 0.67% chance of occurring in any year. Weather events like those seen in Morpeth may occur relatively frequently, with the government estimating events of the same order of magnitude having a 65% chance of happening somewhere in England at least once each year (Defra, 2008a). Changes to UK and European Union agriculture policies have had knock-on effects towards enhanced surface runoff generation at the local field scale, which may also have had effects on river channel flow and downstream flooding of towns/villages (O'Donnell, et al., 2008).

Current government policy relating to flood risk management in rural areas recognise the potential of land use solutions such as the creation of wetlands and the managed realignment of rivers (Defra, 2005). Defra (2005) proposed that priority research should take place to establish the role that rural land management techniques may play in managing flood risk at the catchment level; as well as surface water management plans, in the form of SUDs, being introduced in the urban environment (Defra, 2008b). The European Floods Directive's (2007/60/EC) 'Flood Risk Management Plans,' focus on 'the promotion of sustainable land use practices, improvement of water retention as well as the controlled flooding of certain areas in the case of a flood event.' The recent Flood and Water Management Act 2010 (UK) encourages maintaining or restoring natural processes wherever possible as a method of reducing flood risk, and permits the designation of natural features that can control this risk (POST, 2011). These policies, in combination with concerns about future climate change, highlight the need for the deliverance of sustainable solutions for flood management (Parrott, et al., 2009). The limited time that has elapsed since the emergence of these policies and the present date can explain why implementation of work in this field has been slow to uptake. This is especially the case where flood risk managers are reluctant to try new methods unless substantial evidence (from well-established projects) are available (Ball, 2008).

Although there is still an expectation that anthropogenic climate change will increase the magnitude and frequency of extreme precipitation events, the consequences for inland flooding depend on the generating mechanism, and a host of site-specific factors, not least land-use changes (Wilby & Keenan, 2012). Recent government policy recognises that water management must be seen in the broader perspective, and is inextricably linked to land management (Wheater, et al., 2008). If land management changes are significant in influencing hydrological response at the catchment scale then it will be both important in terms of flood risk assessment and for discovering possible interventions for reducing these downstream flood risks. There is therefore a necessity for guidance concerning the hydrological impacts of land management to inform agricultural policy. The floods that have affected the UK in recent years have reinforced growing concern that changes to agricultural practice may have increased the risk of flooding (Wheater, 2006). It is thought that agricultural intensification may cause higher flood peaks in streams and rivers due to its impact on runoff processes. For example, degradation of soil structure can lead to reduction in infiltration rates and available storage capacities, increasing rapid runoff in the form of overland flow (Heathwaite, et al., 1990; Bronstert, et al., 2002). Although flood hazard is greater in lower lying regions (i.e. areas where population is usually higher), the management of headwaters, with their generally higher precipitation rates and flashier response, is of particular interest for flood runoff generation (Wheater, et al., 2008).

Adaptation to environmental change has occurred throughout human history but is achieving greater prominence as societies recognise their vulnerability to the frequency and magnitude of extreme events (Wilby & Keenan, 2012). Minimizing the effects of flooding continues to be a top priority for the Government and the Environment Agency. In this current economic climate there is a growing need to discover cheaper forms of flood management. The emergence of natural flood management can be traced partly to recognition of the inadequacy of structural flood management options to cope with future increases in flood risk from climate change (see, Evans, et al., 2004; Defra, 2005). It is clear that traditional engineering solutions founded on the assumption of a stationary climate are no longer applicable (Milly, et al., 2008). There has been a growing drive towards working with natural processes to mitigate flooding, but a lack of quantitative evidence able to justify flood management schemes to a wider audience.

1.2 Flooding in Belford

Flooding from the Belford Burn, in Northumberland (North East England), is the focus of this study. The Belford Burn is a small stream that runs through the centre of Belford village, hard up against garden boundaries and walls. It presents a risk of flooding to 34 properties and a caravan park, with the return period of flooding to the houses at lowest-elevation being only 1 in 5-years (Halcrow, 2007).

There have been several previous studies on the Belford burn, the most recent being a prefeasibility study by Halcrow in 2007 – assessing the viability of flood defences for the village. The prefeasibility study used modelling output from a Flood Risk Mapping Study, completed by

JBA (Jeremy Benn Associates) in 2004, to investigate the likely options to improve the conveyance of Belford Burn; aiming to reduce the flood risk in Belford village. Due to tight physical constraints in the village, traditional walls and embankments were unlikely to offer a complete solution. Therefore the study looked at combinations of these traditional solutions with flood storage and / or flood warning and upstream catchment management to reduce flood risk.

The modelling results for the Halcrow prefeasibility study found that the indicative standard of protection for Belford was 1 in 5 years. Several different options were considered for increasing the level of protection to the village, including:

i) Traditional flood defences (walls, embankments, flood gates, bridge improvements)
ii) Flood proofing of individual properties in conjunction with direct flood defences
iii) A combination of flood storage (reservoirs) and direct defences with or without flood proofing

Each of these options was costed (capital and maintenance costs), appraised for their environmental and planning constraints, and an economic appraisal was performed. The preferred environmental defence scheme was Option ii (above). However, the economic appraisal showed that none of the defence options reached unity in the cost-benefit ratio. In fact, increasing the level of protection to 100 years would cost approximately £3.5 million and would involve the construction of a large flood storage reservoir (c. 40,000 m^3).

The analysis undertaken by Halcrow concluded that traditional options and formal flood defences or upstream storage areas could not be cost-effective. The report acknowledged that upland catchment management typically in the form of afforestation, farmland buffer strips, localised storage and management of grazing and cropping patterns, could be a possible solution to the flooding problem in Belford. However, Halcrow's report identified the difficulty in assessing the level of protection from such measures, and that such schemes can only be regarded as experimental at present.

Following the prefeasibility study, outlined above, the Environment Agency funded the application of an upland catchment management programme. This followed early evidence gathered as part of a research project at Nafferton Farm, where corner of field ponds and ditch management was used to mitigate against high flows (e.g. Figure 1.1). The approach presented in the 'Proactive' study was to install passive intervention on farms to mitigate against large amounts of runoff (Quinn, et al., 2007).

Figure 1.1: Corner of field ponds in Nafferton (Quinn, et al., 2007)

Belford's upland catchment management programme began in late 2008 and, with consultation and field work from Newcastle University, has involved the construction of 35 individual mitigation features in the form of small storage features, ditch management, large woody debris, soil bunds installed across fast runoff pathways and some bespoke designs tested during the study. These became, collectively, known as 'Runoff Attenuation Features'.

1.3 Runoff Attenuation Features (RAFs)

A Runoff Attenuation Feature (RAF) is defined as a *man-made landscape intervention that intercepts and attenuates a hydrological flow pathway to provide multiple benefits, including flood management and improving water quality*. Put simply, the design philosophy is to create features that 'slow, store and filter' runoff in the rural landscape. Key design attributes of RAFs are that they:

- are easily accommodated in the landscape;
- do not significantly impact on farming;
- are typically small in size (<500m^2) or located within a ditch or small stream channel;
- are designed to be an extension of the farming and land drainage scheme drainage regime (i.e. they must not be viewed solely as flood engineering projects);
- potentially provide multipurpose benefits, for example in terms of nutrient transport (Barber & Quinn, 2012)

These key attributes differentiate the ‘RAF approach’ prescribed by Newcastle University from several other forms of Natural Flood Management (NFM) interventions that have been proposed to mitigate flooding (e.g. EA, 2010; POST, 2011). Specifically, the above attributes exclude large scale schemes such as extensive wet woodland creation, river engineering and the utilisation of extensive floodplain areas for storage. This is not to state that such schemes do not have merit, but such large scale interventions fall outside of the scope of the RAF approach trialled to date, which has targeted small catchments (~10km^2) (Quinn, et al., 2013). Instead the RAF approach advocates the use of many features located throughout the landscape, with the benefits accrued by the network of features rather than one large scale / dominant intervention. Also, the approach does not seek to replace more traditional flood management options, but rather to add to the number of potential options available to flood managers.

RAFs function in a similar way to sustainable urban drainage systems (SUDS) (see Chapter 2.4.7) in the way they are designed to store a quantity of surface runoff produced on, in this case, intensely developed agricultural land. In general, SUDS are designed to replace physical storage where it has been removed through construction of urban areas to mitigate the impact downstream of the urbanised area. RAFs, however, are designed to reduce the impact of intense development upstream of an urbanised area or point of interest; like Rural SUDs (Letts, 2012).

RAFs may be described as being online or offline. Offline RAFs (Figure 1.1) are essentially temporary ponds placed in fields at points where overland flow routes converge, which allow them to store and slow down large amounts of runoff within hours of the flow being generated by a storm event. This means that for the majority of the time they are empty, which is of huge benefit to the landowner, who does not lose the functionality of the area chosen for the site of the RAF. Offline RAFs located by rivers begin to fill when the water level in the river reaches a certain height, theoretically allowing the base-flow to continue downstream while the peak-flow is removed. Offline RAFs located on flow paths in fields capture overland flow and retard the motion of water moving towards the river network.

By contrast, online RAFs are connected to a ditch/stream network; thus, constantly interacting with flow. These features have more benefit to sediment capture and nutrient transport issues due to the fact they constantly have flow running through them (e.g. Figure 1.2) (Barber &

Quinn, 2012; Barber, 2013). As they are constantly online, however, they fill well in advance of the flood peak reaching them, which limits their effectiveness as flood mitigation measures. Online RAFs such as large woody debris, however, increase roughness in the channel and encourage floodplain attenuation of peak flows.

Figure 1.2: Online RAFs. Left – Phosphorous trap in Nafferton (Quinn, et al., 2007); Right – Wood-chip filter unit in ditch in Belford

1.4 Aims and objectives

1.4.1 Aim

This research will attempt to quantify the impacts of individual Runoff Attenuation Features (RAFs) in the Belford catchment using an evidence-based approach and, using appropriate modelling techniques, will simulate a series of RAFs (acting as a network) to assess the potential benefits of a catchment-based scheme. The modelling techniques will be compared to historical flood events, through hydrometric recordings, as well as established hydraulic and hydrological models to demonstrate the transferability of the RAF approach to similar catchments.

1.4.2 Objectives

In order to deal with the aims of this project, the thesis has been structured into the following series of chapters:

- **Literature Review:** The literature review aims to explore the main causes of flooding in agricultural catchments; examine wider literature on the current application of different forms of Natural Flood Management (NFM), and; identify where gaps lie in the implementation, monitoring and modelling of NFM.

- **Catchment Description:** A desk study; drawing from previous studies in the Belford catchment will be undertaken. The key objectives are to note the current issues that lead to flooding in the village; identify any gaps that these studies may have missed, and; demonstrate alternative methods to obtain the necessary information to fill these gaps in knowledge.

- **Catchment Data:** A multi-scale, nested hydrometric network has been installed in the Belford catchment. Additional monitoring stations will be added to the network with the aim of identifying the rainfall-runoff regime in Belford and helping to measure the impact of RAFs within the catchment. The main purpose of the monitoring network is to quantify the rainfall-runoff regime and capture flood events for analysis and modelling purposes. For this, the experimental design will focus on the dense monitoring of water level, precipitation and evapotranspiration. The stage gauges have been nested within the catchment to observe changes in the propagation of the flood hydrograph as it moves through the catchment. The work will involve an extensive fieldwork campaign, which will cover river and field surveys, flow gauging and manual measurements, and the collection of time-series data logged at all the gauging sites.

- **Results and hydrological data analysis:** The data from the monitoring network will be examined in detail to assess the rainfall-runoff regime in Belford.

- **Analysing Belford Storms:** The analysis of data will be extended to classify the types of storm that cause flooding to Belford.

- **Mitigation methods in Belford:** Example RAFs from the Belford catchment will be assessed, in detail, to understand their construction and purpose within the catchment. Specific RAFs will be chosen for additional hydrometry in order to help identify impacts during storm events. The interaction between RAFs and the rest of the catchment will be analysed in order to develop a conceptual design of RAF networks for use as a potential form of catchment-wide flood mitigation.

- **Pond forensic analysis:** An analytical approach for assessing the impact of individual RAFs will be developed using hydrometric data observed in the Belford Burn and within the RAFs themselves. The method will demonstrate the impact of individual RAFs upon recorded storm events and help to identify the physical attributes of RAFs that have the most impact on targeting peak discharge.

- **The Pond Model:** A bespoke modelling tool, which will use surveyed data and other measured attributes, will be developed for simulating the impact of RAFs during recorded storm events. The main purpose will be to generate a representation of a network of RAFs within the Belford catchment in order to demonstrate the total impact upon the storm hydrograph.

- **Hydraulic and Hydrological modelling and experimentation:** RAF networks will be simulated using both hydraulic and hydrological models to demonstrate transferability to other catchments and other studies. A simple DEM will be generated and configured to contain RAF features (in the form of offline ponds) for use in a 1D/2D hydraulic model. This will allow independent testing of RAF networks against results from the Pond Model. Finally, the Pond Model will be emulated using the simple lumped conceptual rainfall runoff model, TOPCAT. Instead of topographic data, this model will use observed rainfall and runoff alongside potential evaporation to calibrate the existing runoff regime with the model parameters. These parameters will then be altered to represent physical storage and a change in time-to-peak as a result of mitigation within the catchment.

- **Conclusions and Recommendations:** The research will be summarised with the main conclusions and suggested further work presented.

2. Literature Review

2.1 Background to literature review

This chapter presents the relevant background information to the current study. In order to gain a better understanding of natural forms of flood management; flood frequency, land management and the potential sources of runoff are discussed first. Stakeholder engagement and communicating issues of land use within small rural catchments is then discussed. The Pitt review (2007) stated that the Government should give priority to the adaptation and mitigation of flooding in a changing climate. A small section of this literature review looks at the impacts of climate change on flood frequency and severity. The majority of the reviewed material discusses various methods for managing rapid runoff in rural catchments. The drivers for the construction of mitigation vary from preventing muddy floods clogging the drainage systems of towns/villages using retention ponds, to increasing travel-time of peak discharge by driving water out of a river using woody debris, and the more long-term management of runoff through afforestation. The findings from the literature review will then be summarised, highlighting the remaining gaps in knowledge in the context of the present work.

2.2 Flood frequency, scale and land use change

There has been and continues to be concern that land use change and management in rural areas and agricultural developments have contributed to recent flood events, though there is little evidence at the large catchment scale (>10km^2) (O'Connell, et al., 2005) (O'Connell, et al., 2007).

2.2.1 Frequency of events

In the UK, summer is the dominant season for extreme events, albeit at the catchment and local scale rather than the regional scale (Hand et al., 2004; Newson, 1975). Here, extreme events are defined as intense and high magnitude rainfall events. Extreme events are of particular interest because they are accompanied by other hazards, including land-slides, mud flows, and loss of infrastructure and life (Collier, 2007), and there is the possibility that the frequency of extreme events may increase in the future (e.g. Dale, 2005; Frei et al., 2006). In the UK, flash floods having a time to peak of less than 3-hours within catchments of 5-10 km^2 are the main source of danger to human life (Collier, 2007). Recent extreme floods have brought into focus the vulnerability of communities in upland areas. In 2004, during a localised convective event in north Devon, 200mm of rainfall was recorded in 4-hours (Golding, et al., 2005), and 60 properties were flooded in the village of Boscastle (some were destroyed), with the insurance losses estimated at £50M (Gledhill, 2007). There was speculation that the severity of the flood may have been exacerbated by changes in land management (O'Donnell, et al., 2011).

2.2.2 Defining scale

Scale refers to the order of magnitude (as opposed to an exact number) of an area, length or time that defines a process, observation or model (Bloschl & Sivapalan, 1995). Scale varies temporally (in terms of time) and spatially (in terms of area/length) and, as a result, Klemeš (1983) stated that hydrological processes typically span about eight orders of magnitude. For example, the change in reach and duration of unsaturated flow in a 1m soil profile to floods in river systems of a million square kilometres; from flash floods of several minutes duration to flow in aquifers over hundreds of years (Bloschl & Sivapalan, 1995). Temporally, one could work on an event based scale (ranging from hours to days), a seasonal scale (e.g. a hydrological year) or a long term scale (e.g. 30 years). Spatially, the definitions of scale tend to vary depending on the study in question. For ease, the spatial terms of reference discussed in this thesis are outlined in Figure 2.1.

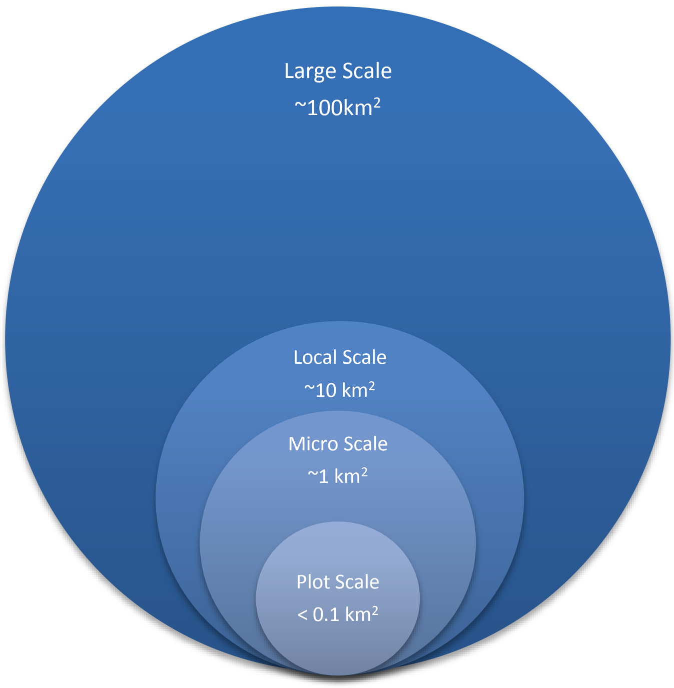


Figure 2.1: Definition of Scales within hydrology

The definitions outlined in Figure 2.1 show a range of scale from plots (e.g. hill slope experiments) to small and large catchments. Following the definitions set out in Figure 2.1 the spatial scale of catchments larger than 1000 km^2 will be referred to as the regional or macro scale. Spatial scale in hydrology greatly affects the ability to gather direct evidence from field-experiments. With an increasing spatial scale a greater amount of natural variability in the landscape makes it almost impossible to attribute land-use change with observations in recorded data (Bloschl, 2001).

2.2.3 Land-use change and case-studies

Modern tillage practices, including the removal of hedgerows to increase the size of fields, constructing under-drainage and ditching works, larger stocking densities and intense cultivation, alter the storage potential and connectivity of the landscape (O'Connell, et al., 2007) (demonstrated by Figure 2.2). Studies into these on-farm management practices have shown the need for reducing the rate of inundation on agricultural land.

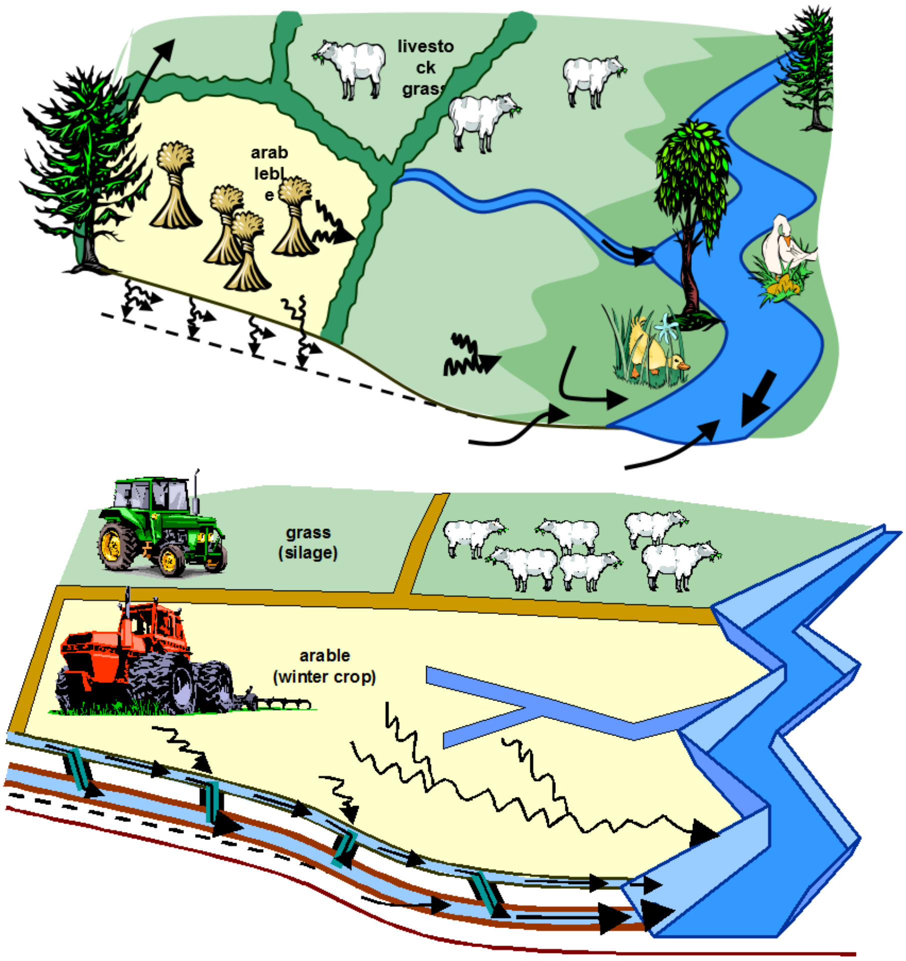


Figure 2.2: Modernisation and intensification of farming (O'Connell, et al., 2007)

Boardman (1995) presented a case study on damage to property in the South Downs, southern England, as a result of flooding and soil erosion, which discussed how changes to land-use had been the main cause of the damage. Land previously set aside for pasture, started to be ploughed during and after the Second World War and by the 1970s many of the farmers in the South Downs had abandoned livestock farming and traditional crop rotations. The introduction of autumn-sown cereals allows greater periods of time (several weeks or months after sowing) for runoff to occur, over the bare soils, during the winter months. Technological developments in farm machinery and production eventually led to enlarging fields by removing hedgerows and other boundaries to increase efficiency. This creates flow networks that become activated during large rainfall events and allow uninhibited flow between fields.

Following two major floods in the Po River in Northern Italy, which occurred in 1994 and 2000, Brath *et al.* (2006) theorised that the flood events were, in part, caused by an increased vulnerability brought about by land use change in the region over the last fifty years. Numerous studies have investigated the impacts that land use change, such as deforestation and urbanisation, have had on the vulnerability of flood prone areas (summarised in Patric & Reinhart, 1971).

An understanding of the impacts caused by land use change, with specific regard to the runoff generated at the local scale and the effect that the increased local runoff has downstream of the farm-land is necessary, as well as how to mitigate these effects using economically and environmentally acceptable methods. In some cases, however, this approach can lead to an over engineering of the natural environment (e.g. channelization and impoundment), which has an adverse effect on the ecological processes involved in the water cycle (Robarts, 1998).

When it comes to quantifying the effect of man-made interventions with respect to changes in flood peak, it will clearly depend on the nature of the examined flood event (Brath, et al., 2006). Hollis (1975) demonstrated that the effect of urbanisation on the peak discharge has a lower impact than increasing return period of the rainfall event. Hollis' conclusion is justified when considering that extreme flood events are produced by storms that induce soil saturation, therefore storage within the soil, allowed by infiltration, diminishes so rapidly that the impact of preventing this infiltration has little effect on surface runoff. An interesting result was obtained by Niehoff, et al. (2002) who discovered that the effect that land use has on storm runoff generation is larger for convective storms with high precipitation intensities, in contrast with long lasting advective storms with lower rainfall intensities.

Research investigating stream water quality in the UK found clear evidence of increased erosion rates from a study catchment (in Devon) since 1950 (Heathwaite & Burt, 1991). The changes were thought to reflect post-1945 intensification of agriculture, which include the modern tillage practices mentioned previously. Heathwaite & Burt (1991) also suggested that reductions in water quality could be attributed to an increase in stocking density from less than four livestock ha^{-1} between 1905 and 1950 to over fifteen livestock ha^{-1} in 1965 (in their study catchments). Intensification of livestock production may increase the level of farm effluents, pesticides such as sheep-dipping chemicals, as well as bacteria and protozoan contaminants, which in combination with increased overland flow due to soil compaction may increase the risk of water quality degradation (Hooda, et al., 2000). Investigations in the Netherlands have revealed that areas of grassland have been increasingly replaced by row crops (such as Maize and sugar beet), which are 15 to 20 times more susceptible to erosion than cereals (Van der Helm, 1987). A particular issue is associated with changing the timing of tillage operations leading to “muddy” floods (Boardman, 1995). These changes in land-use reduce natural attenuation within the catchment (Boardman, et al., 1994).

2.2.4 Mitigation techniques and stakeholder engagement

It is possible to mitigate on-farm impacts through good management practices that delay or attenuate runoff (O'Connell, et al., 2005). It was reported that land management changes have been very effective in dealing with muddy floods in West Sussex (Evans & Boardman, 2003). The runoff that propagated straight over bare arable fields was filtered in some areas using grassland buffers and slowed by zones that disrupted water connectivity. Alternative farming practices implemented at the field scale, such as the sowing of cover crops during the intercropping period and reducing the density of ploughing and sowing in areas prone to concentrated flow help limit runoff generation and erosion production (Gyssels, et al., 2002). However, implementation of some of these practices is dependent on the farmer’s willingness to adopt them (Evrard, et al., 2008).

Stakeholder engagement is an important aspect of managing flood risk with regard to land use management. The Floods and Agriculture Risk Matrix (FARM) is a decision support tool designed to allow farmers and land use planners to investigate the sensitivity of certain areas of land, with regard to flood risk and pollution, and the respective management practices that occur on them, as well as highlighting measures that can be implemented to mitigate the risk (Posthumus, et al., 2008). FARM was originally designed to show the nature of the problems associated with specific farming practices, such as effluent and pollutant export from farmland as well as flood generation, and to suggest strategies to resolve these problems (Quinn, 2004).

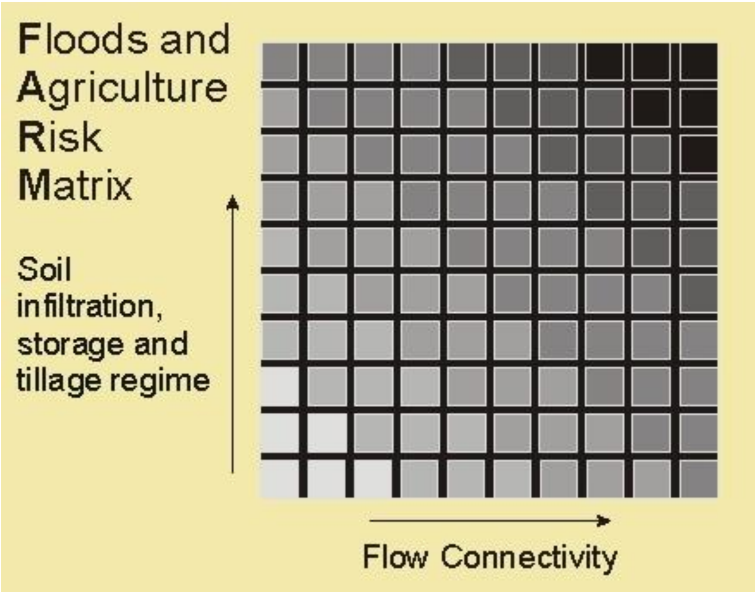


Figure 2.3: The FARM tool (Posthumus, et al., 2008)

The FARM tool follows an interactive interface and captures the likelihood that current farming practices may generate high runoff; thus forcing the user to choose options that will lower the runoff risk (for example, transforming the situation from bad practice, in Figure 2.4, to good practice, in Figure 2.5) (Posthumus, et al., 2008). The computer-based toolkit contains a set of questions associated with each axis; allowing the user to answer the questions according to the current or proposed management of a particular field or farm. Throughout this process, example visualisations (e.g. Figure 2.4 and Figure 2.5) are provided to the user to relate to the level of risk on the FARM matrix. The final position plotted on the matrix depends on answers to all of the questions. If the user ends up with a high risk on either axis of the matrix then changes in practice that could reduce this risk should be considered.

Figure 2.4: The FARM tool demonstrating poor farming practice

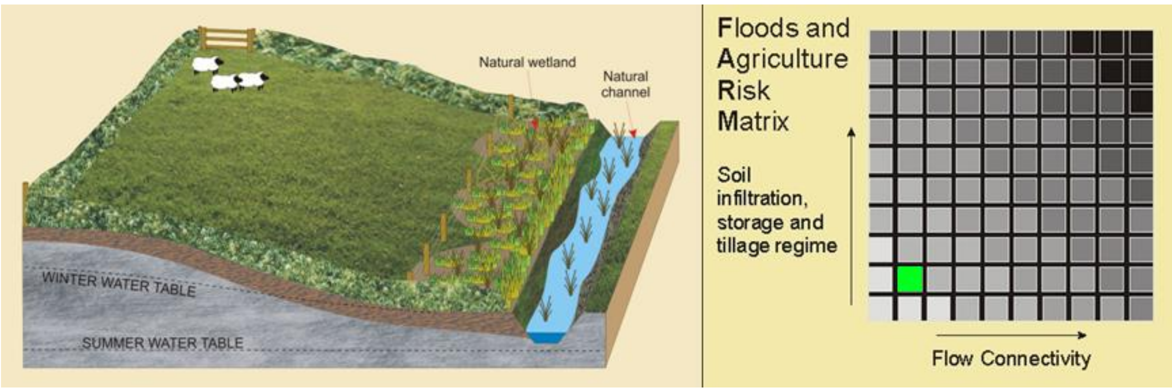


Figure 2.5: The FARM tool demonstrating good farming practice

Stakeholder engagement tools are extremely useful methods of targeting problems and identifying solutions for better land management. Forums for discussion regarding land use and flood risk are also important for identifying areas to target within the catchment disseminating findings, as well as to promote community understanding of the interactions between natural resources on the catchment scale (and how the ecosystems approach to management can benefit natural resources). Johnson, et al. (1996) discuss the establishment of Catchment Care Groups and Catchment Coordinating Committees to act as this type of forum and enpower communities with direct links to land management within their own catchments. Community involvement and grassroots decision making is promoted by Agenda 21 of the Earth Summit (Grubb, et al., 1993).

Dawson, et al. (2011) discuss a method for appraising the benefits of non-structural flood defences, in terms of reduction to economic flood risk, over the extended timescales that they operate. The purpose of this analysis was to communicate long-term reductions in risk to planners and policy makers, so as to avoid decisions that are undesirable in terms of flood risk or lock out the opportunity for alternative actions in the future. They identified that schemes like rural runoff reduction and upstream water storage can lower the overall probability of flooding, in their flood risk calculation, due to changes made to the system over time.

2.3 Natural Flood Management (NFM)

In recent years, emphasis has shifted in the UK to Catchment Flood Risk Management Planning, which will form the basis of delivery of the Flood Risk Management plans required by the EU Directive on the Assessment and Management of Floods (Ball, 2008). It was stated by Evans et al. (2004) that two of the most “sustainable” ways of managing flood risk were better land use planning and catchment-wide water storage. Natural flood management (NFM) is the alteration, restoration or use of landscape features to reduce flood risk (POST, 2011). NFM also aims to provide the framework and the tools necessary to identify and mitigate flood risk. Working with natural processes means taking action to manage flood risk by protecting, restoring and emulating the natural function of catchments, rivers and floodplains (Environment Agency, 2010).

There has been recognition, over recent years, that a holistic approach should be taken to effectively manage river channels at the catchment-scale (Newson, 1997), with the integration of all hydrological processes between land and the river network – as changes in any one aspect of catchment management practice may have impacts of a wide range of catchment processes. Lane, et al. (2003) identify the need for hydrological processes (such as flood flows, sediment capture, water quality and habitat creation) to be linked and managed together as opposed to dealing with them separately. In a recent Parliamentary Review (POST, 2011), a range of NFM approaches were classified based on the location of likely deployment and how the strategy may be distributed on the ground (Figure 2.6). In this classification the RAF approach lies in the top left quadrant, being a **spatially diffuse** approach that works close to the **source** of runoff generation.

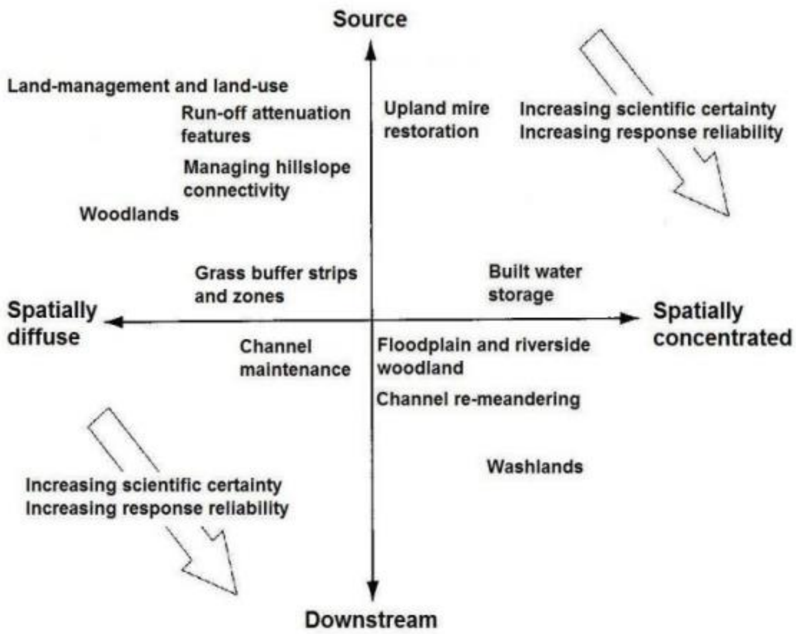


Figure 2.6: Catchment-scale classification of NFM strategies (POST, 2011)

The possibility of increased flood frequency, as a result of climate change, is a driver for the development of future flood management techniques. The provision of hard-engineered flood defences can be expensive, aesthetically unpleasing and damaging to local ecology. Also, many hard-engineered flood defences are built for floods of a certain return period and would be highly expensive to alter for a more severe flood event. This has led to research into soft-engineered flood management schemes, such as retention ponds, floodplain woodlands, woody debris and wetlands.

Ideally a large part of NFM would consider floodplain interactions within a catchment. For most catchments, however, floodplains have attracted intense industrial, housing and agricultural development over the centuries. Naturally and historically, these are the areas that would accommodate high magnitude flood flow and, ironically, must now be prevented from flooding.

2.3.1 Hydrological connectivity

Hydrological connectivity is the term used for describing flow pathways for the movement of water on the surface and through the subsurface of the landscape. It also describes the extent to which water and matter that moves across a catchment can be stored within or exported out of the catchment (Lane, et al., 2003). Amoros & Bornette (2002) explain that hydrological connectivity may take one or more of four types: latitudinal, longitudinal, vertical and temporal. Of particular concern, in this section, are latitudinal and longitudinal surface connectivity which lead to the rapid delivery of runoff to the drainage network, combined with the possible entrainment of particulate matter (e.g. soil) from the land surface. When zones of surface connectivity are disconnected from the drainage network, then the effects of vertical connectivity and latitudinal and longitudinal subsurface connectivity become the principal flow paths, however, much more subtle in comparison (Lane, et al., 2003).

2.3.2 Retention ponds

In the village of Rillaar (Northeast of Leuven, central Belgium) storm runoff had reached a level that was beyond the capacity of the drainage network. Retention ponds have been used in this region to hold runoff for a certain amount of time, which limits the peak discharge to a level that is manageable by the drainage system (Verstraeten & Poesen, 1999). Unfortunately the study in Belgium did conclude that after a few years, the retention ponds can fill up with sediment, which reduced the available storage. This highlights the need for management of the maintenance of these features. It is worth noting, however, that in this region 'muddy' floods are a huge problem to homes and that, due to the multiple benefits of retention ponds, most sediment from the river is prevented from entering the drainage system. The main benefit found in the Belgium study was related to the economics of flood mitigation techniques. The retention ponds provide flood defence for the village directly adjacent to them, but also help protect other villages further downstream.

Further studies into muddy floods in the European Loess belt have been investigating the impact of grassed waterways and earthen dams as a means of controlling runoff and filtering flow (Evrard, et al., 2008). The plot-scale experiments observed a significant reduction in peak discharge (mean of 69%) between the start of the series of interventions and the catchment outlet, shortly after the final retention pond. Evrard, et al. (2008) present a hydrograph of the inflow at the flume, preceding the mitigation, being augmented by the presence of the earthen dams that act in sequence. Runoff coefficients had also dropped (mean of 40%) in the direct vicinity of the grassed waterways, which was linked to increased infiltration. The system in Belgium successfully reduced both downstream discharge and sediment discharge at the outlet of the 3km^2 catchment using cost effective mitigation. Evrard, et al. (2008) concluded that a catchment without intervention would suffer up to seven-times the rate of erosion than one with these measures.

Similar techniques were tested in the Zwettl/Kamp catchment, Austria, as part of the CRUE project. Microponds were used effectively to manage hill-slope runoff (CRUE, 2008). The structure of the microponds is slightly different to retention ponds, due to the lack of outlet flow (provided either by a pipe or through the structure). Instead, these microponds would drain very slowly by constantly percolating into the soil. This method separates the fast runoff stored in the ponds from the rest of the water in the storm hydrograph due to the slow groundwater processes taking place. It means, however, that the micropond would be useless if two extreme rainfall events were separated by, for example, one day. The microponds discussed in the CRUE (2008) report have an average storage capacity of $100m^3$, which means there have to be (and are) thousands located throughout the Zwettl/Kamp catchment in order to have an impact on attenuating hill-slope runoff.

A hydrological modelling study by Niehoff, et al. (2002) experimented with setting 10% of farmland aside for no production, based on Agenda 2000 – a European Union resolution, which required the setting aside of production land on farms from 2000-2006. The study defined the non-production land as almost bare (open) land, with sparse vegetation, because no crops were cultivated in the area, and parameterised the land as having low interception capability. Not surprisingly, the results showed marginal change in the hydrograph response for historical storm events and, in some cases, noted a greater runoff response from the set aside fields (Niehoff, et al., 2002). Admittedly, the vegetation cover on the set aside fields would likely increase over the years; however, this highlights the need for a certain degree of intervention rather than, simply, a reduction of intensification. A series of experimental investigations on Nafferton Farm made some similar conclusions on the amount of intervention required in a catchment to control flow and pollution. If a typical farm or small catchment can sacrifice 2-10% of the landscape to runoff storage and mitigation features, then the properties of the runoff regime would be dramatically altered (Quinn, et al., 2007). The mitigation features are placed in the corners of fields (to capture runoff and sediment) or connected to the stream network to filter stream flow and improve water quality. It has been argued that on-farm interventions like these offer greater flexibility and reversibility than traditional engineered defences, which is important in an uncertain world. Also, these interventions may give wider environmental benefits, capturing sediment and, potentially, reducing diffuse pollution (Barber & Quinn, 2012). Though, there is still little evidence regarding the effectiveness of these interventions at the catchment scale.

Catchment scale modelling simulations of land use management in Pontbren, Wales, have demonstrated the effects of improved and unimproved grassland, and the potential effects of land use management interventions including storage ponds, and tree shelter belts and buffer strips. Results from the Pontbren study have indicated that careful placement of such interventions can significantly reduce the magnitude of peak runoff at the field and small catchment scale (Wheater, et al., 2008).

2.3.3 Floodplain Woodlands

Floodplain woodlands use the same principles as retention ponds by acting as a permanent or semi-permanent wetland for storing flood water produced by extreme rainfall events and delay the downstream passage of a flood peak (Thomas & Nisbet, 2007). Similar to the retention ponds, described previously, floodplain woodland offers a wider range of benefits including improvements to water quality, nature conservation, fisheries and landscape (Kerr & Nisbet, 1996).

The primary function of floodplain woodland is to delay downstream passage of the flood peak, resulting in a lower but longer duration event beyond the intervention, aiding mitigation further downstream (Figure 2.7). This theory is not unique to floodplain woodland alone, however, and provides a sensible estimation of the attenuating effects of many other forms of floodplain storage. The ability of floodplain woodland to delay flood peak is derived from the effect of vegetation roughness. The characteristics of the vegetation negate the type of frictional effects it has upon the flow of water. Trees, for example, create more of a physical barrier than smaller shrubs and bushes because they have greater strength to remain upright during flood flows. Floodplain woodland has the potential to be designed in such a way to provide optimum effect on flood flow by varying the spacing and layout of trees, the level of undergrowth and the degree of trash on the woodland floor, and by introducing trees that grow lower-level branches (Thomas & Nisbet, 2007).

A case study by Thomas & Nisbet (2007) used hydraulic modelling to demonstrate the mitigation effect that two stretches of floodplain woodland within the Parrett catchment (Somerset, UK) had on downstream flooding of towns and villages. The study ran two models (HEC-RAS and River2D) with floods of varying return period whilst applying changes to the value of Manning's *n* to represent the frictional effects of different levels of woodland cover. The model output showed that the woodland had huge effects on reducing the mean water velocity (by 60-70%) and by delaying the time to peak of the flood (by up to 140 minutes). It should be noted, however, that a change of this magnitude induced on the floodpeak is a result of optimum, or near optimum, conditions. The graph in Figure 2.7 shows an ideal outcome from the implementation of floodplain woodlands, but if the threshold of the system is overcome it will greatly impact the effectiveness of the floodplain woodlands as a flood attenuation feature. Unfortunately, in relation to the hydrological scientific objectives, the lack of established floodplain forest within the Parrett catchment meant that much reliance is placed on modelling to justify planting (Ball, 2008).

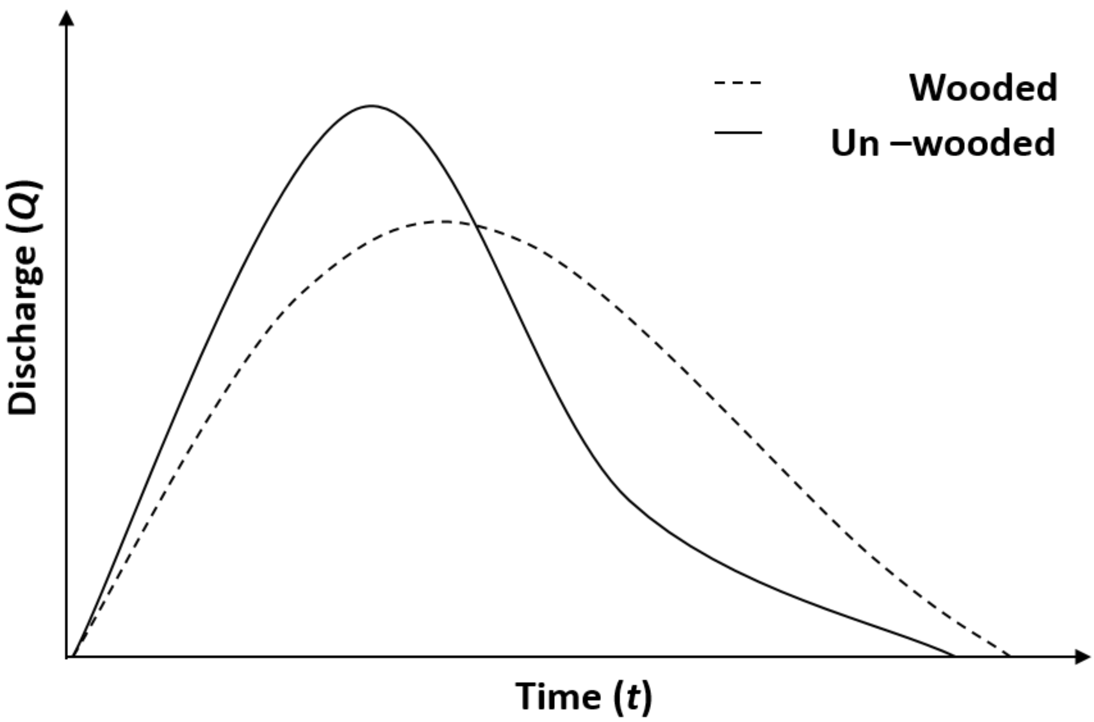


Figure 2.7: Theoretical hydrograph showing the possible effect of floodplain woodland (or other floodplain storage) on flood flow (modified from Thomas & Nisbet, 2007)

2.3.4 Large Woody Debris (LWD)

Dudley, *et al.* (1998) discuss the presence of large woody debris (LWD) and vegetation in riparian zones and the effect they have on sediment transport, nutrient cycling and other geomorphologic processes, and the ability of collections of LWD to form microenvironments for terrestrial and aquatic organisms during extremes in weather. Flume studies by Shields & Gippel (1995) concluded that the presence of LWD in the channel increased the Darcy-

Weisbach friction factor for near-bank-top conditions by 20-30% and decreased bank-top flow capacity to between 5 and 20%.

LWD can have huge effects on flow resistance using soft engineering techniques, which have very little impact on the ecology of the area. During states of high discharge, LWD forces the water level, in proximity to them, to rise and spill onto the flood plain. This process slows the propagation of the flood peak by creating a far more tortuous route downstream. One approach to quantifiably measure the hydraulic effect of woody debris is to use a flow resistance equation, in which it is assumed resistance to flow produced by the debris is expressed as a roughness coefficient or friction factor (using Manning's n). The hydraulic resistance of woody debris varies as a function of flow depth (Gippel, 1995). It has been shown by Beven, et al., (1979) that when LWD is greater in size than the flow depth, the roughness coefficient is abnormally high (Manning's $n > 1$). As the flow depth increases and the LWD becomes submerged, its effect on resisting the flow diminishes.

2.3.5 Wetlands

Runoff generated by precipitation events carries with it materials, be it nutrients, pollutants or even sediments, which cover the land leading to river networks. Wetlands and riparian zones can control the effect this runoff has on river ecology. By filtering the runoff water and sediment flows, wetlands can improve the river water quality and reduce erosion rates as well as help slow down runoff flow, thus mitigating flood risk (Mander *et al.*, 1997; Maitre *et al.*, 2003).

Wetland creation and re-establishing floodplain in the Till catchment, Northumberland, has reported high water treatment efficiency in low to medium flow conditions (93% removal ratio for faecal coliforms, 44% for NH_4^+ and 59% for NO_3^-). However, these effects were lost at higher flows, which has demonstrated the need for further consideration over the design of the morphometry linking the wetland cells (Ball, 2008).

2.3.6 Afforestation

Many upland areas in the UK have been cleared of their natural forest cover due to historical demand for fuel, building materials and to expand land available for grazing. Upland woodlands have the potential to buffer intense rainfall via interception and root uptake.

Certain species can also prevent snowmelt from occurring as rapidly as it would in the open, due to the shade provided by the canopy. Riparian woodland in lower parts of a catchment have to potential to help force river water onto the floodplain and act as a leaky barrier to flood flow by trapping debris and forming natural dams. These woodlands tend to rely on strong species that thrive in wet soil.

Models examining the water slowing and storage effect of woodland at the catchment scale have shown that there can be flood risk benefits, though these depend on the return period of the flood and the distribution and density of the planted area (POST, 2011).

2.3.7 Sustainable Urban Drainage Systems (SUDS)

Sustainable Urban Drainage Systems (SUDS) encompass much of the rationale of NFM, both in terms of the aims they set out to achieve (reducing flood risk, water quality and water harvesting) and the facilities and scales in which they operate (from green roofs on individual buildings to storage ponds and culverts mitigating against entire developments of impermeable land).

The traditional method for draining excess surface water from built-up areas has been through underground pipe systems (Woods-Ballard, et al., 2007). These systems are designed to prevent local flooding by conveying the drainage water away as quickly as possible. Historically, surface water runoff has been combined with sewage flows through a single, combined sewer. However, a significant and unpredictable strain can be forced upon wastewater treatment works during heavy rainfall events, which forces some untreated sewage to spill into receiving watercourses from combined sewer overflows (CSOs) (Woods-Ballard, et al., 2007).

Many traditional drainage systems have not been designed with sustainability in mind and, often, have proved insufficient at handling extreme rainfall events. Issues are often exacerbated when settlements expand, which leaves the drainage systems completely under-designed. SUDS have the capability of working alongside these traditional drainage systems and provide additional protection during storm events where surface runoff has been generated. SUDS can take the form of large empty ponds, which have a certain available capacity for capturing runoff during storm events. In many cases the runoff is delivered to SUDS via outfalls connected to the drainage network of an urbanised development. This allows all excess flow from the drainage system to be deposited into the pond for a period of time. The water is allowed to drain from the SUDS continuously, though the rate at which this occurs is dependent on the river level at the opposite end of the SUDS outflow pipe. This process readies SUDS for the possibility of further rainfall and another potential runoff event. The concept is very similar to the retention ponds (see 2.4.2) in that the water is not stopped from propagating downstream, but it is merely slowed from reaching downstream locations as fast as it would have without their protection. This has the effect of elongating the storm hydrograph along the time axis and reducing the peak flow of the recorded discharge.

2.3.8 Eco-hydrology

The Intermediate Disturbance Hypothesis (Connell, 1978), suggested that the greatest diversity of biotic communities appears at the intermediate level of abiotic (e.g. hydrological) disturbances. Thus, increasing the variability of physical factors, as a result of mechanistic, engineered solutions in water management, reduces the terrestrial storage capacity of river basins for nutrients and sediments. This has the added effect of causing decline in biological diversity and productivity of terrestrial ecosystems and can produce an increase of fertilisation and siltation in river channels, reducing the storage capacity of aquatic systems, which increases the risk of flooding (Zalewski, 2002). This issue highlights the need for a more holistic framework linking ecology with hydrology, especially since these processes are in danger of becoming amplified in years to come based on climate change scenarios.

Human interference with natural systems such as agriculture, forestry, settlements and road construction has led to further complexities in flood modelling. Clearly these direct anthropogenic interferences will have instantaneous effects on flooding. In the long term however, the indirect anthropogenic influences, such as the consequences of deforestation or changes in the availability of water may lead to the enhanced greenhouse effect altering the biosphere and creating more extreme weather events (Bronstert, 2003).

2.4 Summary

In the UK, summer has been identified as the dominant season for extreme flood events (at the local scale). These events can lead to flash floods with a time-to-peak of less than 3-hours in catchments of 5-10km^2. In addition, climate change has the potential to increase the frequency and magnitude of extreme events. Accurately quantifying rainfall runoff response is a necessary step for determining the types of intervention required to mitigate against it.

A vast amount of qualitative evidence from local scale studies has demonstrated the negative impact of post-war agricultural intensification upon downstream flooding. Modern tillage practices have potentially altered the natural attenuation and increased the connectivity of the landscape. There is, however, a lack of evidence gathered from multi-scale nested hydrometric networks to quantify land use changes and their interventions. An understanding of the impacts caused by land use change, with specific regard to the runoff generated at the local scale (during extreme events) and the effect that the increased local runoff has downstream of the farm-land is necessary, as well as how to mitigate these effects using economically and environmentally acceptable methods.

Good management practices, such as inclusion of grass buffers between fields and watercourses; ploughing perpendicular to the direction of slope on an inclined field; and alternative cropping patterns have the potential to delay or attenuate runoff. Mitigation in this form, however, is difficult to quantify. Communicating 'good' practice with land owners and other stakeholders is important, and the use of visual aids (even if they are conceptual) are effective methods of demonstrating better practice and mitigation. The ability to present evidence-based models from similar catchment studies will provide yet another tool for communicating types of mitigation.

Two of the most sustainable ways of managing flood risk are better land use planning and catchment-wide water storage. Natural flood management (NFM) encompasses a range of methods in which to modify hydrological processes at a range of scales. The key function of NFM is to link hydrological processes and take an holistic approach to their management. Under this wider description RAFs have been classified as being a spatially diffuse approach that work close to the source of runoff generation.

It is important to note the varying timescales of different NFM techniques. Woodland restoration will take at least 10 years to become effective, though will require little maintenance once established. On the other hand, retention ponds, whilst providing immediate benefits, may require maintenance to ensure they do not fill with sediment. Large woody debris may require routine inspection to check that the mitigation material is not transported downstream.

The approach used for Belford (Chapter 7) attempts to combine many different forms of flood mitigation to draw from the strengths of a host of techniques, providing that they are installed in the most appropriate locations within the catchment. Ensuring that this is the case will require the successful monitoring of the catchment (Chapter 4) to identify how certain areas respond to rainfall (Chapter 5), and demonstrating the type of storm event that typically causes flooding in the catchment (Chapter 6).

3. Catchment description

3.1 Introduction

The Upper Belford Burn catchment, Northumberland, was chosen for this study because (i) it had an existing flooding problem, with records of flood events dating back to 1877; (ii) previous studies for the Belford Burn had suggested that traditional flood defences would not be viable on the basis of cost-benefit analyses (Halcrow, 2007); (iii) the Environment Agency (EA) had obtained funding for a non-traditional flood defence scheme (in the form of Natural Flood Management); and (iv) an existing monitoring network had been installed by Newcastle University to gather observations from the non-traditional scheme. This Chapter will identify key geophysical and hydrological characteristics of the Belford catchment and explain the nature of the flooding problem in Belford Village. It will be demonstrated over the subsequent three-chapters that a better understanding of catchment processes, through an extensive monitoring regime, can be achieved and compared to the information gathered in the desk study shown in this chapter.

3.2 The Belford Study

3.2.1 Background

The Belford Burn flows 10.5 km from its source, near Bowden Crags, before discharging into the North Sea at Budle Bay (Wilkinson, et al., 2010a). The study area focuses on the Belford Burn catchment upstream of the village of Belford ($5.7km^2$), (Figure 3.1). The Belford Burn drains from the Bowden Crags (185m AOD) in an easterly direction for 4.5 km to the village of Belford. The catchment is predominantly rural and has an average elevation of approximately 115m. The village has experienced a number of flood events in the years preceding this study. The A1 and East Coast Mainline railway are immediately downstream of Belford village and have both been impacted by flooding from the upper Belford Burn catchment, which prompted engineering works to be carried out including concrete flume sections and new culverts that are still present in the existing channel. The Environment Agency (EA) obtained funding for a non-traditional flood defence scheme, which began in late 2008. Figure 3.1 also shows the locations of the River gauging stations, which have been included here for use as reference points within the study area. Rainfall runoff records from these gauging stations date back to January 2008. Monitoring will be further discussed in Chapter 4.

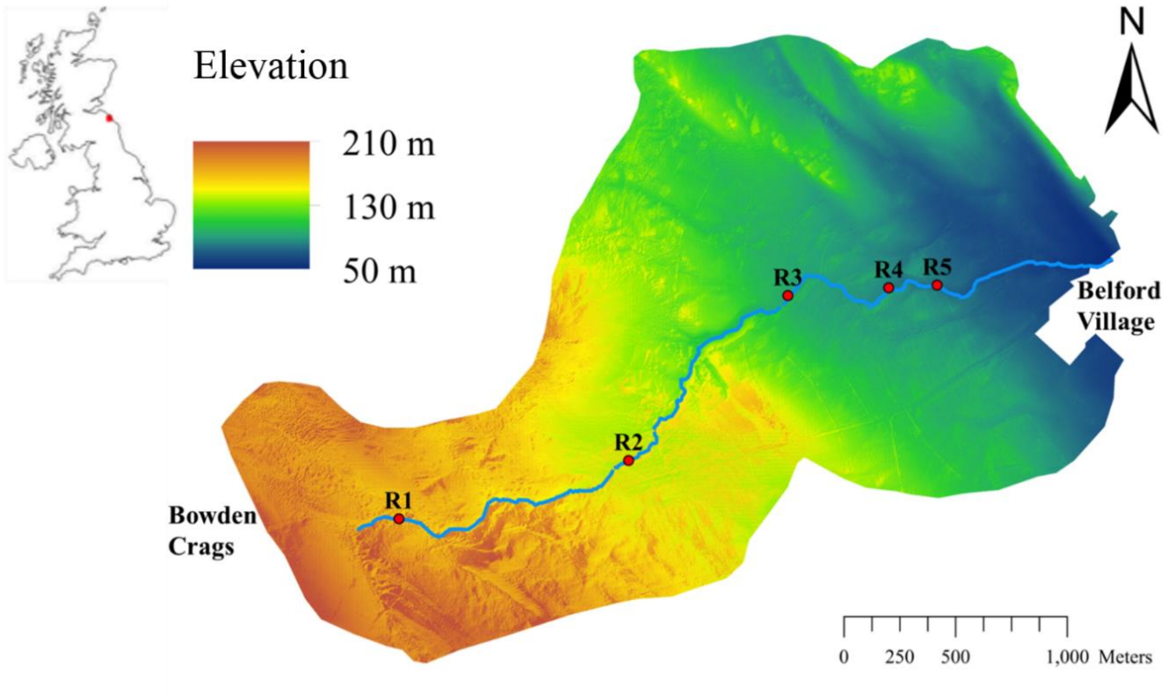


Figure 3.1: Belford catchment topography – showing the location of river gauging points (R1-R5)

3.2.2 Previous studies and information

There have been four previous studies carried out on the Belford Burn, which are summarised here:

i) A study by JBA for Railtrack Plc., investigating the flood event of July 1997, which inundated the East Coast mainline and was estimated to have had a 10-20 year return period
ii) A flood risk assessment by JBA in 2002 associated with the redevelopment of South Garage (a car servicing business) on Belford High Street
iii) A flood risk mapping study in 2004, which concluded that a number of properties are at risk during the 1% annual probability event. It recommended a threshold level survey to more accurately determine the flood risk to the properties.
iv) A prefeasibility study by Halcrow, based on the flood risk mapping performed by JBA, to determine the best solution to Belford's flooding problem (see 1.2).

In addition to these studies, there have been numerous consultations with the EA during Newcastle University's involvement with the project. A further report has been undertaken by Royal Haskoning and Newcastle University, using findings from this PhD project, which presents the Belford 'approach' as a viable method of dealing with flooding in small rural catchments (Quinn, et al., 2013).

3.3 Geology and Soil

3.3.1 Rock formations

Figure 3.2 shows the main geological features of the Belford catchment. The main solid geological strata at the far west of the catchment (where the elevation is greatest) are Fell Sandstone Formation, which forms the Bowden Crags, and Scremerston Coal Member. The geology to the east is predominantly Tyne Limestone Supergroup with interwoven Tyne Limestone formation, Woodend Limestone and Watchlaw Limestone until reaching the area close to the R2 gauging station. Alston Formation becomes the dominating bedrock for the east of the catchment beyond R2. A strip of oxford limestone cuts through the centre of the catchment (in the riparian zone between R2 and R3) and sweeps along the south-east of the catchment. Several faults follow a similar pattern to the Oxford Limestone at this point in the catchment. The presence of these geological features has also created sinks within the riparian zone. The northern segment of the catchment sits upon Great Whin Sill with a small section of Oxford Limestone Member between it and the Alston Formation. A major fault intersects the bedrock from the west to the east of the catchment. The presence of resistant rock outcrops in the far west, the north and north-east of the catchment has led to the formation of steep promontories in contrast with the relatively gentle gradients of the rest of the catchment (see also the topography in Figure 3.1). The exposed rock and steep slopes present at the far extents of the catchment have the potential to lead to a faster runoff response in the more upland areas.

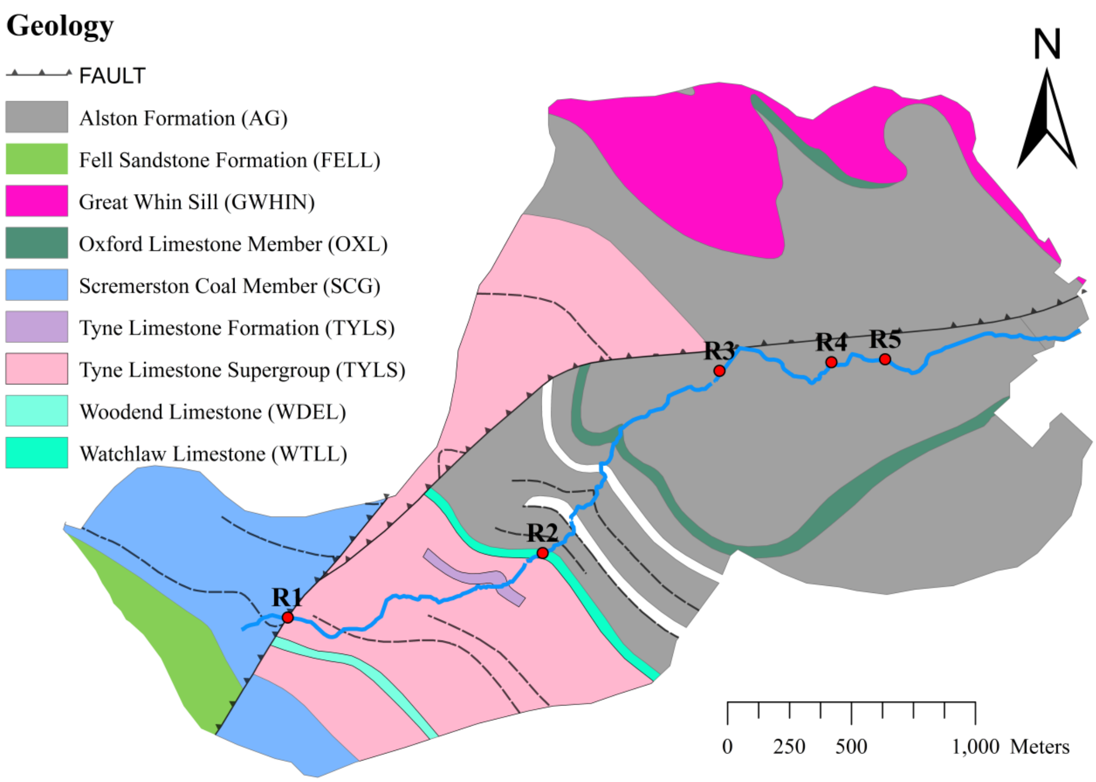


Figure 3.2: Geological map of Belford (Data from NERC – Sourced from EDINA Digimap)

3.3.2 Soil and superficial deposits

The soils in Belford are described as slowly permeable and seasonally wet (using Soilscapes, Cranfield University). There are extensive boulder clay deposits overlain by deep slowly permeable soils (Stagnogleys) belonging to the *Dunkeswick Association* throughout the Belford catchment (Jarvis, et al., 1984). Shallower, better drained soils are found where the Fell Sandstone and Dolerite outcrop. Reconnaissance surveys conducted in Belford have confirmed the accuracy of this mapping (Palmer, 2012), although the soils in the upper catchment in particular are dominated by the fine loamy *Brickfield Series* member of the *Dunkeswick Association*, and there are well-drained soils over a small outcrop of Dolerite in the north west of the catchment. Small areas of peat are present at the top of the catchment, upstream of the R1 monitoring station. This association consists of a deep, wet organic blanket, which is perennially waterlogged. These areas of peat, however, are unlikely to have any significant implications for catchment hydrology (which is likely to be controlled by the response of the boulder clay soils that cover 90% of the catchment area). The dominant soils in the catchment are relatively deep, but have limited infiltration below approximately 60 cm (Jarvis, et al., 1984). The active hydrological zone is therefore largely limited to within 1 m of the ground surface.

3.4 Land-use

Topography has a strong influence on the distribution of land use within the Belford catchment (Figure 3.3). Despite having impeded drainage, Belford's soil is able to support grassland, arable fields and some woodland. The dominant land use at lower elevations, specifically in the southeast of the catchment, is arable rotation – primarily cereals. The upper catchment (to the west) is predominantly utilized for permanent pasture (sheep and cattle), rough grazing and coniferous plantation on steeper slopes. There is a mixture of deciduous and coniferous woodland along the main stream corridor. Three farmers manage the agricultural land within the upper Belford Burn catchment.

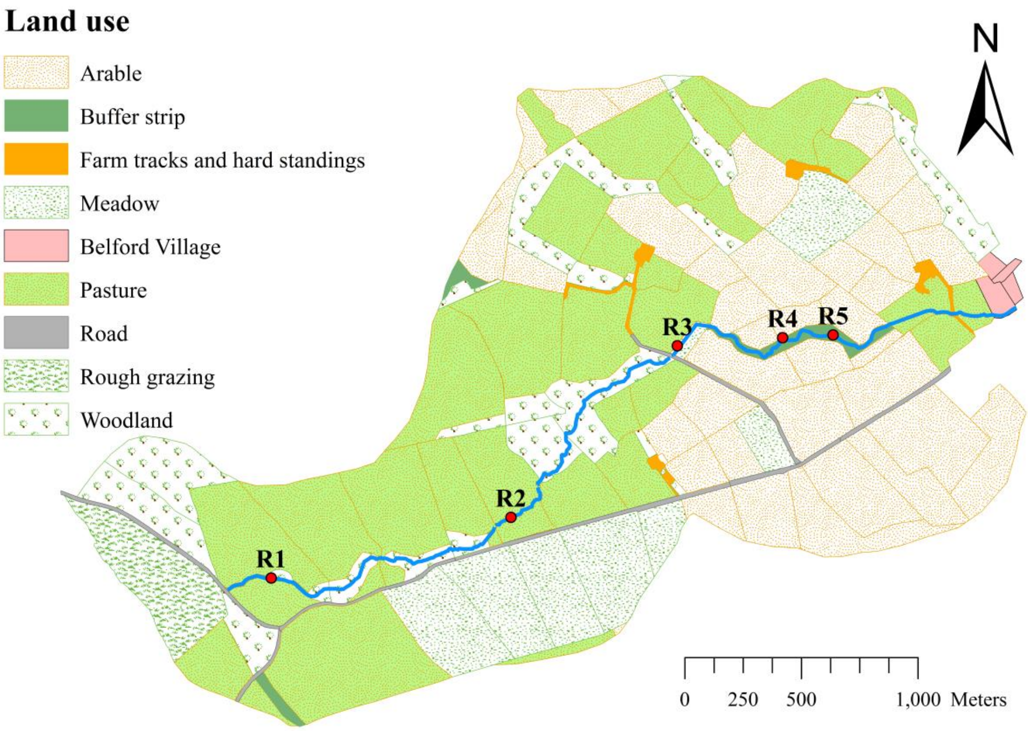


Figure 3.3: Land-use map of Belford (Palmer, 2012)

There are small pockets of woodland scattered throughout the Belford catchment. In recent years there have been efforts to introduce less intrusive species of trees into the woodland in the centre of the catchment. Intrusive sycamore trees have been replaced with slower growing species, including holly, which do not reach as high in the canopy and are known to improve the habitat for birds and small mammals.

3.5 Mean annual statistics

Standard desk studies assessing flood risk for a catchment will consider key descriptors for the area. These descriptors form the variables of a routing model that will generate storm hydrographs based on a unit hydrograph approach (i.e. how the simulated catchment will respond to a certain amount of rainfall acting over a set time-period). The following information (see Table 3-1) is from the Flood Estimation Handbook (FEH) catchment descriptors for Belford:

Characteristic	Description	Value
Physical attributes:		
Area	Catchment area upstream of Belford village (km^2)	5.7
ALTBAR	Average elevation (m)	117
DPSBAR	Mean slope (m/km)	62.5
FPEXT	Fraction of floodplain extent (0:1)	0.0427
URBEXT	Fraction of urban extent (0:1)	0.0
Hydrological attributes:		
SAAR	Seasonal annual average rainfall (mm)	695
BFI HOST	BFI (Base Flow Index) derived from HOST soil classification (%)	0.313
PROPWET	Proportion of the time when SMD (Soil Moisture Deficit) was less than or equal to 6mm (%)	0.45
RMED-1H	Median annual maximum rainfall for 1 hour (mm)	8.7
RMED-1D	Median annual maximum rainfall for 1 day (mm)	33.3
RMED-2D	Median annual maximum rainfall for 2 days (mm)	42.9
SPRHOST	Standard percentage runoff derived using the Hydrology of Soil Types classification (%)	40.75

Table 3-1: Belford Catchment Descriptors [FEH]

The Standard Percentage Runoff (SPRHOST), defined as the percentage of rainfall that causes a short term increase in flow, is 40%, and the Baseflow Index (BFIHOST) is, the long term average of flow that occurs as baseflow, is 0.313 (IH, 1999). The relatively high baseflow index can be attributed to the presence of the permeable rock formations (e.g. limestone) within the geology of the catchment. The time to peak (TP), a measure of the time between the flood-producing rainfall and the resulting flood response, is 2 hours. These attributes are typical of rapid response catchments that are prone to flooding (Environment Agency, 2011). Long term mean annual rainfall is 695 mm, but this value can vary significantly from year to year.

The Belford catchment descriptors (from Table 3-1) were used to generate storm hydrographs in the Revitalisation of FSR/FEH Rainfall-Runoff (ReFH) method (e.g. Figure 3.4). The hydrograph indicates that a high proportion of the total flow generated in the Belford catchment is produced from direct runoff, whilst the baseflow component carries approximately 10% of the total flow during the peak of the event (based on the given catchment descriptors from Table 3-1). This indicates that upland catchment management is a reasonable approach for mitigating flood risk downstream. The proportion of runoff is controlled by a number of factors, but predominantly the standard percentage runoff (SPRHOST) and the proportion of urbanised land within the catchment, which is negligible in Belford. The baseflow component of the routing model is controlled by BFIHOST and PROPWET. The higher these two components, the greater proportion of the total flow will be taken as baseflow. Chalk-dominated landscapes are dominated by baseflow, whereas Belford has a proportion based on the small collections of limestone in the centre of the catchment.

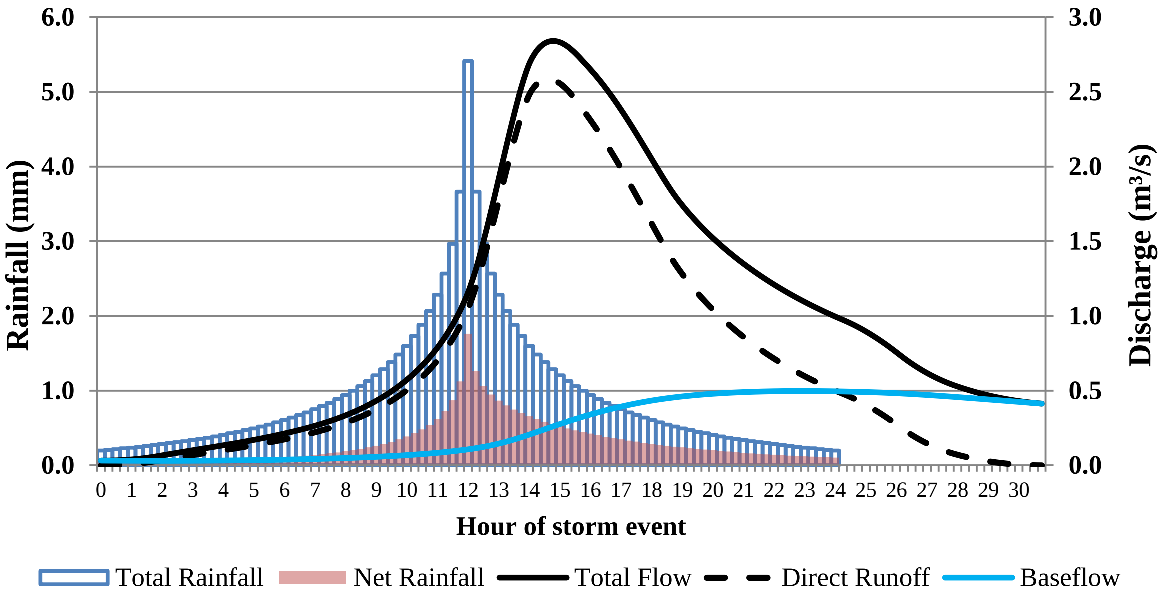


Figure 3.4: Storm hydrograph generated using FEH methods (1:100 summer storm event)

3.6 Flood risk in Belford

Simulated flow from storm hydrographs generated using the FEH catchment descriptors (e.g. Figure 3.4) can be used as input into hydraulic models. The output from an EA commissioned model can be seen below (Figure 3.5).

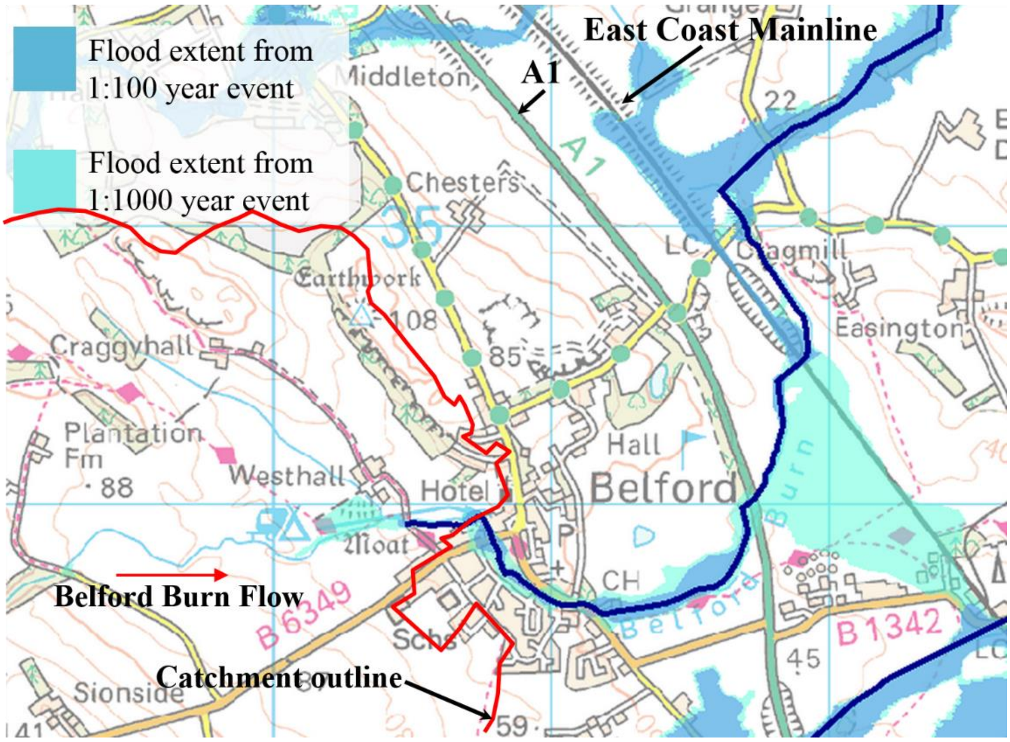


Figure 3.5: EA flood inundation map (Upper Belford Burn catchment outline shown in red)

Flooding in Belford presents a risk to 34 properties, a caravan park and two major transport links, which are the A1 and the East Coast Mainline railway between Newcastle and Edinburgh (Figure 3.5). The East Coast mainline had to be temporarily shut down due to a flood event occurring at Belford in July 1997. The EA have identified 31 properties at risk from flooding, based upon the 1:100 year return period flood, though it has been found that several properties are at risk of flooding as a result of only 1:5-1:20 year return period flood events (Chapter 6 – 6.2).

Notable events in October 2002, January 2005 and July 2007 caused flooding of properties, infrastructure and local businesses. The 2007 flood was reported widely throughout the region, with several other towns and villages being affected (including Morpeth). The Northumberland Gazette published the headline “Sick of sandbags and sympathy” on the 12th July 2007, to highlight the villagers’ angry reaction to the flood.

Figure 3.6: Belford Village during flood event in summer 2007

Housing and other properties have been situated extremely close to the riverbanks. This causes a backwater effect, which in turn leads to the flooding of properties near the centre of the village. Walls within the village lack any real protection against flooding, and gaps in the wall in the centre of the village allows flood water to spill onto the road running through its centre. The former masonry arch bridge was replaced with a new bridge (on West Street) with a reduced soffit level and is prone to blocking (leading to flood water inundating the road – see Figure 3.6). Another contributing factor to flooding in the village of Belford is that the road running through the centre of the catchment (B6349) is poorly drained and has been identified as being a major flow path for surface runoff; however, the flooding that occurs in the village is mainly fluvial.

3.7 Summary

The Belford Burn catchment (5.7km^2), Northumberland in North-East England, was chosen for this study due to its long history of flooding and because the village failed to receive funding for a traditional flood defence scheme. The river channel is greatly constricted by gardens, walls and residential structures within Belford village and the flashy flood response gives rise to properties and businesses being inundated.

This chapter has outlined the knowledge available through a desk study. It highlights that the factors that contribute to flooding in Belford can be broadly classified into (i) geophysical catchment characteristics and (ii) issues within the village itself.

i) Assessment of the FEH catchment descriptors identifies that steep topography in the catchment upstream of Belford, and bare rock formations at the extremities of the catchment, may aid the movement of surface runoff, providing very little natural attenuation of flood flow and producing a flashy response to rainfall (TP is 2-hours). There are also shallow soils throughout the majority of the catchment and where the soil depth increases, the permeability appears to decrease. The catchment descriptors and FEH methodology for generating hydrographs demonstrate the impact of these theories and it has been identified that the catchment response is dominated by surface runoff.

ii) A bottleneck effect is present as the burn enters the village due to the proximity to properties, which have encroached to the stream banks. Also a bridge in the centre of the village constricts high flow.

The Halcrow report (published in 2007) assessed the feasibility of a traditional flood defence scheme in Belford (Halcrow, 2007). It concluded that traditional options and formal flood defences or upstream storage would not be cost-effective (£3.5 million to achieve 1:100 year flood protection). The report acknowledged that upland catchment management could be a potential solution to the flooding problem in Belford. The report showed, however, a lack of confidence in this type of solution; stating, “Such schemes can only be regarded as experimental at present.”

4. Catchment Data

4.1 Introduction

The previous Chapter introduced the methods used by early studies to identify the causes of flood risk in Belford. These methods are usually applied to ungauged catchments where observed data is not available for analysis. The aim of this chapter is to describe the multi-scale, nested monitoring network that has been implemented in the upper Belford catchment and provide an overview of the data monitoring and processing. These data will be analysed in Chapters 5 and 6.

The purpose of the hydrometric network is (1) to quantify the rainfall-runoff regime to understand the hydrological response of the catchment, and; (2) capture flood events in the catchment for later analysis and modelling.

4.2 Observations

The monitoring of rainfall and river stage commenced at the start of the Belford study in late 2007. The monitoring network has expanded over time as more flood management interventions have been constructed. A summary of the locations of the monitoring equipment and a description of the catchment variables being measured is provided in Figure 4.1 and Table 4-1, respectively. Figure 4.1 also shows the locations of all the RAFs (and indicates the RAF type) currently installed in Belford (the types of RAF will be described in more detail in Chapter 7).

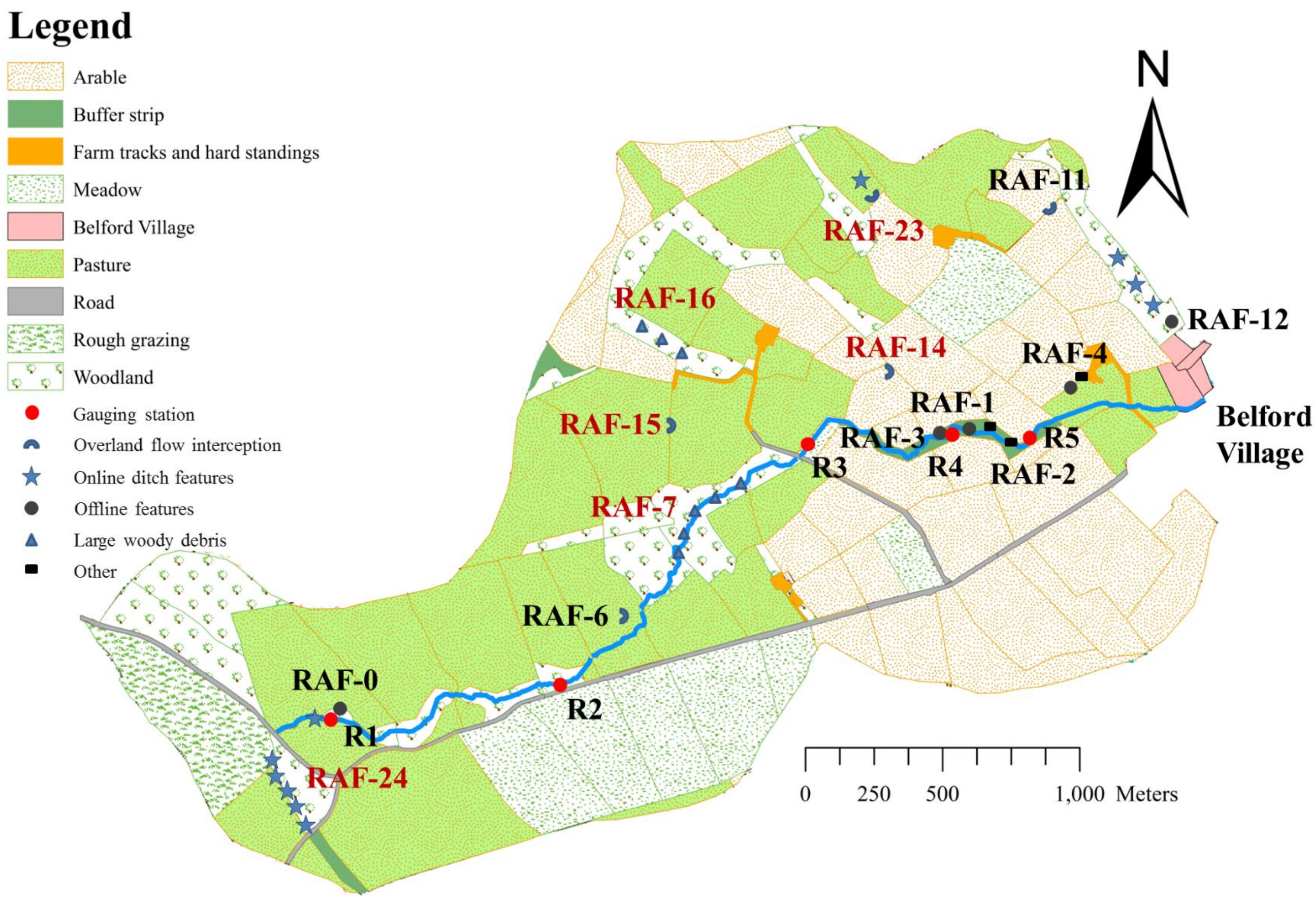


Figure 4.1: Study area with all observations highlighted (RAFs in red have not been monitored to date)

Table 4-1: Catchment Gauge Summary

Observation point	Easting	Northing	Record	Type	Comments
Rainfall and barometric pressure:					
U1	407464	632796	2008 – present	Barometer	Used to obtain stage from river gauges (explained in 4.4)
U1	407464	632796	2008 – present	TBR x 2, Piezometer	Rainfall data have used in generating hydrographs
EA_RG	409266	633167	2008 – present	TBR	Rainfall data have used in generating hydrographs
River Gauges:					
R1	407533	632793	2007 – present	Stage	Data has been used for event analysis in conjunction with rainfall data
R2	408582	633054	2007 – present	Stage	
R3	409297	633790	2007 – present	Stage	
R4	409748	633826	2009 – present	Stage	
R5	409949	633837	2009 – November 2010	Stage	
BEL_Village	410799	633851	2008 – present	Stage	
Runoff Attenuation Features (RAFs):					
RAF_0	407596	632833	2008 – present	Pond Stage	RAF level analysed alongside with river stage to test RAF response during storms
RAF_1	409802	633831	2009 – present	Pond Stage	Offline pond used in Pond-Event analysis
RAF_2	409943	633838	2009 – present	Pond Stage	Online Pond – Data used primarily for water quality, so excluded from analysis in this study
RAF_3	409722	633840	2009 – present	Pond Stage	Offline pond used in Pond-Event analysis
RAF_4	410297	634056	2010 – present	Pond Stage	RAF level analysed alongside rainfall to test RAF response during storms
RAF_6	408610	633138	October 2011 – present	Pond Stage	
RAF_11	410166	634648	October 2011 – present	Pond Stage	
RAF_12	410456	634347	2010 – present	Pond Stage	Online Pond – Data used primarily for water quality, so excluded from analysis in this study

4.3 Data collection

The monitoring of river stage at observation points R1-R3 has taken place since October 2008 and since February 2009 for R4 and R5. Data from these sub-catchments will help us to understand the impact of catchment change or flood management on the flood hydrograph. The upstream contributing areas of R1-R5 are 0.5 km^2, 1.46 km^2, 2.58 km^2, 2.72 km^2 and 2.99 km^2, respectively (Wilkinson, et al., 2010a). Monitoring of the RAFs also began in 2008; as they were constructed (see Chapter 7). River/RAF stage at the monitoring points is measured at five minute intervals. The University raingauges, the barometer and piezometer also store data at five minute intervals.

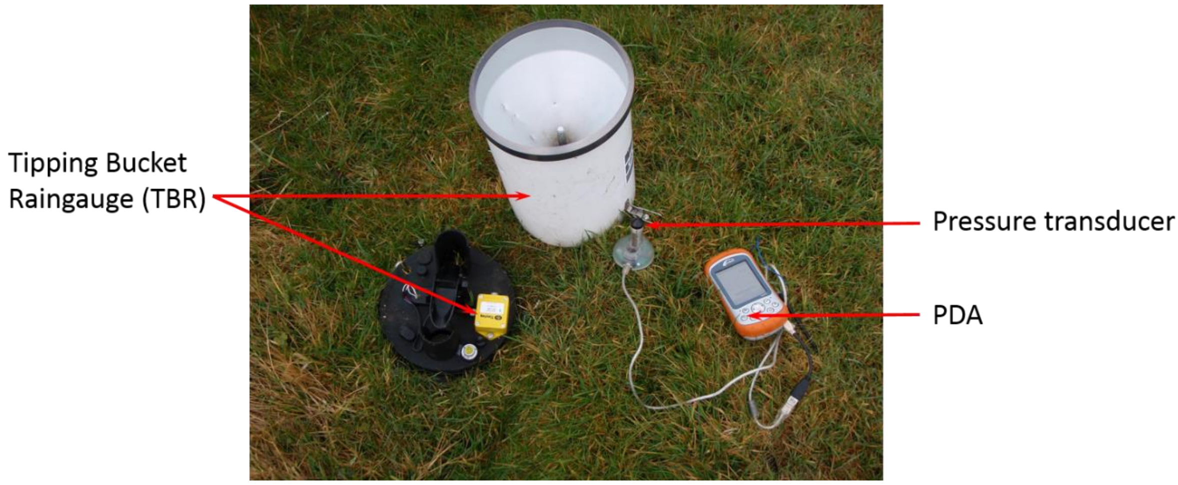


Figure 4.2: Tipping Bucket Raingauge, Pressure transducer and Personal Digital Assistant (PDA)

4.3.1 Rainfall Data

Three Tipping Bucket Raingauges (TBRs) have been installed in the Belford catchment (see Table 4-1); two belonging to Newcastle University and the other belonging to the EA. The University's DT2 (Environmental Measurements Ltd.) logger and Cassella logger each have the capability of continuously recording data over a 6 month duration. Owing to the risk of logger failure and data loss the loggers are downloaded more frequently, typically at 8 week intervals. Having three raingauges in such a small catchment is beneficial if problems occur with the loggers. Several instances of logger malfunction have been recorded throughout the project.

4.3.2 Water pressure and barometer data

Eijkelkamp Agrisearch (Slumberger) pressure transducers ('divers') are used for measuring water pressure/temperature in rivers and storage features, at 5-minute intervals (see Figure 4.1). The main disadvantage of calculating stage using pressure measurements is that a reading for barometric pressure must also be collected. To convert the diver pressure readings to a stage, the values must be compensated for atmospheric pressure (explained in 4.4). Hence, barometric pressure recorders ('baros') have been installed within the catchment, and housed in dry air wells. The devices themselves are inexpensive, unobtrusive, quick to install and simple to download. The divers are housed within a protective tube (Figure 4.3). The tube has a series of holes drilled into it over its length to allow water to enter and exit with a rising or falling stage. The tube has a lid with a detachable lid on top to allow easy access when downloading or cleaning. Drilled through the tube's lid is a bolt from which the diver is hung. The entire gauge is attached in a fixed position to a steel post that is secured to both the river bank and bed. These divers have provided the Belford project with an almost uninterrupted dataset from 2008 to 2013.

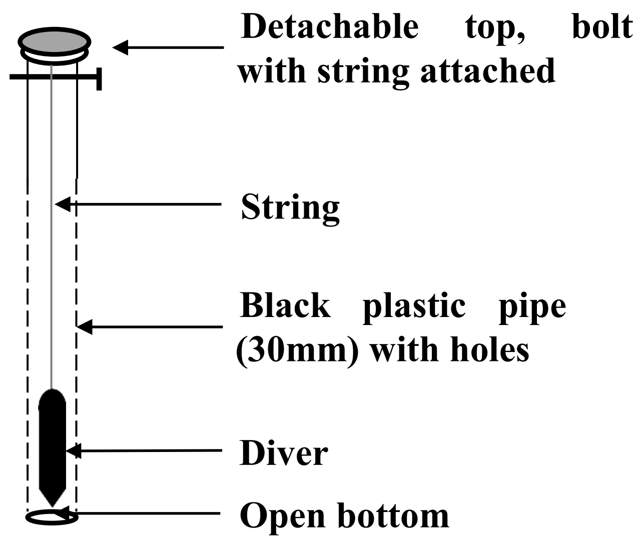


Figure 4.3: Schematic representation of a typical stream gauge

4.4 Stage time series

The diver records water temperature (T) and pressure (P_d). P_d is the sum of the pressure exerted by the water column and atmospheric pressure (P_b). To obtain stage data, P_d needs to be corrected for P_b, which is measured by the barometer. A second issue with the divers is that they are slightly sensitive to temperature and are calibrated only between 15 and 35°C

(Slumberger Water Services, 2010). As *T* is generally <15°C for most of the year, the conversion of raw diver data to stage should take this into account.

When a diver is downloaded, a manual measurement of the stage is recorded. These manual measurements are used in calibration to give the stage time series, y(t) (Ewen, et al., 2010). Equation 5.1 gives the compensation equation for obtaining stage (*y*) in m, where parameters *a* and *b* are calibrated based on the manual measurements.

$$y = a + 0.01(P_d - P_b) - bT_d$$ **Equation 5.1**

where P_d and P_b are in cm; the constant *0.01* converts to metres. *a* adjusts the data relative to the datum and *b* compensates for temperature.

Temperature compensation is required as the instrument sensor is sensitive to temperature. Figure 4.4 shows the effect of the temperature compensation (causing a shift from the blue circles to the red squares). The range of stage plotted in the figure is 600 mm and the average and largest absolute errors (after compensation) are approximately 10 and 30 mm respectively. The average error in obtaining the manual stage measurement is approximated as 5 mm, due to a combination of factors including access difficulties, water ripples, and the general repeatability problems associated with taking manual measurements in the field. Manual measurements have been used to ensure the measurement system is firmly anchored, making it possible to investigate the effects of temperature. Ewen, et al. (2010) found that the maximum errors after compensation occurred during summer low flows, in the Hodder catchment. Overall, these errors generally decrease for sites with larger catchment areas. Ewen, et al. (2010) also noted that the errors decrease with the number of manual measurements made.

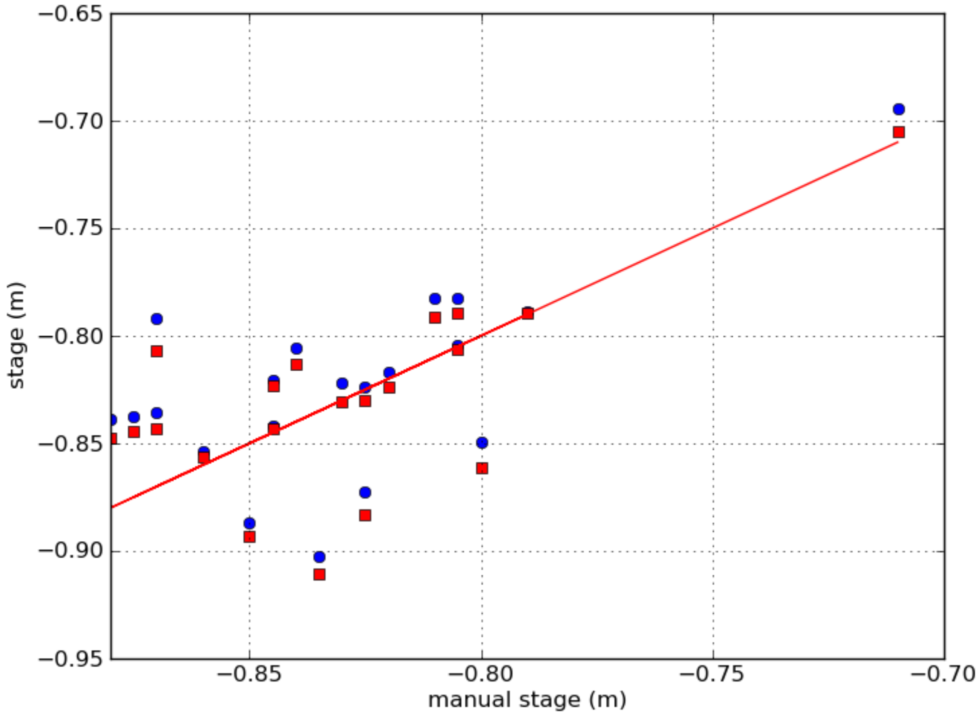


Figure 4.4: Calibration of stage derived from readings against manually measured stage at R1 (red-squares – temperature compensated; blue circles – uncompensated)

In addition to an ordinary diver, an *ISODAQ* FROG RX GSM/GPRS telemetry logger (hereby known as Frog), with an *Impress* depth level pressure sensor, has been installed at the R2 gauging station. Unlike the pressure sensors in divers, the Frog cable has an integrated air tube for atmospheric pressure reference, which avoids the need for a separate barometer gauge and the associated data corrections. The Frog was designed for use in remote areas to remove the need to regularly physically visit the field sites. In Belford, there was no need to utilise the logger in this way; though it was used to warn the research team when the river stage increased above predefined levels. Three levels were chosen (0.35m, 0.65m and 0.9m) based on knowledge of high flows previously recorded at the R2 gauging station. When stage exceeds a chosen level the user receives an SMS message on their mobile phone. This allows the user to judge the best opportunities for manual flow gauging exercises, creating a greater range of data-points.

The data recorded by the Frog are uploaded to an associated webpage (www.timeview2.net), where it can then be viewed and downloaded by the user (Figure 4.5).

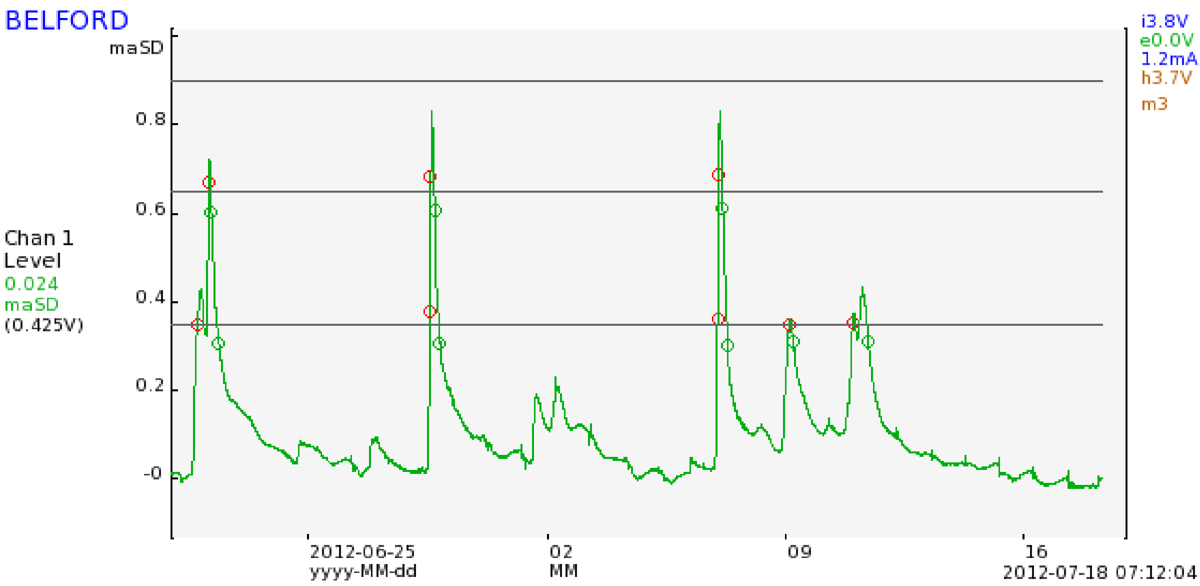


Figure 4.5: 4-week period of river level recorded by 'Frog' (red circles indicate SMS delivery following river height reaching level of specified alarms)

The logger in the Frog is considered more accurate than the divers used throughout the catchment due to the fact that it lacks sensitivity to temperature. Figure 4.6 shows a plot comparing levels recorded by the Frog and the compensated diver stage levels at R2 for more than 14, 000 data points.

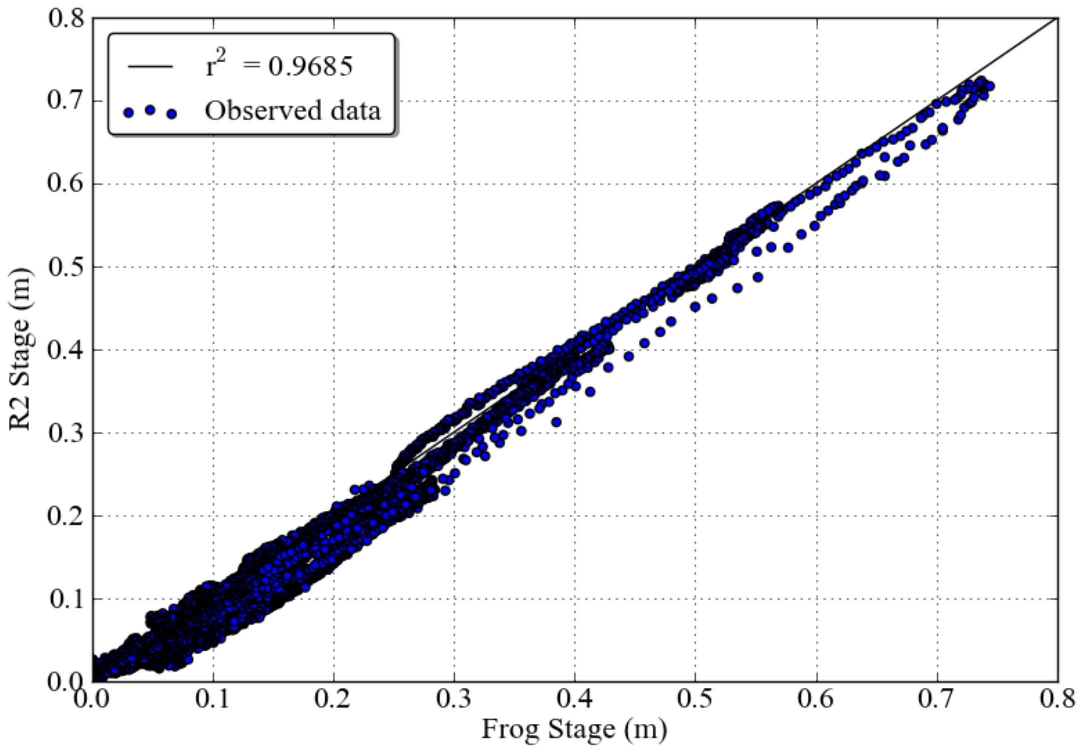


Figure 4.6: Frog and compensated diver stage data comparison at R2

Figure 4.6 shows the data from the frog and diver plotted against each other, with a regression (R^2) of 0.9685 (to the 1:1 line). This regression demonstrates a good relation between the two equipment types for measuring stage. From the plot in Figure 4.6 it may be concluded that the data obtained by the divers are in good agreement with the Frog data with a comparable level of accuracy (standard stage difference between devices < 0.05 m).

4.5 Rating curve

To convert the stage levels to flow, a rating curve is required. The rating curve is constructed by manually measuring the velocity and cross-sectional area over a range of stage levels at a gauging station, so a relationship between stage and flow can be determined. Ideally, the full range of stage would be sampled. Unfortunately, due to the nature of gathering the required information, there are too few data at high flows for the relationship to be based on the calibration between stage and discharge alone. This can often be attributed to the fact that as most events are relatively short lived, it would be unlikely for the field-team to be able to make it to the site at short notice.

4.5.1 Rationale

In order to estimate flow rates beyond the physically measured, the rating curve is usually extrapolated based on the power law. However, this approach can be unreliable – especially when out of bank flow occurs. According to Herschy (1995), the best method for extending rating curves is Stage-Velocity-Area (SVA) method. In the SVA method, the discharge is calculated according to:

$$Q = v.A$$

Equation 5.2

where *v* is the mean velocity of the cross-section (m/s) and *A* is the wetted area of the cross-section (m^2). Both the wetted area and velocity increase with stage although, at higher stages, the rate of velocity increase quickly diminishes and asymptotes at a maximum value. Depending on channel properties for upland channel networks, the maximum velocity lies in the range from 1.5 – 2 m/s (Beven, 1979; Beven, et al., 1979; Herschy, 1995). The relation between stage and mean velocity can be calibrated using velocities of the observed stage/area-discharge pairs, based on Equation 5.2 (Herschy, 1995).

4.5.2 Survey and extrapolate

Channel cross-sectional surveys (e.g. Figure 4.7) allow stage-area look up tables to be generated (e.g. Figure 4.8), which can be used to calculate the mean velocity for each pair of stage/area-flow data. Cross sections from all gauging stations can be found in Appendix B.1.

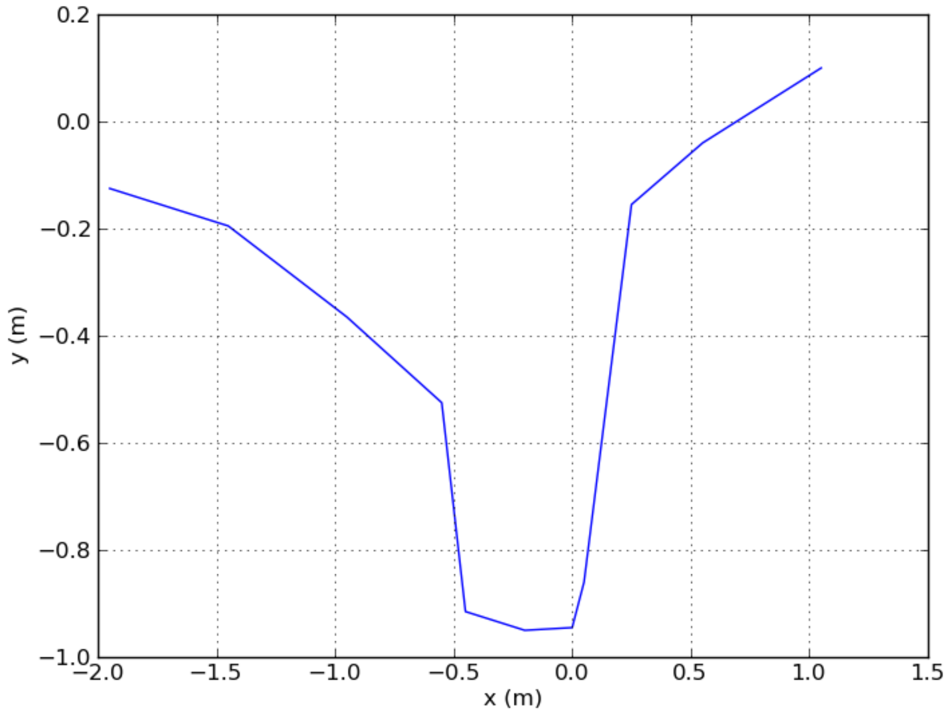


Figure 4.7: Cross-section for R1

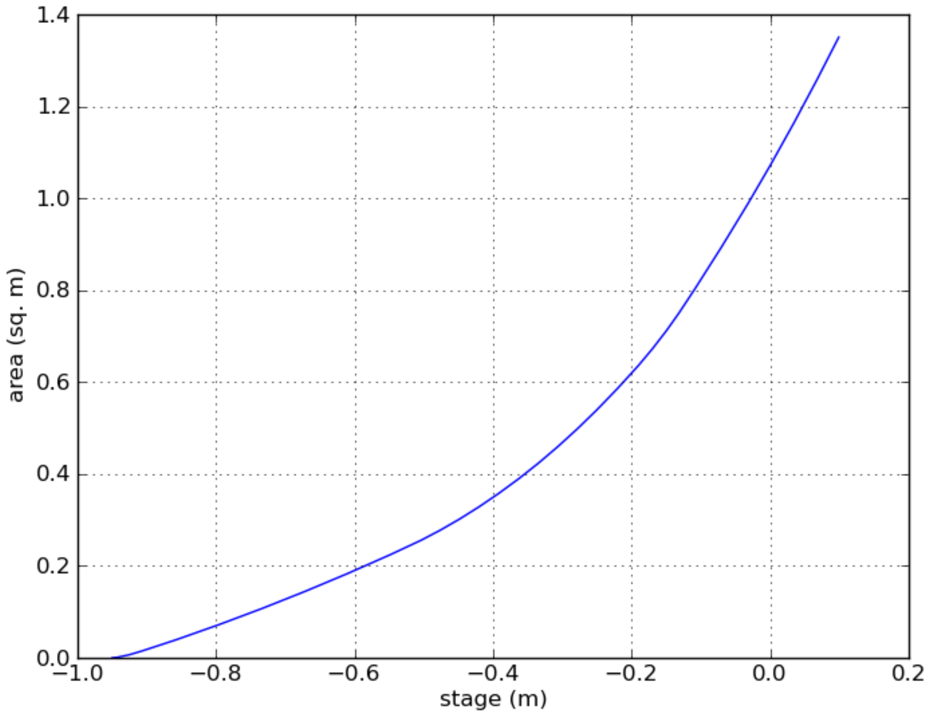


Figure 4.8: Stage-area curve for R1

The stage-area look up table allows the rating points to be paired in terms of velocity and flow. This method fully utilises the available data and avoids the need for estimations of roughness parameters (as in Manning's equation method e.g. Leonard, et al., 2000). However, it does not provide the data for the stage range of interest for high flows. The construction of the rating curve (Figure 4.9), at high flows, can then be based on calibrating and extrapolating the velocity curve, taking into account the stage-area curve (Ewen, et al., 2010). This approach is intrinsically more robust than calibrating the discharge curve directly. The average velocity in the stream is measured using a current meter at points throughout the cross-section at associated depths. The discharge is then calculated using the 'mean section' method (Shaw, et al., 2011). Rating curves from all gauging stations can be found in Appendix B1.

4.5.3 Calibration

Ewen, et al. (2010) developed software that can be used to extend rating curves through the SVA method, that makes use of a calibrated velocity equation, with a sigmoid function that varies smoothly between the upper and lower bounds of velocity (v_{min} and v_{max}) (e.g. 1.5 m/s):

$$X = MIN\left(1, \frac{y - y_{min}}{y_{max} - y_{min}}\right)$$ **Equation 5.2**

$$G = \frac{1}{2}\left[1 + \frac{tanh(2\alpha X^{\beta} - \alpha)}{tanh(\alpha)}\right]$$ **Equation 5.3**

$$v = v_{min} + (v_{max} - v_{min})G$$ **Equation 5.4**

If the velocity is assumed to be zero at the bottom of the channel, then v_{min} and y_{min} are known. This leaves four parameters that can be set or calibrated: v_{max}, the maximum velocity; y_{max}, the stage where the velocity reaches the maximum; and the parameters α and β that control the shape of the transition from low to high velocity (Ewen, et al., 2010).

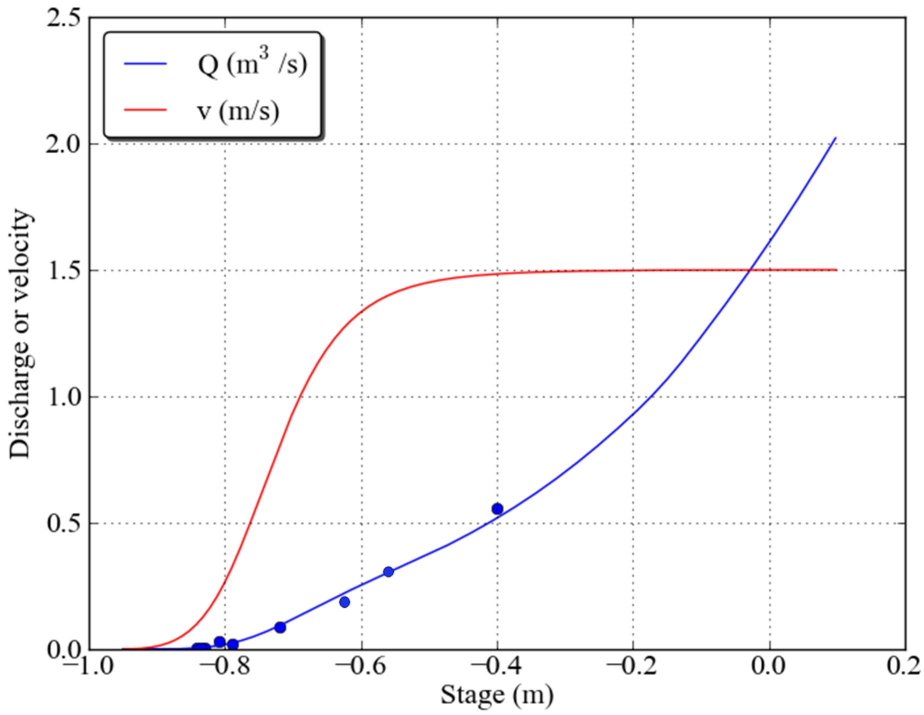


Figure 4.9: R1 rating curve assuming a maximum velocity of 1.5 ms^{-1} (blue circles – measured stage-discharge)

The need to monitor the natural river system has its drawbacks from a practical point of view; not least for the requirement of multiple manual measurements at the gauging locations for the development of the rating relationship, but also for the unforeseen impacts to the natural system. Figure 4.10 shows the damage caused to the river channel as a result of livestock 'poaching'. The cross-section (at the R3 gauge) had to be resurveyed and used with data from that point on, in order to generate the discharge record.

Figure 4.10: Livestock poaching transforms the river channel at the R3 gauging point

4.6 Calculation of daily net radiation

An input of net radiation was required from an automatic weather station (AWS) in order to calculate a time-series of evaporation for the catchment. As there is no weather station in Belford itself; it was decided to use the nearest obtainable data, which is in a town called Boulmer on the Northumberland coast, approximately 25 km south of Belford.

Unfortunately the data from the weather station in Boulmer did not include net radiation. The lack of measured net radiation meant that it had to be calculated from the rest of the available data from Boulmer combined with the temperature, rainfall and barometric pressure recorded at five-minute time-steps in Belford itself. This had to be achieved using the FAO (Food and Agriculture Organisation of the United Nations) certified set of equations, which is shown in Appendix B.2. It has been assumed that aerial patterns in net radiation are close enough over the distance between Boulmer and Belford (< 25 km) to generate a decent estimate.

4.7 Potential evaporation time series

Potential evaporation is calculated following the net radiation calculations from the data series from the nearest AWS in Boulmer. The calculations use the FAO Penman-Monteith equation outlined in Allen, et al. (1998) (http://www.fao.org/docrep/X0490E/X0490E06.htm), which can be found in Appendix B.3. This gives an estimate for "*a hypothetical crop with an assumed* *height of 0.12 m, with a surface resistance of 70 s m^{-1} and an albedo of 0.23, closely resembling the evaporation from an extensive surface of green grass of uniform height, actively growing and adequately watered".* Given that permanent pasture and rough grazing are the dominant land use types; this estimate was assumed to apply over the entire catchment.

4.8 Summary

A multi-scale, nested monitoring network has been installed in the Belford catchment to monitor the catchment response and the effects of land use/flood management changes at a range of scales. The monitoring network expands upon (and refines) the information obtained in the desk study (in Chapter 3). The main purpose of the monitoring network is to quantify the rainfall-runoff regime and capture flood events for analysis and modelling purposes. For this, the experimental design has focussed on the dense monitoring of water level, precipitation and evapotranspiration. The hydrometric network consists of six stage gauges, a frog logger, three raingauges and two barometers. The stage gauges have been nested within the catchment to observe changes in the propagation of the flood hydrograph as it moves through the catchment. The project involved an extensive fieldwork campaign, which covered river and field surveys, flow gauging and manual measurements, and the collection of time-series data being logged at all the gauging sites. It has been demonstrated how these data have been collected and the process involved for the production of the rating curve at gauging locations has been described.

5. Results and hydrological data analysis

5.1 Introduction

This chapter describes the data analysis of rainfall/runoff at a range of spatial and temporal scales. The analysis will assess annual totals before discussing seasonal variation and finally will investigate event-based observations. The purpose of the chapter is to assess how the catchment responds to rainfall and identify the types of storm event that pose a higher risk to flooding in Belford village. The data cover a relatively short period of time. For gauging stations R1-R3 there are approximately 9 months of data prior to the implementation of flood management in the Belford catchment and 12 months of data after the last mitigation was installed. The R4 gauging station was installed to monitor new flood management, and has no data prior to these installations. In the 9 months prior to the installation of the first flood mitigation features there were few mid to high flow events, which has presented difficulties with assessing differences pre- and post-construction. Given the nature of the project, construction of the interventions had to begin very quickly, which explains why a pre- and post-construction analysis is not possible. The analysis has, however, enabled the classification of the types of storm events that cause flooding to Belford village and also highlighted physical differences between sub-catchments.

5.2 Discharge record

The rating curve (see 4.5) converts the stage-time series into a discharge record for the river, which is used to generate the hydrographs for the observation points. The purpose of the analysis is to detect changes in plot to small catchment scale as the hydrograph, measured at the various river gauging locations, propagates downstream through catchment.

5.2.1 Rainfall/runoff analysis

The Belford catchment's rainfall/runoff regime displays considerable inter-annual variability in yield, affecting both the annual runoff production and the Base Flow Index (Palmer, 2012). This may be explained by the much more limited runoff response to average levels of precipitation during the end of the hydrological year (July-September) than in winter months (Figure 5.1: 2008-09, 2009-10 and 2010-11). In contrast, high levels of precipitation at the end of the 2011-

12 hydrological year is concentrated in the growing season (April-September), resulting in an increase in runoff compared to the rest of the hydrological year.

Figure 5.2 demonstrates that, during 2010, runoff events are limited to the winter period, while in 2008, 2009 and 2012 the largest runoff events are in response to extremes of daily or shorter duration rainfall events occurring in the summer months. A prolonged flow (above average) can be observed in Figure 5.2 from July 2011 to April 2012 even when no precipitation has been detected in the catchment (see red box and period up to April 2012). This is likely to have been caused by an error in the diver’s sensor; however, the reference point for the depth at that point in the catchment could have shifted due to sedimentation around the bottom of the diver tube. Time-series data for the other gauges in the Belford catchment are shown in Appendix B.4.

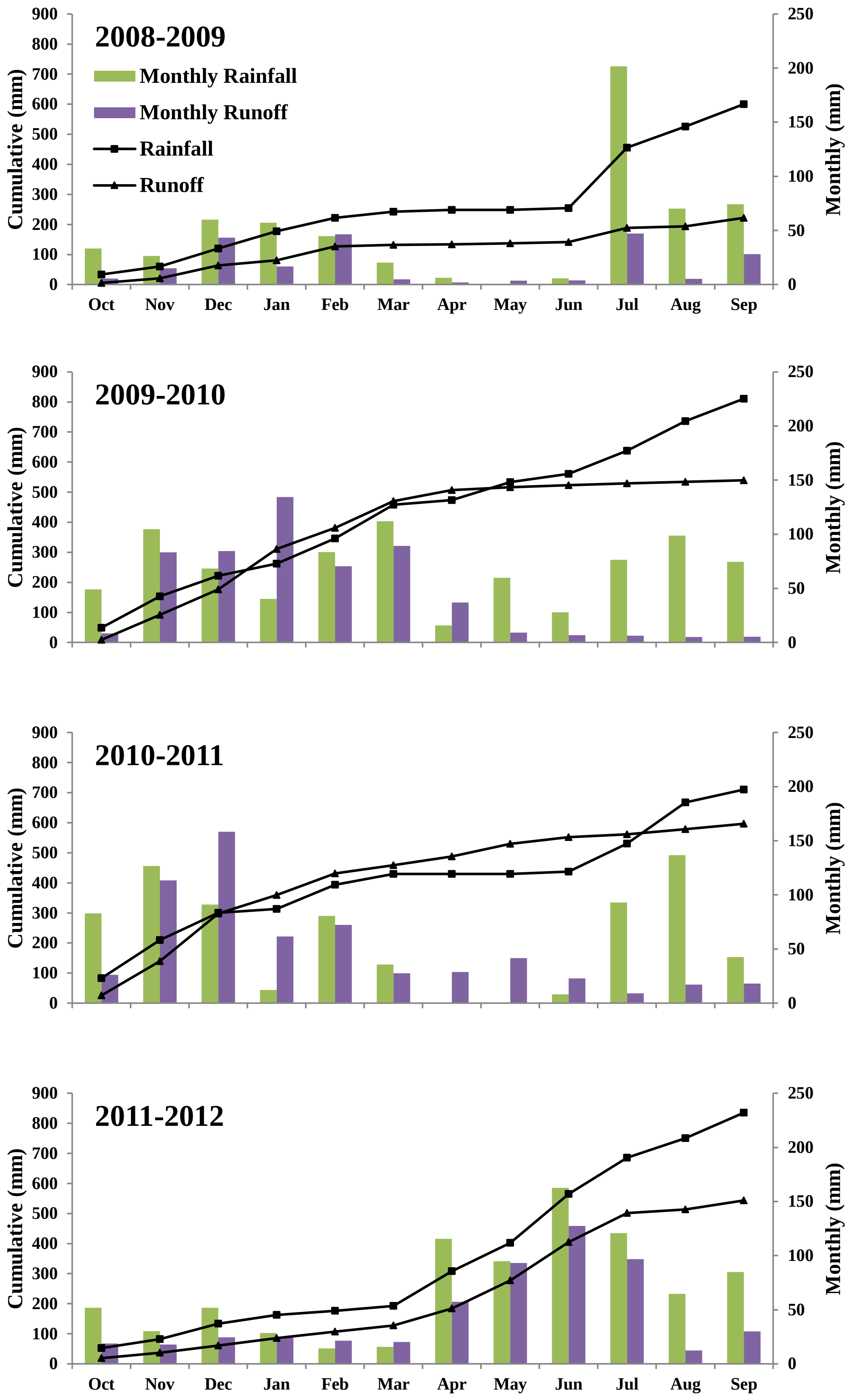


Figure 5.1: Belford monthly and cumulative runoff and rainfall (mm) for water years 2008-2012

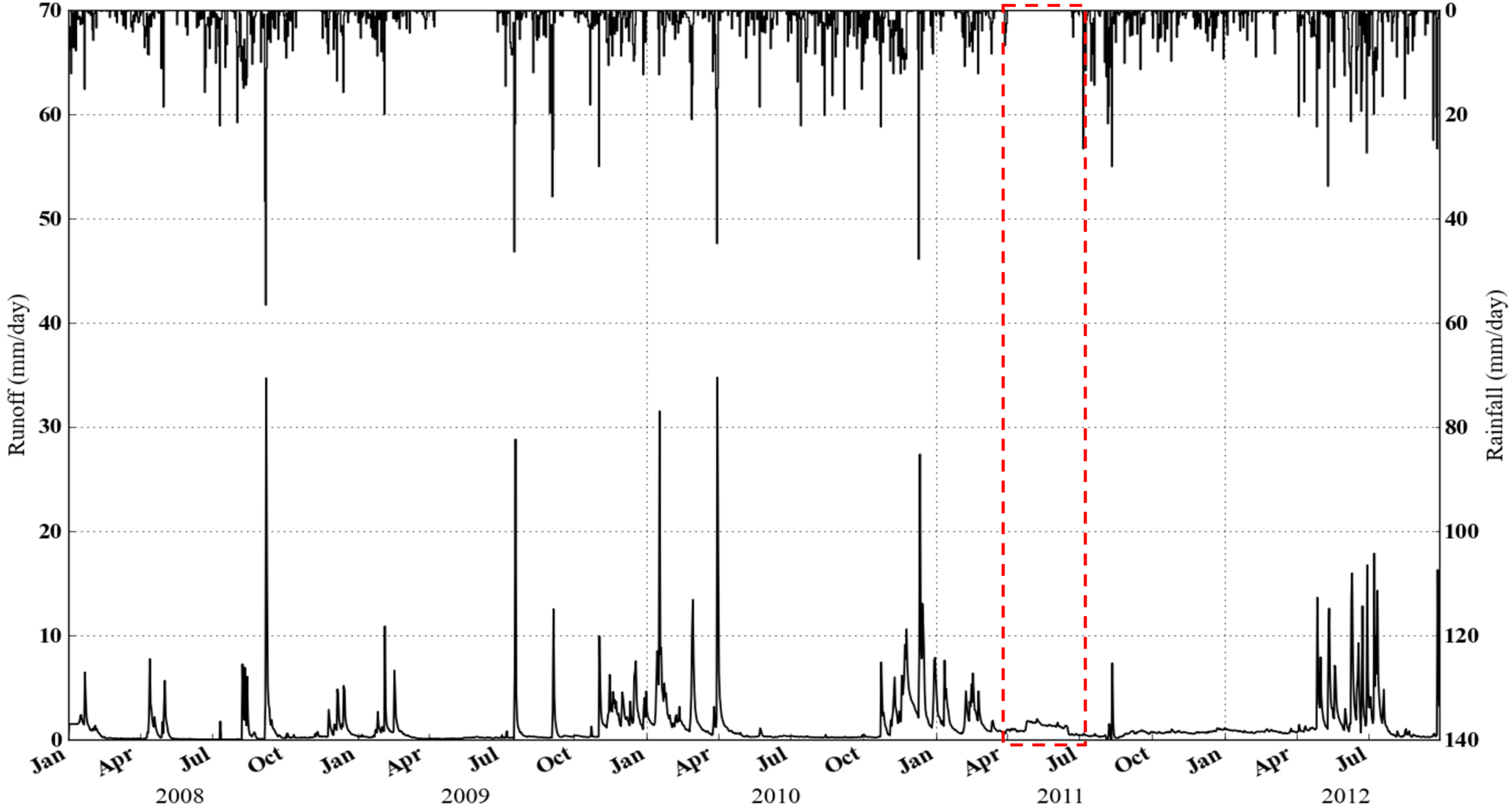


Figure 5.2: Belford Burn rainfall-runoff from 2008-2012 (R3) – Other gauges shown in Appendix B.4

The Belford catchment descriptors identify the SPRHOST (standard percentage runoff) as 40.75%. This is event-based and considers a range of data to determine an average figure, which led to questioning the viability of such a figure in a catchment driven by flashy flood events. Yearly and seasonal runoff coefficients (*R*) can be calculated by dividing runoff totals (mm) by precipitation totals (mm) (See Table 5-1). Here, winter is defined as 1st October to 31st March and summer is consequently 1st April to 30th September.

Table 5-1: Rainfall/runoff totals and runoff ratios for R3

Hydro Year	Season	P (mm)	Q (mm)	R (-)
2008-09	Winter	242.4	131.92	0.54
	Summer	358	90.31	0.25
	Total year	600.4	222.23	0.37
2009-10	Winter	457.8	470.37	1.03
	Summer	353	69.35	0.20
	Total year	810.8	539.72	0.67
2010-11	Winter	429.4	459.18	1.07
	Summer	280.6	137.36	0.49
	Total year	710	596.54	0.84
2011-12	Winter	192.2	127.02	0.66
	Summer	643	416.69	0.65
	Total year	835.2	543.71	0.65
Average	-	492.73	317.03	0.62

The seasonal totals of Discharge (Q) and Rainfall (P) indicate a clear change in runoff response and subsequently runoff coefficients in winter and summer, with the exception of 2011-12 (probably linked to the damaged data period highlighted in Figure 5.2). Although runoff coefficients and SPRHOST cannot be directly compared, the simple analysis reveals that, on average, the runoff coefficient calculated over the four-year study period is approximately 50% greater than the SPRHOST being used by FEH. Does this mean that FEH can underestimate extreme events?

5.2.2 Flow duration and exceedance analysis

'Events' are defined as identifiable increases in discharge from seasonal base flow conditions. Flow duration curves for Belford (developed over the four-year study period) suggest that exceedance of the base flow conditions occur less than 3% of the time (Figure 5.3 and Figure 5.4).

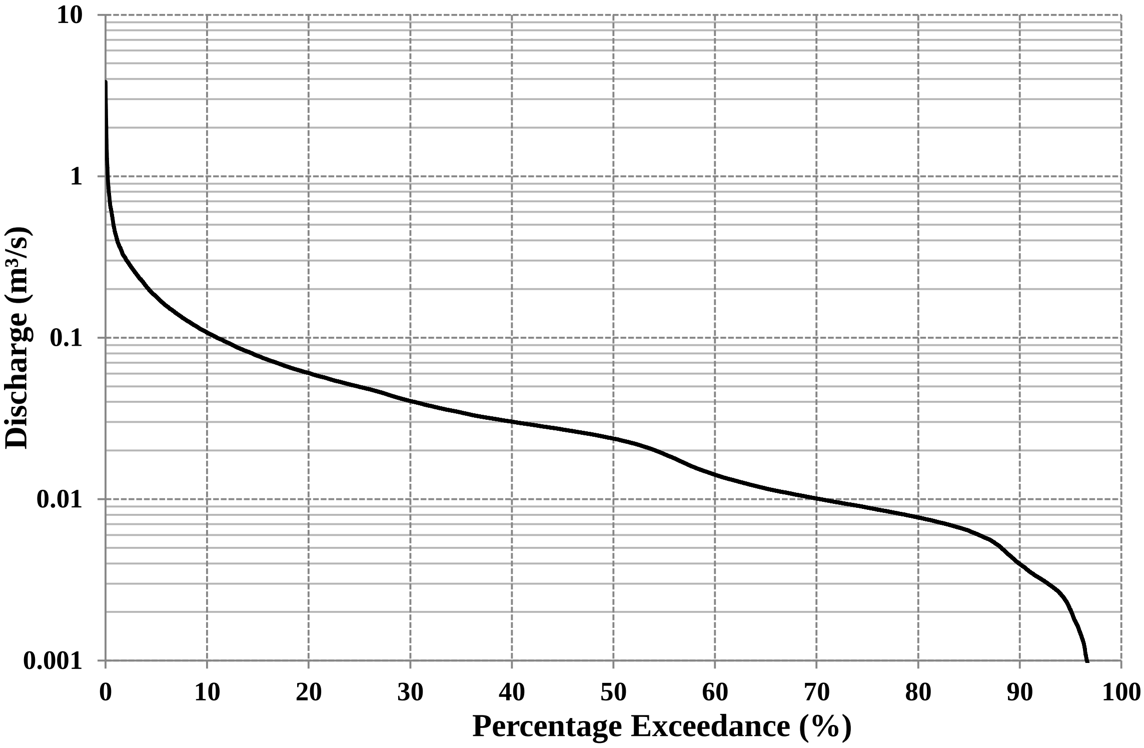


Figure 5.3: Belford mean hourly flow duration curve at R3 (2008-2012)

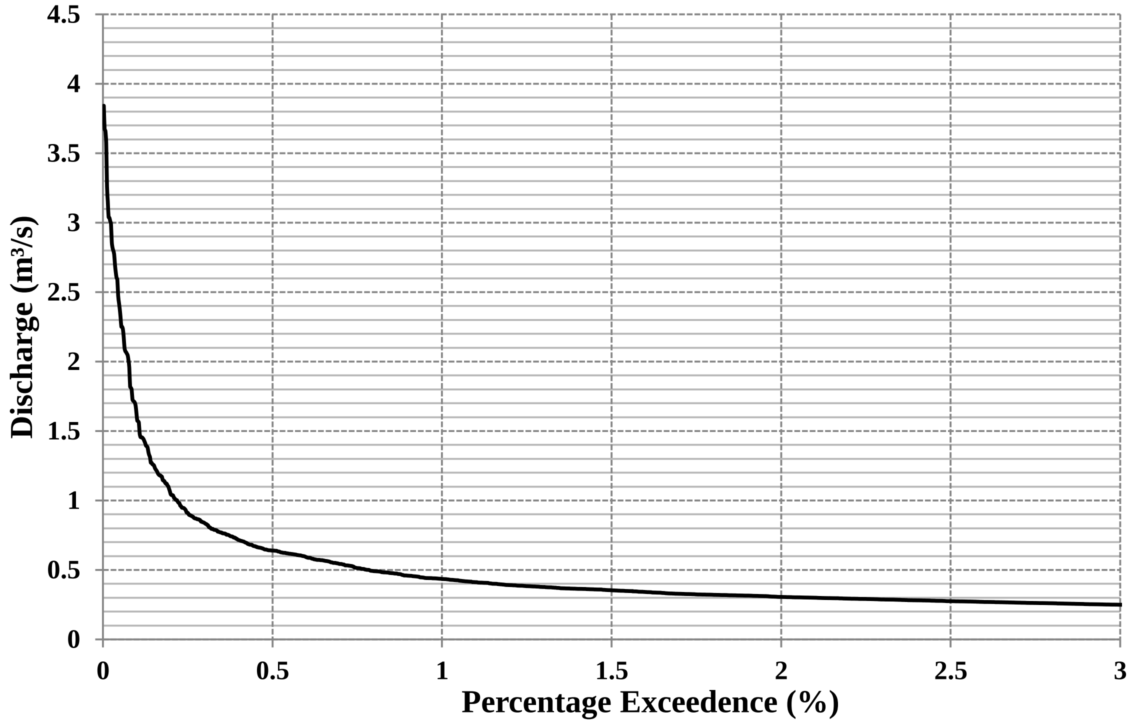


Figure 5.4: Belford mean hourly flow duration curve at R3 (2008-2012) – close-up of extreme values

Table 5-2: Summary of key exceedance values

Percentile Q(% exceedance)	Discharge (m^3/s)
Q5	0.24
Q10	0.15
Q50	0.06
Q95	0.03
Q_{med}	3.01

Table 5-2 summarises the flow duration curves into a few key values. For 95% of the records, generated over the five year monitoring period, flow in the Belford Burn was lower than 0.24 m^3/s. The flow data has also allowed an estimate of Q_{med} (the median annual flow maxima for the hydrological years on record) for the site, which is approximately 3 m^3/s. Very high peak flows relative to low average flow conditions demonstrate that the catchment produces infrequent, but high magnitude responses to precipitation events, which are also characterised as low in frequency (Figure 5.5).

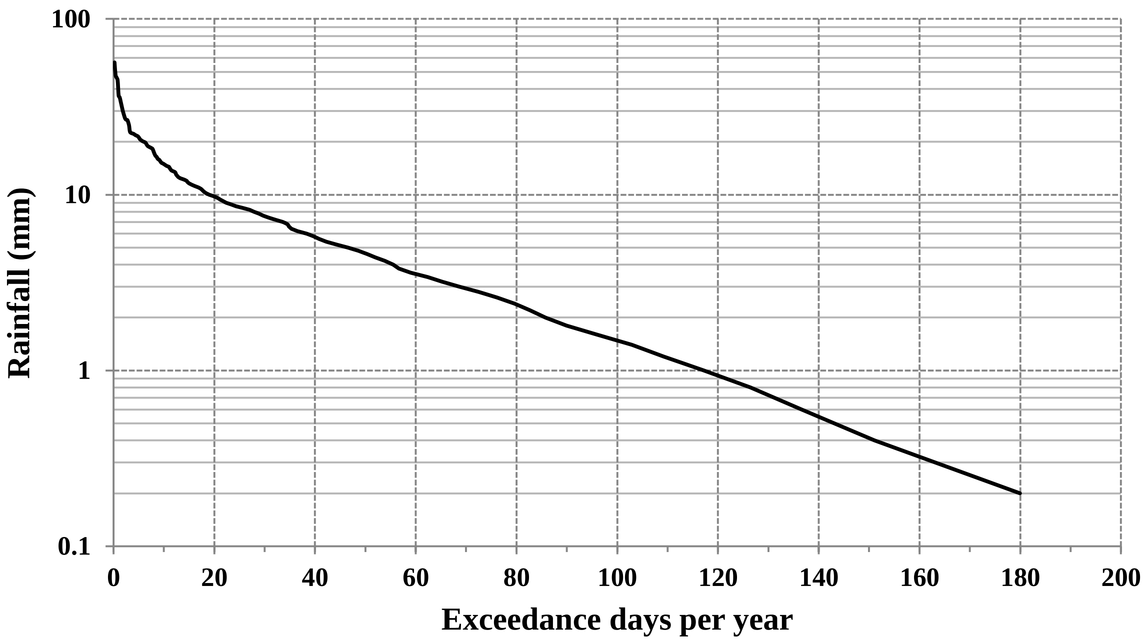


Figure 5.5: Belford daily rainfall exceedance frequency (2008-2012)

5.3 Seasonal patterns in the dataset

During the growing period (April to September) hydrological response to rainfall is limited by the relatively high available storage capacity in the soil, which results in few summer events and prolonged low flow periods (see 3.3.2). During winter, the available storage capacity is much more limited, as the soils remain at field capacity for long periods (Palmer, 2012). The fact that the slowly permeable sub-soils are highly uniform throughout the catchment means that runoff generation is likely once soil saturation conditions are reached. Of particular note is that runoff response to moderate rainfall is much more frequent in winter (see Figure 5.2). Response is rapid in this small catchment, which enhances the magnitude of peak flow and leads to flood hydrographs of short duration. Generally it appears that the catchment has a high capacity to absorb precipitation, but has a rapid, high amplitude response once capacity is exceeded. To demonstrate this, an analysis of the runoff peaks compared with the rainfall intensity preceding the peak was performed. The purpose of the analysis was to identify (1) the type of storms that cause flooding to Belford and (2) the relationship between the peak discharge and the preceding precipitation within the events in differing seasons. The figures that follow present the sum of the rainfall recorded in the hours preceding a peak discharge event to identify whether any patterns exist in precipitation magnitude/intensity and downstream flood peak. The analysis was performed at all gauging points along the Belford Burn, however, the results shown below are from the analysis at the EA gauging station in the village (upstream area = 5.7km^2).

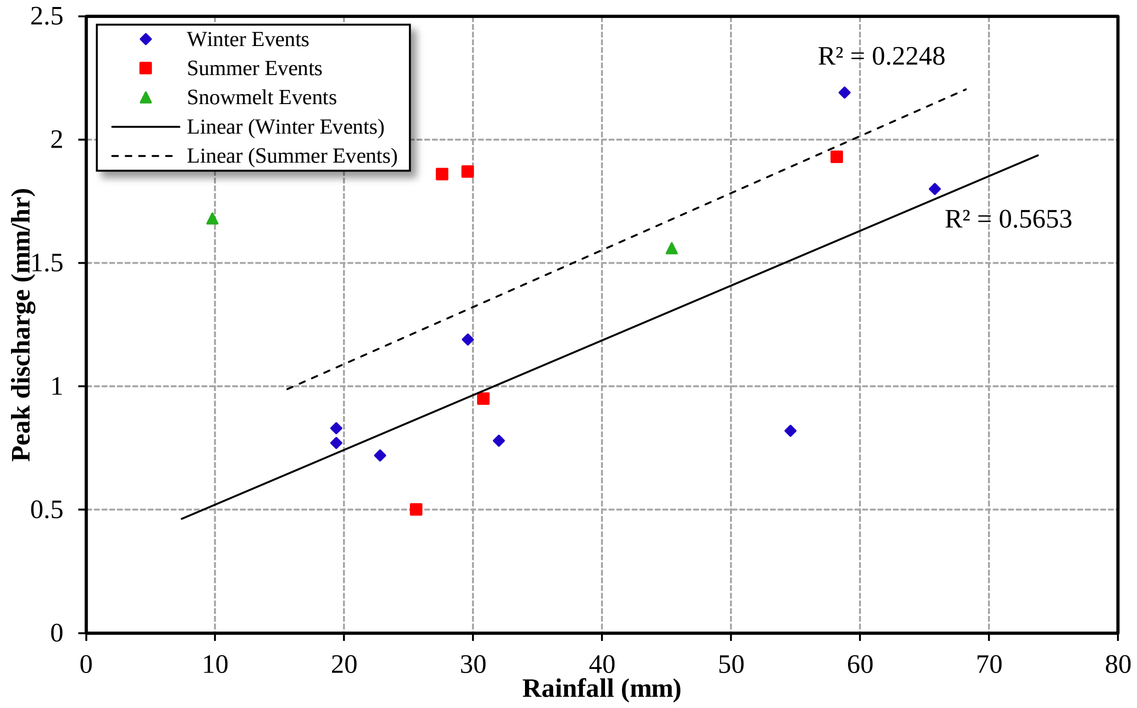


Figure 5.6: Comparison of 24-hour rainfall totals and magnitude of runoff recorded at EA gauging station

The analysis of 24 hour rainfall events preceding peak discharges has demonstrated a relatively strong relationship (with an R^2 value of 0.56) between preceding rainfall and peak discharge at the EA flow gauge for winter events (Figure 5.6). The 24-hour rainfall totals do not show a strong relationship for the summer events. To determine a better trend in the summer event data, a much shorter preceding rainfall period of 6-hours had to be chosen (Figure 5.7), and other time-periods were analysed for completeness (see Table 5-3). Figure 5.7 affirms the theory that there is a fast catchment response during summer events once antecedent dryness has been overcome; generating very flashy, high magnitude peaks in the runoff response for Belford. The results indicate a strong relationship at the EA flow gauge, using 6-hour rainfall totals, during summer events (with an R^2 value of 0.94).

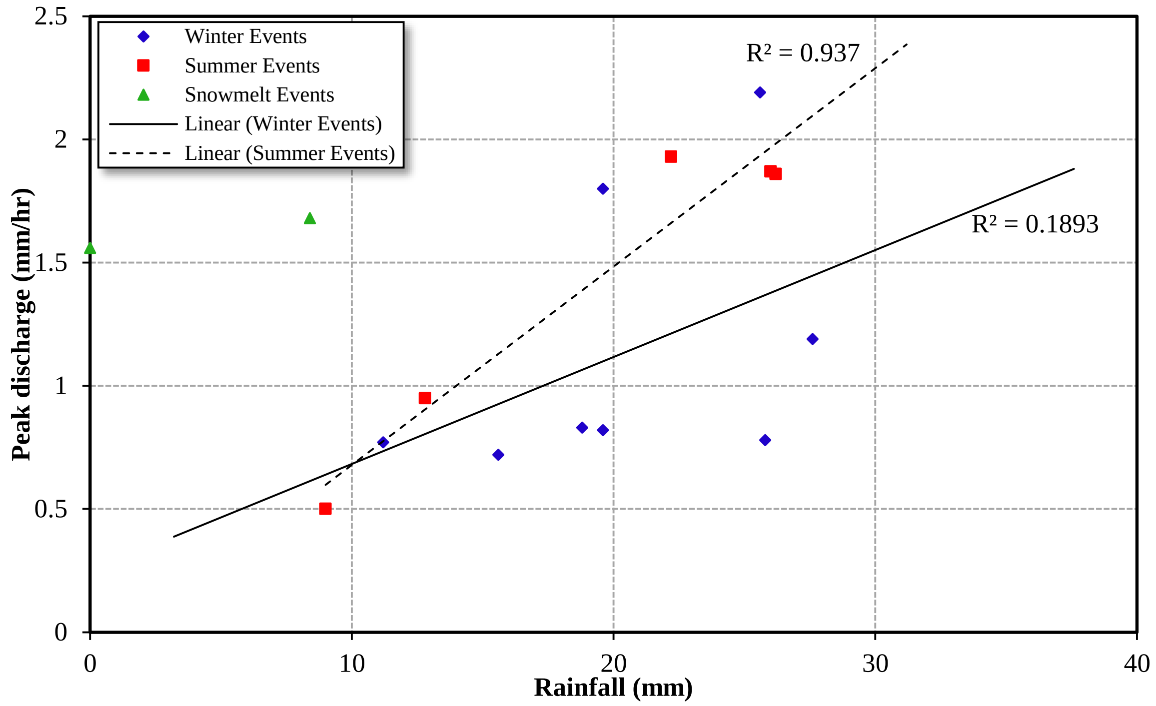


Figure 5.7: Comparison of 6-hour rainfall totals and magnitude of runoff recorded at EA gauging station

This analysis, although highlighting catchment response during storms, reveals little about the degree of flooding within the catchment. The storm events must be analysed in more detail (see Chapter 6). It does reveal, however, that Belford is susceptible to flooding from short duration, intense periods of rainfall that occur within multiday events. Table 5-3 shows further the trend results from further analyses into storm duration. The 2-hour totals for summer rainfall also show a strong relationship with peak discharge at the EA gauging station.

Table 5-3: R^2 values from analysing other durations of storm event

	R^2 value for 2hr rainfall	R^2 value for 12hr rainfall	R^2 value for 48hr rainfall
Winter	0.153	0.427	0.493
Summer	0.887	0.420	0.328

5.4 Hydrological overview

Over the five-year monitoring period (2008-2013) the average annual rainfall is 735 mm. Typically the highest average monthly rainfall totals occur throughout July to September, but the wettest individual months have been recorded between March and June (see Figure 5.8). The lowest average temperatures occur between December and February and have led to several snowfall events during this period. Within the monitoring period, the hydrometry has also recorded several storm events, of varying return period, which will be discussed in the next chapter.

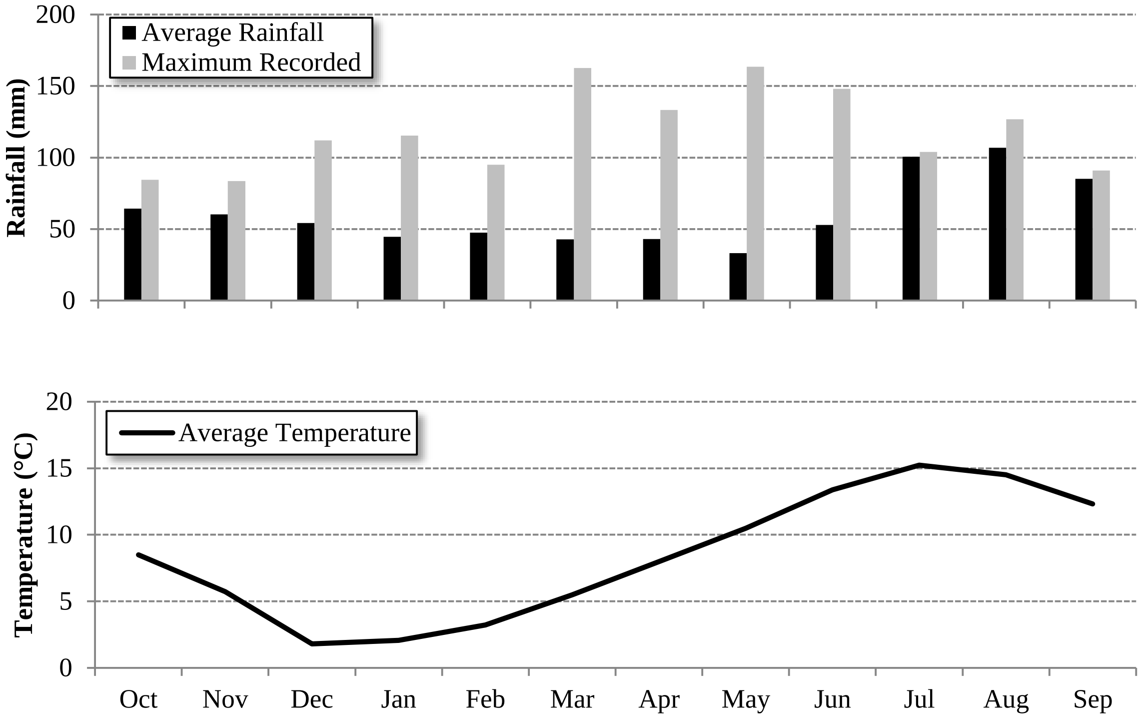


Figure 5.8: Top: Average and maximum recorded precipitation Bottom: Average monthly temperature. Recorded at R1 in Belford (2008-2012)

5.5 Summary

So far, the rainfall and runoff has been expressed as totals for years, months and storm events to determine patterns between them. A contrast between winter and summer rainfall timings has been observed, suggesting different catchment responses to the two types of storm event (see 5.3). Analyses have also highlighted that extremes in rainfall occurring at different times of the year will have different runoff responses, depending on crop cover and available soil storage.

It has been identified, over the five-year monitoring period, that exceedance of low-flow conditions occur rarely in the catchment. A small percentage of high magnitude rainfall events have the potential to enter the top 5th percentile of flows. In fact, 95% of the time the flow remains below 0.24 m^3/s. This confirms that flooding in Belford is driven by high magnitude rainfall (often as part of a longer-lasting, synoptic rainfall event) and flashy catchment response leading to short-lived but equally high magnitude flow events in the river and surrounding area.

The fast catchment response, achieved when soil capacity is exceeded (see 5.3), highlights that fast runoff sources and areas of high hydrological connectivity must be mitigated on land or in the river network to reduce peak discharge at the catchment outlet. Land management in agricultural catchments can be difficult to achieve, due to varying needs of the land-owners. The solution, therefore, potentially lies in targeted management of runoff and its sources.

6. Analysing Belford storms

6.1 Introduction

The final stage in the rainfall/runoff analysis is to break down typical storm events and observe how catchment response varies from gauge to gauge. This will allow a more forensic analysis of the catchment and help identify the areas that generate most runoff.

6.2 Determining return periods of events

Chapter 3 (3.5) discussed the use of design hydrographs for modelling purposes. The use of design hydrographs is important for flooding studies as it provides a benchmark that is easily replicated. It also sets a standard to achieve a solution (e.g. through mitigation). Here, the 1:100 year winter design storm is shown (Figure 6.1) and eventually used as a comparison against historical events recorded during the five-year monitoring period. The hydrograph was generated following the methodology of the 'Revitalisation of the FSR/FEH rainfall runoff method' (Defra/EA, 2005).

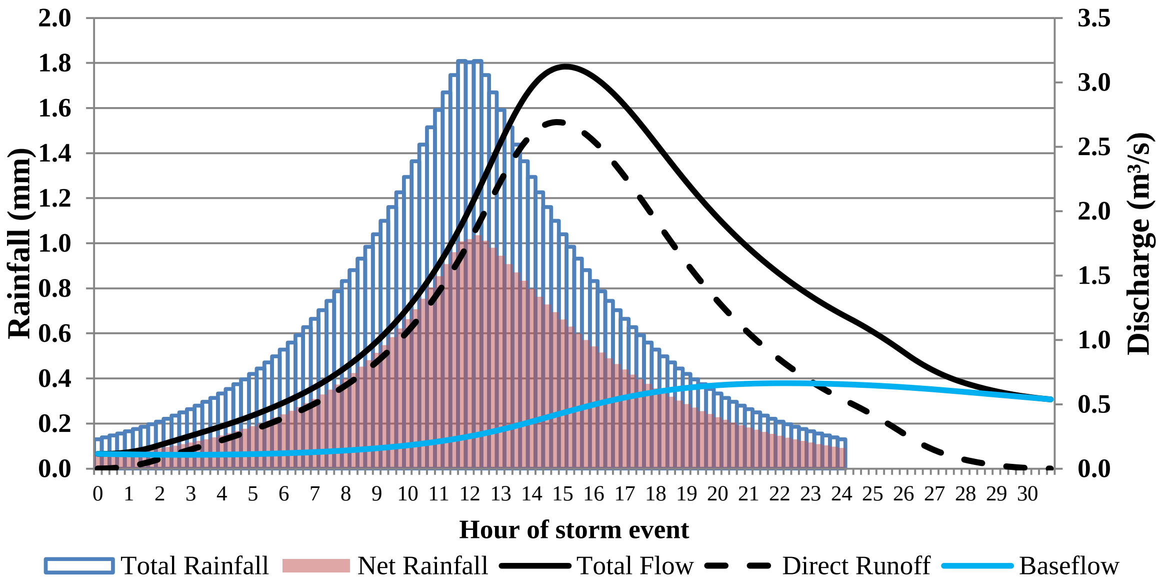


Figure 6.1: Design storm for 1:100 year winter event

A selection of Belford storm events will be shown in this Chapter to demonstrate the impact to the catchment in varying seasons and storm types. Table 6-1 shows the rainfall totals during Belford storm events and their corresponding rainfall return periods. The standard Flood

Estimation Handbook (FEH) approach for classifying storms and degree of flood protection is by classifying the return interval of the storm event. Interestingly, in an analysis based around design storms for the Belford catchment, it was discovered that by following an FEH approach that the design storm hydrographs for the 1:100 year event are directly comparable to the observed data from Belford despite being known to be much lower return period events.

Table 6-1: Rainfall Totals for Belford Storms (Rainfall return periods in square brackets)

Event	Rainfall (mm) (6hrs)		Rainfall (mm) (12hrs)		Rainfall (mm) (24hrs)		Rainfall (mm) (48hrs)	
Winter								
Sep-08	19.6	[2]	27	[2]	65.8	[20]	66	[6]
Feb-09	18.8	[2]	19.4	[1]	19.4	[1]	19.4	[1]
Sep-09	19.6	[2]	28.6	[2]	54.6	[10]	63.6	[5]
Nov-09	27.6	[5]	29.6	[2]	29.6	[1]	33	[1]
Feb-10	11.2	[1]	13.8	[1]	19.4	[1]	20	[1]
Mar-10	25.6	[5]	36.2	[5]	58.8	[12.5]	62.6	[5]
Apr-12	15.6	[1]	19.6	[1]	22.8	[1]	25.4	[1]
Sep-12	25.8	[4]	29.8	[2]	32	[1]	32	[1]
Summer								
Jul-09	22.2	[2]	41.4	[10]	58.2	[12]	61	[5]
May-12	9	[1]	22	[1]	25.6	[1]	25.8	[1]
Jun-12	26.2	[5]	27.2	[2]	27.6	[1]	34.4	[1]
Jul-12	26	[5]	26	[2]	29.6	[1]	30	[1]

FEH catchment descriptors and analysis of the observed data in Belford have helped generate the following return-period frequency curves for 24- and 48-hour rainfall events (Figure 6.2).

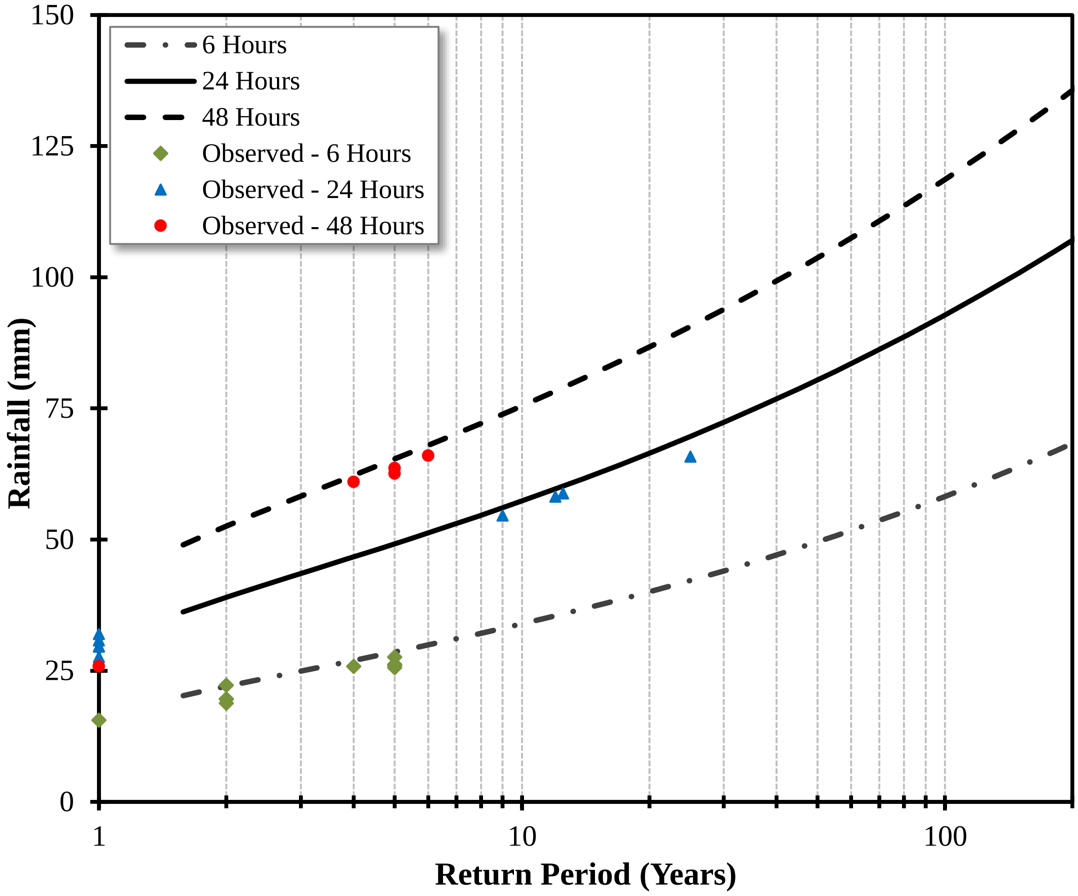


Figure 6.2: Rainfall return period frequency curves for Belford (6, 24 and 48 hours) with observed data

Figure 6.2 and Table 6-1 demonstrate the large frequency of high magnitude storm events over the 5-year monitoring period. The FEH method uses national datasets to translate the rainfall totals into runoff magnitudes in river networks within catchments. For Belford, the Revitalized Flood Hydrograph (ReFH) model was used to estimate the runoff at the R4 River gauging point for a range of FEH rainfall return interval events. The ReFH model uses BFI_HOST and SPR_HOST (the mean annual statistics from Chapter 3 – 3.5) to calculate runoff.

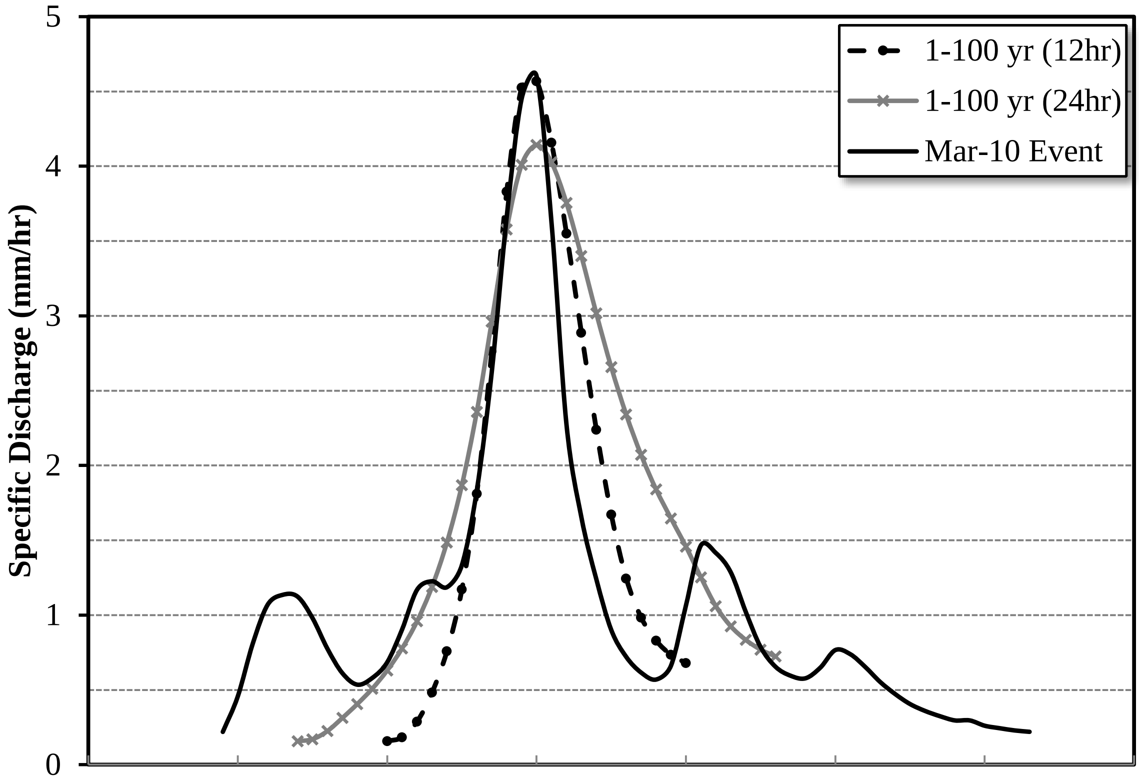


Figure 6.3: Comparison of Belford data and FEH design storms (Generated using the ReFH model)

Figure 6.3 shows the March 2010 storm event directly compared with the 1:100 year design storms for 12- and 24-hours in the winter season. The purpose of this is to demonstrate that a traditional FEH-style approach may not work in small-catchment studies. The March 2010 storm event was classified as ranging from a 1:5 year to a 1:12.5 year rainfall return period for 12 and 24hrs, respectively. The FEH design storms for the 1:100 year scenario, following the same method of classification, are noted as being highly comparable in appearance. (NB there are insufficient data to reliably estimate the actual return period of the observed storms). It is concluded that using design storms may over predict the actual return period of an event and conclusions drawn from analyses based on design storms will be potentially unreliable.

6.3 Is it possible to track flood waves?

In order to clearly present the hydrograph data, for selected storm events, the discharge is expressed as flow per unit area to allow a direct comparison against the rainfall and multiple sites along the river. The data in these graphs are, therefore, standardised due to the fact that the units of specific discharge are mm hr^{-1} (in comparison to discharge which uses the upstream area of the gauging station to give a measurement in m^3s^{-1}). This means that differences in land use, geological conditions and the farm's drainage network may be detected by noting where natural storage is lost or gained between gauging stations. Essentially, if a catchment had exactly the same characteristics over its entire area and a rainfall event occurred that was uniformly distributed; the resulting specific discharge hydrograph would be identical for all sites along a river's reach; although time-delayed depending on the distance between gauging stations. In reality, however, small differences in land use, geology, slope, spatiality of rainfall etc. alter the specific discharge recorded between sites.

Figure 6.4 shows a schematic of the catchment between gauging stations R1 and R4. The key assumption here is the fact that pastoral land and riparian woodlands have greater natural attenuation than intensely cropped arable land. Evidence to support this has been shown in the literature review in Chapter 2.

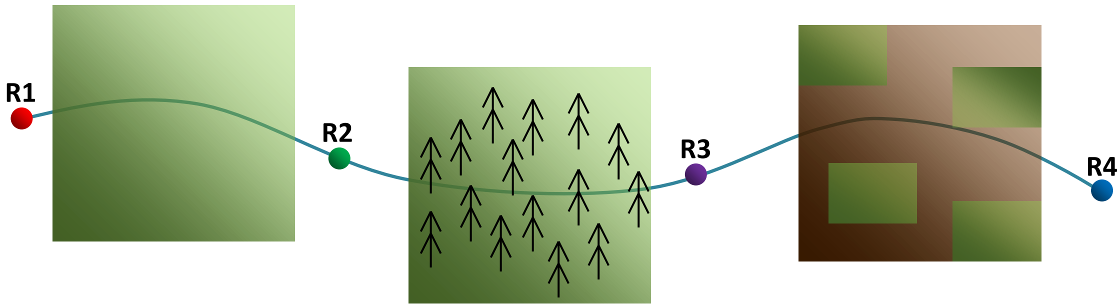


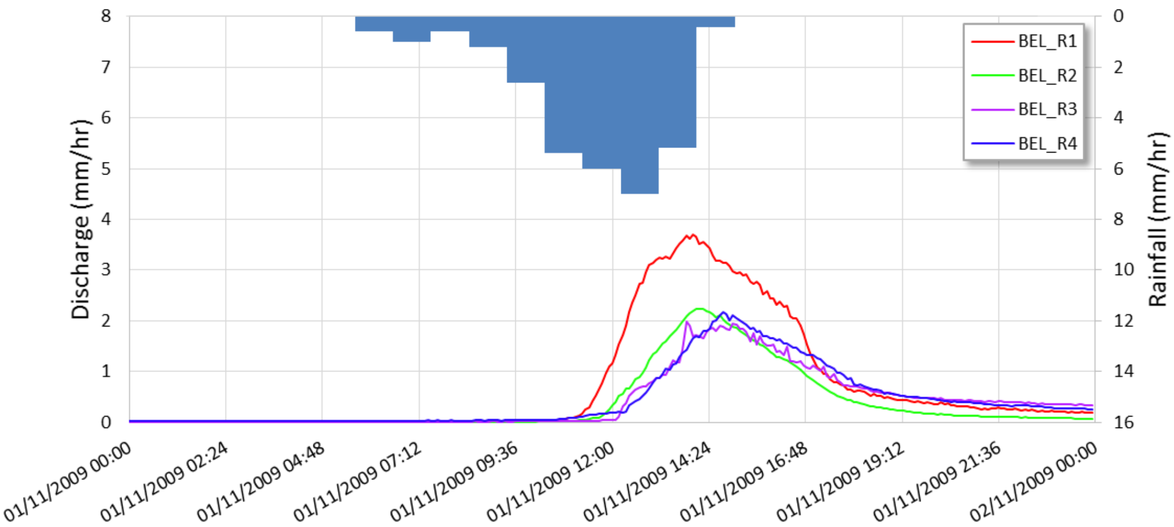
Figure 6.4: Schematic of river reach (highlighting changes to land-use between gauging stations)

6.4 Rainfall signature and catchment response

It can be seen in the figures that follow that the signature of the rainfall is clearly present in the shapes of the hydrographs. It has been noted that response to rainfall is particularly apparent in the upstream site (R1), while it usually requires antecedent dry conditions to be overcome in order for the downstream sites to become as responsive. The catchment response is of a scaled nature in short-lasting but intense rainfall events; with fast response occurring in the uplands and a slower response as catchment area increases. This indicates a significant increase in antecedent catchment storage between R1 and the other gauging stations. For longer-lasting events, however, this storage becomes depleted, allowing a faster response further into the catchment.

6.4.1 Short duration storm events

Short-lasting storm events are characterised by intense rainfall events occurring over a period of 4-12 hours. Most of these events, in Belford, occur when there is antecedent storage available in the shallow soils that cover most of the catchment. There have been cases in summer months, however, where the antecedent dryness in the soil has been overcome following a month of above-average rainfall.



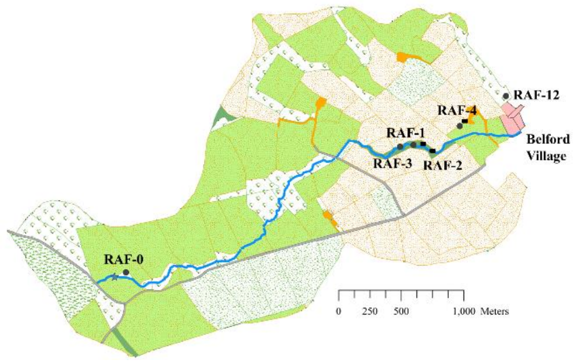


Figure 6.5: November 2009 storm event (catchment map, at time of event, inset)

The November 2009 storm event (Figure 6.5) has been highlighted for comparison with some of the larger events due to the fact that it helped demonstrate the potential impact of the mitigation in Belford. The study of the November 2009 storm event demonstrated a strong, positive impact of RAFs on the reduction of downstream discharge (see 8.3). The coefficient of discharge during the peak of the storm reaches 0.55 at the R1 gauging station and approximately 0.3 for the other gauging stations.

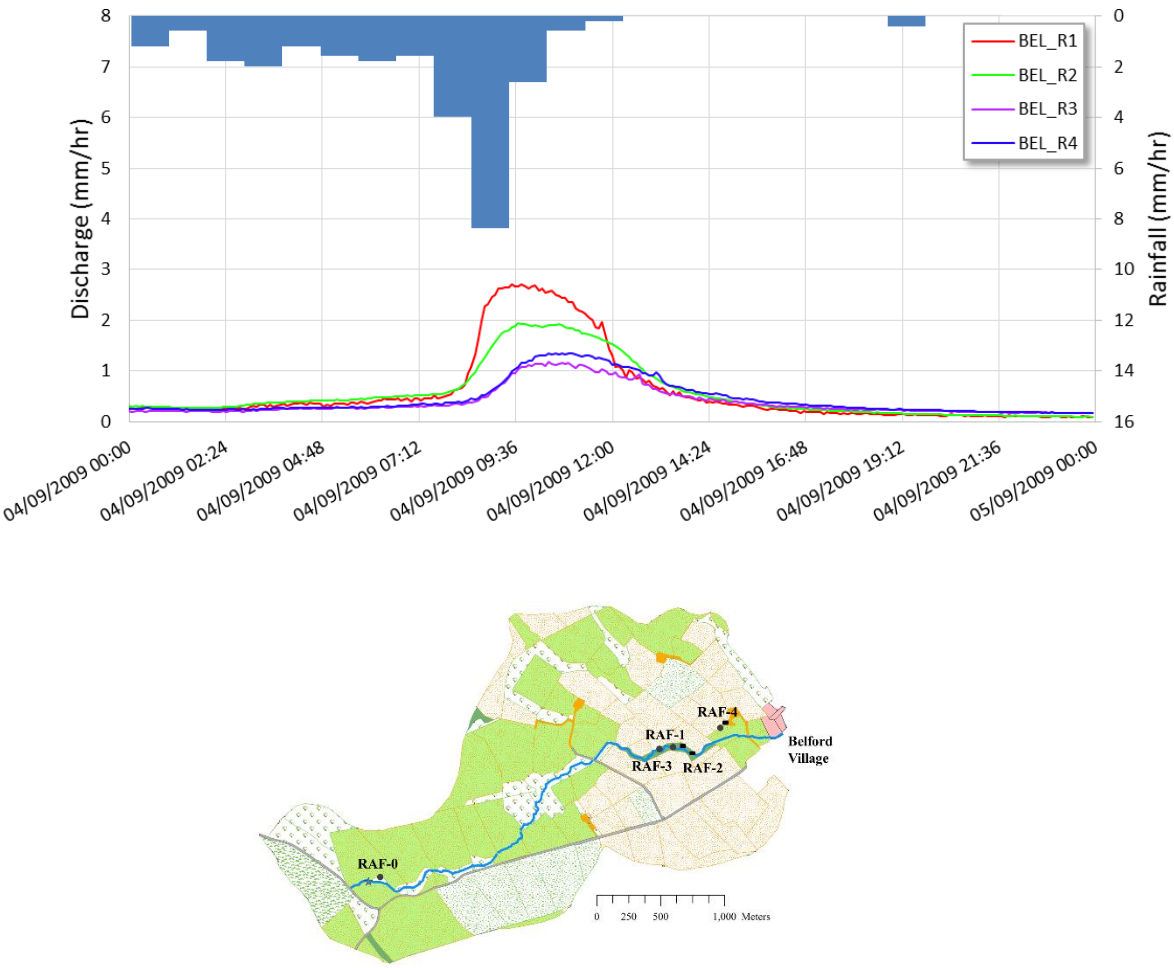


Figure 6.6: September 2009 storm event (catchment map, at time of event, inset)

Both the November 2009 storm event (Figure 6.5) and the September 2009 storm event (Figure 6.6) demonstrate the impact of a localised rainfall event occurring at the top of the catchment (near the R1 gauging station). The response at R1 is very flashy, which could be attributed to the steep slopes at the top of the catchment, and it appears that a lot of natural attenuation occurs between this point and R2. Dry catchment conditions in the month leading up to this storm event may have contributed to a lack of response downstream of the R1 gauging station.

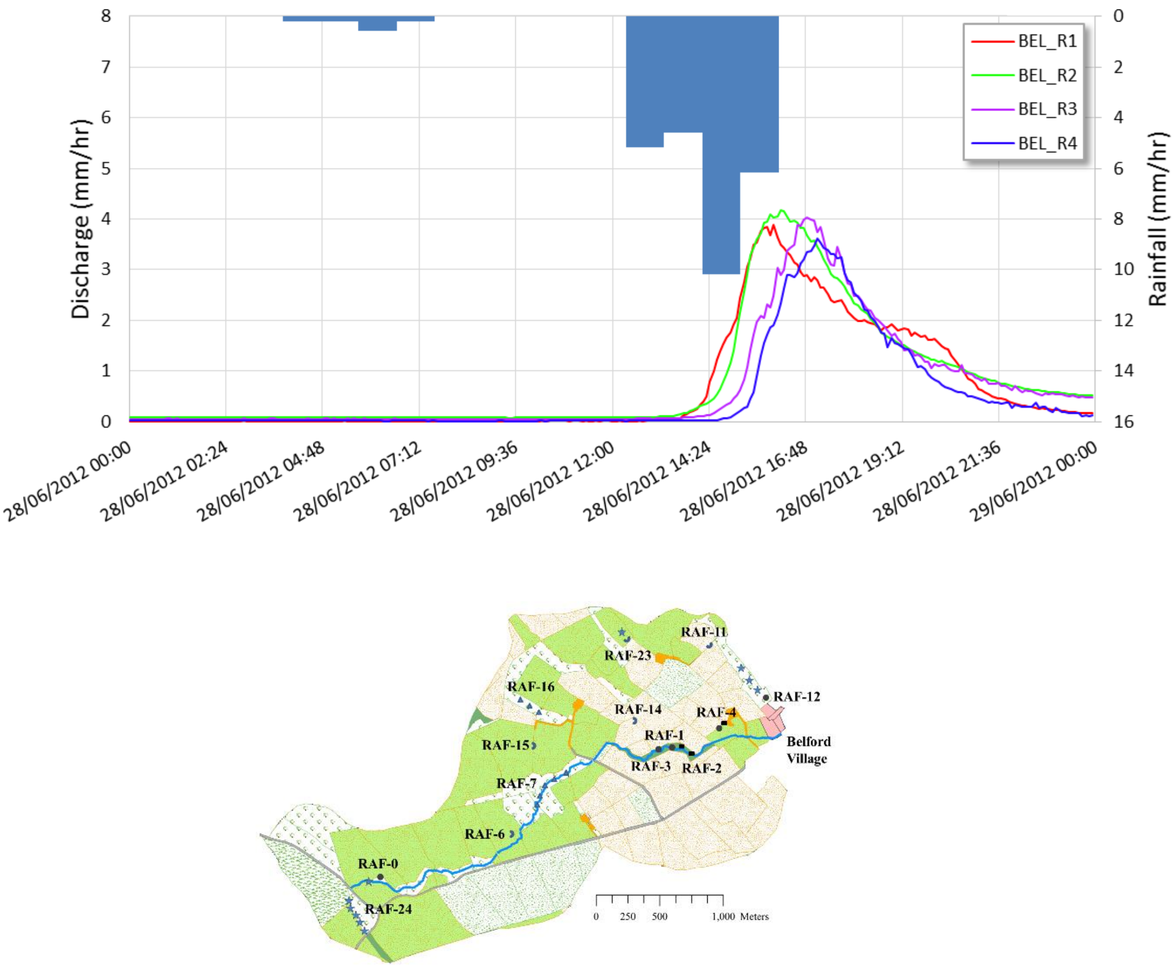
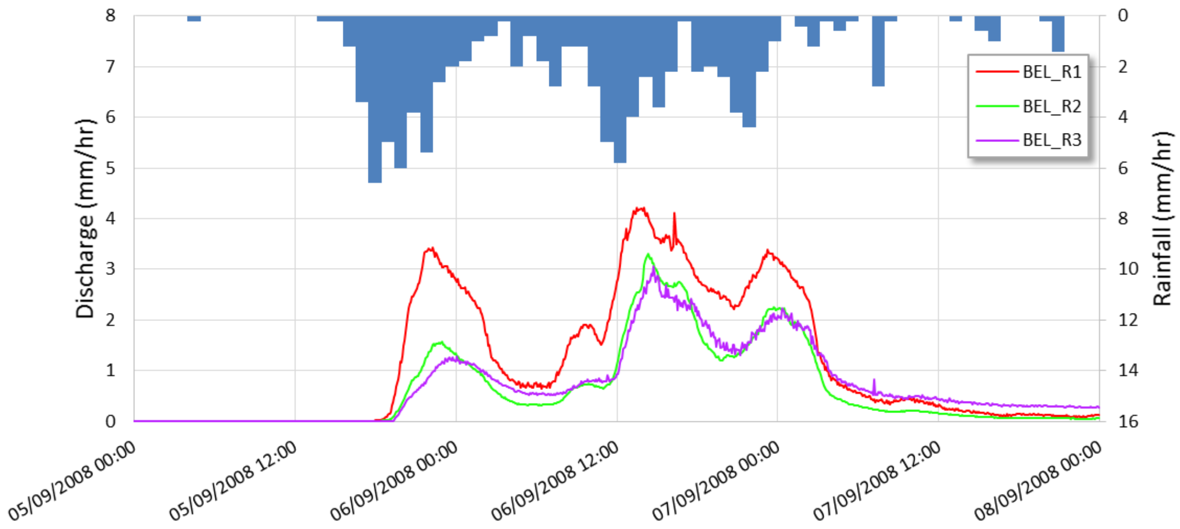


Figure 6.7: June 2012 storm event (catchment map, at time of event, inset)

The June 2012 storm event (Figure 6.7) reached a high peak discharge after 4-hours of intense rainfall. Similar in magnitude to the July 2009 storm event and the January 2010 snowmelt event (see Chapter 6 – 6.4.2 and 6.4.3, respectively), the data from July 2012 will be used to demonstrate that it is not only magnitude of storm events, but also duration, that increases the risk of flooding to Belford. The June 2012 storm event, although short and intense, followed a month of substantial rainfall, which indicates the evident lack of antecedent storage throughout the catchment. This explains the strong signature between the rainfall and runoff. The runoff coefficients for all sites were approximately 0.4 for this event, which is notably smaller than runoff coefficients recorded in other storm events. This highlights a difference in catchment response for short events in the summer months. The rainfall total for June 2012 was 162.6 mm, which is the highest monthly total recorded during Belford’s monitoring period (starting in January 2008).

6.4.2 Multi-peak storm events



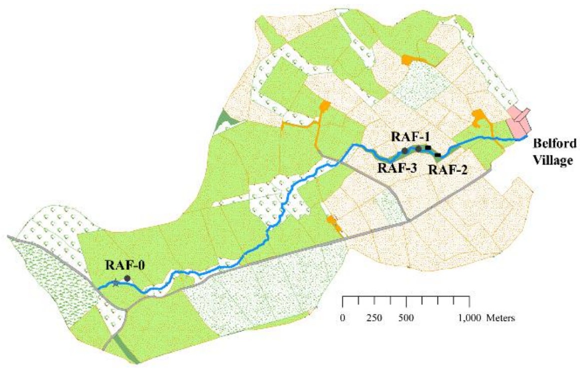


Figure 6.8: September 2008 storm event (catchment map, at time of event, inset)

The September 2008 storm event came approximately one-month after the construction of RAF-0 and just days after the completion of RAFs 1-3. The event was the result of two days of heavy rainfall. The storm produced heavy rainfall over a vast area of the North-East and caused severe flooding of the nearby town of Morpeth. The hydrograph for the event (shown in Figure 6.8) shows a great deal of discharge generated at the top of the catchment (R1) (with a runoff coefficient greater than 0.7 at the peak of the storm), which seems to dissipate to a lower magnitude as the peak reaches R2 and R3 (to approximately 0.5). The rainfall return period of the event was 1:25 years for 24-hours.

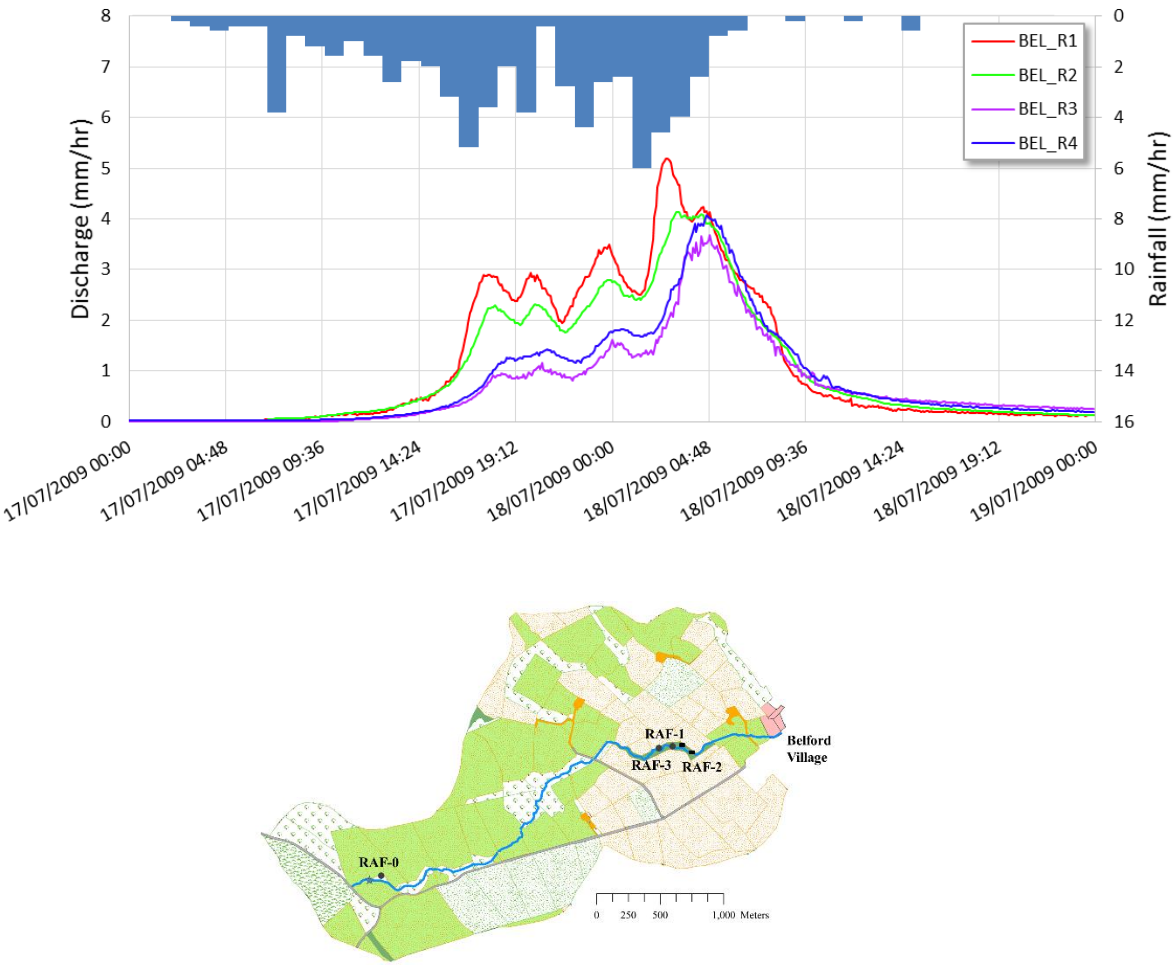


Figure 6.9: July 2009 storm event (catchment map, at time of event, inset)

The July 2009 storm event is an interesting example (see Figure 6.9), as it is the first major storm event to include more downstream gauging stations. The rainfall return period of the event was just under 1:15 years for 24-hours. The coefficient of discharge during the peak of the storm rises to 0.85 at the R1 gauging station and approximately 0.65 for the other gauging stations. For the first time in the data, a change in the pattern can be seen as the flood-wave propagates downstream. The R4 gauging station exhibits a greater specific discharge than R3. A major contributing factor to this scenario could be the change in land-use between the upland and lowland areas of the catchment, which changes from pastoral to arable just downstream of the R3 gauging station.

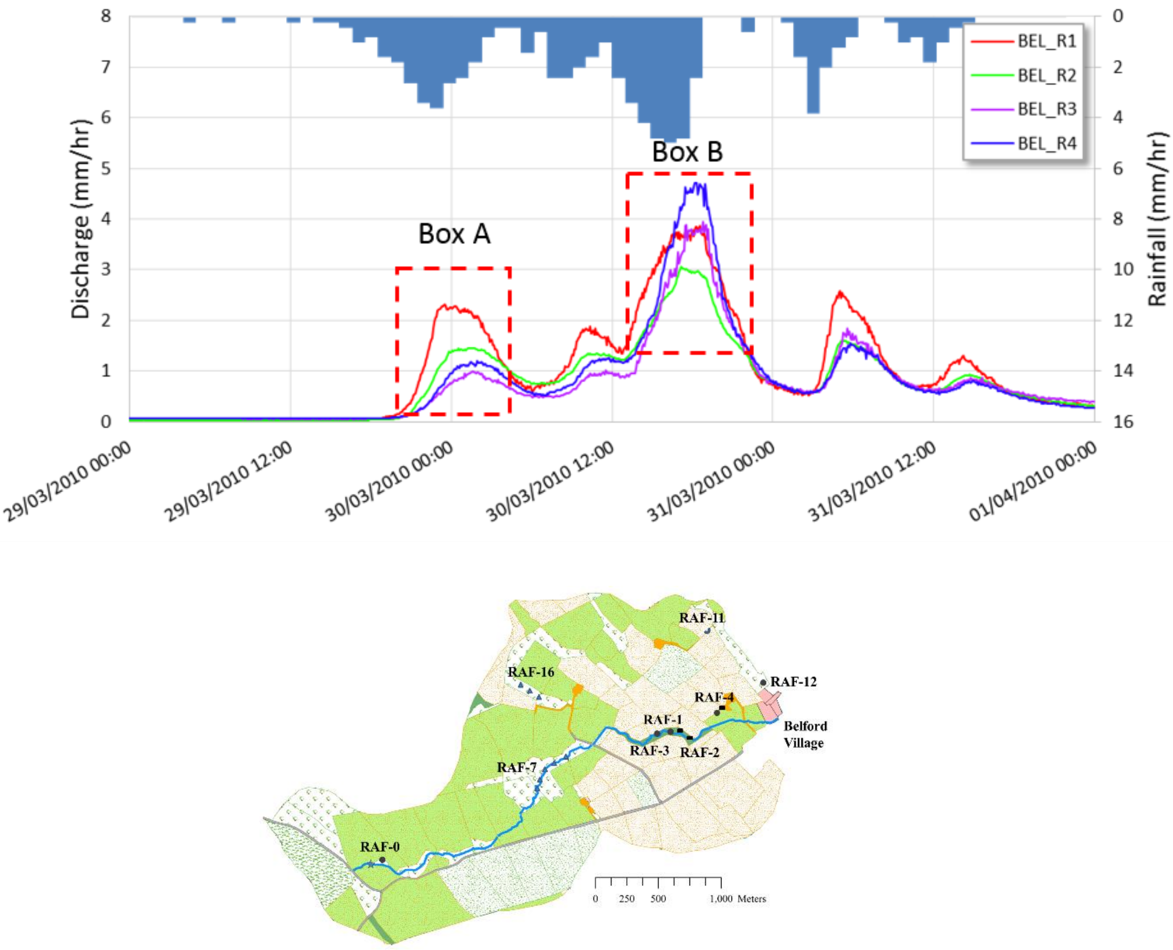


Figure 6.10: March 2010 flood event (catchment map, at time of event, inset)

The March 2010 storm event (Figure 6.10) followed a 1:15 year rainfall return period event for 24-hours. An interesting observation was made regarding specific discharge between R2 and R3 when changing from low flows (see Box A and B in Figure 6.10) to high flows. Boxes A and B show the position of R2 and R3 specific discharge, respectively. In Box A, R2 has greater specific discharge, whereas in Box B the specific discharge at R3 has a greater magnitude than R2. This identifies the soil capacity being exceeded during a storm event. Once this occurs the hydrograph follows the signature of the rainfall to a much higher degree and runoff is present throughout the catchment (see Figure 6.11).

Figure 6.11: Fast flow pathways in Belford during March 2010 flood event. Photograph taken on 30/03/2010 at 12:00 (Note: This is overland flow; not the river)

Substantial photographic evidence from field visits during storm events have captured runoff sources, which builds upon the hydrological data gathered from the observation points along the river. However, field investigations in Belford have indicated that it may not be simply the soil capacity that buffers flow during the wetting-up period. Evidence from hydrograph analysis has indicated that a considerable capacity for flow generation exists between R2 and R3. The prefeasibility study performed by Halcrow (2007) referenced the discovery of a swallow hole along the route of the Belford burn during a site visit by the EA in 2006. However, during latter surveys by Halcrow geological staff, working on the prefeasibility study, no swallow holes were encountered. In spite of this; further field investigation and analysis of geological maps were performed as part of this PhD study and have led to the conclusion that some water is escaping the river network through the river bed in Blagdon Dean, which is the riparian zone between R2 and R3. The geological maps showed both the presence of faults and fissures in the bedrock and a strip of Tyne Limestone Formation both intersecting the river in Blagdon Dean. Site visits have confirmed the presence of a swallow hole within Blagdon Dean (see Figures 6.13-6.14). The geological features present in Blagdon Dean have caused areas of the riparian zone to sink several metres below their original elevation. A small fork of the Belford Burn flows directly into this swallow hole, which then disappears into the ground (Figure 6.13).

The hydrograph also indicates that when there is more flow in the river, the effect of the geology is greatly reduced (see Box B in Figure 6.10). There are a number of possible explanations for this. Firstly, water level in the limestone fissure may have reached capacity by the time the higher flow is present in the river. Secondly, the fissure may require more time for infiltration, which higher flows do not allow. Thirdly, the capacity of the fault may be only capable of carrying a small amount of discharge away from the river; therefore, at higher flows the impact of the fissure appears much lower.

Figure 6.12: Sink in Blagdon Dean. Part of Belford Burn flows directly inside.

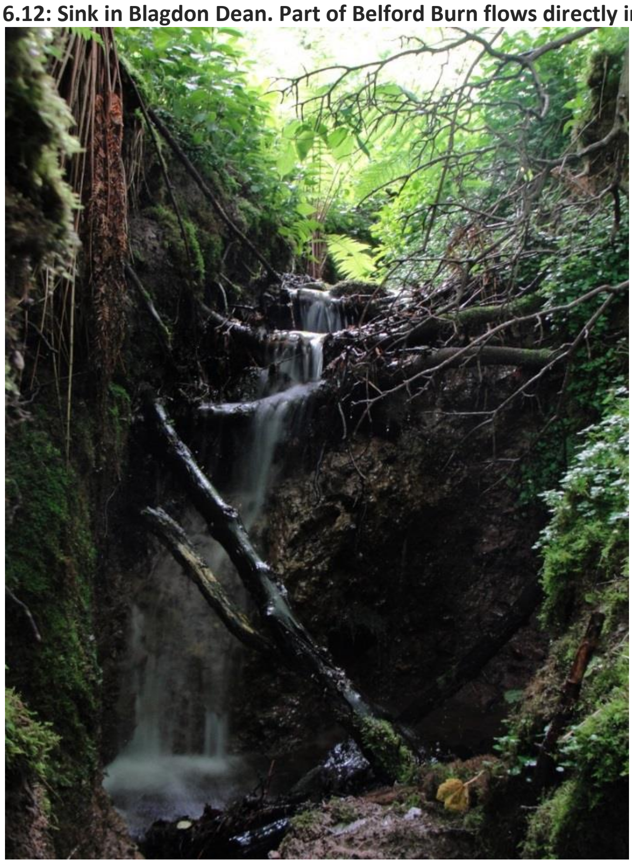


Figure 6.13: Major drain of the Belford Burn flowing into swallow hole

6.4.3 Rain on snow events

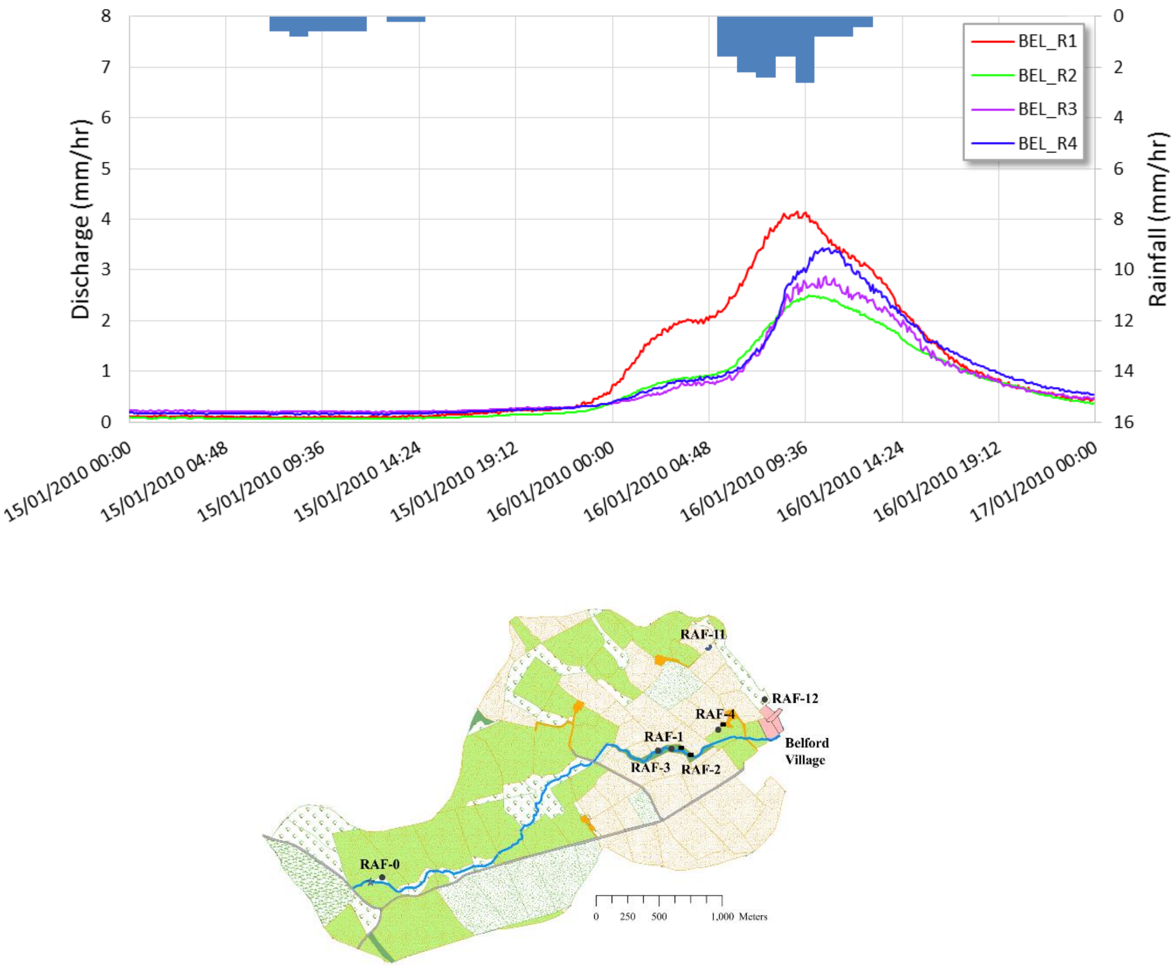


Figure 6.14: January 2010 snow melt (catchment map, at time of event, inset)

The January 2010 event (Figure 6.14) was the result of a combination of raised temperatures and low levels of precipitation falling on a snow-covered upper catchment. The recorded levels and discharge within the river network became a benchmark event for the study, as it was the smallest recorded event that caused flooding in the village. The precipitation magnitudes, in the form of rain and snow, are very difficult to quantify for this type of event. This is due to the raingauge funnel either getting blocked by snow or freezing as a result of the low temperatures.

6.6 Summary

The analysis of storm hydrographs from different gauging stations has indicated that the Belford catchment has a greater response to rainfall at the top of the catchment, where the soil is shallow and the slopes are steep. Over long-lasting events or following periods of prolonged, low magnitude rainfall the soil capacity is exceeded and the catchment shows a more uniform response to rainfall at all of its gauging locations. Geological features between R2 and R3 gauging locations have been shown to potentially buffer flow between the two sites, however, this phenomenon becomes less influential at higher river flows. The observed storms from Belford have been classified by return period and it has been concluded that an FEH approach may under predict the return period of a design storm, making it potentially unreliable for analysis.

7. Mitigation approach in Belford

7.1 Introduction

The Halcrow report (discussed in Chapter 1 – 1.2) identified that an engineering solution to the flood problem in Belford would come at an extremely high cost; c. £3.5 Million to protect 34 properties (Halcrow, 2007). In this chapter the application of different forms of mitigation will be discussed in detail.

Common methods of protecting urban areas from flooding are through the construction of flood defences, like walls, levees or large storage reservoirs. Traditional flood defences were not applied to Belford owing to the high cost, lack of space for flood walls/embankments (Figure 7.1) and failing to meet the criteria for Grant-in Aid funding because of the small number of properties at risk (see Chapter 3). Growing pressure from the Belford residents urged the EA to respond with funding from their Regional Flood Levy Team. In addition to the more traditional forms of mitigation (channel dredging/widening and some improvements to the drainage network within the village) the funding was used for the installation of mitigation features upstream of the village (as well as the hydrometry installed within the catchment).

Figure 7.1: Belford Burn in flood (Note the severe lack of space)

The theory behind the mitigation is to introduce storage and attenuation in the catchment area upstream of the village. Research, performed at the plot scale (<1km^2), on Nafferton Farm discovered small reductions in discharge as a result of soft-engineered wetlands and leaky barriers installed within a ditch network (Quinn, et al., 2007). The approach taken in Belford aimed to apply these features on a larger scale in order to reduce rapid runoff from farmland and reduce discharge in the headwaters of the catchment (Wilkinson, et al., 2010a). The features installed in the Belford catchment became known as Runoff Attenuation Features (RAFs). After consultations with land owners, and some initial speculation, the first RAFs were installed in the Belford catchment, which included a permeable timber barrier, installed in the top of the catchment, the offline RAFs 1 and 3, and the online RAF 2. A storm event followed the construction of these first features, which (visually) demonstrated their effectiveness to the land owner. Since then, the land owner has been much more open to the concept, and the project expanded. There are now 35 RAFs installed within the Belford catchment (Figure 7.2), with a total storage of more than 8,000 m^3 (see Table 7-2); and more have been proposed by the EA. In the context of flooding, RAFs can be typically defined as one of four types:

1. **Offline diversion ponds:** Water is diverted from the stream network into a pond structure creating temporary storage. The pond is typically drained by an outflow structure in the form of a plastic pipe which is constructed to allow the feature to drain in 4-24 hours.

2. **Permeable timber barriers:** Rather than strictly adhering to the design criterion specified for each type of feature, a more opportunistic and ambitious structure was chosen to save space and soil within certain areas of the catchment.

3. **Overland flow interception**: Involves creation of a bund (soil, wood or stone barrier) across a flow path to create storage. These features are designed to drain slowly; the barrier may be ‘leaky’, have an outlet drainage pipe installed, or often incorporate both of these options.

4. **Large woody debris and online ditch management**: Logs and branches are situated across the stream, or interlaced in ditches, to increase hydraulic roughness. Online-barriers constructed from natural materials are located in drainage ditches and streams to cause backwater effects. Often widening drainage ditches is inexpensive and additionally creates an effective sediment trap and new ecological habitats.

7.2 Design of a RAF network

7.2.1 The Belford RAF Network

The majority of the Belford RAFs have divers installed at the deepest points within them in order to understand how they fill and empty during and after a flood event (Table 4-1). The function of the RAFs varies based on position within the catchment. Some have been constructed to reduce peak river flow; others have been installed to intercept and slow fast overland flow and; some have been positioned in order to filter flow and improve water quality at the catchment outlet.

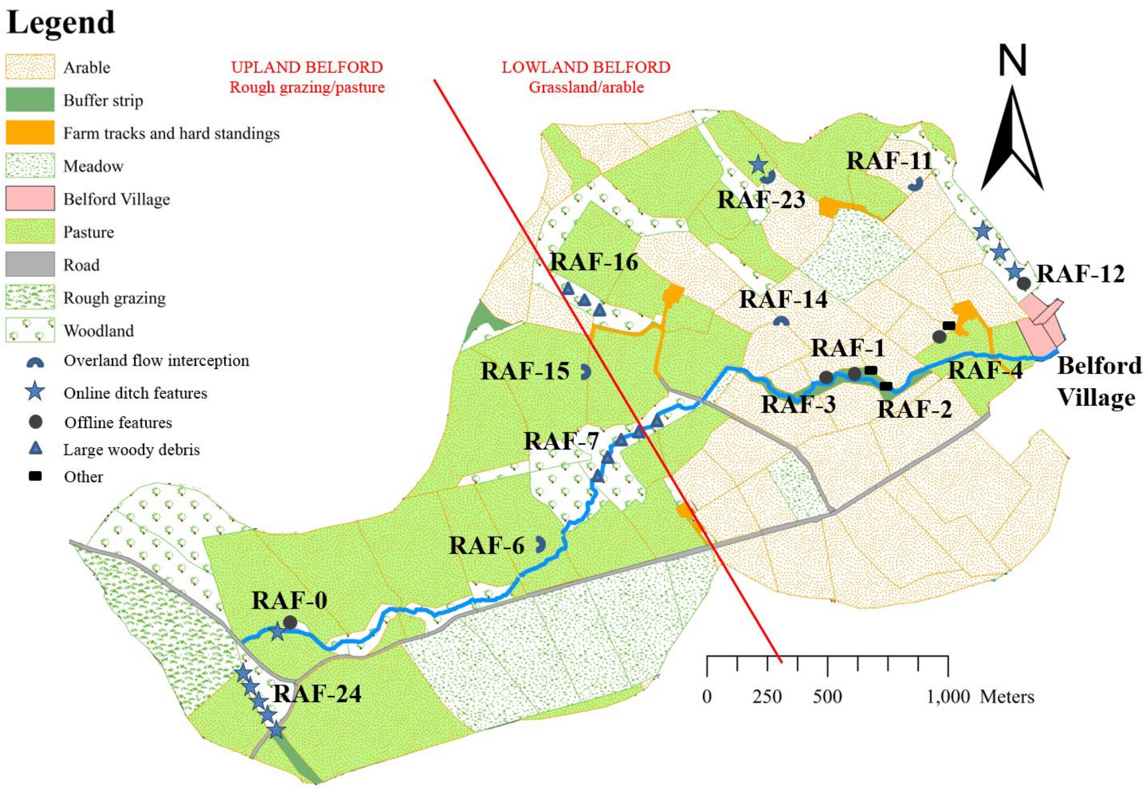


Figure 7.2: Upper Belford Burn catchment with locations of RAFs

The location of RAFs within Belford required careful negotiation with land owners but did not, particularly, follow a set template. At the beginning of the project, those involved could only approximate the impact of the proposed features, which ensured that many different types of RAF were considered for construction and testing. The results of topographic analysis would be compared with satellite images and site visits to ensure the proposed locations were sensible prior to opening discussions with the land owner. Sizes and function of RAF were decided based on the particular chosen location, which meant that RAF designs were entirely dependent on the location (even when considering individual aspects of the design e.g. material choice). This approach has had benefits in understanding the best features to consider for certain locations and has helped to develop a methodology for constructing RAFs in other catchments (see 7.2.2).

Total cost for the Belford project included the research carried out by Newcastle and some scoping studies. The cost to the EA regulators who attended meeting are not included. Table 7-1 only includes construction costs (i.e. no maintenance). The table infers that a RAF network scheme could be as cheap as £37K, but in reality a mixture of features are used and occasionally higher costs are worthwhile. Hence, the cost for the Belford scheme at the time of this report is in the range of 70-100K. The higher costs include several features that proved to be more expensive. Clearly the multiple benefits of the features have not been analysed. Finally, the comparison of the Belford RAF network scheme costs have not been compared to the construction costs of a single flood storage reservoir of 10,000-20,000m^3 capacity, but it is assumed that this would be much higher and that the regulatory framework and maintenance regime would be much more complex.

Table 7-1 Features built in Belford and estimates of typical capacity and cost. (Consultancy and research costs are not included)

Feature type	Number built	Typical min, max storage m^3	Estimated cost
Overland flow interception	5	300-1000	1K-5K
Online ditch features	9	50-150	1K-3K
Offline ponds	5	200-3000	2K-6K
Large woody debris	8	50-150	1K-3K
Other opportunistic sites	3	100-3000	1K-10K
TOTAL	**30**	**Estimate for Belford 8,000m^3**	**£70K-100K**

7.2.2 "Where should a RAF be located?" (Quinn, et al., 2013)

This sub-section is based on the findings of Quinn, et al. (2013), which included myself as a contributing author.

There is no fixed design for locating a RAF other than identifying fast flow pathways and choosing an appropriate site to target the peak flow of a storm. Typically, the identification of priority sites for RAFs should consider the whole catchment above each flood risk site and determine the flood generation processes likely in those areas. Farmers often possess this knowledge, which can be augmented with simple flow accumulation rules to identify suitable locations (Heathwaite, et al., 2005). Walk over surveys are also beneficial; often evidence of overland flow can be observed in swales and valley bottoms. Identifying specific RAF locations can also be undertaken using GIS and topographic survey data. The schematic shown in Figure 7.3 illustrates an idealised network, and also provides design rules regarding the appropriate scale at which features should be installed.

It is appropriate to raise a number of salient points that should be considered in the planning stages. It is imperative that the construction of a RAF network involves a partnership between regulatory bodies, farmers and land owners, the local community and other relevant stakeholders. Experience has shown that a transparent and inclusive process avoids later complications and quickens the regulatory process. Keeping as many people informed as possible, for example through Town Hall meetings, often encourages additional uptake of the approach by farmers, land owners and land managers.

There may need to refine individual features to improve their functioning during flood events; often visual observation during an extreme event can be used to determine whether an inlet feature or drainage rate requires modification. Any indication of feature scouring or collapsing can also be reported. It is beneficial to engage those in close proximity to the sites, including farmers or local farming advisors, to conduct visual surveys.

There is anecdotal evidence that land owners and those living within the catchment prefer the use of natural materials in the construction of RAFs, purely from an aesthetic standpoint. Many different materials were used in Belford, which can be broadly grouped into 4 categories:

1. In a more upland area with livestock and thin soils, a farmer preferred the use wood or stone as soil bunds require more space and use up valuable soil.
2. In a lowland area with arable crops the use of soil bunds was preferred by the farmer as the soil was thicker and readily available. There was more available space in the field and the farmer aided in construction.
3. Online ditch features can be created using treated wood or willow spilings. Experience of using recycled plastic has proven to be less desirable for aesthetic reasons (Letts, 2012). The widening of ditches yields material that can be used to construct bunds either in the ditch or to raise the ditch bank height.
4. Clearly large woody debris requires tree trunks. Experience, has shown that the EA team would prefer the removal of non-native species (sycamore in Belford) with replacement

with native species (such as oak). The brash created by the tree felling can be used to roughen the local floodplain.

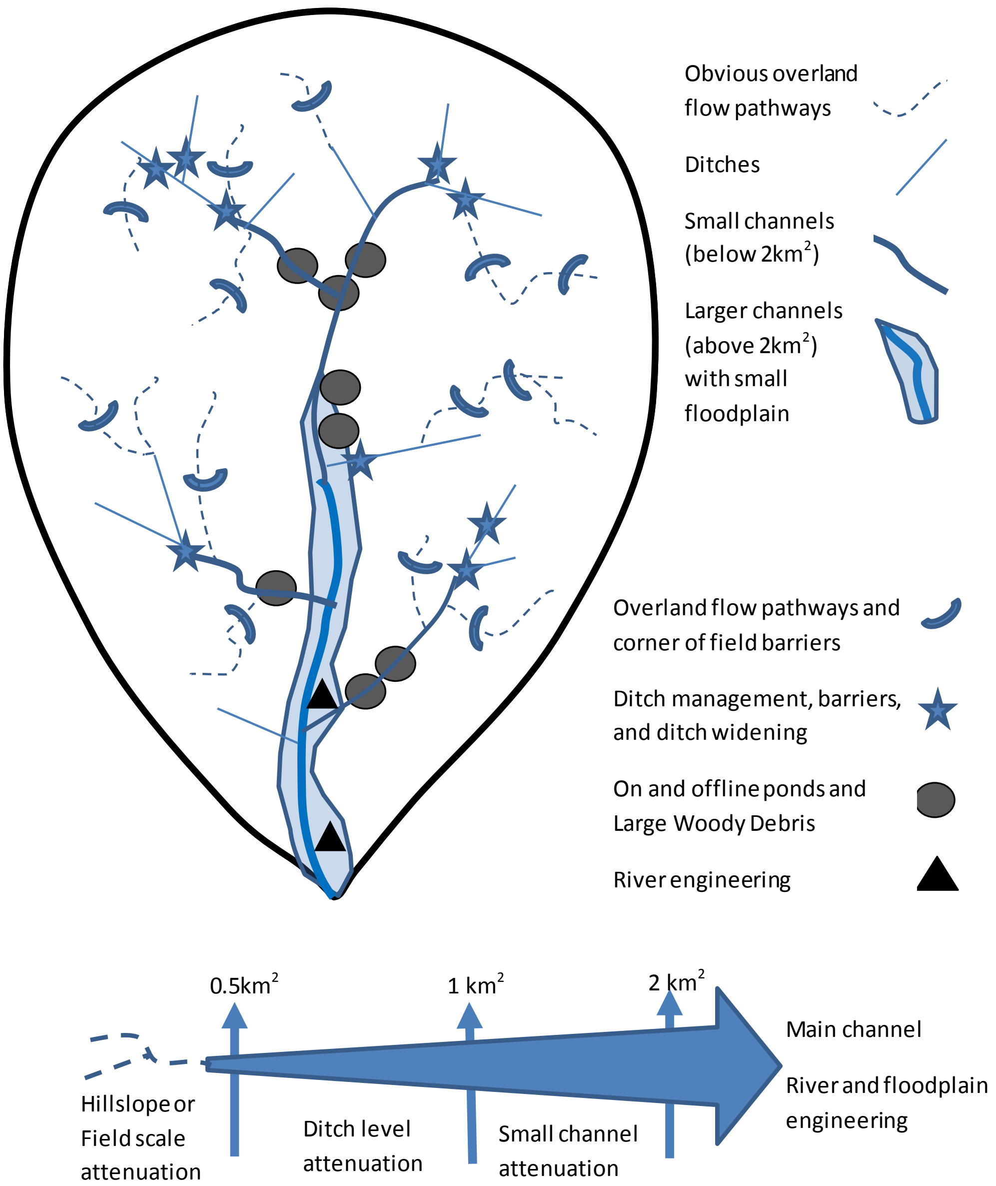


Figure 7.3 Design of a RAF network (Quinn, et al., 2013)

7.3 Determining potential storage of RAFs

7.3.1 RAF surveys

In order to validate the rationale for the Belford flood mitigation (see 7.4), accurate surveys were required for all of the storage features installed in the catchment. Many of the storage features in Belford were surveyed using Global Positioning System (GPS) surveying equipment known as a Real Time Kinematic (RTK) GPS device. This device uses both Global Positioning software and mobile phone reception to obtain points with reference to the British National Grid that can be uploaded into ArcGIS (Figure 7.4). A novel approach of pacing the length and breadth of the RAFs whilst storing the three-dimensional coordinates, at multiple locations, allowed the surveys to accurately represent the shape and capacity of the installed features.

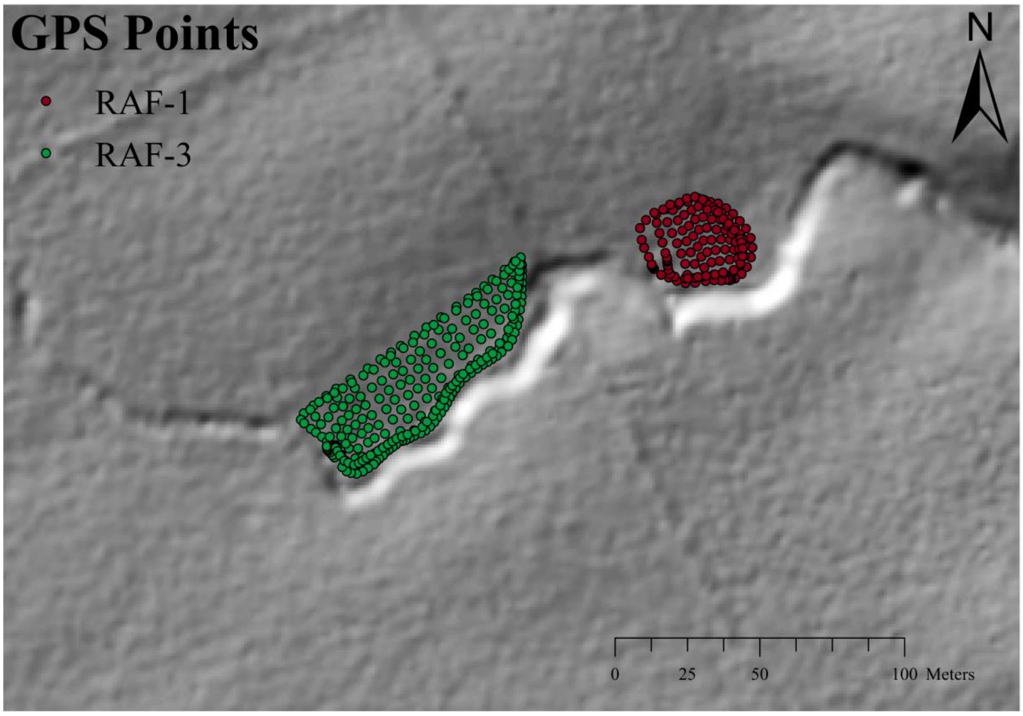


Figure 7.4: RTK points of RAFs 3 and 1 (left to right) uploaded to ArcGIS as a shapefile

The results from the RTK can be accurate to 0.01 m, depending on reception. The points are then uploaded and geo-referenced in ArcGIS. Once the points had been put in place and referenced to their proper position alongside Belford Burn, it was possible to create a Triangulated Irregular Network (TIN) surface. A TIN surface interpolates between all the points in a 'shapefile' and creates a surface representing the shape and capacity of the ponds (Figure 7.5).

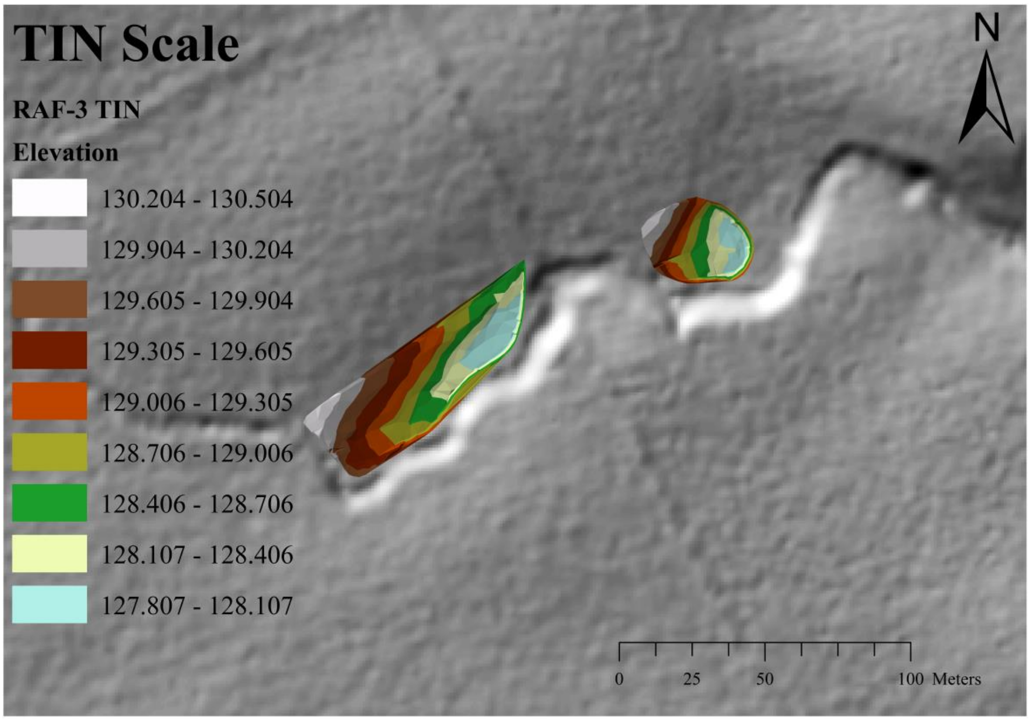


Figure 7.5: Shapefile of RAFs 3 and 1 (left to right) converted into a TIN

With the new TIN surface in place, it is possible to discover the potential storage dynamics of the RAFs using the 3D Analyst Tool in ArcGIS. A set of look-up tables were made, using this information, to show the relationship between RAF stage and volume in order to make any analysis of the RAFs more simple. The volumes were calculated for a number of RAF levels (every 0.05 m) using the 3D Analyst tools in ArcGIS. These surveys were critical for the development of the analytical method for determining the impact of RAFs during storm events, and the creation of a bespoke modelling tool called the Pond Model (described in Chapters 8 and 9, respectively).

7.3.2 Analysis of the survey data

The creation of the lookup charts allowed the observed levels in the RAFs to be converted into a volume (Figure 7.6). It will be demonstrated, in Chapter 8, that these lookup charts can be used to help assess the effectiveness of storage ponds during observed storm events. An understanding of the volume entering or leaving the RAF at every time-step allows an estimate of the inflow/outflow relationship to be made. The change in volume over the time-step (dv/dt) is measured in the same units as the discharge in the river (m^3s^{-1}).

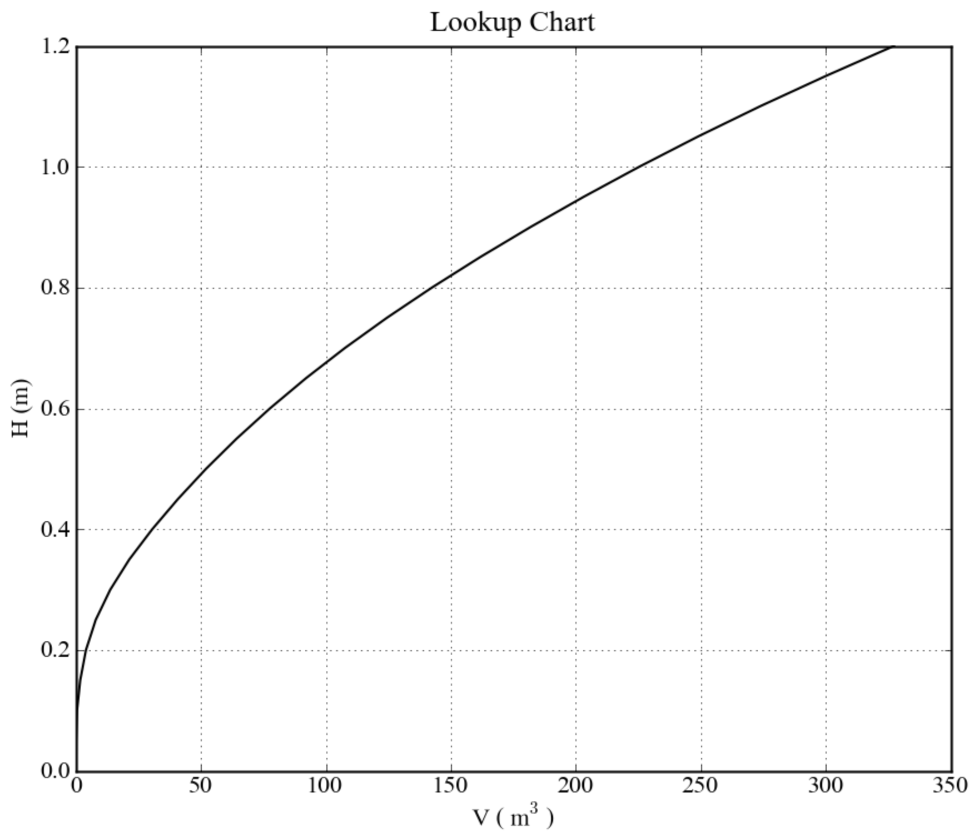


Figure 7.6: Pond-volume lookup chart for RAF-1

Table 7-2 shows the surveyed volumes of some of the RAFs currently installed in Belford. The volume of each of the RAFs has been converted into a potential storage (in mm). The reason some RAFs have not been surveyed is due to their proximity to woodland, which blocks the GPS and mobile-phone signal from the RTK device. It will be possible to survey these RAFs using other techniques, although the advantage of the RTK is that it can be operated by a solo user.

Table 7-2: Summary of RAF locations and storage capacities

* Volume measured with Level and Staff, so considered a reasonable estimate. All other volumes measured using more accurate RTK GPS device.

Site	Easting	Northing	RAF Type	Hydrometry	Volume (m^3)	Contributing catchment area (km^2)	Potential Storage relative to upstream area (mm)	Potential Storage relative to total catchment area (mm)
RAF-0	407596	632833	Offline Pond – adjacent to river	Yes	1000*	0.50	2.000	0.175
RAF-1	409802	633831	Offline Pond – adjacent to river	Yes	330	2.72	0.121	0.053
RAF-2	409943	633838	Online Pond – connected to river	Yes	410	2.99	0.137	0.072
RAF-3	409722	633840	Offline Pond – adjacent to river	Yes	370	2.65	0.140	0.065
RAF-4	410297	634056	Offline Pond – in prominent flow-path	Yes	3000	1.00	3.000	0.351
RAF-6	408610	633138	Offline Pond – in prominent flow-path	Yes	1000*	1.30	0.769	0.175
RAF-7	409137	633707	Woody Debris – in river (6 structures)	No	100*	2.10	-	-
RAF-11	410166	634648	Offline Pond – in prominent flow-path	Yes	500	0.30	1.667	0.088

RAF-12	410456	634347	Online Pond – Connected to ditch	Yes	150*	0.35	0.429	0.026
RAF-14	409589	634084	Offline Pond – in prominent flow-path	No	450	0.70	0.643	0.079
RAF-15	408803	633896	Offline Pond – in prominent flow-path	No	200	0.50	0.400	0.035
RAF-16	408908	634092	Woody Debris – in ditch (3 structures)	No	-	0.15	-	-
RAF-23	409553	634523	Offline Pond – in prominent flow-path	No	500*	0.15	3.333	0.088
RAF-24	407449	632455	Wooden Screens – in ditch (5 structures)	No	150*	0.15	1.000	0.026
				Total =	**8160**			**1.458**

7.3.3 Inlet features and effective 'timing' of mitigation

Combining the observed data, recorded by instruments in the RAFs, and the surveyed measurements of RAF volume allows the development of a 'theoretical solution' to the flood issue. If it were possible to coincide when the offline RAFs begin to fill with the passing of the flood peak, then theoretically the total available storage of the RAFs could be removed from the peak of the hydrograph (like the one inset in Figure 7.7). This theory does mean that if all the inlet heights were adjusted to have maximum impact on a large flood event, like March 2010 (Figure 7.8), then they would have little or no impact on storms of a smaller magnitude. The question is, whether the features *need* to have an impact on storm events of lower magnitude than the March 2010 flood? If the answer to that question is 'yes' it highlights the need for a range of offline storage features; some designed for flood events of low magnitude and some designed for much larger magnitude events.

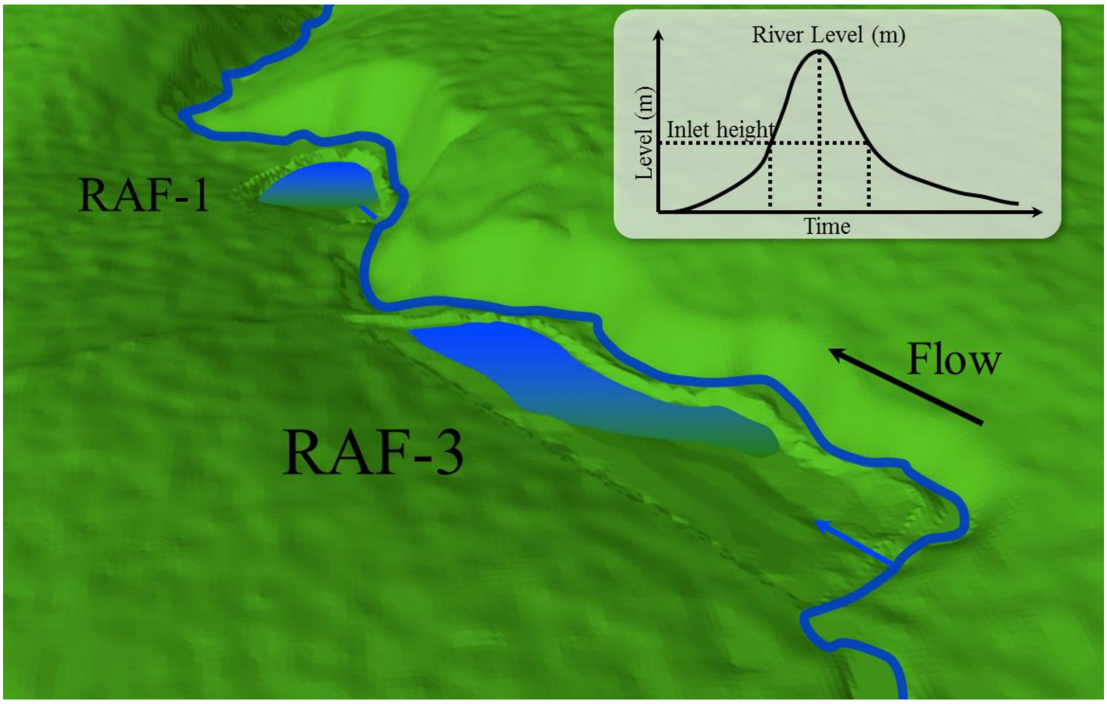


Figure 7.7: Schematic of Belford Burn showing RAFs filling as a result of river-level increase

7.4 Rationale behind mitigation in Belford

For RAFs, the magnitude and duration of the flood-peak has a huge impact on the type of intervention required. The desired impact of the RAFs should be determined at an early stage in a project, so that it becomes possible to identify what magnitude of storm they are being designed to attenuate. The greater the flow magnitude, the higher chance a particular RAF will have of filling during a storm event. It will be identified, in analyses, that single RAFs (<500 m^3 capacity) will have piecemeal impact on the flood hydrograph during high magnitude events, which is why RAFs must work as a collective unit (or network) during these large events to have significant impact.

Before describing the RAF types in great detail, the rationale for the Belford mitigation will be introduced. In Figure 7.8 the hydrograph and rainfall for a large winter event (March 2010) is shown for Belford. The discharge is expressed as flow per unit area (mm/hr) to allow a direct comparison against the rainfall. The signature of the rainfall is clearly present in the hydrograph shape, indicating a rapid response. During the early part of the storm the rainfall intensity approaches 4mm/hr, with the discharge 1mm/hr, indicating significant antecedent catchment storage. This contrasts with the response at the time of the main peak when discharge (4.5mm/hr) is marginally lower than rainfall (5mm/hr), indicating that storage has been depleted and overland flow is generated (which has been verified from walkover surveys during large events, Figure 7.9).

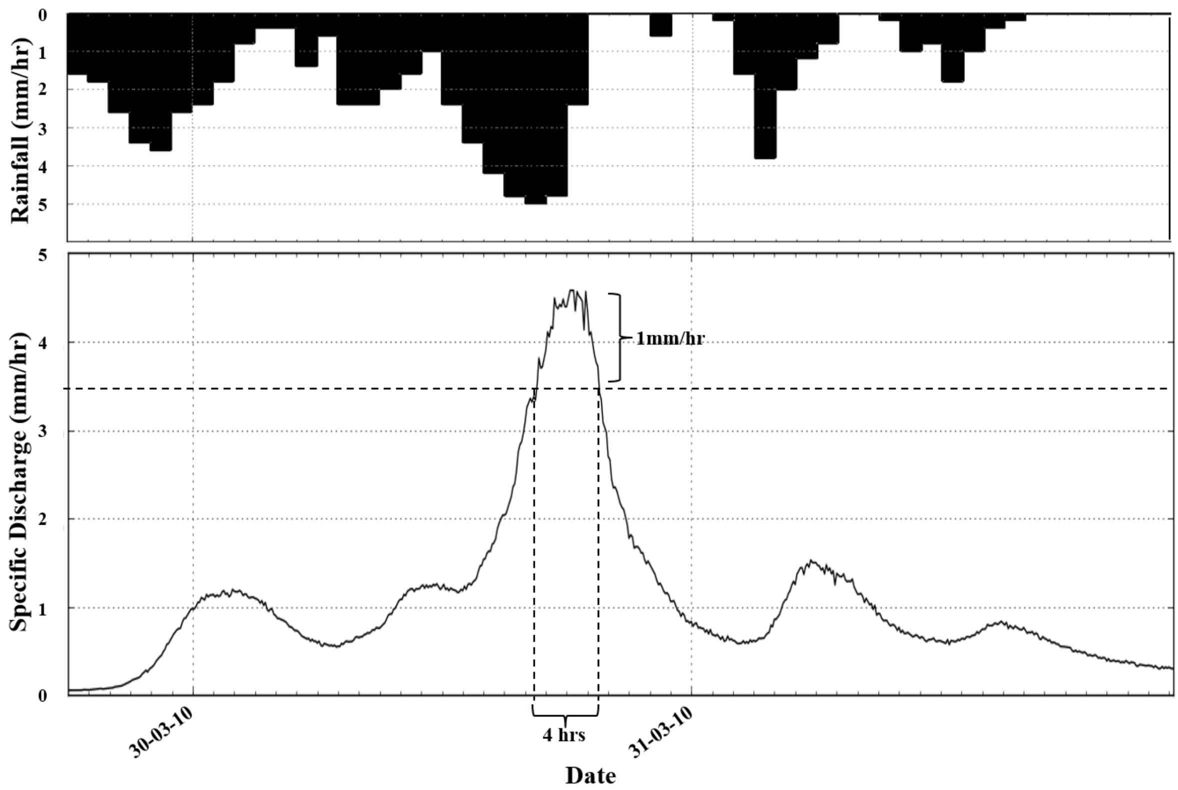


Figure 7.8: Storm event in March 2010 – showing the need to target key components of flow

The horizontal dashed line in Figure 7.8 corresponds to the rate of discharge at which historic flooding has occurred in Belford. The volume of runoff above this line is approximately 20,000m^3 over a duration of 4 hours (calculated by multiplying the catchment area 5.7km^2 by the number of hours and converting into m^3). This simple analysis has been performed to provide an approximate guide to the volume of flow that would need to be managed to reduce flood hazard in the downstream settlement of Belford for this event. In interpreting such an analysis, a number of cautionary notes are required:

- The amount of flood management required will depend on the magnitude of the flood peak and the duration above the desired level of protection
- The flood management interventions need to be active at the time of the flood peak (Nicholson, et al., 2012a)
- Attenuation effects (e.g. due to large woody debris) need to be carefully considered; attenuation and storage work together

To return to the simple hydrologic analysis; during extreme events the drainage network expands with ephemeral flow pathways generated by overland flow linking fields and hillslopes to the ditch and stream network. The RAF approach advocates targeting this expanded drainage network, through the installation of features that attenuate and store runoff.

Figure 7.9: Overland flow generated during the intense storm (taken at 12:00pm 30/03/2010)

It should be pointed out, from Table 7-2, that the total storage surveyed in the RAFs (~8,000 m^3) does not yet match the target storage highlighted in the rationale (~20,000 m^3). However, this value does not include the attenuation and storage created by woody debris and within channel barriers which is more difficult to quantify (specifically, the backwater storage and attenuation effects of these features is a function of stage, which is event specific).

7.5 Examples of RAF operation during storm events

Although RAFs are not considered as purely flood management features, hereon they will be discussed in the context of flood mitigation. As discussed in Chapter 1 (1.3), RAFs can be described as ‘online’ or ‘offline’ mitigation based on their interaction with water within a catchment. Offline RAFs disconnect flow pathways by capturing surface runoff on fields and/or by diverting the peak flow from an adjacent river section to temporarily store flood water during an extreme rainfall event.

RAFs, which intercept runoff and increase floodplain/channel interactions during high runoff periods, are ideally positioned in areas of high surface connectivity or areas where the river and floodplain are able to interact. For this reason, the location of RAFs is generally based on the topographic information available for the site; however, this must be coupled with field surveys to ensure no other factors (e.g. land-drains, ditches or geological conditions) affect the capture of surface runoff.

7.5.1 Offline diversion ponds

Offline RAFs function by diverting flow from the main channel during peak-flow events. An inlet structure situated on the riverbank, which is approximately 1m wide, controls the filling of the pond. RAF-1 has a maximum capacity of 330 m^3. RAFs like this, located adjacent to rivers, remove peak flow through filling when the water level in the stream reaches a certain height (taking 3-4 hours to fill) (Figure 7.10). The RAF, therefore, has the potential to, both, reduce flood peak and increase the lag-time of the flood hydrograph at that point in the river (Wilkinson, et al., 2010a). This RAF was constructed by scraping the soil from the centre of the pond and using that soil to form a bund around its perimeter.

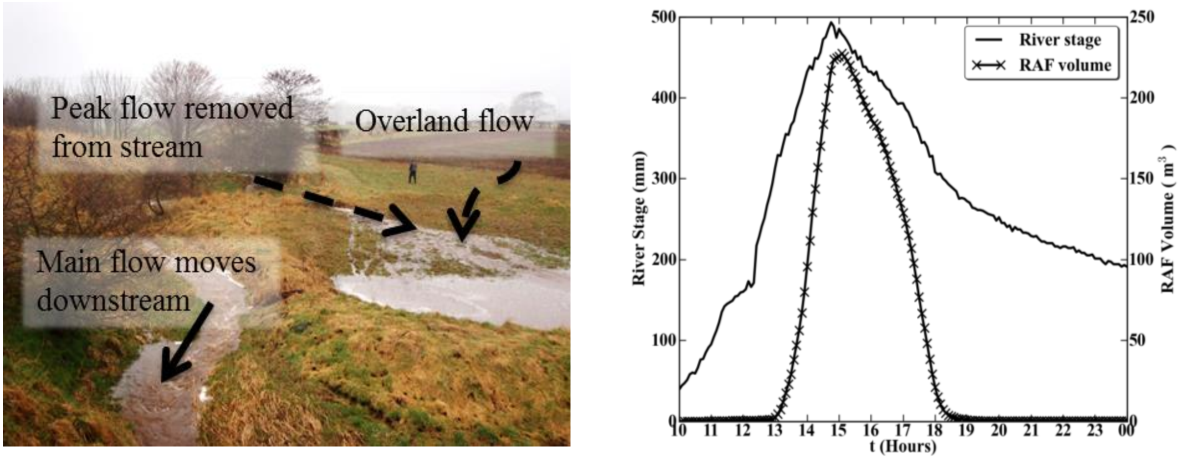


Figure 7.10: RAF-1 during storm event. RAF begins to fill when inlet (set at 300mm) is overtopped (see graph on right)

Figure 7.10 shows RAF-1 during a storm event together with the data gathered during the storm. The graph (in Figure 7.10) shows the relationship between the water level in the stream and the volume of water stored in the RAF. The graph also indicates the inlet height of the RAF (300 mm above the river bed). When the water level in the stream rises above 300 mm, the RAF begins to fill. The RAF drains through an outlet pipe (with a 0.25 m diameter) and is designed to be completely empty before another extreme rainfall event hits the catchment. It can be seen on the graph that the RAF takes just over three-hours to completely drain.

Features like these should be inspected after high flow events for scour at the inlet channel and sedimentation within the feature itself. Surface runoff in agricultural regions can transport huge amounts of top-soil from the fields to natural water courses, field drains, sewers and other pipe networks, and even into properties within the floodplain. This sediment is valuable to farmers, as it is usually the most fertile soil, but potentially damaging to properties, roads and highways, pipe networks and field drains, which can lead to huge costs to local government. Figure 7.11 shows a small amount of fine sedimentation in RAF-1 following the March 2010 storm event. It is argued that this sediment will be nutrient-rich and valuable to farmers. This highlights the need for management of these features to avoid loss of capacity (see Verstraeten & Poesen, 1999), however, if enough sediment collects in RAFs it can be harvested for use as top-soil on nearby fields.

Figure 7.11: Sedimentation in RAF-1 following March 2010 storm event

7.3.2 Permeable Timber Barriers

The first RAF constructed was a pilot pond (RAF-0) to demonstrate the concept to stakeholders and regulators. This RAF is located at the top of the catchment and capable of storing approximately 1,000 m^3 of floodwater, both from the stream and from surface runoff generated in the small catchment area leading up to the feature (Figure 7.12). The RAF diverts peak flow from the stream using a control structure, in the form of a V-notch weir, and stores it during a storm event. During high magnitude storm events the flow diverted into the RAF from the stream can be as much as 15% (see Appendix A.1 – Pond Model output). The pilot pond was constructed by driving timber vertically into the ground, to avoid the necessity to make a pond using the shallow soils of the upland catchment. The small contributing area upstream of the RAF generates overland flow, which is also targeted by the RAF.

Figure 7.12: RAF-0 – Full of water following a storm event in September 2008 (left): from Wilkinson, et al. (2010a)); demonstrating permeability (right)

Water slowly drains through the timber structure of the RAF (see Figure 7.12 (right)), allowing it to continue moving through the catchment (along a tortuous path). The intention of the RAF's design is for it to be completely empty before a further extreme rainfall event. The RAF will fill on the rising limb of a flood wave over 8-10 hours (depending on the severity of the storm) and, once the flood wave has passed, will drain over a 5-6 hour period. This means that for the majority of the time the RAF (and others like it) is empty, which is of huge benefit to the landowner, who does not lose the economic functionality of the area. This fact also readies the RAFs for further rainfall, which may be present in a double-peaked storm event.

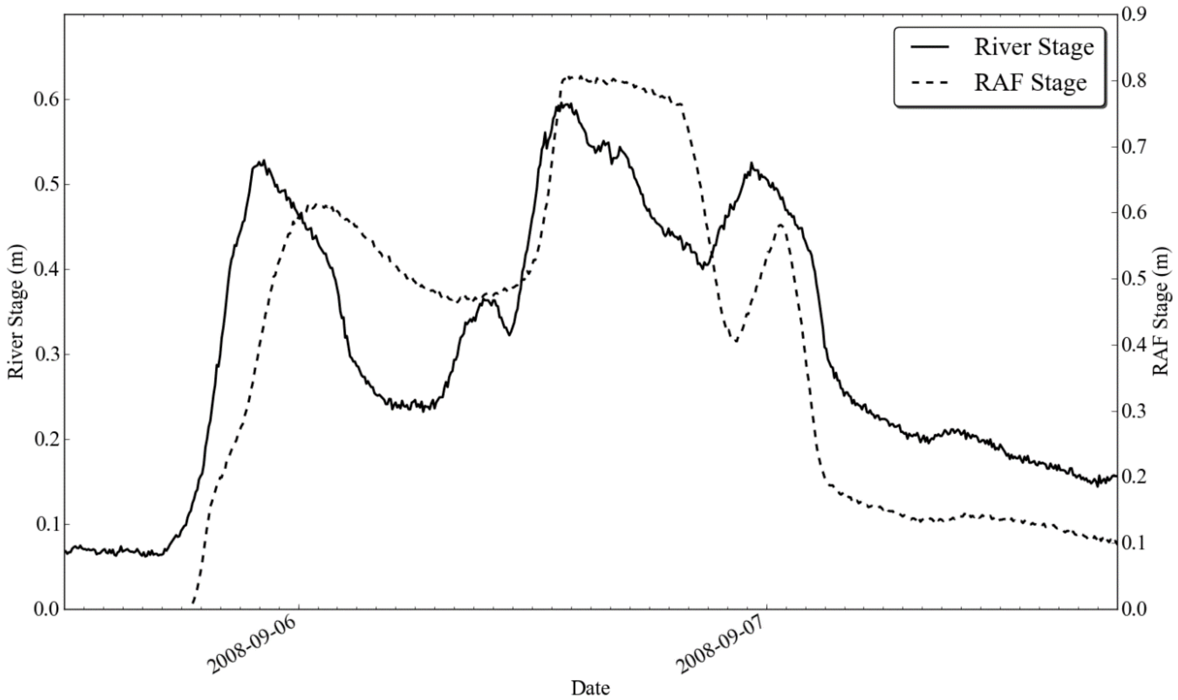


Figure 7.13: Data from RAF-0 during September 2008 storm event (modified from Wilkinson, et al., 2010a)

Wilkinson, et al. (2010a) present a graph of the pilot pond functioning during a double-peaked storm event in September 2008 using data obtained from the instrumentation within the RAF (Figure 7.13). Capturing data in RAFs during storm events is, thus, critical for presenting the evidence of RAFs to stakeholders and extremely useful for suggesting improvements to their function. Data from the September 2008 storm event was used to modify the pilot pond, so that water could drain slightly faster from the structure. Due to the nature of the project in Belford and the strong local support for flood mitigation in the village, a news article was published in the Berwick Advisor with the heading "Pioneering ponds save Belford from Flooding" (on 17/09/2008). It is important to note that, at this point, there were no analysed data to prove that this was the case, though it did show a growing perception of the benefits of RAFs by the local community. This was a huge help in stakeholder and landowner engagement at this early stage in the project.

A similar feature has been installed in a livestock field adjacent to the R2 gauging station (see Figure 7.14). This RAF (RAF-6) uses the same construction method and materials as the pilot pond, but aims to intercept purely overland flow from the long flow pathway (~400 m) produced in the steep catchment area approaching the site.

Figure 7.14: Permeable timber barrier (RAF-6)

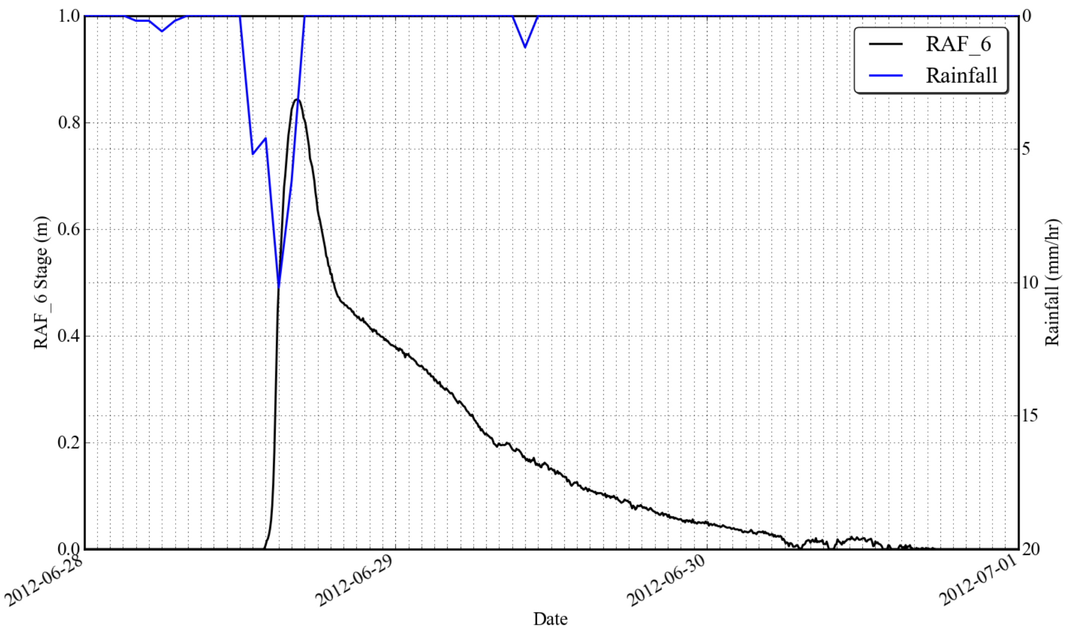


Figure 7.15: Data from RAF-6 during storm event (late-June 2012)

Data from RAF-6 in Belford (Figure 7.15) shows a quick response to a short, intense rainfall event. This identifies that overland flow must have occurred, as water has no other means of collecting in the RAF. It can be inferred that RAFs designed to capture overland flow are targeting the peak of the event, due to the fact that they can only fill as a result of overland flow (generated once soil capacity has been exceeded). The event, which occurred in June 2012 and caused no flooding to Belford, saw 28 mm of rain fall in 12-hours. The 1,000 m^3 capacity RAF achieved a water-level approximately three-quarters of its maximum. The gradient of the RAF level graph indicates that the permeable structure is better at leaking under greater hydrostatic head. There is a clear gradient change between the drainage from 0.83-0.47 m and 0.47-0.15 m. This equates to a rate of change in depth of 0.104 m/hr and 0.023 m/hr, respectively. Due to the lack of survey data for this RAF there is no way to express this change in depth in terms of discharge out of the RAF. It has been estimated, however, that the majority of the storage of this RAF is only available at greater depths. This indicates that the outflow discharge may be significant at greater depths.

7.3.3 Overland flow interception RAFs

An overland flow interception RAF is purely for the interception and storage of overland flow during the high runoff period where there can be, effectively, 100% runoff. Despite having underdrainage, the site of this RAF was found to contain a dominant overland flow-pathway under intense rainfall. The contributing area leading up to the RAF is 11 ha (0.11 km^2). A GIS tool, using Light Direction And Ranging (LiDAR) data, was used to identify the best possible location for the feature (Figure 7.16) (Wilkinson & Quinn, 2010). The GIS tool helped communicate the location and description of the feature to the landowners and also provided an accurate estimate of the potential storage, which was found to be 500 m^3 (following an RTK survey). The GIS tool is extremely useful in communicating and gaining support for features from the farmer and local regulators.

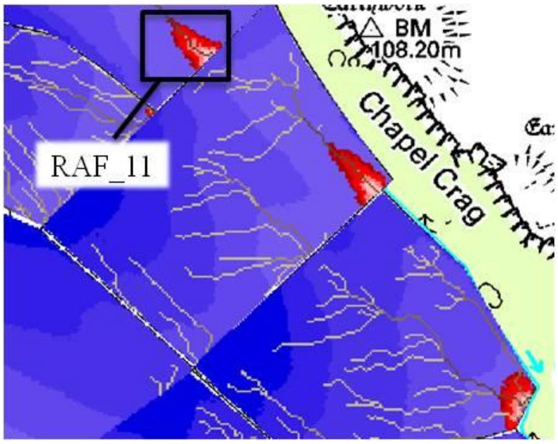


Figure 7.16: RAF-11 during a storm event (left). The GIS software identifying the location of the RAF (right: from Wilkinson & Quinn (2010))

The RAF was constructed using a locally sourced soil and boulders to form a bund over the natural gully in the field. The bund itself also provides the land owner with a track to drive vehicles and machinery over the waterlogged zones of the field during wetter periods. The RAF has a 0.22 m diameter outlet pipe to allow it to drain in a matter of hours. Features like these are ideal for disconnecting fast flow pathways during the peak of storm runoff, which relieves the river network throughout the catchment during a storm event. The outflow of these features should allow them to drain in 8-10 hours to prevent seasonal waterlogging of productive fields. It has been noted, in the Belford project, that features like this should have a restricted height to avoid large localised scour and pressure increases in any field drains that may run beneath them.

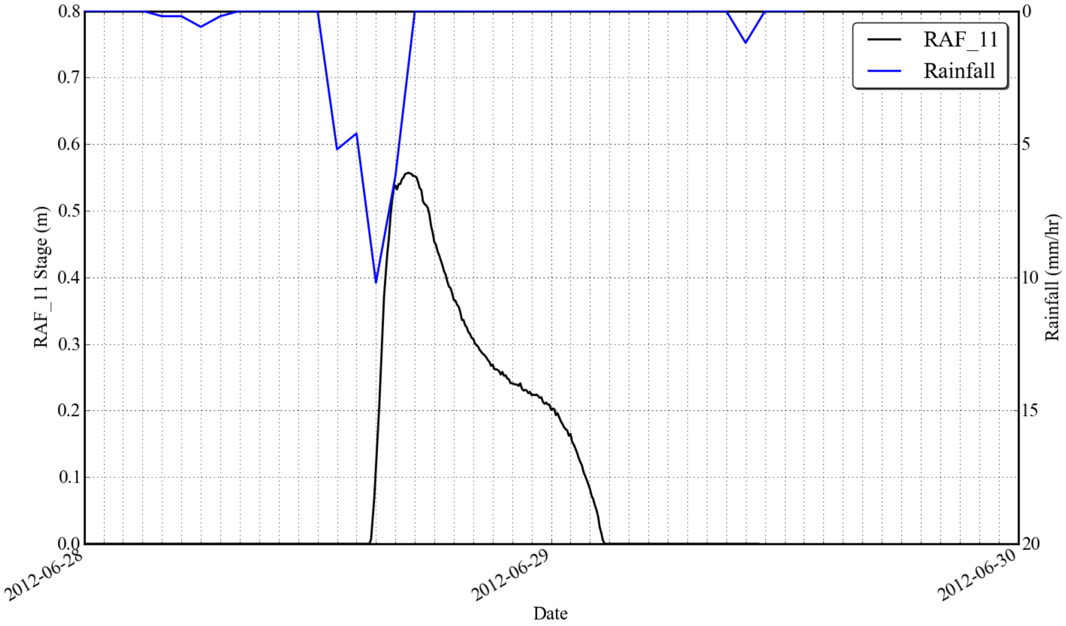


Figure 7.17: Data from RAF-11 during storm event (late-June 2012)

Data from RAF-11 in Belford (Figure 7.17) shows a targeted response (as designed) to a short, intense rainfall event. The event, which occurred in June 2012 and caused no flooding to Belford, saw 28 mm of rain fall in 12-hours. The 500 m^3 capacity RAF achieved a water-level of approximately half of its maximum. The field leading up to the RAF was dense with wheat-crop, which may have reduced the impact of the rainfall in this particular event. Had the field been bare, as it has been in winter months, the surface runoff may have been greater in magnitude. It has been estimated by Palmer (2012) that 0.99 tonnes of sediment were deposited in RAF-11 during a single runoff event in January 2011, the equivalent of 91 kg ha^{-1}.

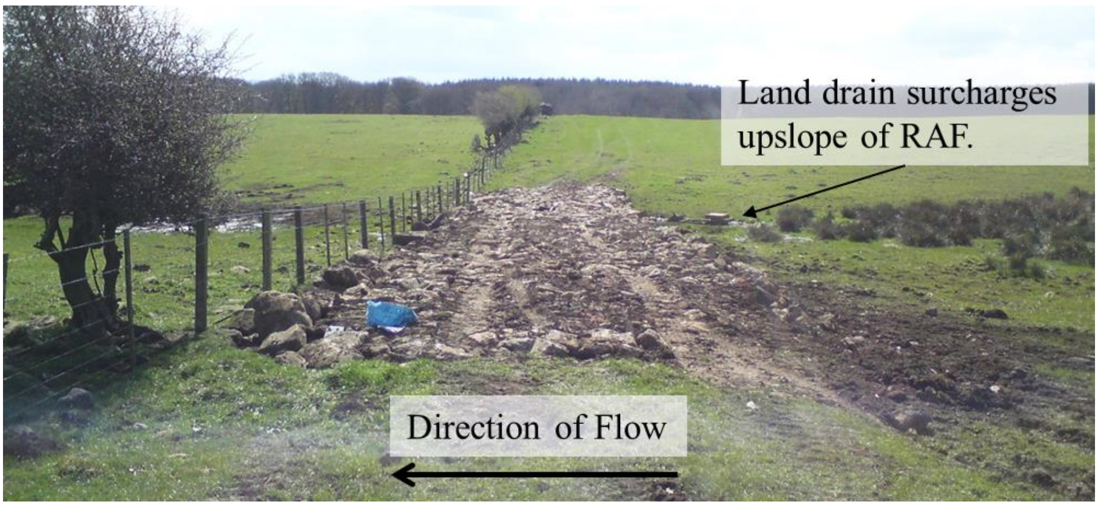


Figure 7.18: RAF-15 in Belford

RAF-15 (Figure 7.18) is constructed of boulders and coarse aggregate making the structure permeable. The RAF is capable of intercepting overland flow from the contributing upstream area. Interestingly, the RAF also receives inflow from the tile-drain upslope surcharging during storm events. The RAF has a physical storage of 200 m^3; however, the structure itself possibly provides increased roughness to overland flow. The RAF itself has no hydrometry, so there is no data for analysis.

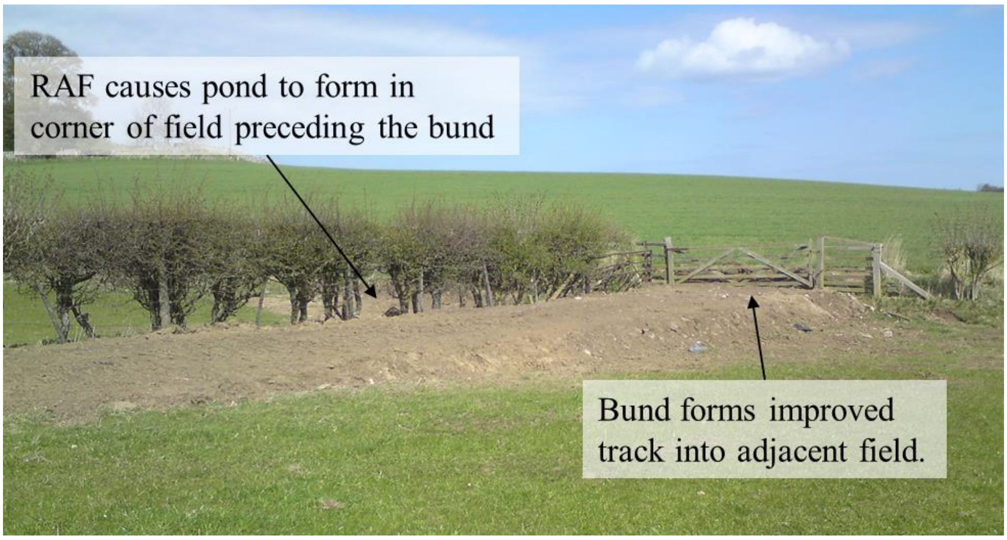


Figure 7.19: RAF-14 in Belford

RAF-14 (Figure 7.19) is an interesting example of an overland flow disconnection pond due to how it drains. Instead of draining through an outflow pipe or percolating through the structure of the RAF, as can be seen in other examples; this RAF drains using an existing field drain. The construction process connected the ground surface with an entrance to the tile-drain beneath. Since the tile-drains in Belford are limited in capacity (approximately 0.03 m^3s^{-1}), this characteristic allows the drains to function at capacity whilst the RAF is allowed to fill on the surface. The construction of this RAF has improved drainage of this field following storm events in a previously wet area of the farm, which has improved the local productivity of the land. The RAF can store 450 m^3 of overland flow and allows continuous access to the adjacent field, even during storm events.

7.3.4 Large woody debris and online ditch management

Large woody debris (LWD) has been installed in the riparian area of the catchment (Figure 7.20). The construction of LWD came with an opportunistic decision to have sycamore trees removed and replaced with suitable less intrusive tree species. The LWD and associated floodplain barriers were split into six locations (15 m apart) over the reach of the stream through the wooded riparian zone (Wilkinson, et al., 2010b).

Figure 7.20: Woody debris installed in Belford Burn (RAF-7)

During states of high discharge, LWD forces the water level, in proximity to them, to rise and spill onto the flood plain, where further woody debris is installed to increase friction (see Figure 7.20 (right)). It is estimated that each LWD feature could create up to 100 m^3 storage (depending on the surrounding floodplain elevation). A small amount of scour has been recorded at the LWD locations as the trunks have settled into position and it is possible for brash within the river to cause the LWD to become more like 'beaver-dams'. These are seen as beneficial in ecological terms and in the case of Belford, the LWD have been left untouched when this occurs due to the minimal impact on storage during high flows. Build-up of brash does, however, often lead to an increase in localised scour. There has been no data collected for this area (apart from visual evidence). It can be inferred that the features force water onto the floodplain during high flow and the increased roughness, caused by the brash, slows the propagation of the flood peak downstream.

A similar technique has been used at the top of the catchment. Small, online wooden barriers have been installed above the river, which are designed to allow normal flow to pass underneath them and divert peak flow from the river onto the adjacent land (see Figure 7.21).

Figure 7.21: Wooden diversion feature (RAF-24)

The land on both sides of the river has been set aside for tree plantation, and has a potential storage capacity of 150 m^3. Floodwater diverted by the wooden feature is expected to be forced into the surrounding land, where it will slowly permeate into the soil. This process will slow the movement of water between this point and the rest of the catchment. Due to the positioning in the channel, features like this can be prone to scour and will require inspection after high flow events.

7.6 Summary

This chapter has described the function of Runoff Attenuation Features (RAFs) and outlined the rationale behind the use of RAFs within the Belford catchment. The rationale identifies that there is the potential to drastically reduce peak discharge in the river and overland by targeting runoff sources through effective timing of mitigation and suggests that if all rainfall/runoff, that lies above a specified threshold, can be temporarily stored; flood hazard can be reduced. The effective timing of mitigation features has been discussed, which is linked with the rationale behind the project. Evidence from RAFs during storm events and the development of an analytical method for assessing their impact will be discussed in the chapter that follows.

The design of RAF networks and the interaction between RAFs and the rest of the catchment has been introduced in the context of the Belford study. Using knowledge gained from the Belford study, Quinn, et al. (2013) were able to identify the best possible use of RAFs within new catchments; identifying the best locations and types of RAFs for varying contributing upstream areas. Surveying methods have been discussed, which demonstrate the ability to calculate storage reintroduced into the catchment through the application of a RAF approach. Finally, example RAFs from the Belford catchment have been discussed, in detail, to explain their construction and purpose within the catchment. These have been coupled with example data recorded from the hydrometry within them, purely to demonstrate their ability to function during storm events.

8. Pond forensic analysis

8.1 Introduction

In this chapter a novel method for analysing the impact of individual RAFs, using observations from the divers installed in the river network and offline RAFs, will be presented. After analysing storm events in the form of hydrographs (Chapter 6) the data were combined with observations of water-stage, both from the stream and offline ponds, in order to gain a better understanding of the impacts of RAFs upon the river network during a storm event. Graphical plots were developed to demonstrate the impact of individual RAFs on the discharge in the stream directly adjacent to them.

8.2 Detecting changes to catchment response

As discussed in previous chapters, the lack of pre-change data for Belford has made it difficult to quantify the effects of mitigation – especially since RAFs were being installed throughout the project runtime. For this reason, some initial attempts were made to use a model framework that could detect yearly variability in long-term datasets.

It was hypothesised in Ewen, et al. (2010) that the part of a hydrograph least sensitive to natural variability is the falling limb (i.e the recession), on the basis that it is not affected by precipitation variability. A conceptual catchment model that is based on recession analysis should therefore be less sensitive to natural variability; thus representing storage dynamics throughout the catchment.

One way of expressing the recession curve is by pairing values of discharge with their rate of change during a recession. Observations using 3 sequential hydrological years during the monitoring period have been examined to determine whether catchment response has varied from year to year. Figure 8.1 shows the regressions from 2008-2011 for the R2 River gauging point. For this upper part of the catchment the analysis observes a generally slower catchment response, chronologically (from 2008 to 2011) due to the lower rate of change in discharge (dq/dt) when plotted against peak discharge (Figure 8.1). The analysis, however, lacks robust evidence and relies on complete datasets for the analysis. The theory was presented that this method could help identify whether mitigation installed over the study period could be detected. Unfortunately, due to various problems with hydrometry, the only successful gauge in the analysis was R2. Although the R3 dataset was also without interruption, the analyses proved inconclusive, with more scatter and no discernable pattern. Another factor that made this analysis unsuccessful, was that only a small number of mitigation features were installed upstream of the R2 gauge.

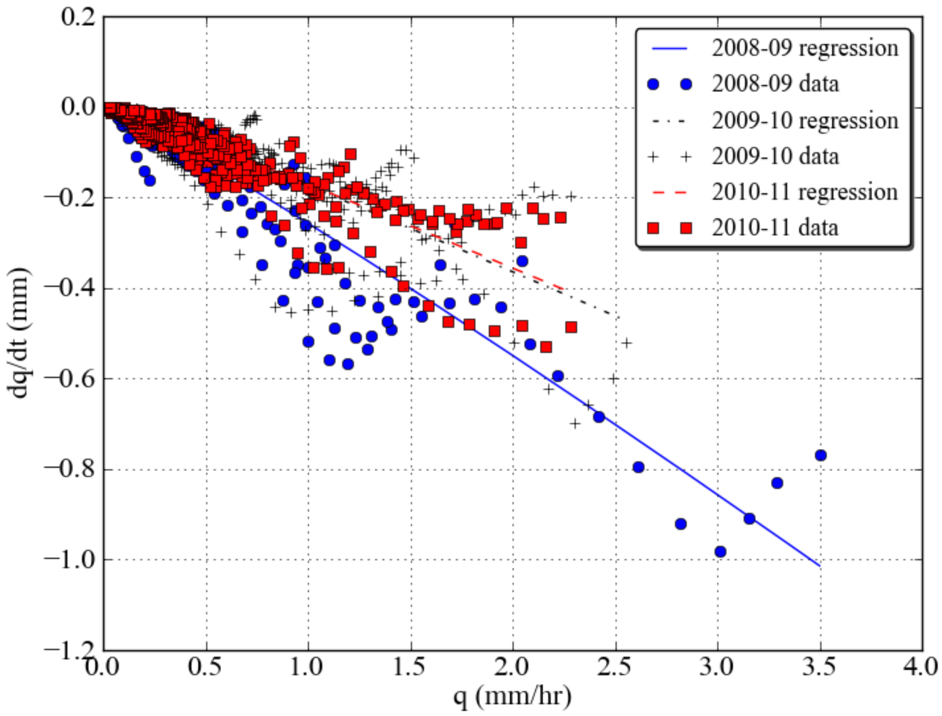


Figure 8.1: R2 yearly recession data with regressions

Due to the large amount of flaws in the above method, not to mention significant gaps in some of the downstream gauges, it was decided to narrow the window of analysis. This was in order to track changes more clearly and avoid having to rely on large, fully-complete datasets.

8.3 Quantifying impacts of individual RAFs

The process for determining the impact of individual RAFs has been divided into two aspects; Firstly, to assess the actual downstream impact of a storage body; and secondly to determine which attributes of an offline pond, adjacent to a river, make it able to control how much impact it may have upon the downstream discharge.

8.3.1 RAF storage and downstream impact

Table 4-1 indicates the hydrometry present within the RAFs in Belford (see 4.2). Divers within the RAFs record the stage of water (m) on five-minute time-steps. Pond level-volume relationships were necessary for converting the diver data in the ponds from level (m) to volume (m^3). The fill rate is change in volume (*V*), from one time-step (*n*) to the next, divided by the duration of the time-step ($V_n - V_{n-1} / dt = dV/dt$) ($m^3s^{-1}$). A similar technique was used to determine reservoir operation in the upper Tone River in Japan (Yang, et al., 2004) (see Equation 8.1):

$$\frac{dV}{dt} = Q_{in} - Q_{out}$$

Equation 8.1

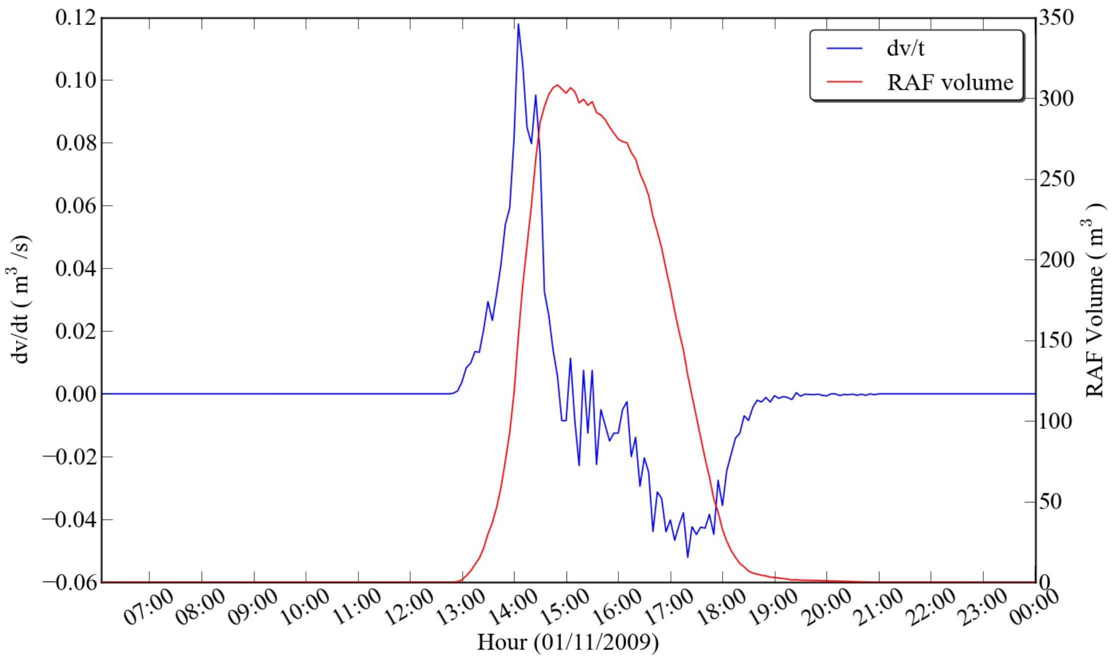


Figure 8.2: Pond volume against time (red); Change in pond volume between two time-steps (blue)

Figure 8.2 shows the observed rate of change in volume in the pond (i.e. a discharge) plotted against the observed pond volume during a storm event (November 2009). Once the inflow, initial conditions, RAF characteristics (e.g. relationship between the water level and volume – discussed in Chapter 7 – 7.5), and the operational rules of the storage body are known, the outflow from a RAF can be calculated (Ponce, 1989).

The study on the upper Tone River in Japan attempted to simulate outflow from a reservoir, in order to route the flow downstream in the river network (Yang, et al., 2004). In the case of the

Belford study, however, the analysis is able to detect impact on the river without having to simulate outflow. The downstream discharge is simply calculated by subtracting the function *dV/dt* from the observed upstream discharge. Figure 8.3 shows the discharge in the river upstream (Q_{us}) and downstream (Q_{ds}) of the RAF. Q_{us} is observed, and Q_{ds} is calculated using:

$$Q_{ds} = Q_{us} - \frac{dV}{dt}$$

Equation 8.2

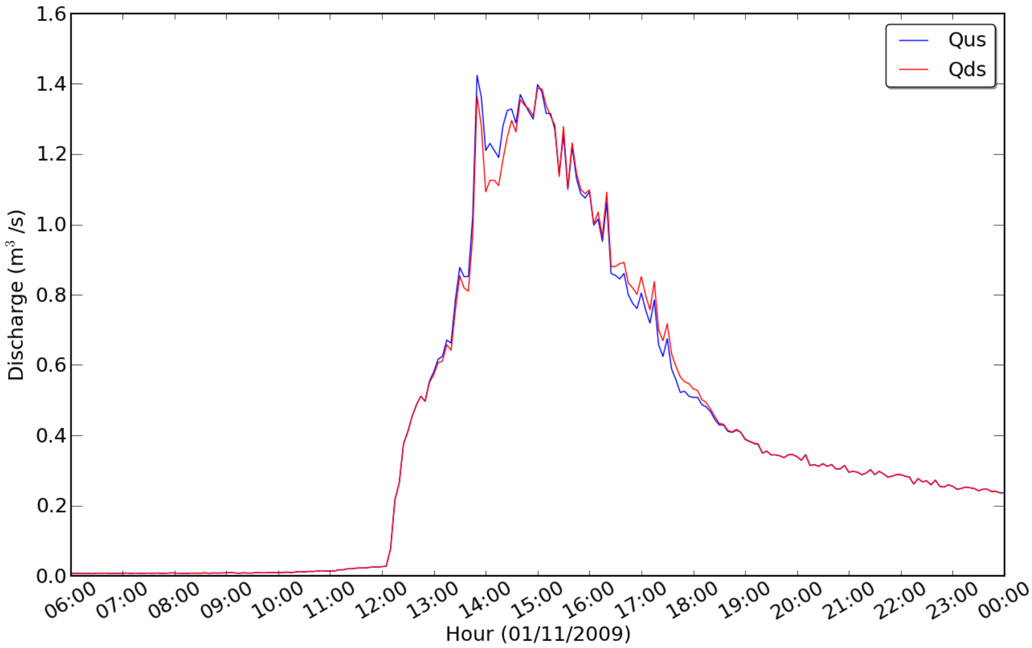


Figure 8.3: Impact of the RAF on downstream discharge. Blue line shows discharge without the RAF, red shows discharge in the river with the RAF in place.

The above figures show the effect that one RAF can have on the discharge in the river adjacent. In the case shown, November 2009, the RAF (Pond 3) reduces the river discharge by approximately 10 per cent during the peak of the event – the fill-rate of the pond reaches $0.12m^3s^{-1}$ while the discharge in the river reaches $1.2m^3s^{-1}$. Note: $1.2m^3s^{-1}$ is a minor event and would not cause flooding of Belford Village. These plots have identified, however, the degree to which the ponds provide effective attenuation, and have also given clues as to how to improve them. For example, the size of the pond and the height of the inlet channel above the stream have both been identified as important variables to test in the next stage of the analysis; the pond size because it dictates the potential storage compared to rainfall and specific discharge data, and the inlet height because it controls when the pond can start filling.

By comparing RAFs 1 and 3 during a particular event, it is possible to show how different inlet heights and storage capacities can change the impact of a RAF. The surveys of the two RAFs indicated the differences in volume – with RAF-1 being 330m^3 and RAF-3 being 370m^3 (see Table 7-2). The following graphs (in Figure 8.4 and Figure 8.5) indicate the inlet heights of RAFs 1 and 3 respectively. It is clear from the figures that the inlet channel height for RAF-3 is higher than that of RAF-1. The information provided by these observations helps show how storm events can be 'targeted'; eventually with the aim of designing inlet-controls that only start to allow water to enter in high magnitude events.

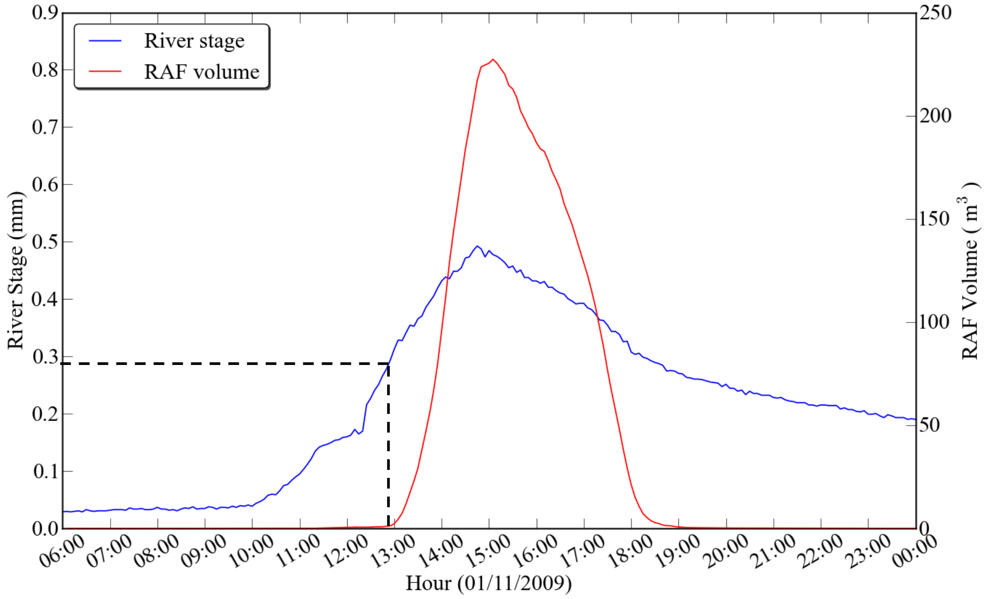


Figure 8.4: River stage at R4 against RAF-1 volume (Inlet height marked as approximately 0.28 m)

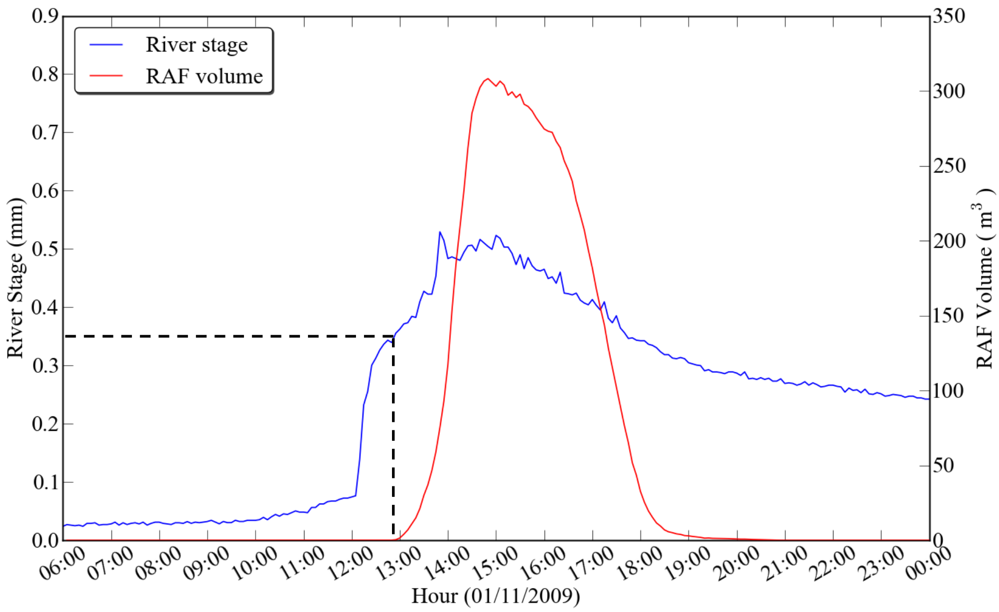


Figure 8.5: River stage at R3 against RAF-3 volume (Inlet height marked as approximately 0.33mm)

Figure 8.6 shows an analysis graph for RAF-1 during the November 2009 storm event. It can be seen, by comparing to RAF-3 (Figure 8.2) during the same event, that a reduced impact is detected in the effective discharge being removed from the river adjacent to the RAF. In this case, this should not be attributed to potential volume due to the fact that neither RAF reached maximum volume (as the November 2009 storm is only a minor event). It can be speculated, however, that the reduced effectiveness could be a result of the height at which the inlet channel is situated to remove water from the river (see Figure 8.6 and Figure 8.7).

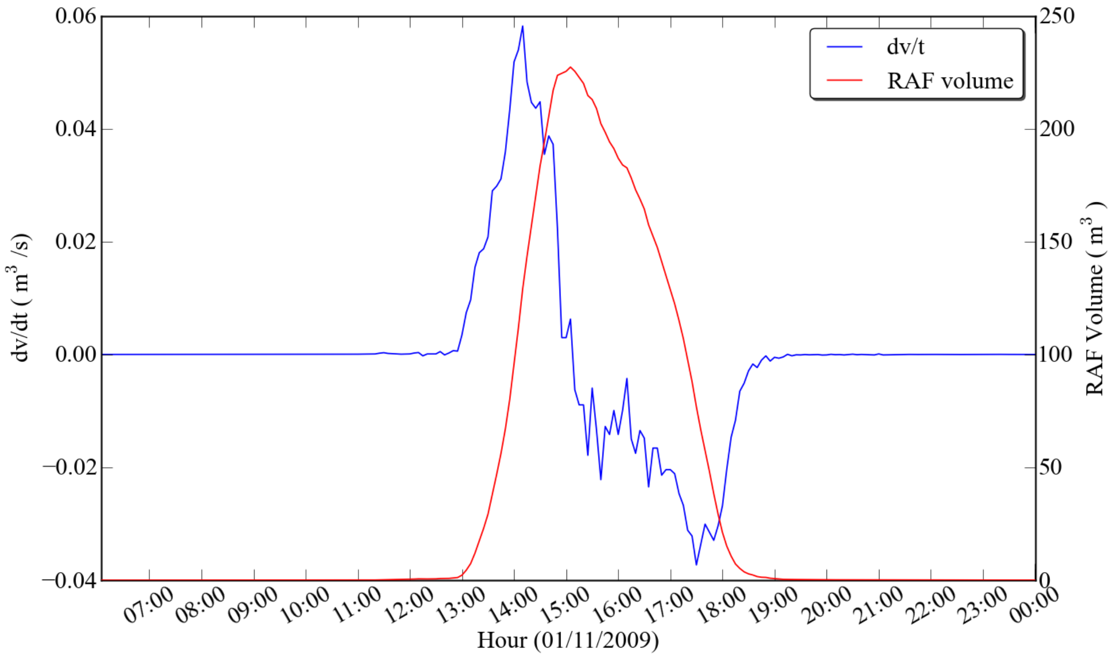


Figure 8.6: RAF-1 function during November 2009 event

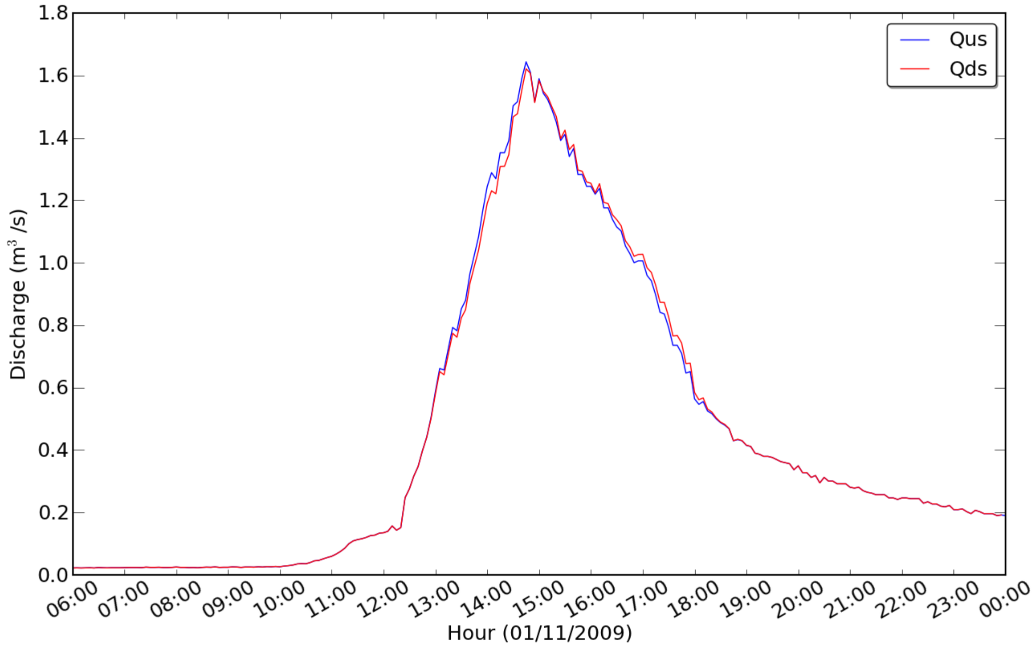


Figure 8.7: Impact of RAF-1 on downstream discharge during November 2009 event

8.3.2 Short-duration storm events

The following storm event did not cause the RAFs to completely fill, which makes it easier to assess the impact on downstream discharge. Figure 8.8 and Figure 8.9 show the impact on RAF-3 during a small storm event in September 2009. It demonstrates a reduction in peak discharge by up to 5%.

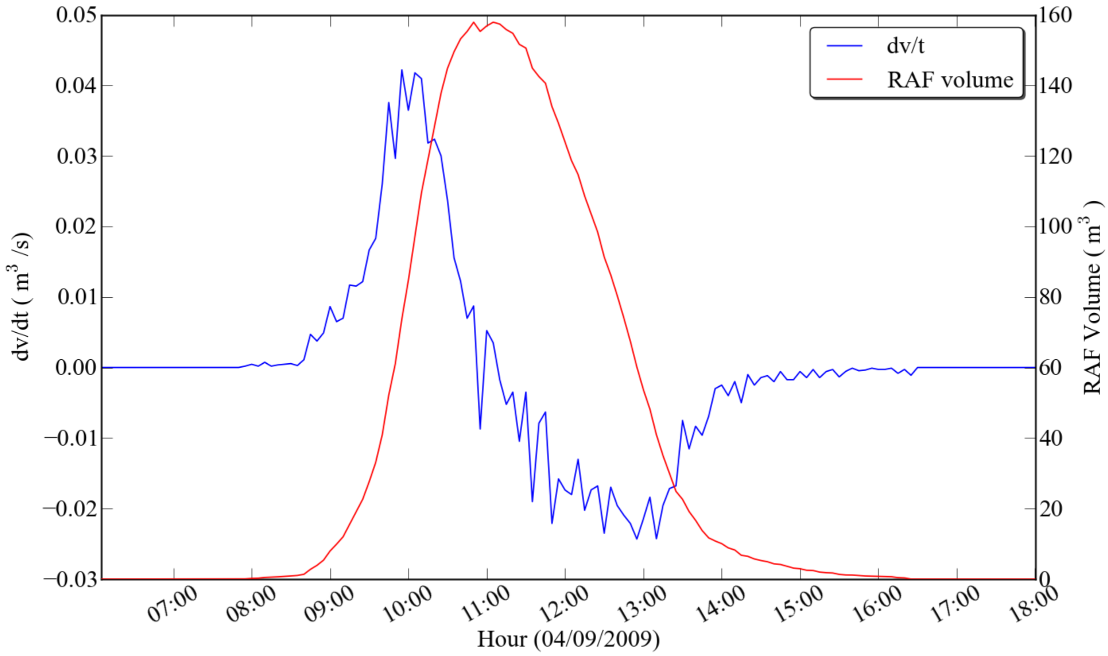


Figure 8.8: RAF-3 function during September 2009 event

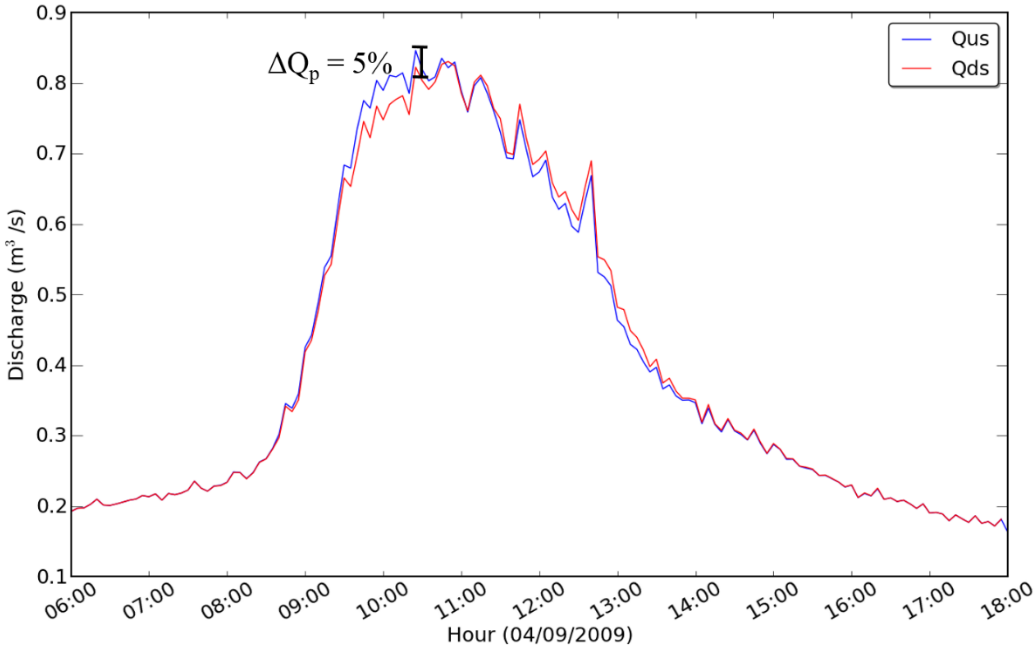


Figure 8.9: Impact of RAF-3 on downstream discharge during September 2009 event

Figure 8.10 and Figure 8.11 shows RAF-3 during a high magnitude, but short duration storm event in June 2012. The steep rise in *dv/dt* indicates a sharp reduction in peak discharge of up to 20%. Unfortunately, the RAF only impacts the rising limb of the hydrograph; as the RAF becomes full prior to the peak of the storm.

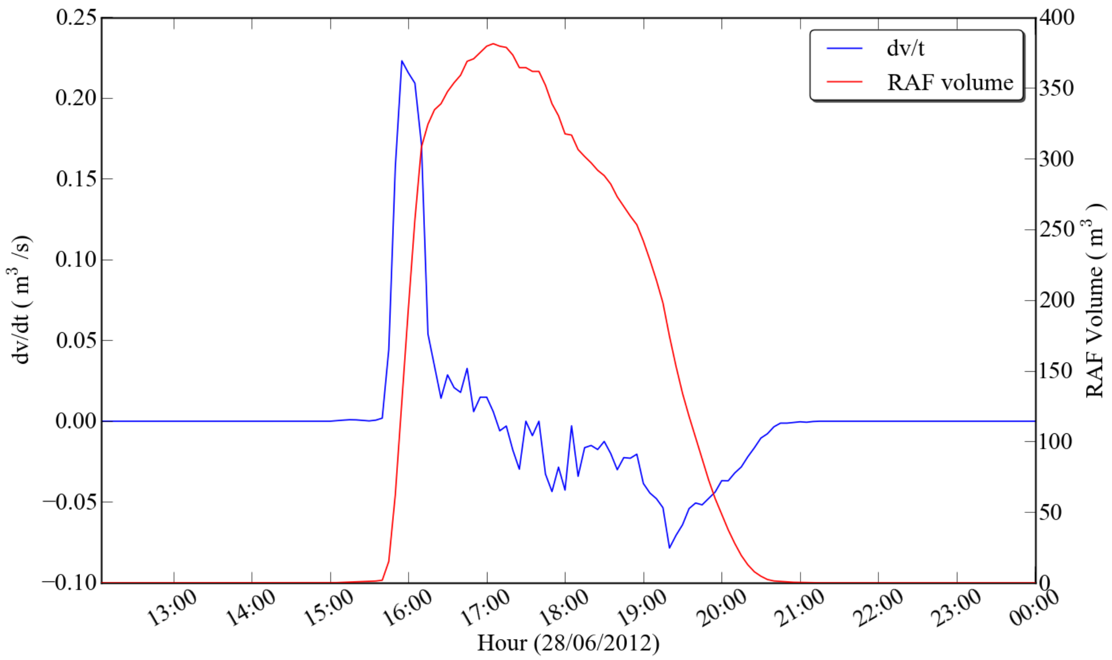


Figure 8.10: RAF-3 function during June 2012 event

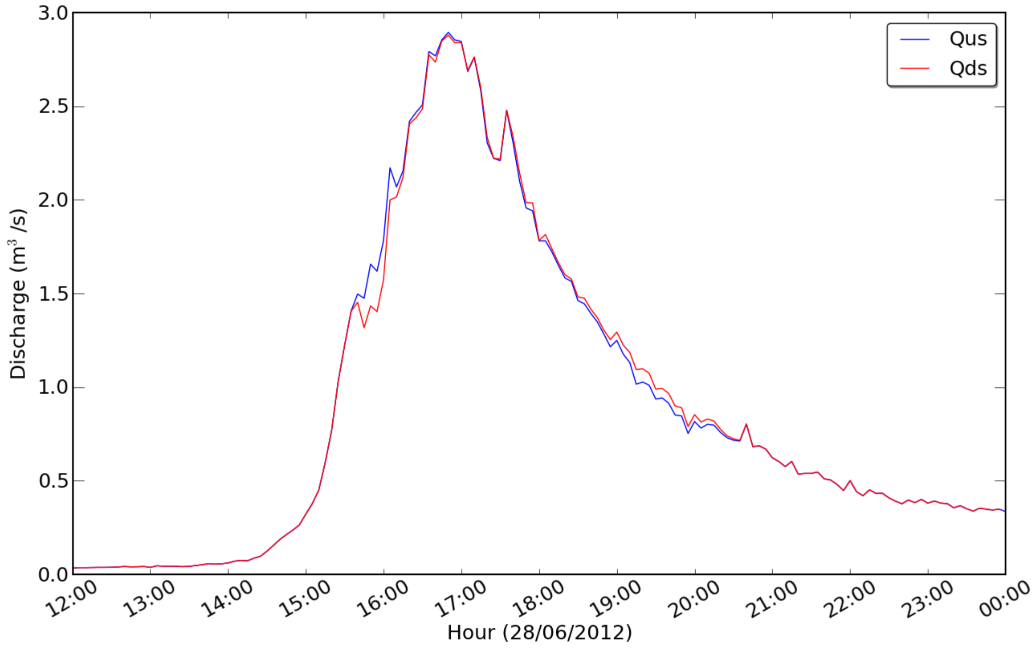


Figure 8.11: Impact of RAF-3 on downstream discharge during June 2012 event

8.3.3 Multi-peaked events

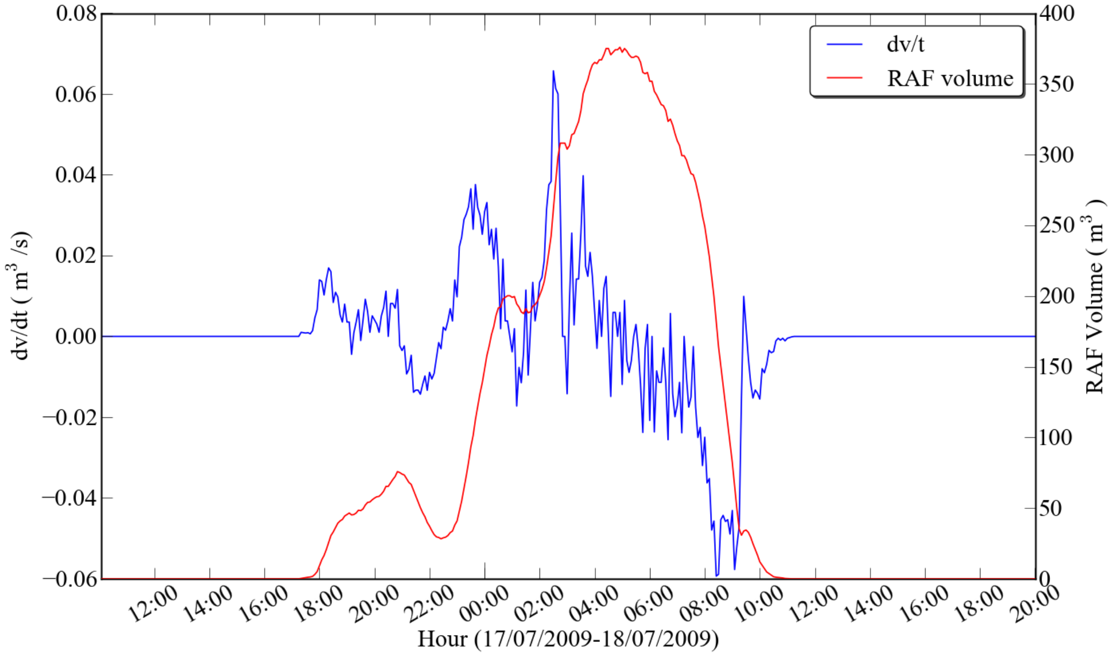


Figure 8.12: RAF-3 function during July 2009 event

Figure 8.12 and Figure 8.13 demonstrate the issue regarding the height of the inlet channel to the offline ponds. In this particular event (July 2009) the offline pond (RAF-3) has already begun filling during an initial low peak that passes by it, which reduces the available storage by the time the second, larger, peak arrives. When analysing pond impact during the July 2009 event it must be noted that the peak discharge recorded at the R3 gauging station was over twice as large as the November 2009 event (see 8.3.1) and the duration is nearly twice as long as the January 2010 event (see 8.3.4). It can be seen by both figures that RAF-3 is full long before the peak flow arrives, which means there can be no impact, in terms of change in discharge, made by the individual offline pond during the peak.

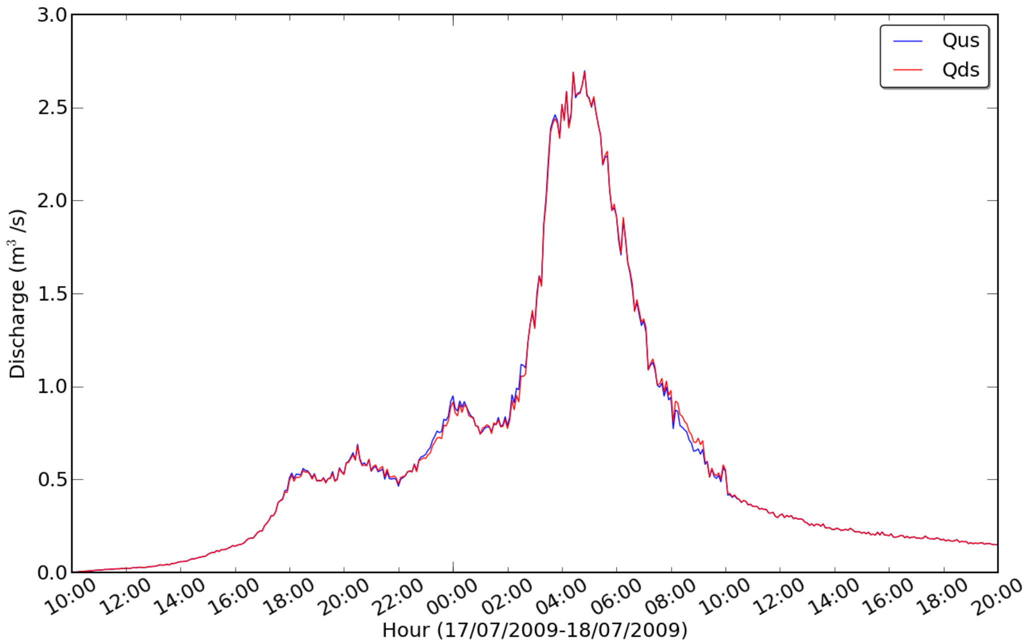


Figure 8.13: Impact of RAF-3 on downstream discharge during July 2009 event

The duration of multi-peak storm events combined with the inlet height of the RAFs being too low results in a loss of impact on river discharge. Figure 8.14 and Figure 8.15, from the March 2010 event; also show no impact on river discharge. This is, again, due to the RAF being full prior to the peak flow moving past the RAF. This raises the following question; do full RAFs have an attenuating effect?

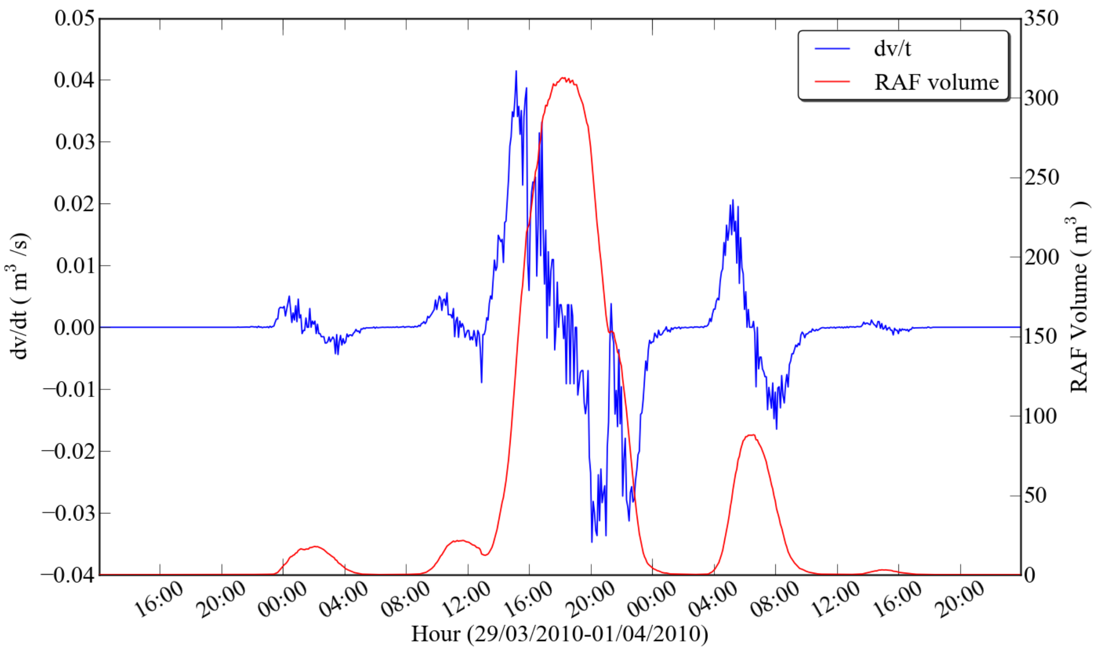


Figure 8.14: RAF-1 function during March 2010 event

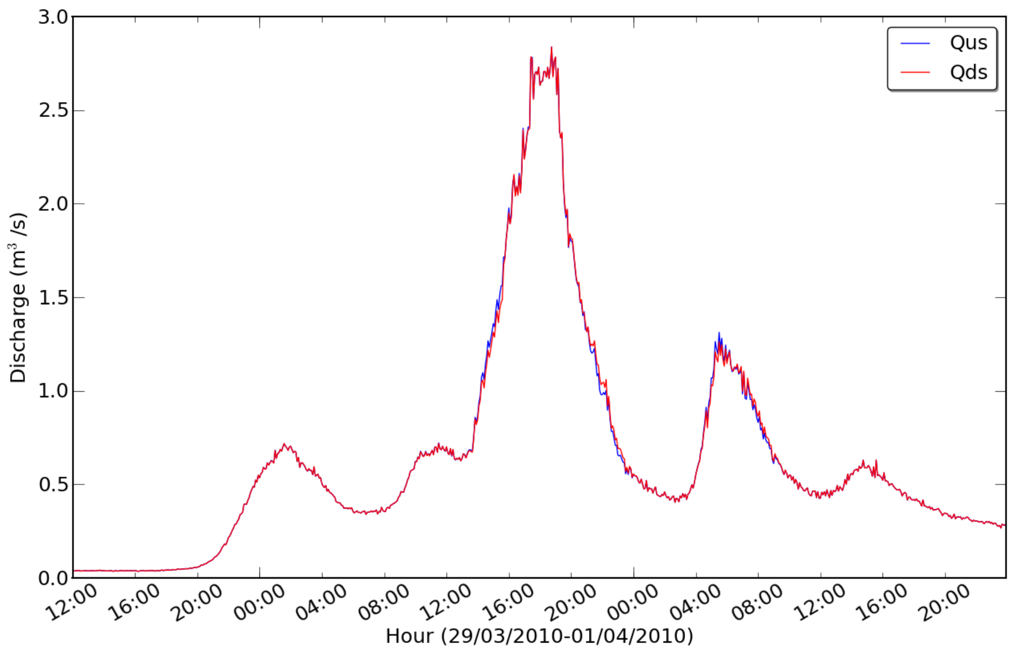


Figure 8.15: Impact of RAF-1 on downstream discharge during March 2010 event

8.3.4 Rain on snow events

As it was mentioned in the previous section; it is not only magnitude but duration of storm events that can control the effectiveness of RAFs. The January 2010 snowmelt and June 2012 storm events were both very similar in magnitude. It can be seen in Figure 8.16, however, that due to the duration of the January 2010 event the impact of the RAFs have been greatly reduced.

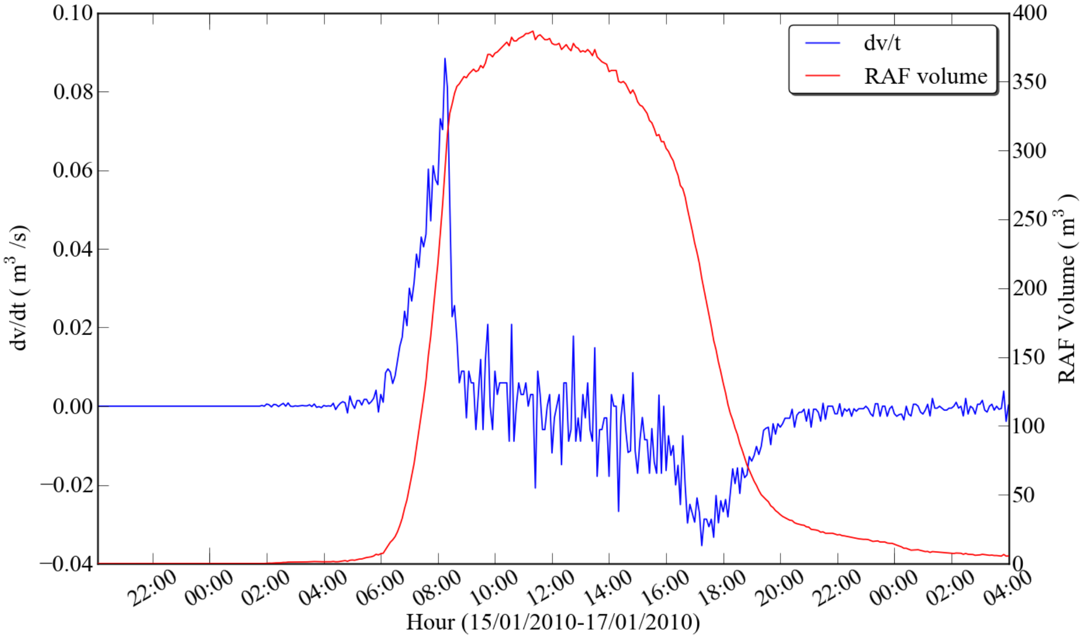


Figure 8.16: RAF-3 function during January 2010 event

Comparing Figure 8.16 and Figure 8.10 it can be seen that RAF-3 is active for over 12-hours in the January 2010 event, yet only 5-hours in the June 2012 event. As a result, the RAF has a reduced impact on river discharge in the case of January (Figure 8.17).

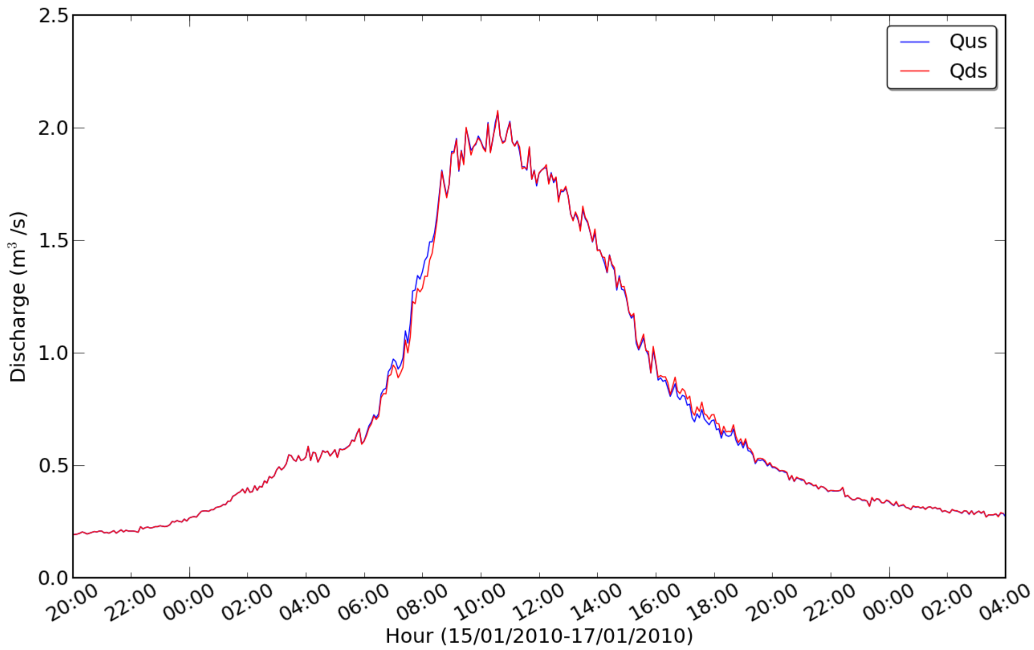


Figure 8.17: Impact of RAF-3 on downstream discharge during January 2010 event

8.4 Summary

An analytical method of determining mitigation impact of individual RAFs has been described in detail. The 'timing' of the mitigation has been introduced using comparisons between two similar RAFs during a short-duration storm event. In the pond forensic analysis, the offline RAFs have proved to have a relatively minor effect on downstream discharge for the majority of high-magnitude, large-duration flood events (see Figure 8.12 to Figure 8.17). It has been noted that the RAFs begin filling too early during storm events due to the height at which the inlet channel is set for water entering the RAF from the river. The ideal moment for the RAFs to begin filling would be at the moment the peak discharge begins to flow adjacent to them. Identifying the lowest peak discharge before Belford Village is put at risk from a flood event is a key component of identifying the best height at which to set the inlet channels to the offline ponds. The procedure for setting the correct inlet height for a certain feature is difficult using this analytical method. Many high-magnitude storm events cause the features to fill prior to the peak of the storm, which has raised the question as to whether RAFs have any effect when they are full. Arguably, the lengthening of flow pathways and increased roughness may be adding some attenuation. If each feature is capable of having, sometimes very, minor impact on the discharge in the River; what will happen if they are assessed as a network of features?

9. The Pond Model

9.1 Introduction

This chapter describes the development of a physically-based model (called the Pond Model) to represent the impact of RAFs upon river discharge. The necessity to do this has come from the forensic analysis of RAFs, in the previous chapter, and the desire to learn more about the function of RAFs as viable mitigation features. The Pond Model will be shown to represent the function of existing RAFs, in the Belford catchment, and then be used to simulate impacts once changes are made to the physical attributes of the RAFs. It will also demonstrate the impact of constructing several RAFs, in sequence, along a stretch of the river. Finally, an overland flow version of the Pond Model will be shown with the ability to demonstrate the impact of overland flow interception ponds.

9.2 Background to model structure

Here, the development of a physically-based model for assessing the impact of existing RAFs in the Belford catchment will be described. This was developed using Python to create an automated and reliable model structure. The model uses observed river stage and surveyed RAF dimensions to predict the amount of water entering a RAF at a given time-step. Once this is known, the volume of water within the pond is converted to a water level and then converted to flow through the outflow pipe using a hydrostatic equation (Nicholson, et al., 2012b). The Pond Model can also be used for simulating changes to the physical attributes of existing RAFs (including the effective volume, inlet height and outlet-pipe diameter) and for predicting the impact of proposed RAFs.

9.2.1 Inflow condition

The water flowing into a RAF is simulated using a sharp-crested weir equation. The sharp-crested weir equation chosen was that demonstrated in Liang, et al. (2007). The connection is a lateral node with water spilling over the riverbank onto the adjacent floodplain where the 1D and 2D flows are linked through spills in the middle. It is assumed that the flow within the river has the dominating direction, whereby the 1D flow assumption is still applicable despite the overtopping of the levee. For this connection an equation is required to calculate the flow exchange Q_{in} between the 1D and 2D models. Liang, et al. (2007) choose a simple sharp-crested weir equation to calculate the absolute value of exchange flow rate:

$$|Q_{in}| = \begin{cases} 0 & if\ z_{s1} \leq z_{sw} \\ f_r C_d \frac{2}{3} b \sqrt{2g} (z_{s1} - z_{sw})^{1.5} & if\ z_{s1} > z_{sw} \end{cases}$$ **Equation 9.1**

where z_{s1} and z_{s2} are the water levels in the river and on the floodplain, respectively, z_{sw} is the elevation of the weir (bank) crest (Figure 9.1), b is the length of the overtopped part of the bank (or the width of the inlet channel), C_d is the discharge coefficient (=0.8) and f_r is the drowned flow reduction factor, which is determined from the following equation:

$$f_r = \begin{cases} 1.0 & if\ z_{s2} \leq z_{sw} \\ \left[1 - \left(\frac{z_{s2} - z_{sw}}{z_{s1} - z_{sw}}\right)^{1.5}\right]^{0.385} & if\ z_{s2} > z_{sw} \end{cases}$$ **Equation 9.2**

Q_{in} is positive if the water level on the 2D side is higher than that on the 1D side, and negative otherwise. In total, there are five possible conditions over the weir, including; no flow; free flow from the 2D to 1D domain; free flow from the 1D to 2D domain; drowned flow from the 2D to 1D domain; and drowned flow from the 1D to 2D domain (Liang, et al., 2007).

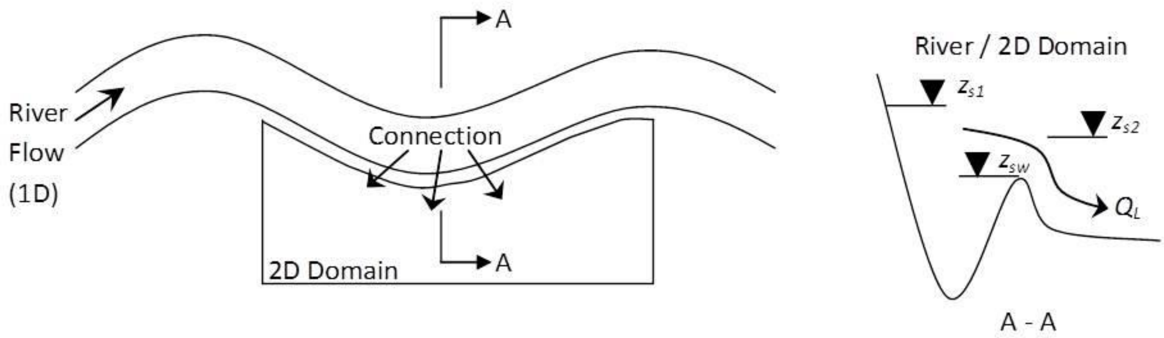


Figure 9.1: Schematic of connection node between 1D and 2D domains [from Liang, et al. (2007)]

The aim of the Liang, et al. (2007) study was to combine computational efficiency with the ability to be able to adequately cope with the 2D phenomena occurring on the floodplain. For the purposes of the Pond model, Q_{in} is simply a function of river level, the dimensions of the inlet channel and the reaction to water that may be present in the RAF (controlling drowned flow).

9.2.2 Volume increase

The flow into the pond (Q_{in}) is immediately converted into a temporary volume by multiplying Q_{in} by the length of the time-step, in seconds (which, in the Belford simulations, is 300-seconds or 5-minutes). This converts from discharge (m^3 s^{-1}) into volume (m^3). The volume is then converted into water level within the RAF using the look up tables obtained from surveys (see 7.3). The water level is then used by the outflow condition for the next time-step in the simulation.

9.2.3 Outflow condition

The outflow condition from the RAF is simulated using a generic formula, which is based on hydrostatic flow through a small orifice in order to ensure model transferability to similar RAF types. It makes the assumption that the water inside the RAF is static, which is similar to assumptions made in engineering studies on lakes and reservoirs despite discharge currents being present in the water body.

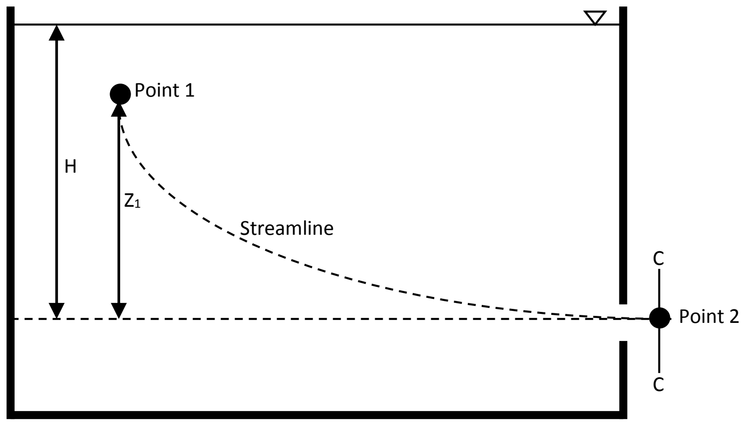


Figure 9.2: Diagram indicating flow through a small orifice

An orifice used for flow measurement is an aperture with a sharp edge (bevelled side facing downstream). Consider a very large reservoir draining through a sharp-edged orifice (Figure 9.2). If the flow is frictionless and the fluid has a constant density, provided the flow is assumed to be steady, Bernoulli's equation gives:

$$\frac{P_1}{\rho g} + \frac{u_1^2}{2g} + z_1 = \frac{P_2}{\rho g} + \frac{u_2^2}{2g} + 0 \qquad \textbf{Equation 9.3}$$

Therefore, since the reservoir is very large, and the pressure at the vena contracta is atmospheric pressure:

$$\frac{\rho g(H-z_1)}{\rho g}+0+z_1=\frac{P_2}{\rho g}+\frac{u_2^2}{2g}+0 \qquad \textbf{Equation 9.4}$$

$$u_2=\sqrt{2gH}\ \ (\textit{Torricelli's formula}) \qquad \textbf{Equation 9.5}$$

If the area of the orifice is 'a', then:

$$Q_{out}=a_2.u_2=C_c.a.u_2=C_c.a\sqrt{2gH} \qquad \textbf{Equation 9.6}$$

where C_c is the coefficient of contraction of the water jet the vena contracta (defined as C_c= area of water jet / area of orifice). In practice, there will be friction losses; a practical equation is:

$$Q_{out}=C_d.a\sqrt{2gH} \qquad \textbf{Equation 9.7}$$

where C_d, the coefficient of discharge through the outlet pipe is given by:

$$C_d=C_cC_v \qquad \textbf{Equation 9.8}$$

and C_v, the coefficient of velocity, is given by:

$$C_v=\frac{actual\ velocity\ at\ vena\ contracta}{ideal\ velocity\ at\ vena\ contracta} \qquad \textbf{Equation 9.9}$$

Sample values of C_d for a negligible approach velocity for a bevelled small orifice and a Borda's (re-entrant) mouthpiece are shown in Figure 9.3 (Marriot, et al., 2009). Typically, C_d can range between 0.61 and 0.75.

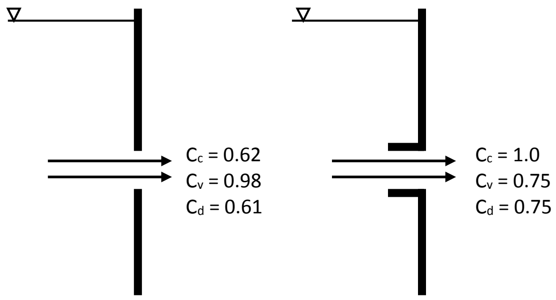


Figure 9.3: Diagram showing typical values for C_d: Left – Bevelled orifice; Right – Borda's (re-entrant) mouthpiece

Due to the fact that RAFs are empty prior to a storm event, the outflow pipe will not be fully submerged throughout the entire simulation. Equations to represent partially submerged pipe flow were used in the Pond Model to more accurately represent outflow at low water levels. Figure 9.4 shows water at level y in the outflow pipe. The angle (ϑ) between the pipe centre and the water level at the edges of the pipe is used to determine the submerged area of the pipe.

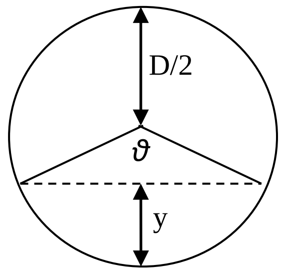


Figure 9.4: Partially submerged pipe

$$\theta = 2\cos^{-1}\left[\frac{\frac{D}{2} - y}{\frac{D}{2}}\right] \; if \; y < \frac{D}{2}$$ **Equation 9.10**

$$\theta = 2\pi + 2\cos^{-1}\left[\frac{\frac{D}{2} - y}{\frac{D}{2}}\right] \; if \; y > \frac{D}{2}$$ **Equation 9.11**

The equations above allow the angle between the centre of a partially submerged pipe and the water level to be determined. The effective area of the pipe can then be calculated using the below equation (from Chadwick & Morfett, 1986):

$$A = \frac{D^2}{8}(\theta - \sin\theta)$$ **Equation 9.12**

The effective area (A) is then used in the outflow equation, described previously, to more accurately represent flow through the outflow pipe at low water levels. The outflow from the RAF is a function of the water level and submerged pipe area.

9.2.4 Model output

The Pond Model reads input data in the form of recorded river level. Once the trigger-condition is met, whereby the water level in the river exceeds the height of the inlet channel ($z_{s1} > z_{sw}$), the input data is converted into a discharge into the RAF (Q_{in}) by Equations (9.1) and (9.2). This discharge forms a temporary volume of water within the RAF. The temporary volume is then converted into water level inside the RAF (H) using a surveyed stage-volume curve. Once H is known, the outflow (Q_{out}) is calculated using Equation (9.7). The difference between inflow and outflow is then subtracted from the upstream discharge in the river (Q_{us}), which has been obtained using a Rating Curve and the observed river level, to produce a simulated downstream discharge in the river (Q_{ds}) (see Equation 9.13). The output from the Pond Model is a collection of graphs designed to give as much detail as possible to the user regarding both the functioning of the RAF and the impact it has had upon downstream discharge.

$$Q_{ds} = Q_{us} - (Q_{in} - Q_{out})$$ **Equation 9.13**

9.3 Testing the model output

The equations for inflow and outflow were automated in Python in order to deal with the large dataset of the Belford project. The Pond Model calculates flow into and out of the RAF – based on the stage height in the adjacent river. It produces a text file with output from the model and plots the data in four graphs (see Figure 9.5 to Figure 9.8). Figure 9.5 demonstrates the response of the RAF based on River stage (and associated inlet height). Figure 9.6 compares the simulated volume to the observed volume in the RAF. Figure 9.7 plots the calculated inflow and outflow against each other – the difference between these two graphs is subtracted from the observed discharge in the river upstream of the pond (Q_{us}) to show the impact of the pond on downstream discharge in the final plot (see Figure 9.8). Figure 9.8 shows the impact to the River by comparing upstream (Q_{us}) and downstream (Q_{ds}) discharge.

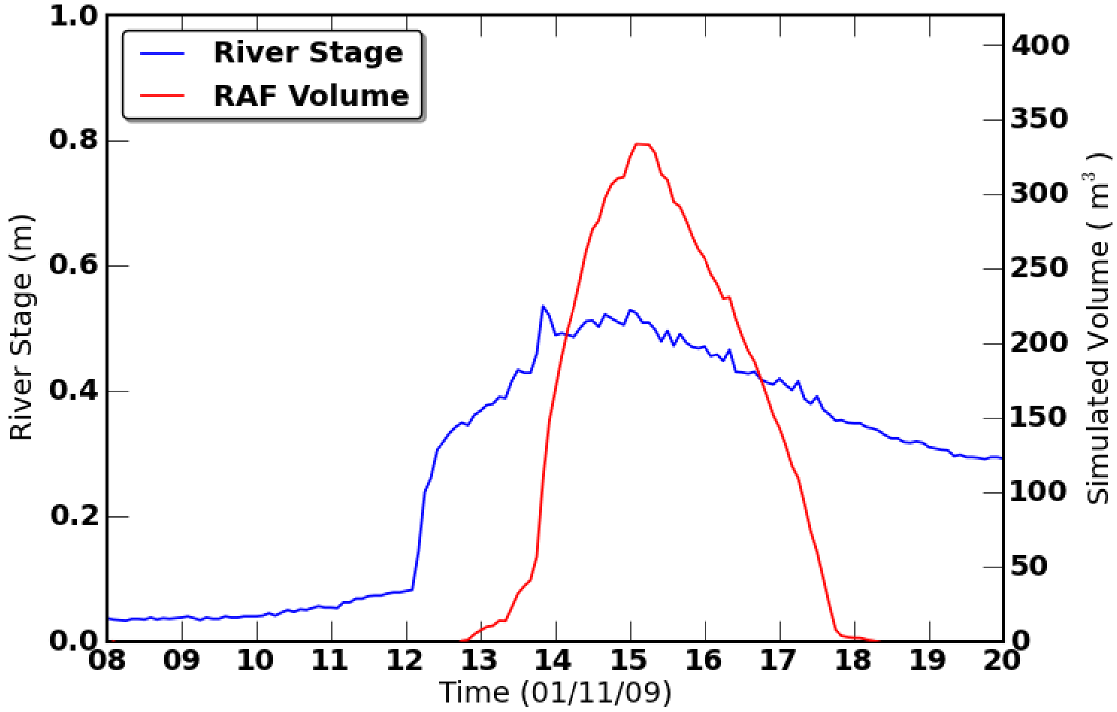


Figure 9.5: Observed river stage (at the R3 flow gauge) plotted against simulated volume (in RAF-3) (for the November 2009 Event)

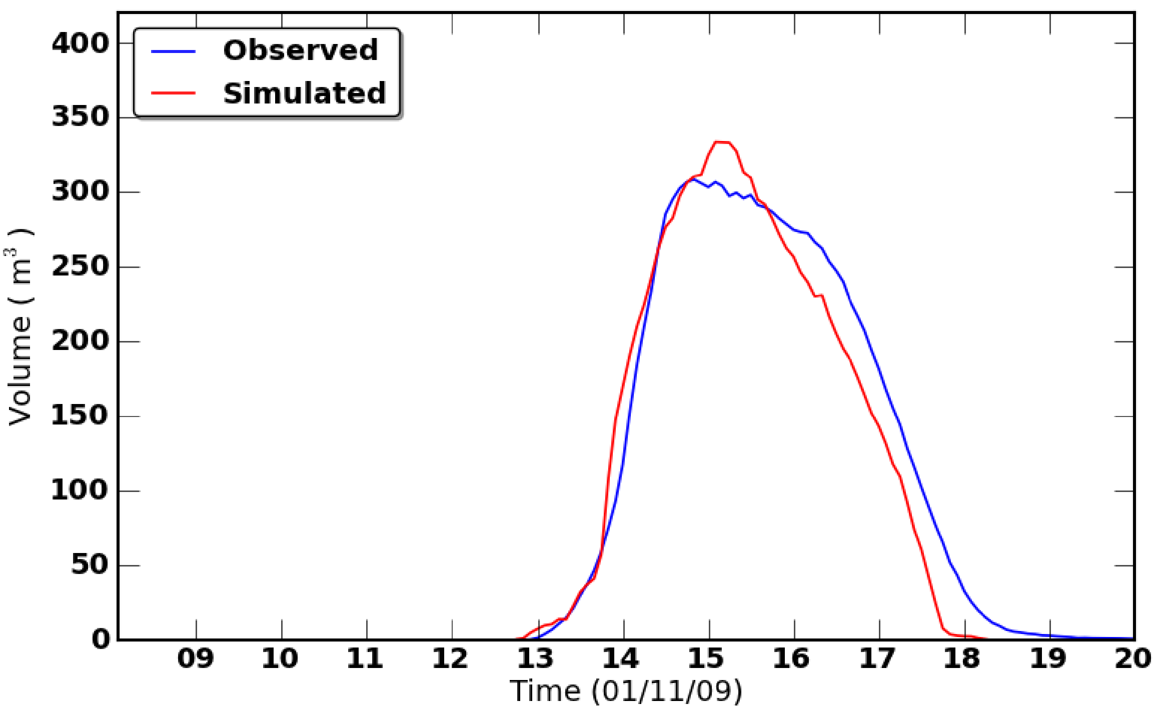


Figure 9.6: Observed versus simulated volume (in RAF-3)

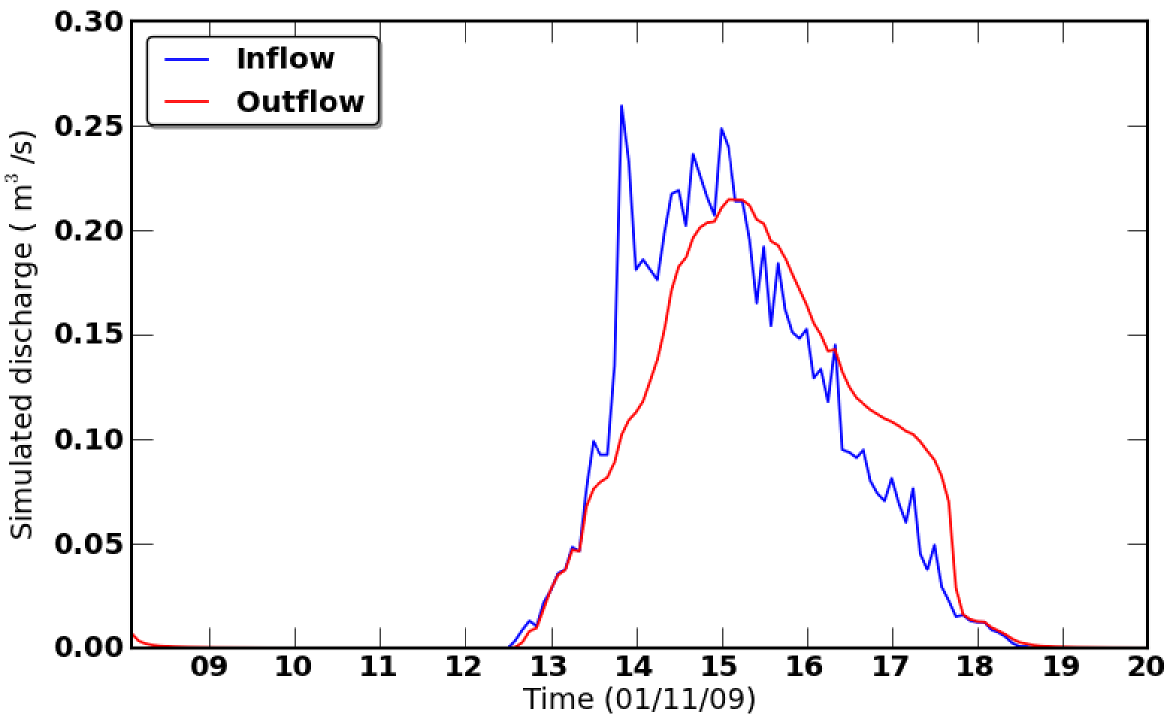


Figure 9.7: Simulated inflow plotted with the simulated outflow (for RAF-3)

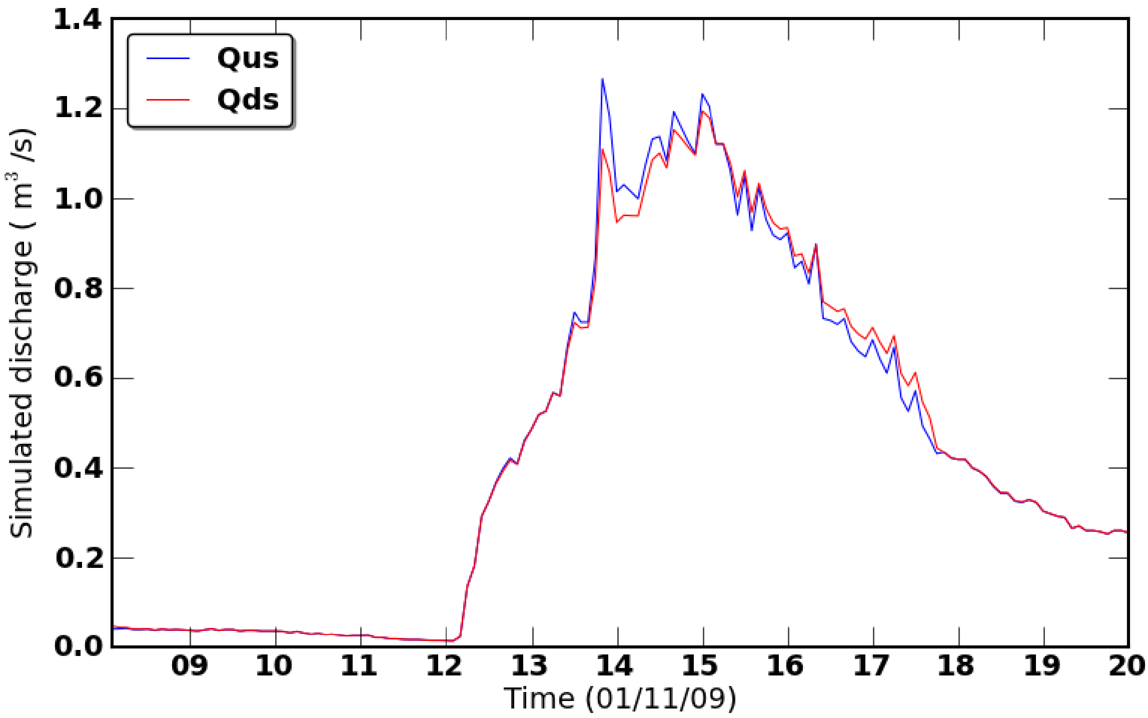


Figure 9.8: Observed upstream discharge (Q_{us}) plotted against simulated downstream discharge (Q_{ds}) to demonstrate the impact of RAF-3 (c. 10% at peak River discharge)

The following figures demonstrate independent model testing on more storm events (of varying magnitude and duration) for RAF-1 in Belford. Note the match between the observed and simulated volumes. The Pond Model uses a combination of observed data and surveyed information, so requires little calibration. It demonstrates a reasonable level of performance in its simulation against the observed. This generates confidence that other features, which have not been surveyed, can be modelled using approximations for inlet heights, outflow conditions and volume-level relationships in order to simulate their function. The Pond Model represents a full RAF with a 'flat-topped' graph in the simulated volume plot. This scenario means the model will assume that inflow is equal to outflow, and the pond will cease to have an impact upon downstream river discharge.

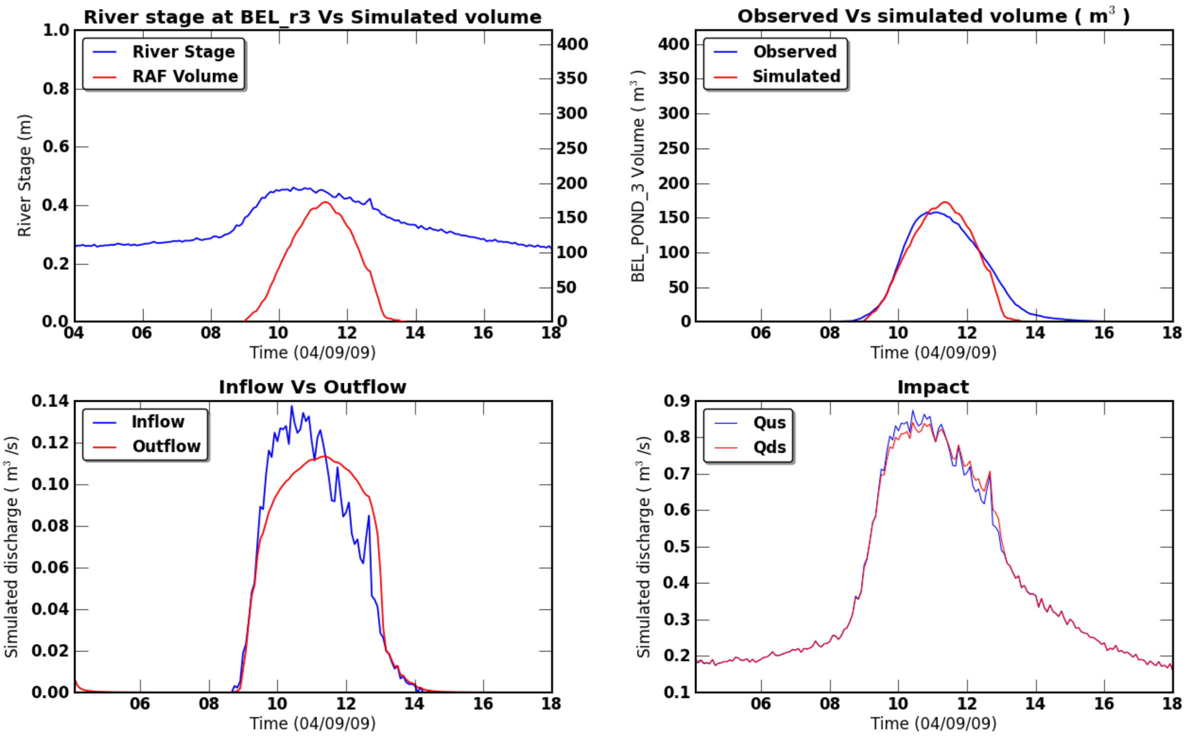


Figure 9.9: Pond Model Output: RAF-1 (September 2009 Event)

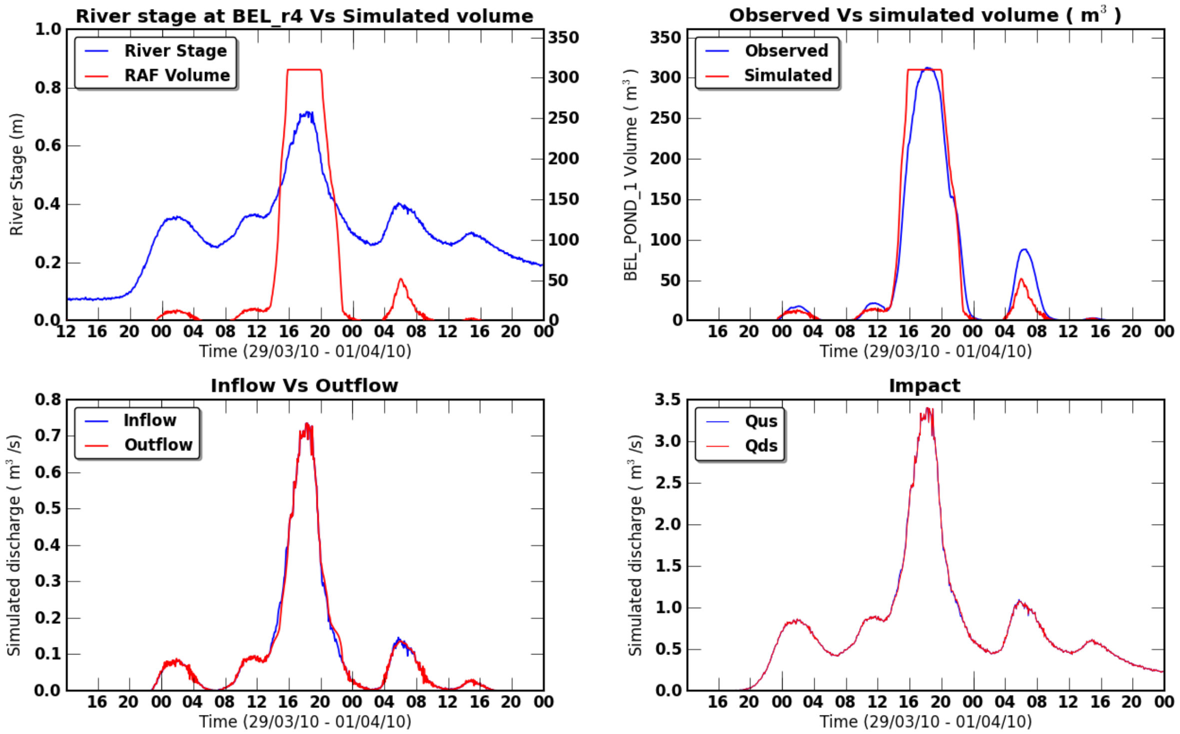


Figure 9.10: Pond Model Output: RAF-1 (March 2010 Event)

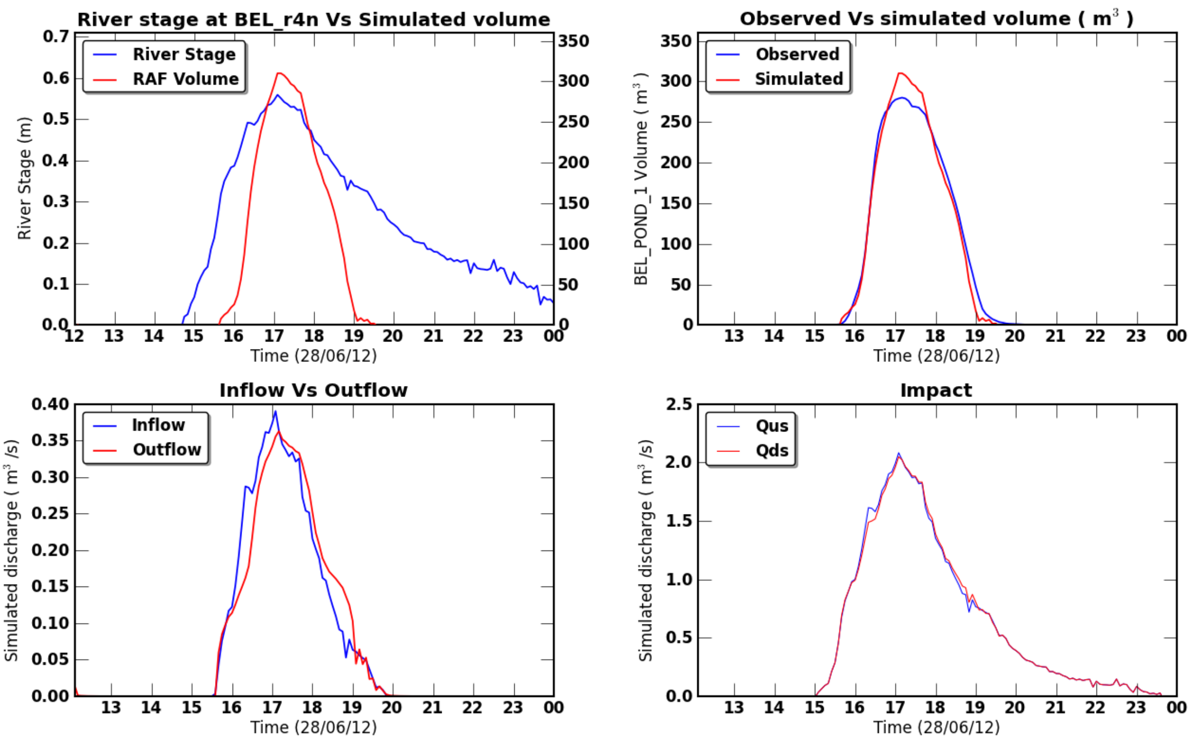


Figure 9.11: Pond Model Output: RAF-1 (June 2012 Event)

9.4 Improvements to existing RAFs

One of the main purposes of the model is to identify how the RAFs can be improved by altering their physical attributes (such as inlet height and width). It has been discussed in supervisory meetings that altering the volume and outflow conditions of existing RAFs would be out of the question, based on the amount of disruption and extra cost this would involve. Changes to the inlet conditions, however, would be cheap and simple to achieve. The following figures show simulations of the Pond Model, for RAF-3 in Belford, starting with the current conditions and moving through changes to the inlet.

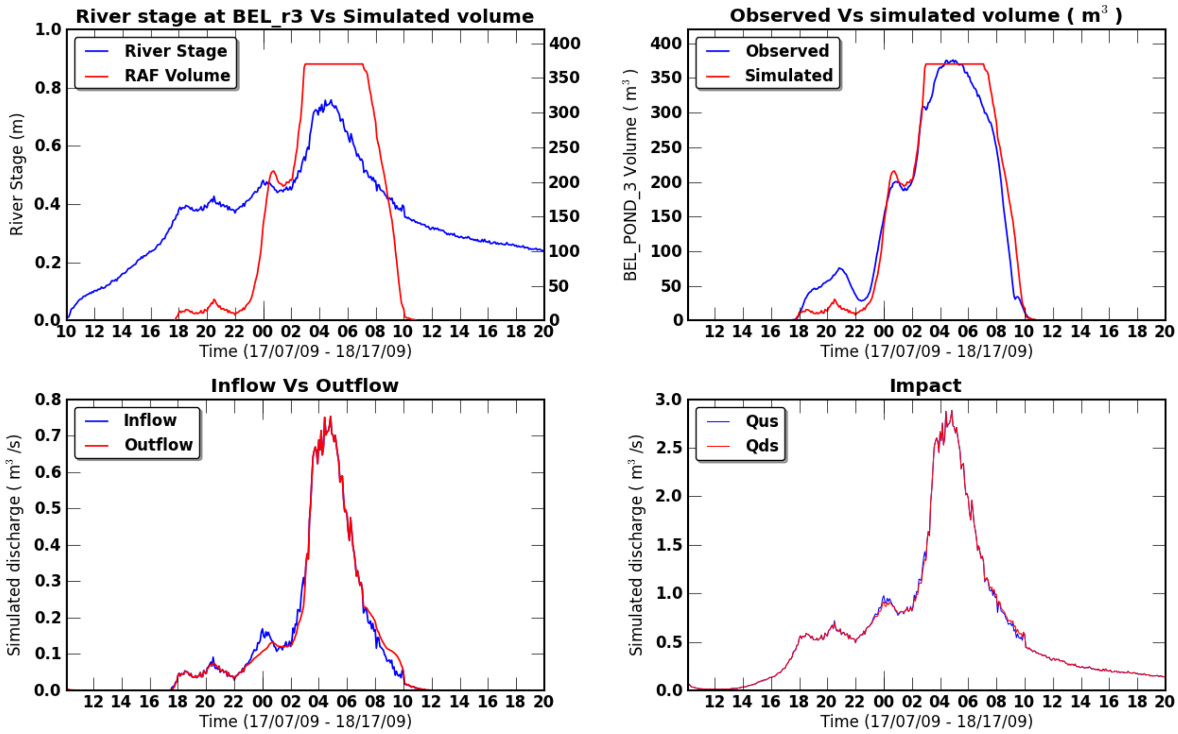


Figure 9.12: Pond Forensics Output (July 2009) – Calibration (Inlet h=0.33m, w=1.1m)

Figure 9.12 shows the baseline scenario for RAF-3 (with the inlet height and width set to 0.33 m and 1.1 m, respectively). Note that the RAF starts filling 8-hours prior to the arrival of the peak. Raising and narrowing the inlet height can have the effect of delaying this process. Figure 9.13 shows that by raising the inlet height by c. 0.1 m (to 0.45 m) the time at which the RAF starts to fill is delayed and a noticeable effect on downstream discharge is witnessed. The RAF does, however, become full (indicated by the 'flat-top' on the simulated volume graph), which reduces its impact during the peak of the event.

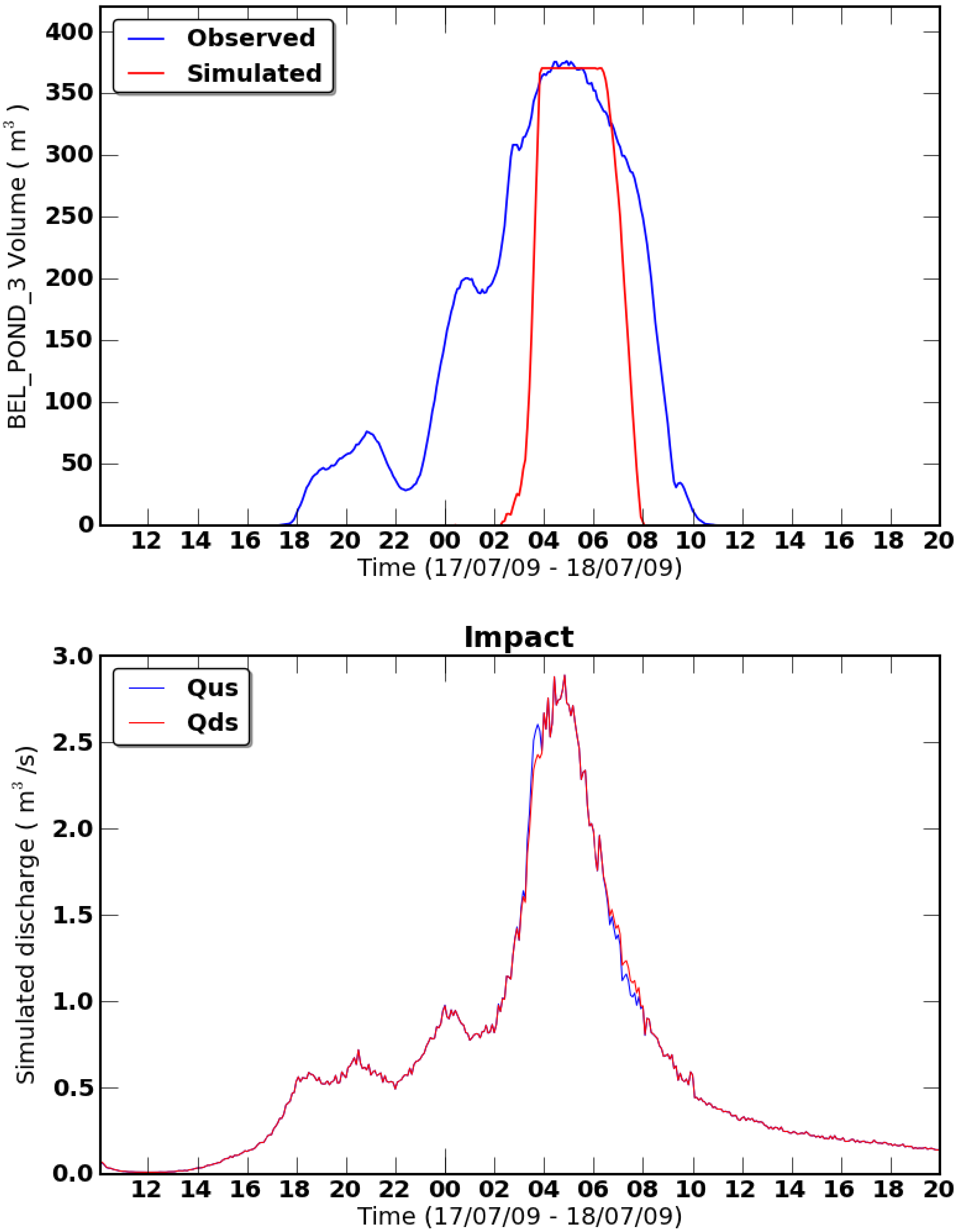


Figure 9.13: Pond Forensics Output (July 2009) – (Inlet h=0.45m, w=0.9m)

Figure 9.14 indicates the effect of increasing the inlet height by too much. This results in targeting only the very peak of the event, and does not allow enough time for the RAF to fill from the River. As the RAF only reaches c. 200 m^3, it can be deduced that the RAF only uses 2/3 of its capacity.

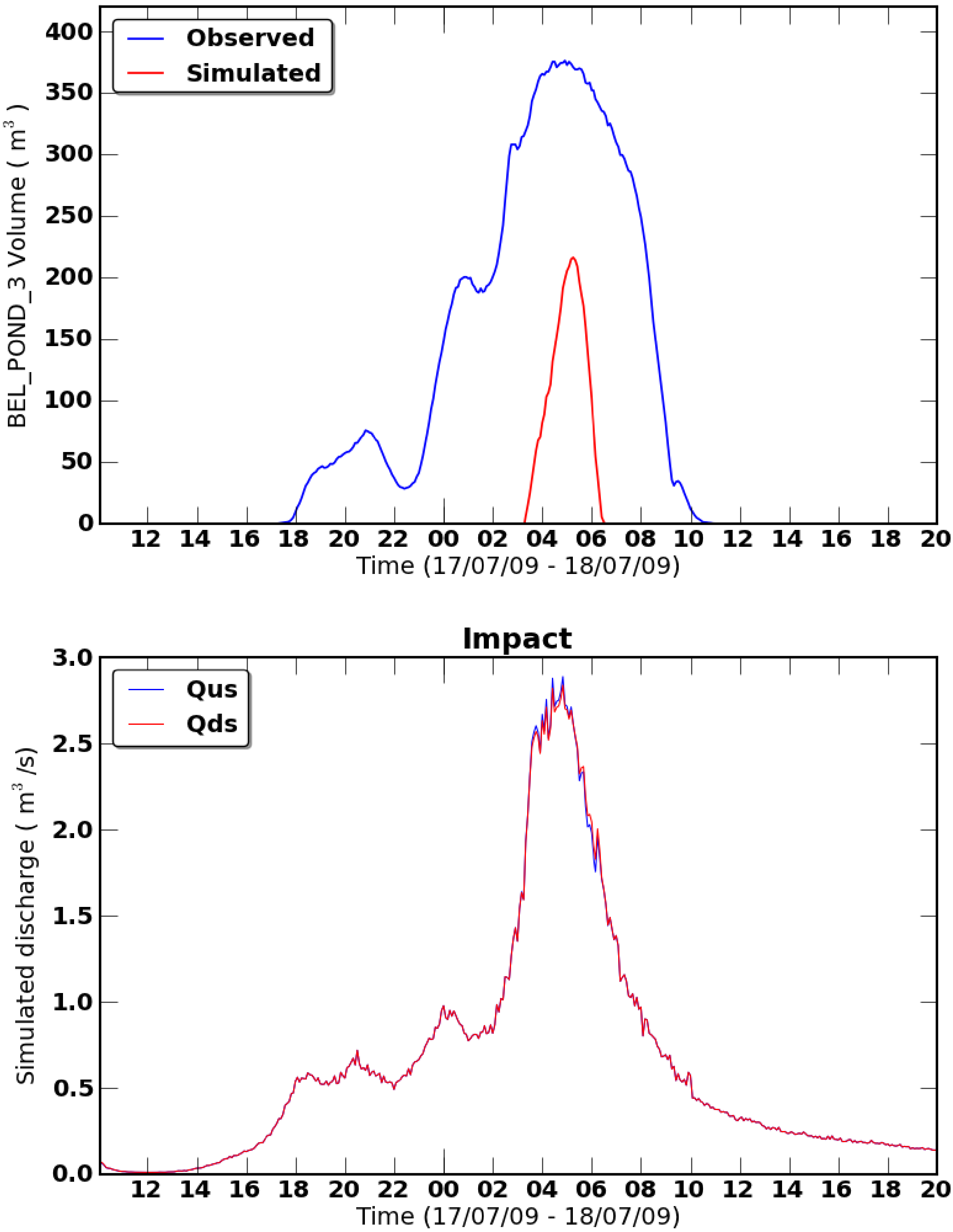


Figure 9.14: Pond Forensics Output (July 2009) – (Inlet h=0.575m, w=0.9m)

The final example (shown Figure 9.15 and Figure 9.16) has targeted the main flood peak, whilst using most of its available storage capacity (without flat-topping). This is an idealised solution and, clearly, will only ever apply to this particular storm event (to the same degree). It demonstrates, however, that peak flow can be targeted with small individual features.

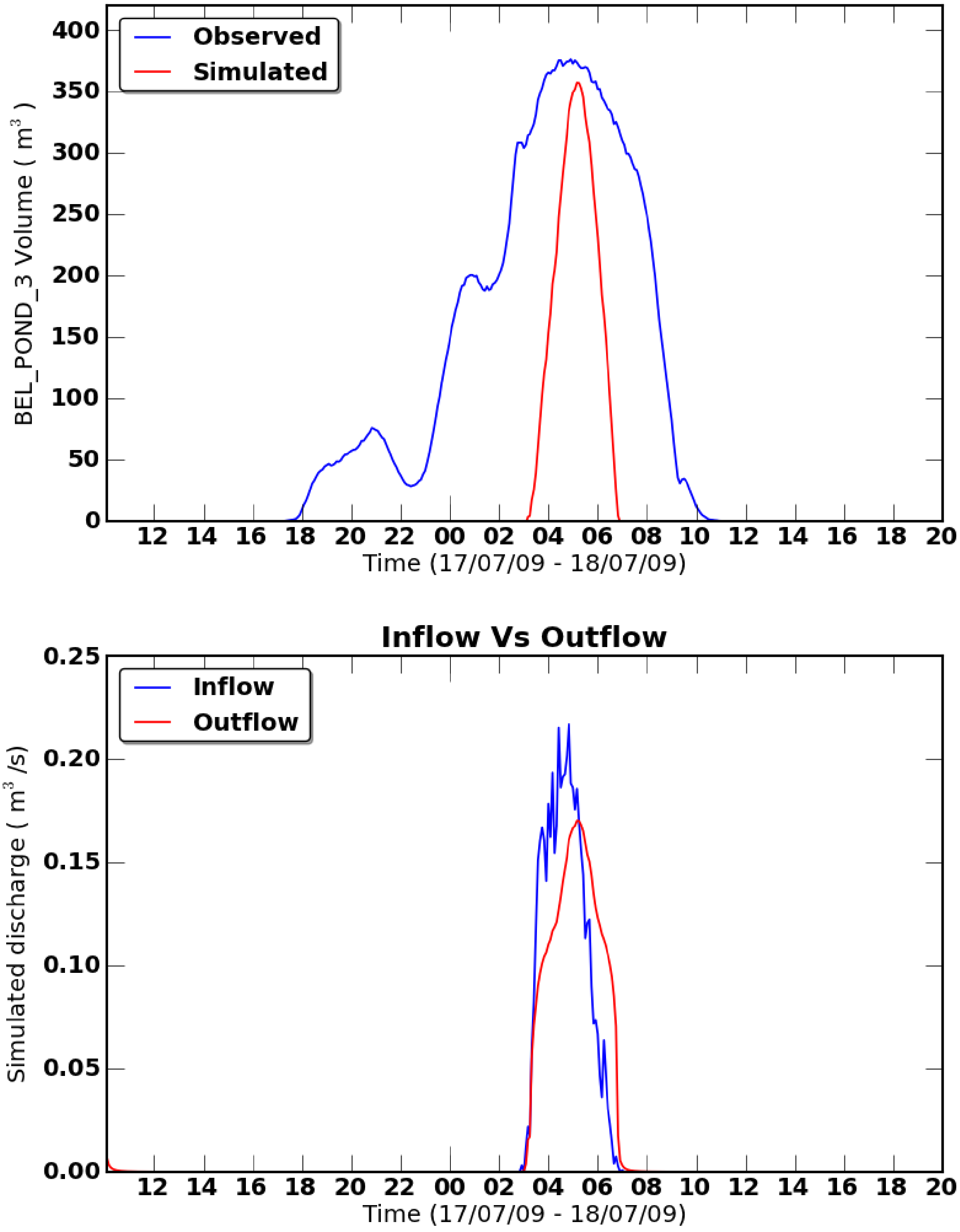


Figure 9.15: Pond Forensics Output (July 2009) – (Inlet h=0.55m, w=1.0m)

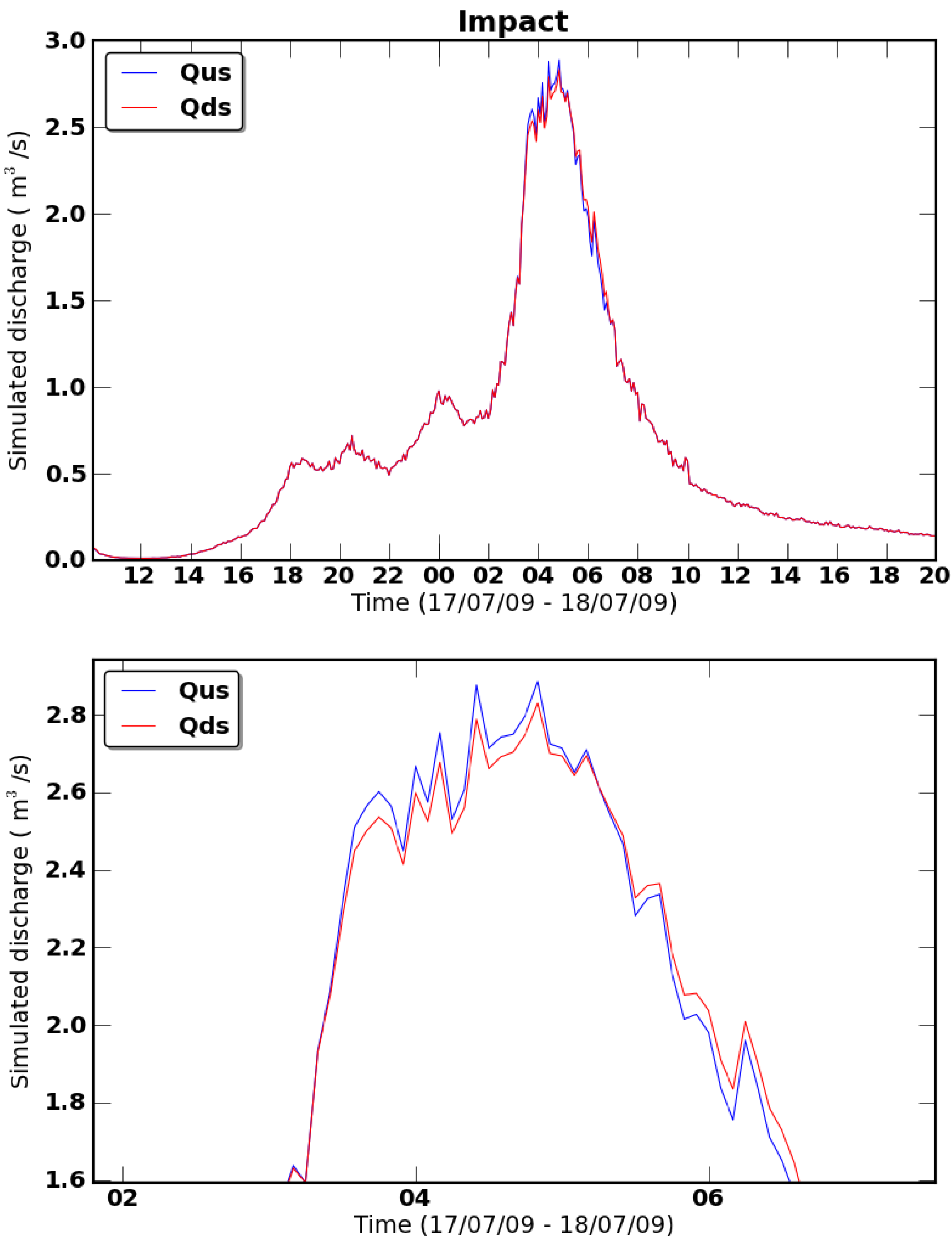


Figure 9.16: Pond Forensics Output (July 2009) – (Inlet h=0.55m, w=1.0m) (Zoomed in on the peak)

It can be seen (in Figures 9.12 - 9.16) that gradually increasing the inlet height of the RAF has the ability of targeting the peak of the storm event. Achieving the optimum inlet height has the effect of separating the inflow and outflow, so that a difference can be measured in the river discharge. For an individual RAF to be effective during a storm event, it must not be allowed to become full until the peak of the event has passed. Figure 9.16 indicates a very small, targeted reduction (approximately 2%) in peak River flow due to the effects of a small RAF located adjacent to the River and highlights the necessity for design inlets to be considered prior to the construction of RAFs.

9.5 RAF Design: Simulating additional volume

9.5.1 Background

In order to make the Pond Model capable of simulating changes to the attributes of the RAFs currently installed in Belford, it was necessary to be able to alter the dimensions of the RAFs using a separate tool, named 'Volume Generator'. Accompanying the Pond Model with this extra tool means that the model will be transferrable to other sites where RAFs are not currently constructed – even other catchments, once it has been tested. It will, therefore, be a useful stakeholder engagement tool for use on similar projects.

The volume generator tool allows the user to change the dimensions of the input shape, which is currently a trapezoidal prism (Figure 9.17) because of its likeness to the natural shape of RAFs. The dimensions that can be changed are the length (a) and width (b) of the base, and the maximum height (h_{max}) of the space. The final dimension (c) is calculated by the volume generator – using the assumption that the bank slopes must have a ratio of 1:2 (for slope stability). Thus:

$$c = (b + 4h)$$

Equation 9.14

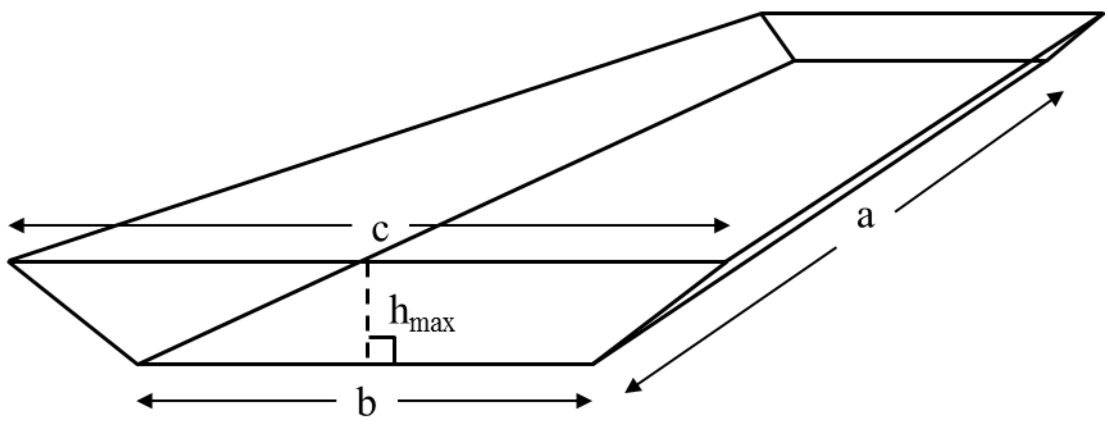


Figure 9.17: Trapezoidal prism

The volume of a trapezoidal prism (Figure 9.17) is:

$$V = a.\left[\frac{1}{2}(b + c).h\right]$$ **Equation 9.15**

Substituting the equation for c into the above formula gives:

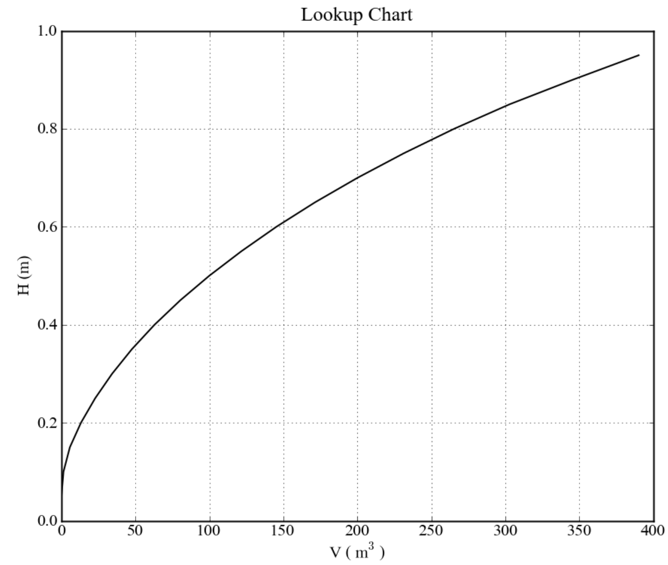
$$V = a.\left[\frac{1}{2}(2b + 4h).h\right]$$ **Equation 9.16**

for the volume at a given h, and:

$$V_{max} = a.\left[\frac{1}{2}(2b + 4h_{max}).h_{max}\right]$$ **Equation 9.17**

for the maximum storage volume of the space.

The volume generator model generates a text file containing a polynomial (quadratic) equation describing the relationship between volume (m^3) and level (m) within the pond, which is used as input for the 'Pond Model with Volume Generator' model, and produces a graph of volume against level. An example of the text and graph are shown in Figure 9.18:



-2.08397169e-07   1.65026556e-03   8.23652235e-04

Figure 9.18: Sample output from the Volume Generator (look-up chart with accompanying quadratic)

The numbers generated in the text-file (shown below the graph in Figure 9.18) represent the values a, b and c, respectively, in the standard quadratic formula:

$$y = ax^2 + bx + c$$

Equation 9.18

where, *y* represents pond level and *x* represents simulated pond volume.

With the Pond Model updated, with the simple representation of a simple storage volume, it became possible to experiment with various pond volumes and inlet conditions to see how each RAF can be designed to have a greater impact during the measured storm events.

9.5.2 Pond Model with Volume Generator

To demonstrate the impact of changing the dimensions of the RAF and increasing the inlet height, the following plots show the current state of the RAF (RAF-1), in the July 2009 storm event, followed by the simulated improvement of the RAF by increasing the maximum storage to 800m^3 and increasing the inlet height from 0.28m to 0.45m (see Figure 9.19 and Figure 9.20).

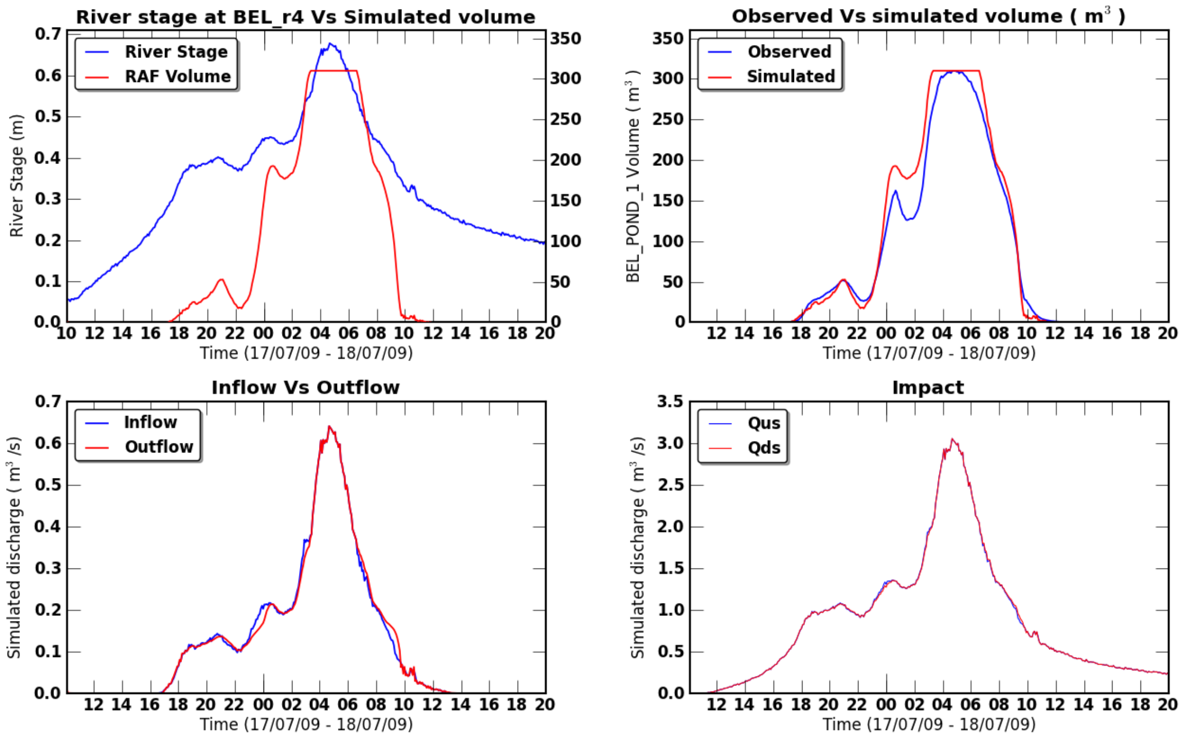


Figure 9.19: Initial conditions of RAF-1 (Maximum volume = 310m³, Inlet h = 0.28m, w = 1.0m)

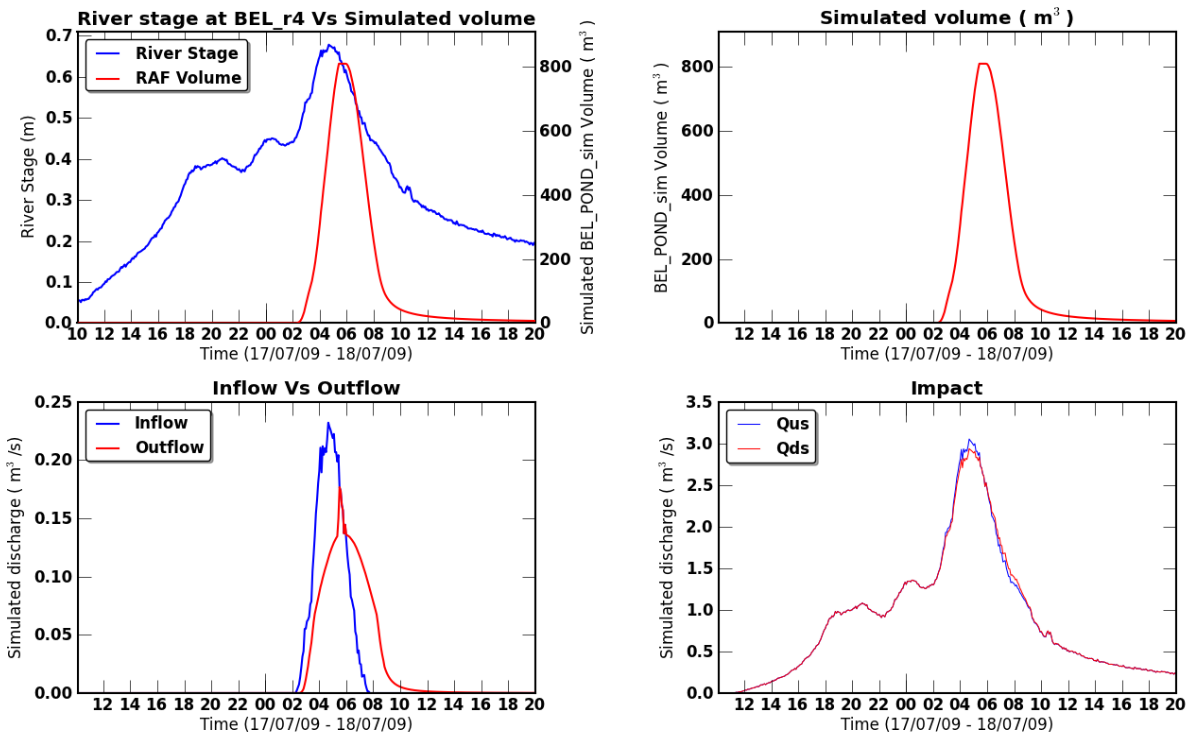


Figure 9.20: RAF-1 (Maximum volume = 800m³, Inlet h = 0.45m, w = 0.9m)

It can be seen that the RAF in the second set of plots does not ‘flat-top’, which indicates that it does not completely fill and hence overflow. This evidence demonstrates the RAF has a small impact on the storm event – reducing the flow in the river downstream of Pond 1 by nearly 5% at the very peak. This is, admittedly, a small amount, but it will soon be shown that by combining several of these RAFs into a network, the discharge in the river can be greatly reduced.

9.5.3 Aggregate Pond Model

It was decided to make the model represent the total effective storage for a catchment (see Table 7-2). If we use the Pond model (with volume generator) in its current state, we are only able to identify the total storage required for the catchment. The timing of the storage in its divided form must be assessed using statistical techniques to map storage versus inlet height over a number of scenarios. Below is an example of how the model can demonstrate the total storage required for Belford (based on results from the July 2009 flood event):

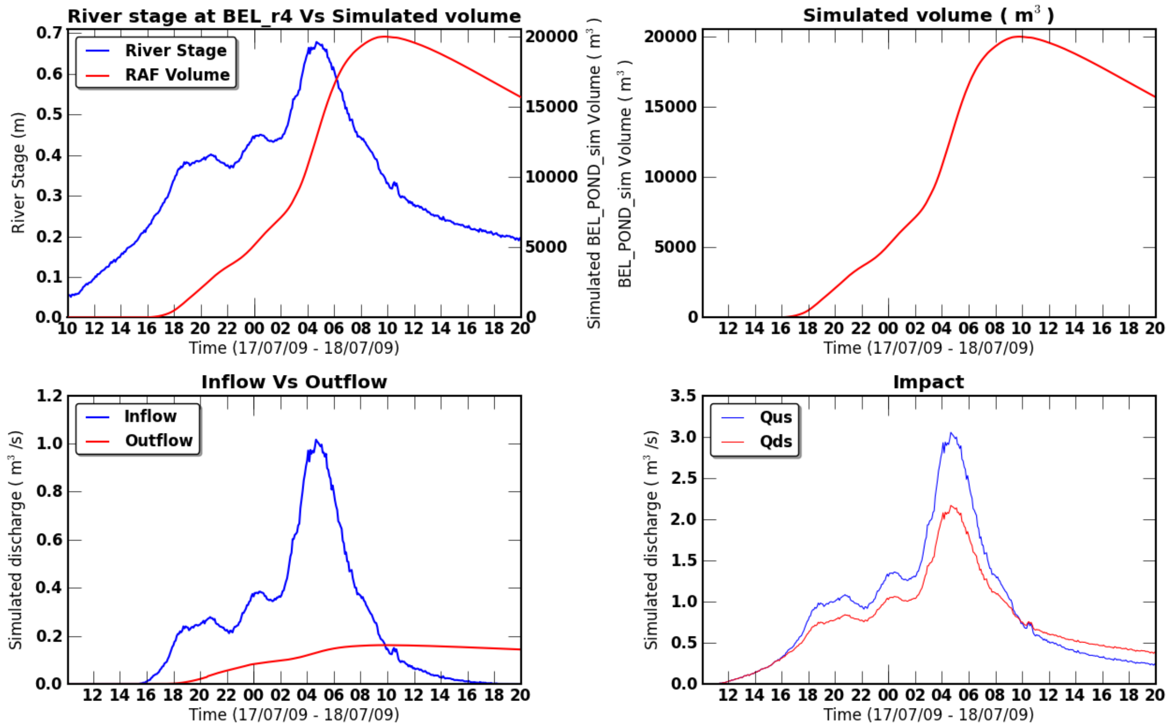


Figure 9.21: Aggregate Pond (Maximum volume = 20,000m³, Inlet h = 0.2m, w = 1.3m)

In the above scenario (Figure 9.21), a large storage body of capacity 20,000m^3 has been simulated in the location of RAF-1. The storage body has dimensions a=200m, b=100m, h=1.0m and has an inlet height of 0.2m (w=1.3m). We can see that the peak discharge has been reduced by the effective storage by approximately more than 30%. With this scenario, the RAF would exist in a single location and act more like a storage reservoir. The reduction in peak discharge supports the rationale behind the Belford study (see 7.4), but lacks realism. The RAF itself will require approximately 40-hours to completely drain. This would render the RAF partially, if not fully, disabled in the event of more rainfall. This information gives confidence, however, when estimating the total number of RAFs required within a catchment. Table 9-1 shows the impacts of a range of large RAFs for comparison. Note the large footprint required; which is due to the maximum height of the feature being limited to 1 m (avoiding reservoir status).

Table 9-1: RAF sizes and percentage impact on peak discharge

RAF Volume (m^3)	Inlet dimensions (m)	Footprint (m)	Percentage impact on Q_p (%)
1,000	h = 0.44; w = 1.0	30 x 33	4.5
5,000	h = 0.32; w = 1.0	100 x 50	12.5
10,000	h = 0.25; w = 1.0	100 x 100	18
20,000	h = 0.20; w = 1.3	200 x 100	29

9.6 Pond Network Model

9.6.1 Model development

The Aggregate Pond Model presents an estimate of the total storage required to impact a particular flood event. It does not, however, provide a realistic solution to the flooding problem. Based on experience gained from the Belford project, it would be impossible to secure such a large amount of land in one location. Large storage ponds are very expensive to construct and manage (Halcrow, 2007). In addition, there is a high risk of catastrophic failure when providing one large RAF. Imagine a catastrophic failure occurring at the peak of a storm event – far worsening the situation downstream. It was decided, therefore, to develop a model capable of demonstrating the effect of multiple RAFs constructed in series on a reach of the Belford Burn (Figure 9.22).

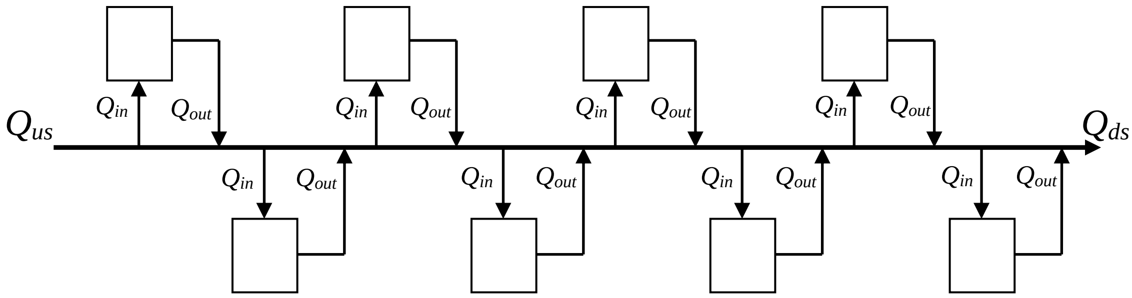


Figure 9.22: Schematic of RAF network

The model structure assumes that there is no hydraulic routing between RAFs in the network, as it simply represents the physical storage in a series of RAFs.

9.6.2 Historical storm events

The Pond Network Model allows us to plot a specified number of identical RAFs adjacent to a reach of a river. The output for a case in Belford is shown below. The storm being captured is July 2009. The RAF volume is 550m^3 (with a footprint of 20 x 25 m) that triggers through an inlet positioned at 0.45m above the river bed and is 1.0m wide. Figure 9.23 shows the impact that growing RAF numbers have upon the stream discharge. The colour-ramp of the graphs goes from blue to green, with the bluest line being the initial discharge before entering any RAFs and the greenest line being the discharge in the river after the final RAF in the network (the lines indicate increments of 5 RAFs for clarity).

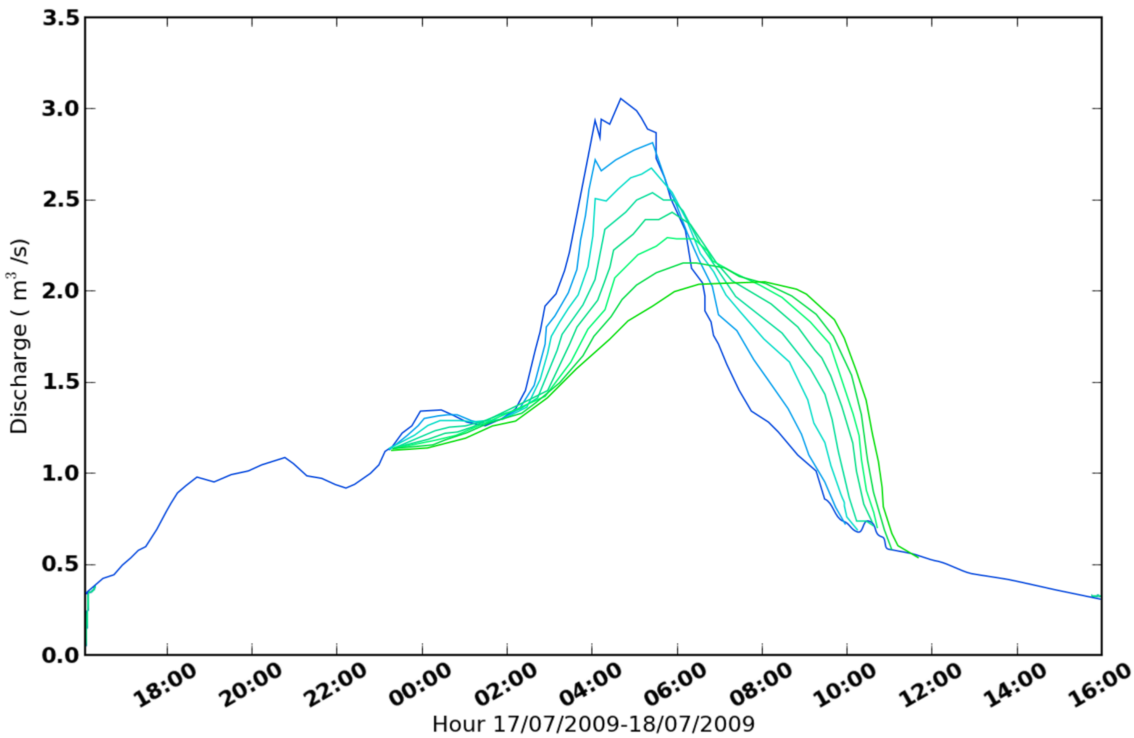


Figure 9.23: Pond Network of 35 RAFs (July 2009 event) - Each line represents impact of 5 RAFs

The output from the Pond Model shows an altered shape to the hydrograph as a result of 35 RAFs positioned in sequence along a short reach of the river in Belford (see Figure 9.23). Water from the peak river flow is temporarily stored and then slowly released as the river level drops; augmenting the recession. The output shows a reduction in river discharge of more than 30%, which demonstrates the benefits of having all the RAFs in the catchment working together at the peak of the event. The effective volume required for the 35 RAFs being simulated is approximately 20,000m^3. There are currently 3 RAFs (of this type) installed in the Belford catchment, with a total surveyed 'physical' volume capacity of 1,500m^3. Transient storage, due to increased roughness and tortuosity, may also exist in the RAFs installed in Belford, though it is difficult to quantify. Adding the storage from the rest of the RAFs installed at Belford gives an estimated total storage of approximately 8,000m^3 (see Table 7-2).

The output in Figure 9.23 also indicates the level of discharge attenuated by 5, 10 and 20 RAFs; approximately 10%, 15% and 25%, respectively. These results indicate the necessity to determine the level of protection required at the catchment outfall (see Figure 7.8). Combining this information with the output from the Pond Model may make it possible to estimate the potential cost of achieving the required protection.

The Pond Model is generating quantitative evidence demonstrating the impact of RAFs upon downstream discharge. The example above takes an existing RAF installed in the Belford catchment and simulates the effect of replicating it several times along a reach of the river. The Pond Model also has the ability to assess the functioning of existing RAFs so that changes can be made to their design to improve efficiency. For example, it was determined that RAF-1 in Belford would benefit from having its inlet channel raised, which was carried out in summer 2012. The Pond Model can also simulate the effect of these changes upon historical storm events and assess the impact of adding entirely new RAFs to the existing network.

The rationale presented in Chapter 7 (7.4) showed a storm event in March 2010. Figure 9.24 shows the same pond network that was used in the previous example. Note that not only the magnitude, but the duration of the March 2010 event was much larger in comparison to the July 2009 event (the graph windows are 24-hours for both cases).

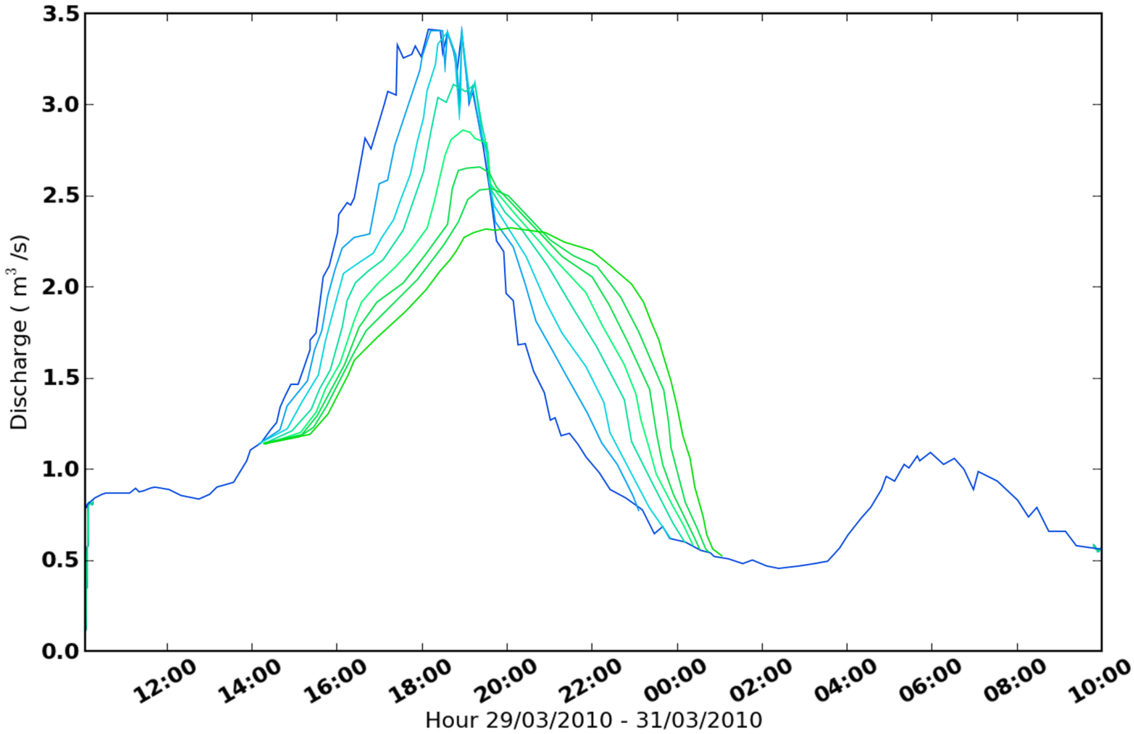


Figure 9.24: Pond Networks of 35 RAFs (March 2010 event) - Each line represents impact of 5 RAFs

Figure 9.24 shows the pond network operating during the March 2010 storm event. It can be seen that a small network of 5 and 10 RAFs, respectively, have almost no impact on removing peak discharge from the river. The smaller networks do, however, shorten the duration of the peak. The model output affirms the need to know exactly how much storage is required to reduce flood levels at a point of interest. The rationale (see Chapter 7 – 7.4) stated that 20,000m^3 of storage would be required to prevent flooding in Belford village. A network of 35 RAFs with 550m^3 capacity equates to 19,250m^3. The Pond Network Model has proved the ability of a network of RAFs to reduce downstream discharge to safe levels on a number of historical storm events.

9.6.3 FEH design storms

Traditional flood defence schemes are designed to protect a point of interest against storm events of certain return periods. It is necessary, therefore, to assess the performance of RAFs and the Pond tool using design storms. The event chosen is the 1:100 year winter rainfall event, produced using FEH techniques. The event is simulated using the Pond Model (Figure 9.25) to demonstrate the impact of 35 RAFs on river discharge.

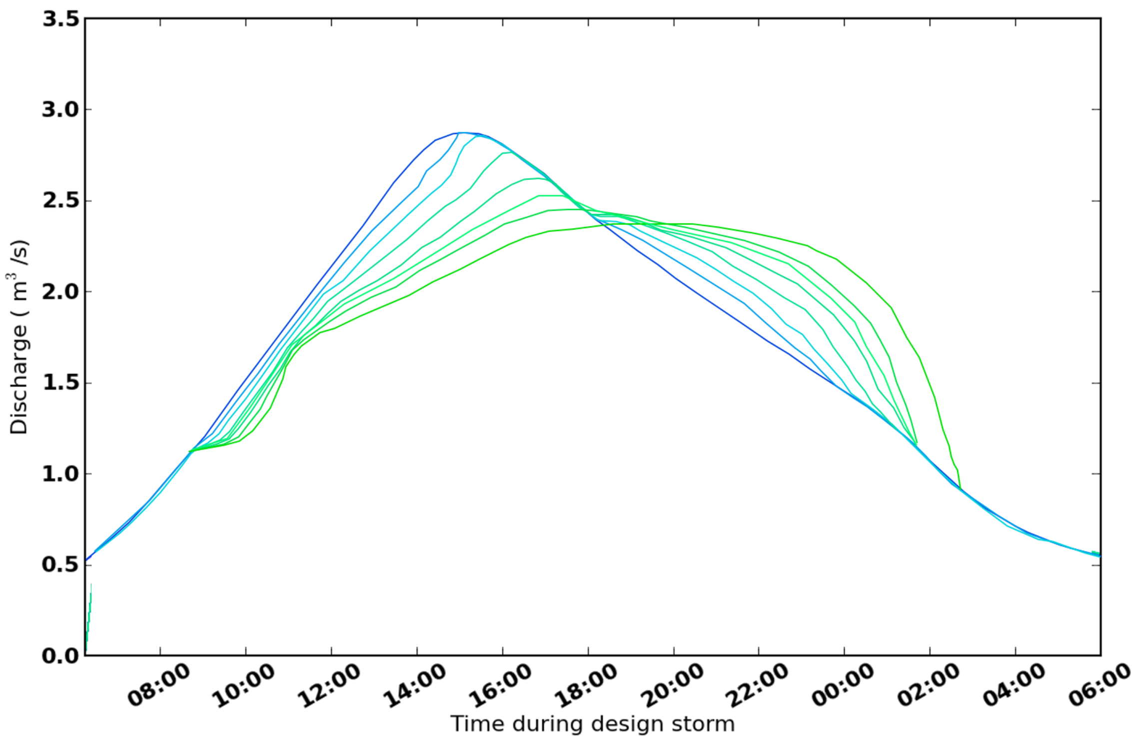


Figure 9.25: Output from Pond tool showing cumulative impact of 35 RAFs (max. vol. = 550 m^3) on the 1:100 year winter flow event - Each line represents impact of 5 RAFs

It can be seen that the RAF network reduces the discharge in the river by approximately 18%. The impact of the RAFs is reduced (compared to the July 2009 event) based on the duration of the storm; however, the presence of the RAFs still provides protection at the point of interest during this design storm. Clearly, there is some degree of uncertainty based on the assumptions made by the Pond Model and alternative shapes that the storm hydrograph could present itself. Double-peaked events or a storm event that is less normally-distributed may present a problem to the RAF network.

9.7 Quantifying mitigation levels

If the percentage reduction in discharge is plotted against the number of RAFs required to force that reduction, the resulting graph reveals an estimate for the best level of protection for a point of interest. Figure 9.26 shows the comparative percentage change in peak between the July 2009 and March 2010 storm events alongside the 1:100 year winter design storm. It can be seen that the percentage reduction is greatest for the July 2009 event, which can be attributed to the shape of the hydrograph for this event. The magnitudes for the July 2009 event and 1:100 year winter design storm were similar, though the duration for the 1:100 year winter design storm was more than twice as long. If the only events being considered were the July 2009 storm event and the 1:100 year design storm; the message presented by Figure 9.26 would suggest that any number of RAFs will generate a reduction in peak discharge. If the March 2010 event is considered in this analysis, however, the message identifies that a storage threshold must be overcome before any significant reductions in peak discharge can be observed. This threshold is approximately 10, 000 m^3 for Belford.

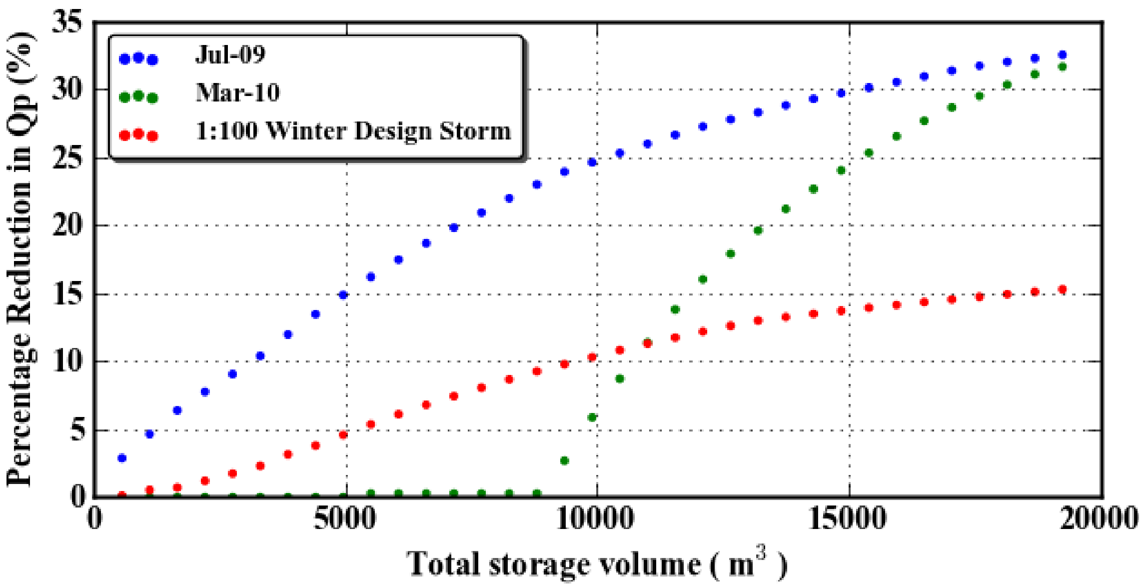


Figure 9.26: Percentage change in peak for Jul-09 and Mar-10 events, and 1:100 year winter design storm

It has been demonstrated by the Pond Network Model that it requires numerous RAFs to start impacting the peak of the March 2010 storm event. The magnitude of this event was almost 15% greater than the July 2009 event with a longer duration by approximately 2-hours. The resultant effect of these factors was a lack of reduction in peak discharge for the first 16 RAFs in the sequence. It is important to note, however, that the duration of the peak discharge was reduced due to the impact of these 16 RAFs. Figure 9.26 shows an agreement in peak discharge reduction as a result of 20 RAFs. The shorter duration of both the July 2009 and March 2010 storms means that the overall reduction generated by 35 RAFs is much greater than the impact simulated during the 1:100 year winter design storm.

These graphs could be used to allow planners to estimate how many RAFs (how much storage) would be required in order to generate a desired percentage reduction in peak discharge. The information generated by these graphs could be combined with estimates of costs of RAFs, which could also be useful for planners when given a budget for flood alleviation. In essence, the analysis shows the positive benefits of a network of, relatively inexpensive, RAFs to create storage. The original premise of this study was that at least 20,000m^3 of storage is required to reduce flood hazard in Belford. Based on experience gathered in the Belford project; a total of 35 RAFs at a cost of £1,500 each (capacity 550 m^3) would require an expenditure of £52,000 (excluding maintenance costs). The construction of a large flood reservoir (e.g. 20,000m^3) would cost in the region of £2.5 million (Halcrow, 2007). The Belford project cost approximately £250,000 on a combination of consultancy and mitigation. A similar site may not require 35 RAFs but should maximise the number if possible.

The Pond Model should be used in conjunction with topographic analysis and local knowledge to determine the location of opportunistic sites throughout a catchment. The space required (20 x 25m) is minimal for small catchments; however, it is possible that locations directly adjacent to the river channel are not possible for a number of reasons. A combination of RAF types and locations should be considered for mitigation against flooding.

9.8 Overland flow and the Pond Model

The Belford study has revealed a great deal about the function of offline diversion ponds. Chapter 7 (7.5) discusses the other types of RAF installed in the Belford catchment. Many of the RAFs, in Belford, capture overland flow by disconnecting fast flow pathways. Here, the evidence from observed data will be discussed alongside a simple tool for estimating the impact of a network of overland flow features on a sample sub-catchment.

9.8.1 Results from overland flow features

Appendix C.1 contains evidence gathered from two of these overland RAFs (RAF-6 and RAF-11 – see Chapter 7 – 7.5), which have been monitored from November 2011. RAF-6 has a relatively large contributing area (0.5 km^2) and steeper slopes leading up to it, compared to RAF-11. The land approaching RAF-6 is used for cattle grazing, which keeps the grass short and the soil compacted.

9.8.2 Forensic analysis of RAF-11

A more forensic analysis of RAF-11 was undertaken to assess the impact it had had on the runoff generated by the June 2012 storm event. The analysis incorporated aspects of the Pond Forensics approach and the Pond Model. All the data in the model is observed apart from the calculation of RAF outflow, which uses the hydrostatic equations (from Chapter 9 – 9.2), and the subsequent estimate of RAF inflow, which is obtained using the following equation:

$$Q_{in} = \frac{dv}{dt} + Q_{out}$$ **Equation 9.19**

where *dv/dt* is the change in volume during the time-step (measured in m^3/s) and the volume (*v*) is obtained from a lookup table – following surveying the RAF.

The results, shown in Figure 9.27, demonstrate a significant reduction in peak overland flow (>50%) generated in the small contributing area preceding the RAF. This, combined with the observed data of these types of features filling during storm events, indicates a high local impact targeting overland flow.

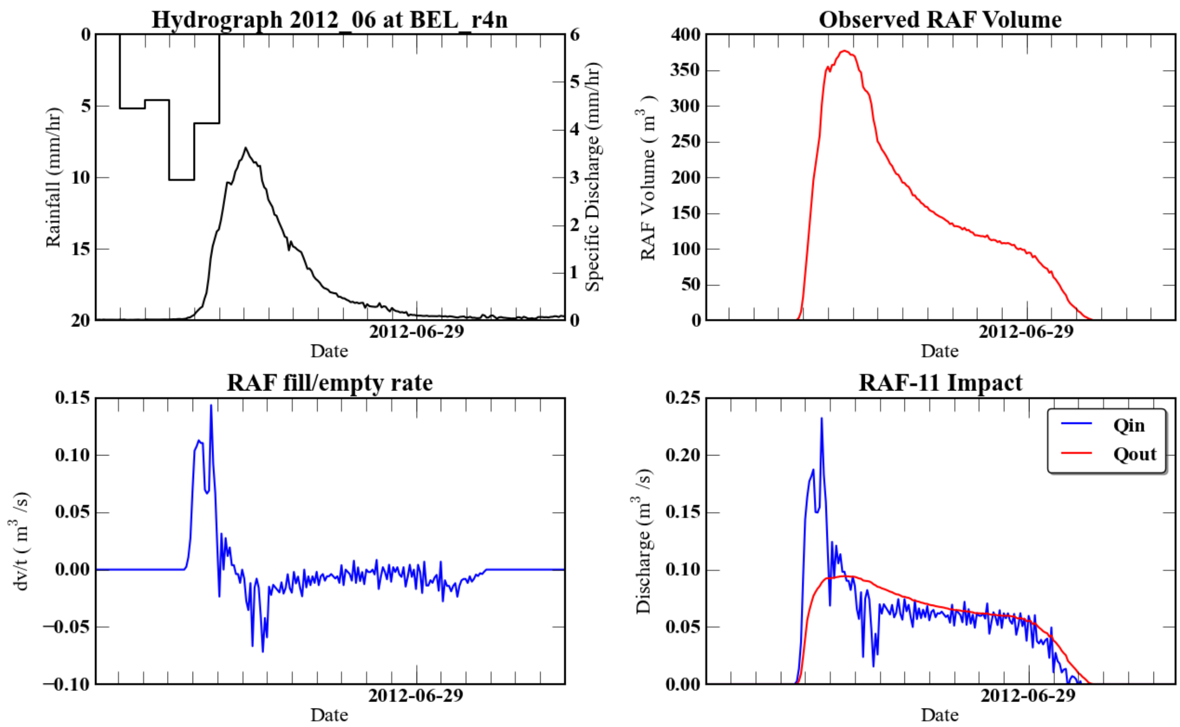


Figure 9.27: Pond Forensics output from RAF-11 during June 2012 storm event

9.8.3 Model development

Following the mounting evidence from the high potential of using overland flow RAFs, it was decided to construct a model capable of targeting overland flow attenuation over small catchment areas. This tool converts specific discharge (q), taken from an appropriate gauging station, into overland flow (Q_{over}) (in m^3s^{-1}) using the effective catchment area (A) (in km^2), upstream of a RAF, and an estimated runoff coefficient (α) (see Equation 9.20). A schematic of the model structure is shown in Figure 9.28.

$$Q_{over} = \frac{q.A.\alpha}{3.6} \qquad \text{Equation 9.20}$$

where, the constant (3.6) is a function of converting q (mm/hr) and A (km^2) into discharge (m^3s^{-1}). If a network of RAFs is being considered, the flow into the subsequent RAFs is a combination of the overland flow (Q_{over}), generated in the effective area upstream of them, and the outflow from the previous RAF in the network. The outflow from the RAFs is the same as that used in the original Pond Model (see Equation 9.7). Just like the Pond Model, the flow into the pond (Q_{in}) is immediately converted into a temporary volume by multiplying Q_{in} by the length of the time-step, in seconds (which for the Belford simulations was 300-seconds or 5-

minutes). This converts from discharge ($m^3\ s^{-1}$) into volume (m^3). The volume is then converted into water level within the RAF using the lookup tables obtained from surveys (see Chapter 7 – 7.3). The water level is then used to calculate the outflow from the RAF for the next time-step in the simulation (see schematic of model structure in Figure 9.28).

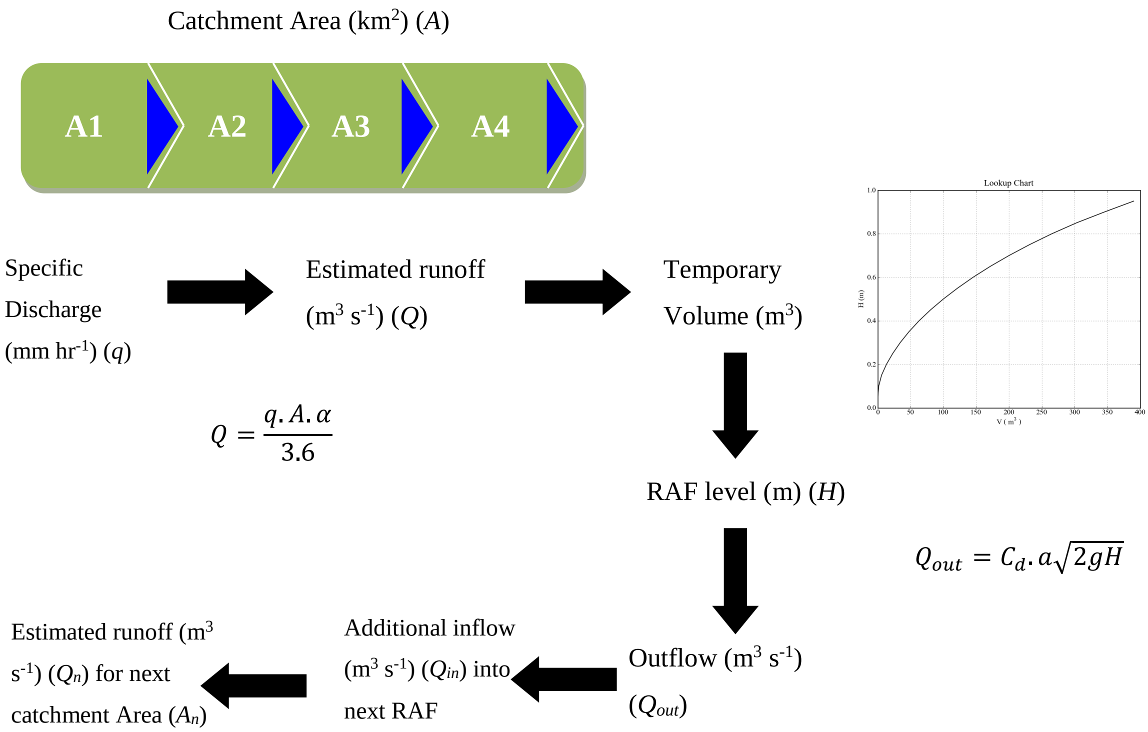


Figure 9.28: Schematic of overland flow tool

The overland flow tool has been used here to simulate four RAFs within a $1km^2$ catchment. The specific discharge (from the R2 gauging station) from the July 2009 storm is converted into runoff using the equation for Q (shown in Figure 9.28). The RAFs have been evenly distributed throughout the catchment area, so that each one has an effective upstream area of $0.25km^2$. RAFs 1-4 have $650m^3$, $950m^3$, $1{,}900m^3$ and $2{,}350m^3$ capacity, respectively. In order to target peak runoff, the outlet pipes have increased in diameter as the catchment area has increased. This is to minimise the risk of the RAFs overtopping during the storm event. The RAFs, capturing overland flow, should only begin to fill when the outlet pipe is surcharged as a result of the magnitude of the surface runoff. The results from the simulation are shown in the graphs that follow:

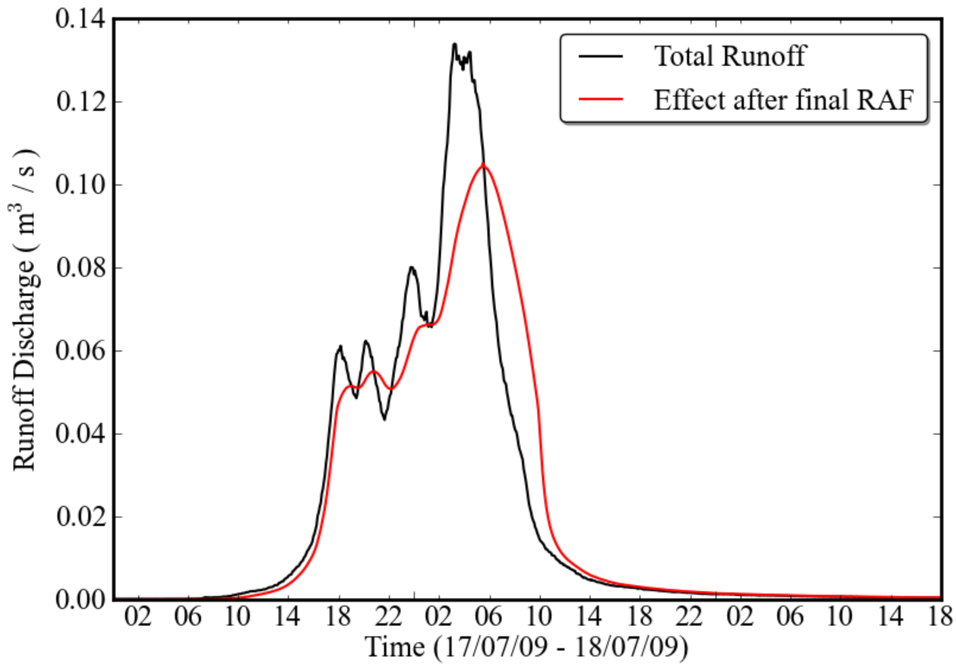


Figure 9.29: Impact of RAF-1 upon surface runoff

The impact of the first RAF is a reduction of 30% of the peak estimated runoff (over $0.25km^2$): same as the first simulation. This can either propagate over the next catchment area (shown in blue on the second graph) or can be attenuated, further, by a subsequent RAF. The combination of RAFs 1 and 2 reduces the estimated runoff by 27% (over $0.5km^2$).

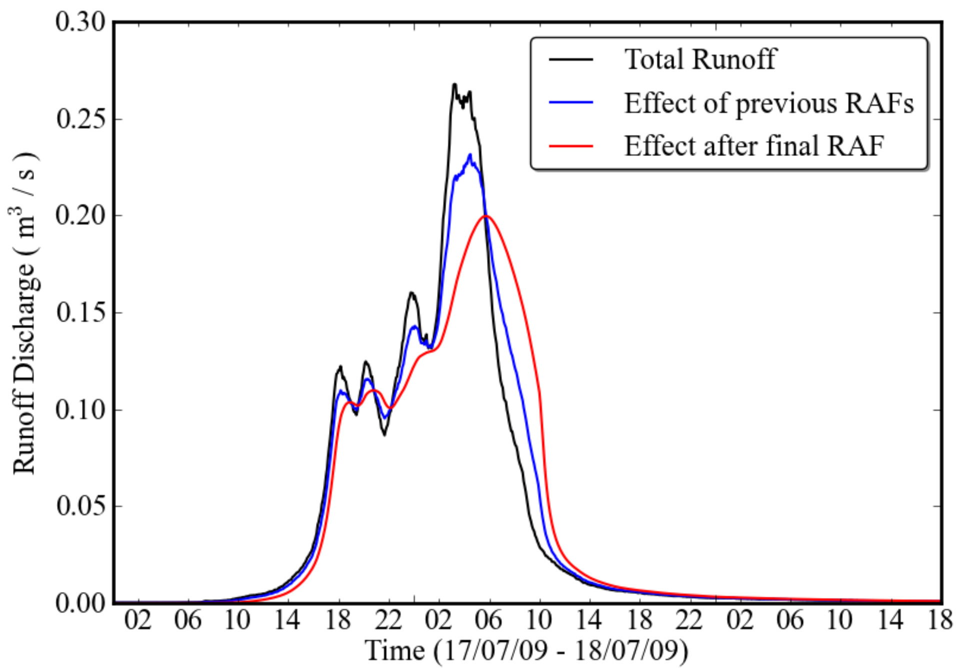


Figure 9.30: Impact of RAFs 1 and 2 upon surface runoff

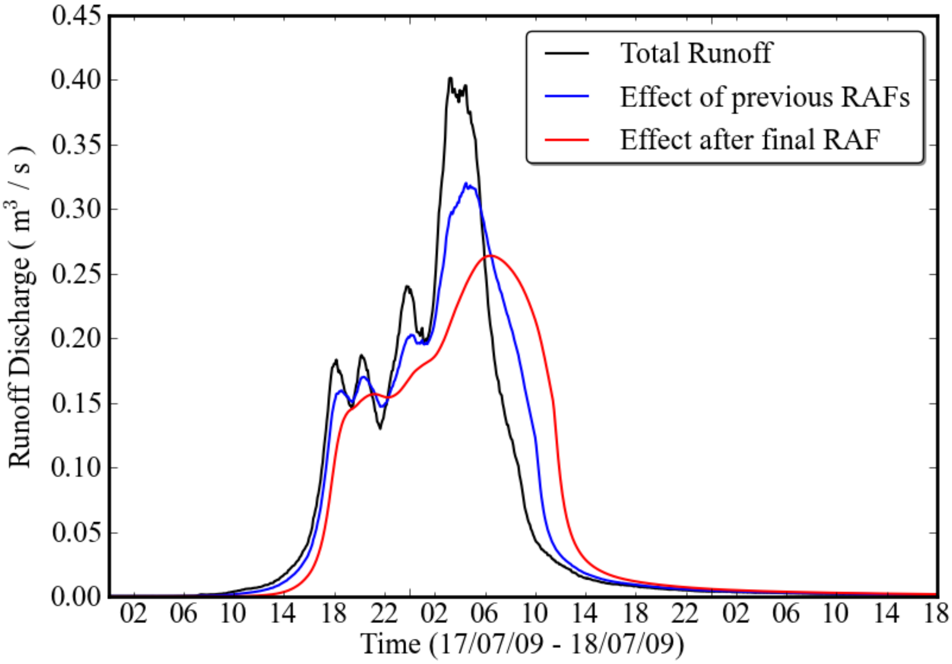


Figure 9.31: Impact of RAFs 1-3 upon surface runoff

The impact of the first two features can be seen (in blue) on the above graph. After the outflow from the two features propagates over the third catchment area and is combined with the additional runoff, it can be seen that just over 20% of the runoff has been attenuated. The combination of RAFs 1-3 reduces the estimated runoff by 36%.

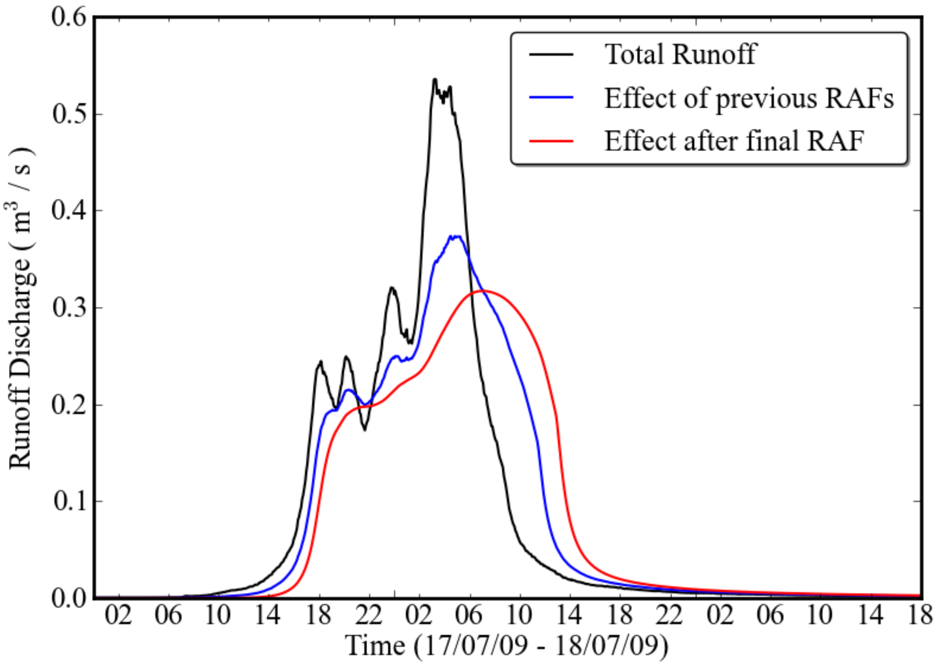


Figure 9.32: Impact of RAFs 1-4 upon surface runoff

At the outfall of the sample catchment, the attenuation effects of RAFs 1-3 have reduced the estimated discharge by more than 30%. The addition of RAF 4 to the sequence further reduces the peak runoff to 42%.

The overland flow component of the Pond Model is, admittedly, very simple and makes a number of assumptions about the generation of runoff. It does, however, indicate that reductions in surface runoff can be measured through the installation of RAFs in the sample catchment. Between 4,000 and 6,000 m^3 of storage (within the 1 km^2 sample catchment) is enough to reduce peak runoff by 30-40%, respectively. The model is physically reasonable and has observations that back-up the generation of overland flow. The representation of the RAFs in the above example may be slightly exaggerated, in terms of the physical storage provided, but reinforces the concept of installing numerous features to capture overland flow.

This methodology highlights the importance of overland flow interception RAFs in Belford. The nature of the landscape leading up to these RAFs, which is often steep and gullied, means that relatively little space is required to generate the high storage volumes without removing vast areas of productive farmland (7.5.3).

9.9 Summary

This Chapter has introduced a novel tool for representing the function of RAFs, both as individual mitigation features and as a network of flood defences (connected to rivers or situated on the landscape). The tool has demonstrated a high level of similarity to the observed data and has identified physical defects with RAFs installed in Belford, which have been updated (e.g. raising the inlet in RAF-1). After the model had been validated against observed data, it was used to demonstrate the level of intervention required on a number of historical storm events as well as design storms. The Pond Model has been successful at representing the offline RAFs installed in the Belford catchment (both those adjacent to the river and those present in dominant flow paths away from the river). It has done this using a combination of survey data and basic hydraulic equations, which have closely represented the filling and emptying regime of the features.

The Pond Model can be used to show impacts, of a network of RAFs, directly on storm hydrographs or take the information from the hydrographs to demonstrate percentage reduction in peak discharge per RAF. For the latter; multiple events can be chosen for the analysis, so that it can be identified whether a threshold of available storage is required (see

Chapter 9 – 9.7). The Pond Model toolkit can be used to demonstrate the benefits of RAFs to stakeholders/land owners, both in terms of desired flood protection and projected cost of installation.

Based on local observations; the Pond Model has generated a conceptual picture of how relatively small RAFs can impact, both, peak flows in channels and overland flow generated during storm events. It does not yet, however, include representation of roughness and tortuosity (altering the route of flow of the water).

10. Hydraulic and hydrological modelling, and experimentation

10.1 Introduction

This Chapter describes the application of separate modelling technologies to represent the RAF network theory in the Belford study through; (1) the application of the hydraulic model, NewChan, on a reach of the Belford Burn between the R3 and R5 flow gauging stations; (2) the development of virtual experiments to validate the Pond Model for networks of RAFs, and; (3) the application of the hydrological model, TOPCAT, to emulate the function of the Pond Model for networks of RAFs.

1. Originally, the purpose of utilising NewChan (Liang, 2008) was to detect the cumulative impact of the small network of existing RAFs (1-3) upon river discharge. During this time, however, the Pond Model was developed and adapted to simulate networks of RAFs. The hydraulic modelling (with NewChan) was, therefore used as a validation exercise for the much simpler Pond Model. Although NewChan, has not been validated against field measurements, it has been extensively tested for application in numerous flood inundation studies elsewhere (e.g. Liang, 2008; Liang, et al., 2008; Kesserwani, et al., 2010). In addition to engaging in a comparative study between models, there was a desire to create a transferrable procedure for estimating and modelling the impact of soft-engineered RAFs on other catchments. In theory, a hydraulic model can be set-up for a catchment without the need for a detailed monitoring network. The availability of topological data and either rainfall or upstream flow information would be enough to perform initial modelling scenarios for a reach of a river.
2. The application of NewChan was then altered to perform virtual experiments on a simplified DEM, generated in excel, to compare the results from the Pond Network Model, which have otherwise not been validated using observations. It also became apparent that the Pond Model, although being capable of transferring volumes of water to and from a one-dimensional (1D) river, was incapable of representing the two-dimensional (2D) fluid mechanics at the inlet and outlet of each RAF. Thus, this section of research compares output from the Pond Model and NewChan.
3. The development of a simple, transferrable methodology for assessing the impacts of a network of RAFs upon a river may require a different approach – especially for sites that have no access to high quality topological data. For this reason, the simple lumped

conceptual rainfall runoff model, TOPCAT (Quinn, et al., 2008), will also be used to estimate the impact of mitigation in the Belford catchment. This technique will be transferrable to catchments of a similar size to Belford (<10 km^2).

10.2 Background and governing equations of NewChan

In order to improve the computational efficiency of hydraulic models, the full 3D Navier-Stokes equations are often simplified according to what the model is required to demonstrate. For modelling open-channel flows the fully 3D hydrodynamic equations can be integrated over the cross-sectional area to give a set of 1D equations to be used in 1D computational models. Similarly, the depth integrated form of the Navier-Stokes equations is used in 2D models to predict flow over floodplains or in lakes and estuaries.

Advancement of computer technology and wider availability of topographic data has led to a trend to adopt 2D simulations for an increasing number of applications. The higher accuracy of 2D models and their ability to provide more information about velocity distributions and inundation extent makes them more attractive to modellers. 1D models do have their advantages, especially when applied to narrow riverine reaches (such as Belford); requiring less information to run and lower computational strain. It is also worth noting that several hydraulic structures including weirs, culverts and gates are more straightforward to model in one-dimension (Liang, et al., 2007).

Selection of either 1D or 2D models will depend on whether the physical flow field is primarily 1D or 2D in nature. In practical situations, however, 1D and 2D phenomena can occur in the same reach of flow, for example; when bank-full flow depth within a river is reached and the water spreads onto the adjacent floodplain.

Liang, et al. (2007) discuss the pseudo 2D method, whereby a 1D model simulates flow in the river channel, while water spilling onto the floodplain is represented using storage cells. The method is extremely time-consuming to set-up, requiring a mass of information prior to simulation, including flow direction, which is usually unknown for flow over a floodplain. Above all else with the pseudo 2D method, large errors can be generated with the assumption that 2D hydrodynamic processes can be represented in the form of a stage-volume relationship. Conversely, if the entire simulation were run in a 2D model it would generate extremely long runtimes if using a fine 2D grid, and if a coarser mesh was used it could lead to greater inaccuracies.

The NewChan model (developed by Liang, 2008) uses an explicit finite-volume Godunov-type scheme, which is combined with a Harten, Lax and van Leer with contact wave restored (HLLC) approximate Riemann solver to solve the two-dimensional governing equations (outlined below). These equations are solved directly onto a Cartesian grid lattice, which enables automatic capture of trans-critical flows, including shock-like flow discontinuities, to a very high resolution. The grid mesh will be regular and rectangular, and can be refined, for modelling flow through the complex floodplain and RAF network. The finer the grid mesh the more accurately the experiment area will be represented in the computer model. Second order accuracy is achieved in the model using a Runge-Kutta time stepping scheme, and a simple local boundary modification is applied to deal with irregular domain geometry. The model was originally developed for simulating complex flows in open channels but will now be used for modelling 1D flow in the Belford Burn overtopping into RAFs on the floodplain in the 2D domain to simulate the function of the RAFs during high flow events.

10.2.1 One-dimensional equations

Unsteady flow of water in a wide, rectangular channel of gradually increasing cross-section with a sufficiently gentle bottom slope can be modelled using the one-dimensional Shallow Water (St Venant) equations (Kesserwani *et al.*, 2008) – shown below:

$$\frac{\partial h}{\partial t}+\frac{\partial q}{\partial x}=q\frac{b'}{b}$$

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{q^2}{h}+g\frac{h^2}{2}\right)=-\frac{q^2}{h}\frac{b'}{b}+gh\left(S_0-S_f\right) \qquad \textbf{Equation 10.1}$$

where h (x , t) = water depth; q (x , t) = flow unity discharge; b (x) = channel bottom width; S_0 = bed slope; S_f = friction slope; g = acceleration due to gravity. The right-hand side of Equation (10.1) contains the sources and sinks of momentum arising from bed slope, the width variation and the friction losses due to roughness of the channel walls. The bed slope is the spatial partial derivative of the bottom elevation z

$$S_0 = -\frac{\partial z}{\partial x}$$ **Equation 10.2**

The friction slope is defined, in terms of Manning's coefficient of roughness, *n*

$$S_f = \frac{q^2 n^2}{h^2 R^{4/3}}$$ **Equation 10.3**

where $R = bh / (b + 2h)$ = hydraulic radius. These 1D shallow water equations can also be effectively solved using a finite Godunov-type scheme (Alias, et al., 2011).

10.2.2 Two-dimensional equations

The two-dimensional (2D) shallow-water equations can be derived by depth-integrating the 3D Reynolds averaged Navier-Stokes equations (Liang, 2008). In matrix form, the hyperbolic conservation law formed by the 2D non-linear shallow water equations can be expressed as

$$\frac{\partial u}{\partial t} + \frac{\partial f}{\partial x} + \frac{\partial g}{\partial y} = s$$ **Equation 10.4**

where *t* denotes time, *x* and *y* are the Cartesian coordinates, and **u**, **f**, **g** and **s** are vectors representing the conserved variables, fluxes in the *x*- and *y*-directions, and source terms, respectively. Ignoring Coriolis effects, viscous terms and source stresses, these vector terms are given by

$$u = \begin{bmatrix} h \\ uh \\ vh \end{bmatrix} \quad f = \begin{bmatrix} uh \\ u^2 h + \frac{1}{2} g h^2 \\ uvh \end{bmatrix}$$

Equation 10.5

$$g = \begin{bmatrix} vh \\ uvh \\ v^2 h + \frac{1}{2} g h^2 \end{bmatrix} \quad s = \begin{bmatrix} 0 \\ -\frac{\tau_{bx}}{\rho} - gh\frac{\partial z_b}{\partial x} \\ -\frac{\tau_{by}}{\rho} - gh\frac{\partial z_b}{\partial y} \end{bmatrix}$$

where h is the total water depth; u and v are the depth-averaged components of velocity in the x and y directions, respectively; g is the acceleration due to gravity; ρ is the density of water; z_b is the bed elevation above the datum; $\partial z_b / \partial x$ and $\partial z_b / \partial y$ represent the bed slope in the two Cartesian directions; and τ_{bx} and τ_{by} are the bed friction stresses, representing the effect of bed roughness on the flow and may be estimated using the following empirical formula:

Equation 10.6

$$\tau = \rho C_f u \sqrt{u^2 + v^2} \text{ and } \tau = \rho C_f v \sqrt{u^2 + v^2}$$

in which the bed roughness coefficient C_f can be evaluated using $C_f = gn^2/h^{1/3}$, where n is the Manning coefficient.

Rogers, *et al.* (2003) discussed the fact that the shallow water equations (equations (10.4) and (**10.5**)) may not preserve a still water state of u = 0 and $v = 0$ but $h \neq 0$ in a domain with a non-uniform bed profile when they are solved using a finite-volume Godunov-type method incorporated with Roe's approximate Riemann solver. Adjustments have to be made to the vector terms in equation (10.5) to give:

$$u = \begin{bmatrix} \eta \\ uh \\ vh \end{bmatrix} \quad f = \begin{bmatrix} uh \\ u^2h + \frac{1}{2}g\left(\eta^2 - 2\eta z_b\right) \\ uvh \end{bmatrix}$$

Equation 10.7

$$g = \begin{bmatrix} vh \\ uvh \\ v^2h + \frac{1}{2}g\left(\eta^2 - 2\eta z_b\right) \end{bmatrix} \quad s = \begin{bmatrix} 0 \\ -\frac{\tau_{bx}}{\rho} - g\eta\frac{\partial z_b}{\partial x} \\ -\frac{\tau_{by}}{\rho} - g\eta\frac{\partial z_b}{\partial y} \end{bmatrix}$$

If we take the momentum equation for the *x*-direction as an example, the new formulation (equation (10.8)) is essentially derived from the following relationship:

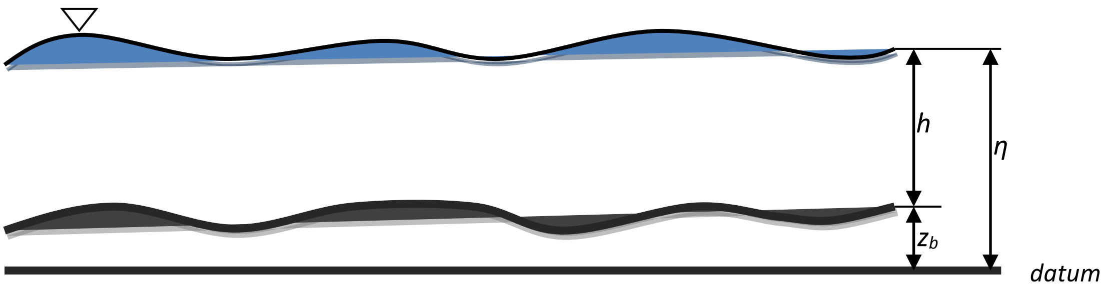


Figure 10.1: Bed topography for the shallow water equations [from Liang Q. (2008)]

Equation 10.8

$$\frac{g}{2}\frac{\partial h}{\partial x} + gh\frac{\partial z}{\partial x} = \frac{g}{2}\frac{\partial\left(\eta^2 - 2z_b\eta\right)}{\partial x} + g\eta\frac{\partial z}{\partial x}$$

Figure 10.1 shows η is defined as the surface elevation above the datum, therefore the water depth can be calculated as $h = \eta - z_b$.

Liang (2008) goes on to explain that the hyperbolic property of the new conservation law contained within the shallow water equations (Eqns. (10.4) and (10.8)) can be evaluated by examining the eigenstructures of the flux Jacobian. The new shallow water equations are mathematically balanced for the flux and source terms so that the fluid's still shallow water state can be automatically maintained. This can be expressed by imagining the general case of motionless steady state of fluid where $u = 0$, $v = 0$, $h \neq 0$ and η is a constant in a domain with spatially varying bed bathymetry. The continuity equation is directly satisfied as $u = 0$ and $v = 0$, so by substituting these values into the vectors in equation (10.8) the momentum equation for the x-direction reduces to $(g/2)[\partial(\eta^2-2\eta z_b)/\partial x] = -g\eta(\partial z_b/\partial x)$. Under the wet-bed conditions $h \neq 0$, the momentum equation for the x-direction can be further simplified to $-g\eta(\partial z_b/\partial x) = -g\eta(\partial z_b/\partial x)$, which is balanced; meaning the initial steady state will be unconditionally conserved. As a result of this, there will be no necessity to apply specialised numerical techniques to the source terms, provided that the term for bed slope ($\partial z_b/\partial x$) are altered to the discrete form of $(z_{bi+1/2,j} - z_{bi-1/2,j})/\Delta x$, which is the case in the current numerical model. Δx is the grid size in the x-direction, and $z_{bi+1/2,j}$ and $z_{bi-1/2,j}$ are the bed elevations at the right and left interfaces of the grid cell respectively. This technique can also be applied to the momentum equation in the y-direction.

As mentioned previously, the 2D shallow water equations are solved using a finite volume Godunov-type numerical scheme incorporated with an HLLC approximate Riemann solver. In the current application, the 1D and 2D modelling components are dynamically coupled so that the 1D model is used to represent the unsteady flow in the channel and the 2D model is for predicting flood flows in the RAFs and floodplains. The dynamic coupling is achieved by mass exchanges (evaluated by weir equations) through extra source terms in the 1D and 2D continuity equations.

10.3 Model input

LiDAR data was obtained to provide topographic information for the wider modelling domain. This covered the entire catchment up to, and including, Belford Village. The LiDAR data was merged with the RTK GPS data from the surveyed ponds (Chapter 7 – 7.3) to be used as topographic input for the hydraulic analyses of the RAF network (see Figure 10.2).

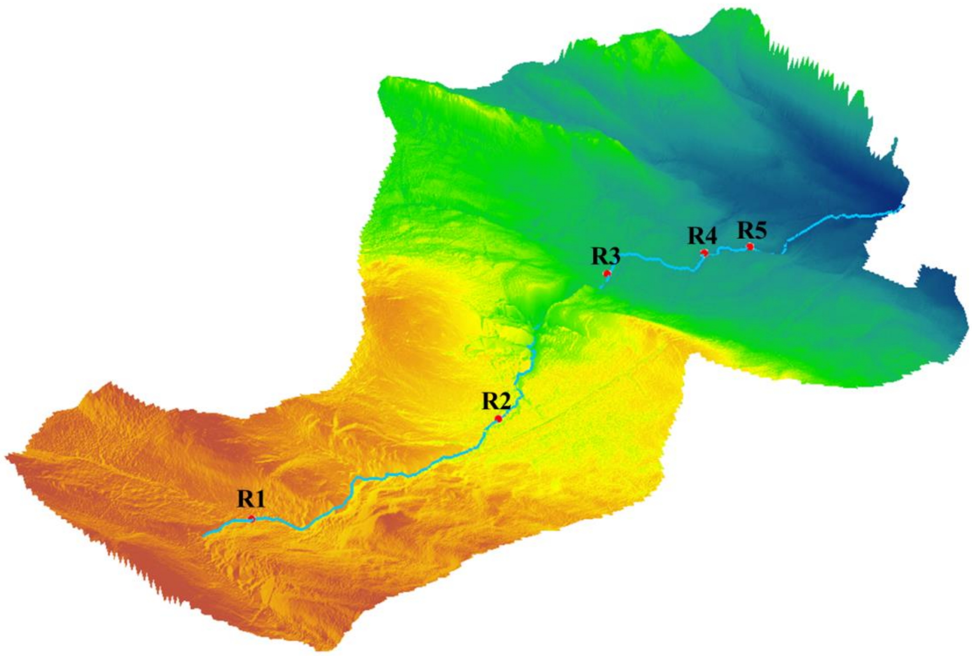


Figure 10.2: LiDAR data for Belford (shown in perspective)

In addition to the topographical data, NewChan also requires numerous river cross sections for identifying the location and dimensions of the 1D river channel. The 'River' file describes the locations of river cross sections along the study reach based on their surveyed easting and northing. It also gives a surface elevation of both the centre of the river and the riverbank at that cross section, specifies the width of the river and gives the value of Manning's n at that point. A 'set-up' file specifies the parameters for the simulation (i.e. time step, the domain for the simulation, location of boundaries, length of the simulation, and the time period for output files). The 'DEM' and 'friction' (which shows the distribution of Manning's n over the 2D domain) files dictate the layout of the domain, in terms of z-elevation and roughness, respectively. The 'inflow' file contains the discharge (in m^3s^{-1}) at the boundary of the DEM for a particular storm event, which flows in a direction specified by the user (north, south, east or west).

10.4 Configuring the hydraulic model to Belford

10.4.1 River cross-sections

The River file guides the 1D river-flow through the 2D domain, therefore the more cross-sections that exist in the model, the more accurately the 1D/2D couple will be represented. Originally, 17 cross-sections were collected between R3 and R5 (to the same order of accuracy as used in generating the rating curves – Chapter 4 – 4.5). It had been identified that it would be necessary to undertake further surveys to improve accuracy. This was due to the River file bypassing RAF inlets in initial simulations. A further 10 cross-sectional surveys were captured using the RTK GPS device and logging the location of the riverbank and the centre of the river, as well as measuring the width at each location. The cross-sections could be logged in this way due to the nature of the 1D component of NewChan. NewChan assumes a rectangular cross-section with only 2 dimensions (height and width), as well as having x, y and z coordinates associated with each cross-section. Figure 10.3 shows the comparison between 17 and 27 river cross-sections.

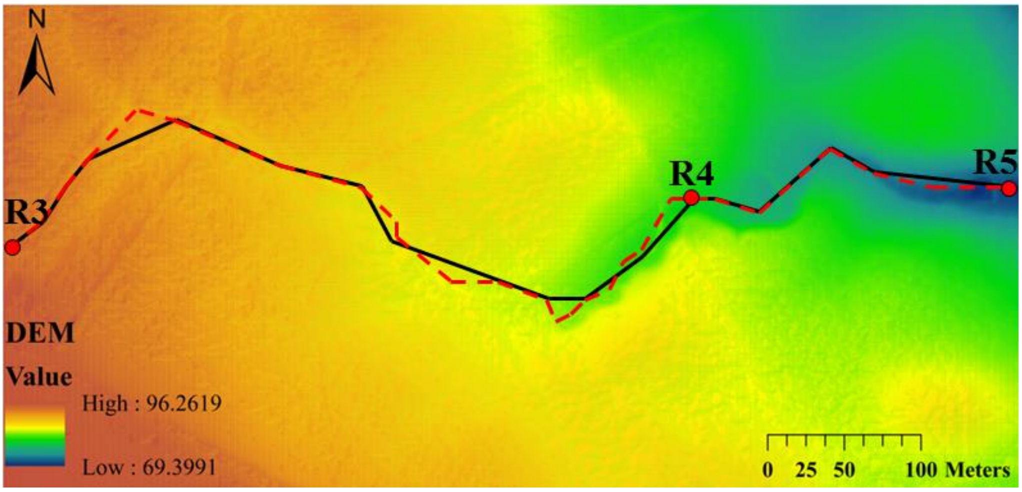


Figure 10.3: 17 cross-sections (in solid black) compared with 27 cross-sections (in dashed red)

The output from the NewChan model gives (1) ASCII files that represent inundation on the floodplain and can be viewed in ArcGIS; (2) data files containing simulated river stage and velocity at the 27 cross sections; and (3) generates stage data at specified locations, which, in this case, were the locations of the divers within the RAFs being simulated.

10.4.2 Outflow condition using DEM sensitivity

The NewChan code lacks the ability to represent outflow from a pipe within a RAF. This presented problems when trying to simulate the RAFs in the domain. To allow simulation of an outflow condition, the DEM was physically altered to 'cut' through the embankments at the location of the outlet pipe (Figure 10.4). This created a V-Notch weir through embankment, which the water in the 2D domain would be able to spill through and drain back into the river. If the outflow condition was too fast through the V-notch; the two nodes either side of the dropped (red) node could be raised to reduce the angle within the V, thus limiting flow through the gap (see Figure 10.4). The manipulation of the DEM has created a reasonable representation of the outflow from the RAFs during storm events.

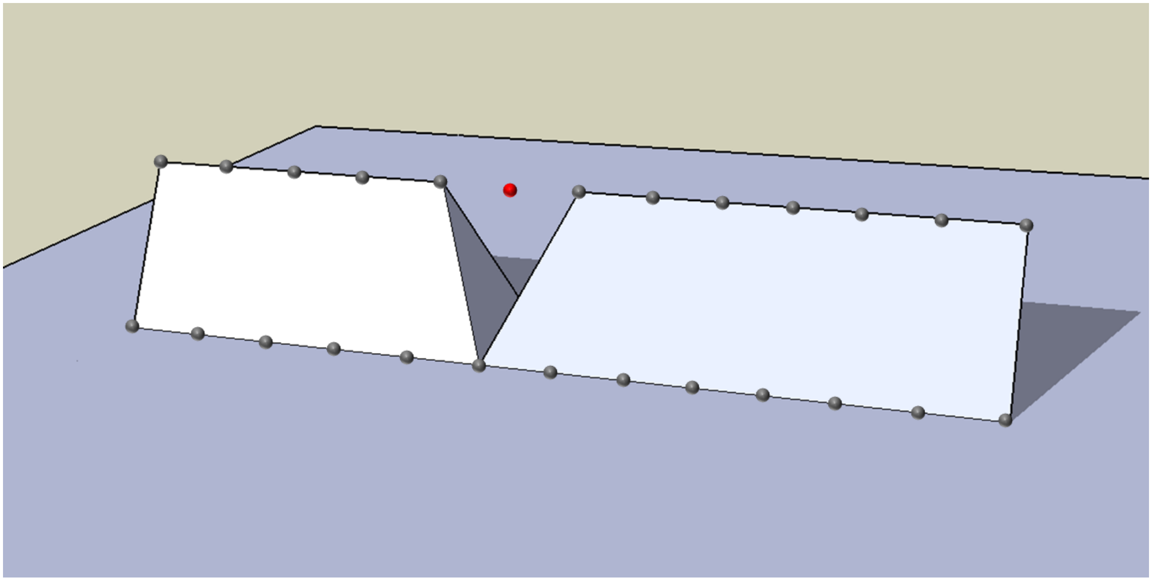


Figure 10.4: Idealisation of DEM treatment. Red node dropped to ground level to construct V-Notch through embankment

10.4.3 Model validation

Validation attempts to simulate RAF function within the complex Belford DEM proved very difficult. The complexities of the DEM and fine resolution that was being simulated meant that changes to any aspect of the DEM (with regard to z-elevation and the variation of Manning's n) was extremely time consuming, both prior-to and during simulation. Nonetheless, some of the modelled output was useful and can be compared to the observed data and the Pond Forensics Tool. Sample ASCII data from the simulation, of the July 2009 storm event, is shown in the Figures 10.5-10.7.

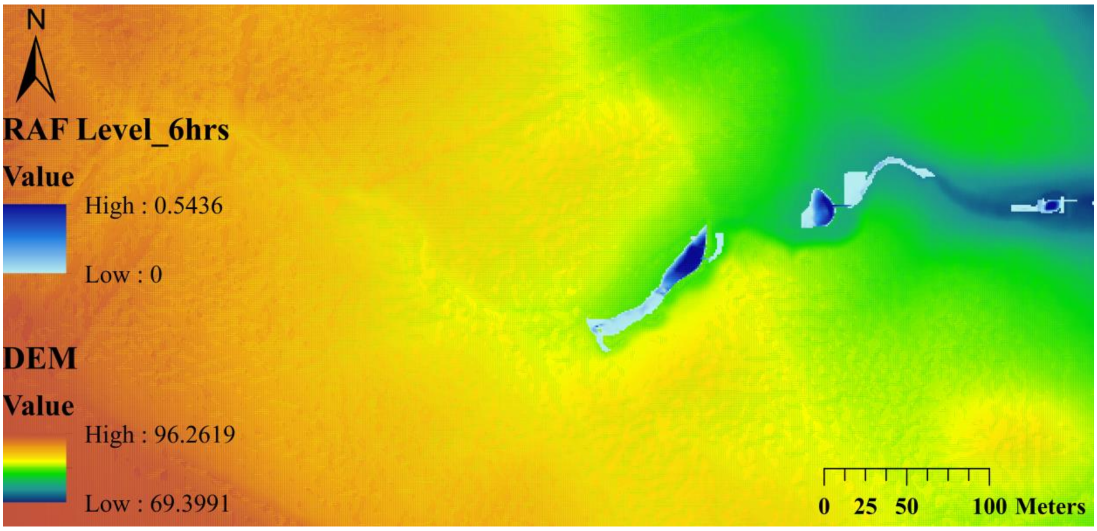


Figure 10.5: NewChan output after 6hrs of July 2009 storm event

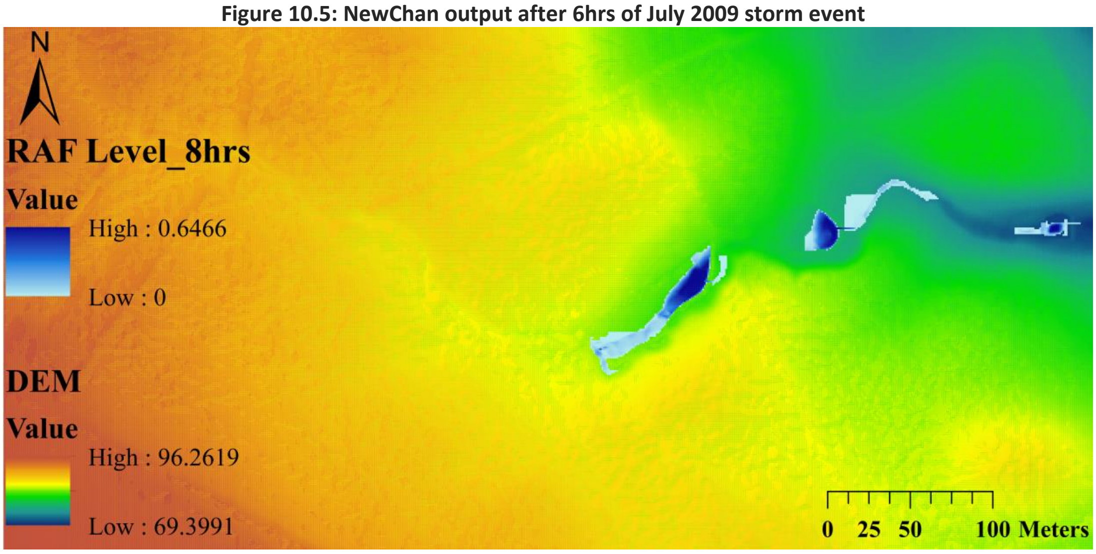


Figure 10.6: NewChan output after 8hrs of July 2009 storm event

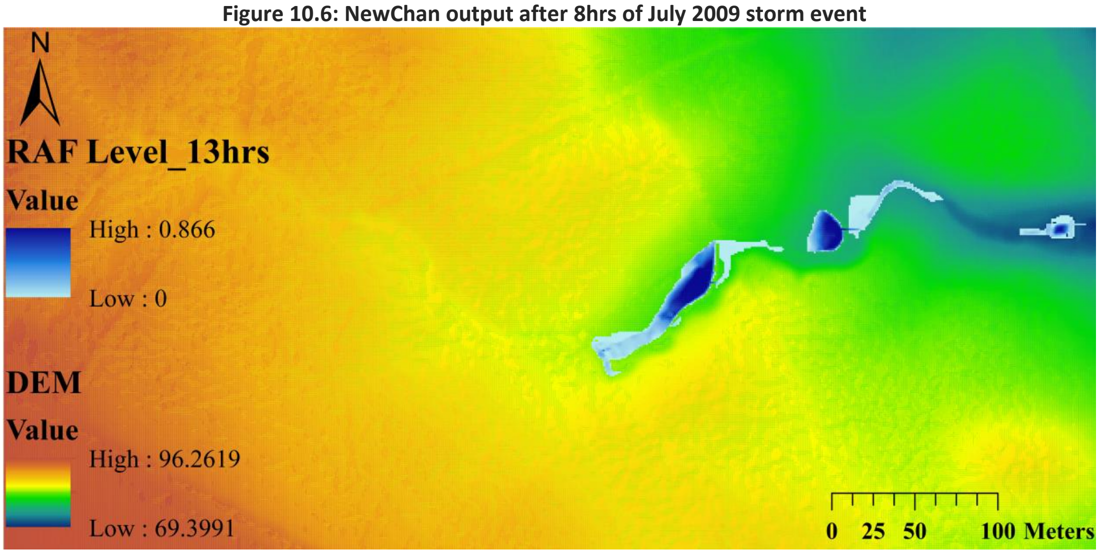


Figure 10.7: NewChan output after 13hrs of July 2009 storm event

10.4.4 Comparing models and observations

The ASCII results are a useful tool for demonstrating the function of RAFs to third-parties, especially where people are unconvinced of their effectiveness during storm events. The results from the simulation can also be viewed in graphical output for the purpose of comparing to the observed data and other simulations. The comparison of the NewChan output with observed data and simulations from the Pond Model, for the July 2009 storm event, is shown in Figure 10.8 and Figure 10.9. (Results from the November 2009 and March 2010 storm events can be found in Appendix A.2).

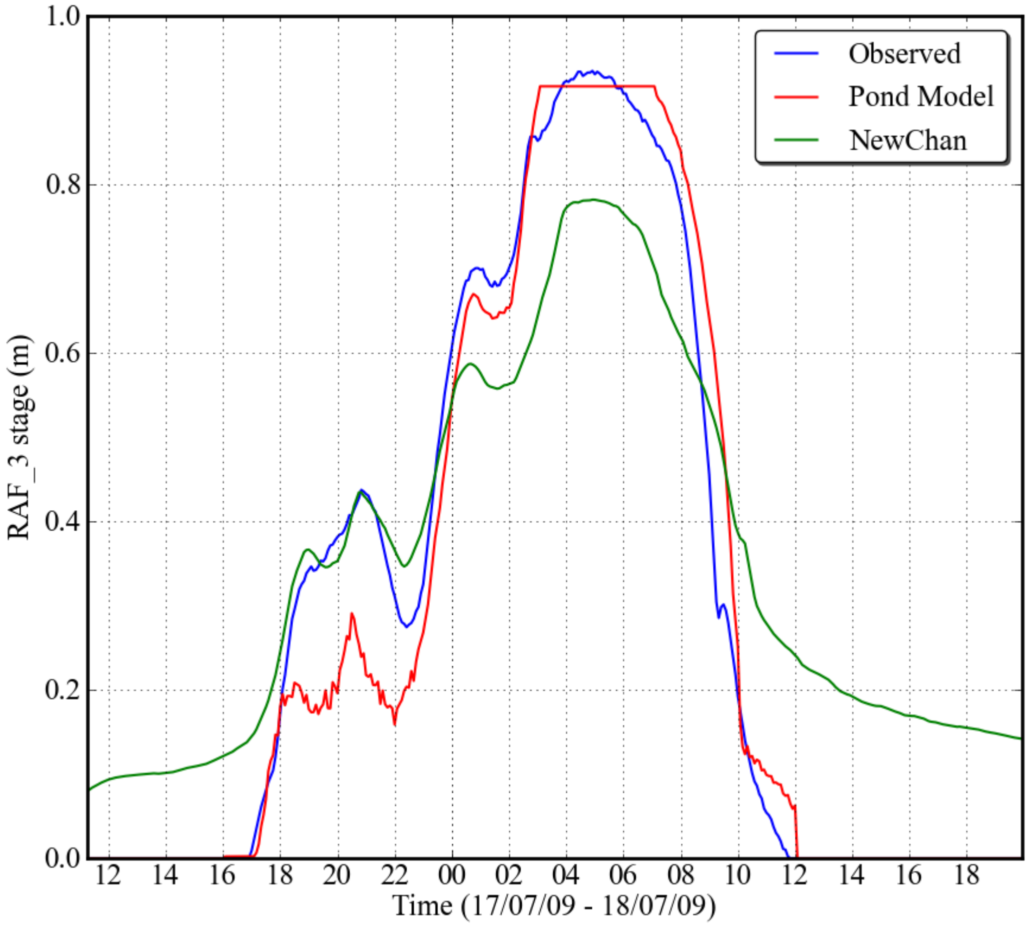


Figure 10.8: Comparison between models and observed data for RAF-3

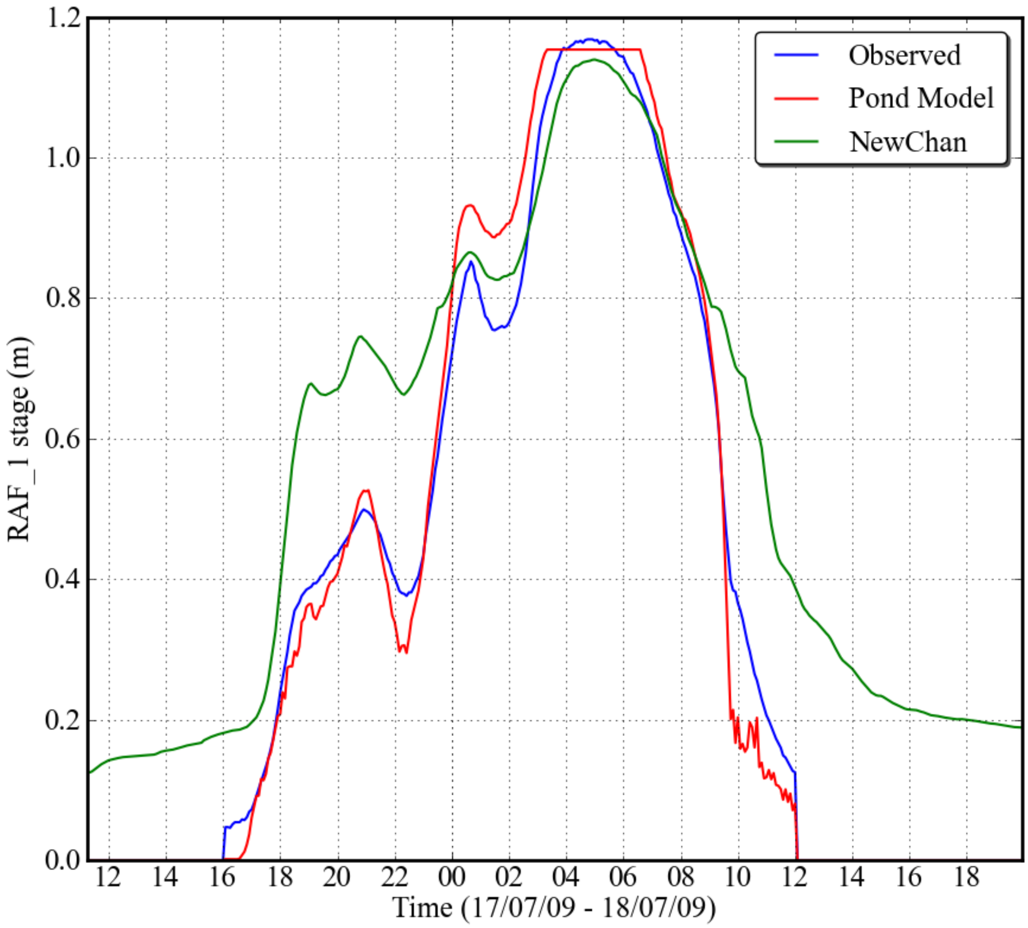


Figure 10.9: Comparison between models and observed data for RAF-1

It can be seen that the RAFs in NewChan fill at a faster rate at low river levels when compared to the observed data. This may be attributed to a lack of calibration achieving the correct river stage using the inflow data during simulations. The 1D component of NewChan is attempting to simulate the river levels downstream of the inflow input. It is doing this, in a simplified way, using rectangular channel cross-sections, which lack a lot of the physical detail in the actual river. It is likely that this change in the physical representation of the channel, between the simulation and reality, is forcing the early filling of the features. Despite the visible ponding in the RAFs observed at low river flows, however, the shape of the RAF stage during simulation is close to both the observed data and the simulations from the Pond Model. These similarities can be seen in the other simulated storm events in Appendix A.2. The comparisons have provided further evidence that the Belford RAFs are functioning during storm events and reinforces the capability of the Pond Model.

10.5 Virtual experiments

Despite the simple representation of outflow of the Belford RAFs, the NewChan hydraulic model has been effective in accurately representing the response of RAFs during storm events (see Figure 10.8 and Figure 10.9). As a result of this, it was decided to attempt a series of simulations on a simplified DEM to show the impact that different combinations of RAFs can have upon stream discharge adjacent to a small settlement; comparing the output with identical scenarios in the Pond Model. The benefits with using a simplified DEM mean that edits to the DEM can be done very quickly (in Excel) and the simulations, themselves, require less run-time.

10.5.1 Experimental configuration

The simple DEM contains a straight river of 500 m. The DEM can be described as an open book on a decline starting at the western extent – with the river flowing through the centre of the two pages towards the east (i.e. the Northern and Southern extents slope downwards towards the river in the centre) (see Figure 10.10).

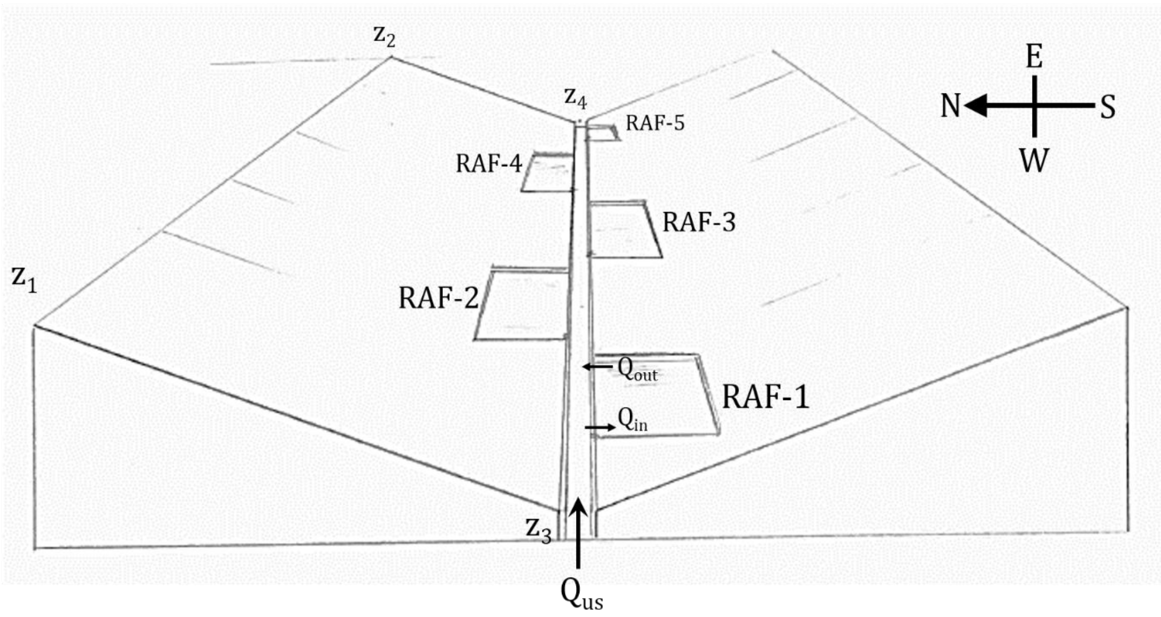


Figure 10.10: Virtual experiment layout (Elevation, $z_1 > z_2 > z_3 > z_4$)

The Raster output from NewChan enables the extent of the floodwater to be viewed; with the colour-ramp (in the bottom-left corner of each figure) demonstrating the degree of the depth of flow. As the Pond Model simulates the balance of water flowing between river and RAFs; for

NewChan, it was decided to manufacture raised cells in the DEM, on the outside of RAFs in order to force the water leaving each RAF back into the river channel without being stored on the floodplain (Figure 10.11); thus creating a more direct comparison between models. Figure 10.12 shows the NewChan output for a full RAF during a simulation.

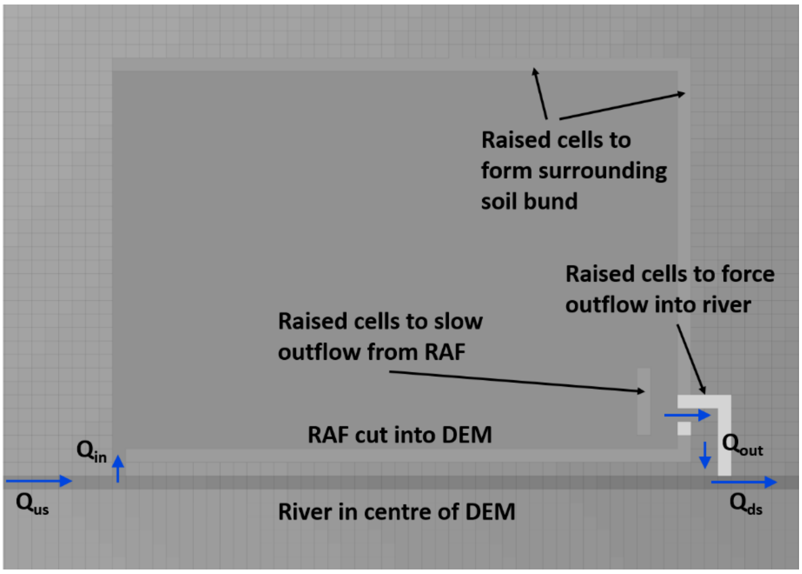


Figure 10.11: Raster idealisation of RAF set-up in NewChan

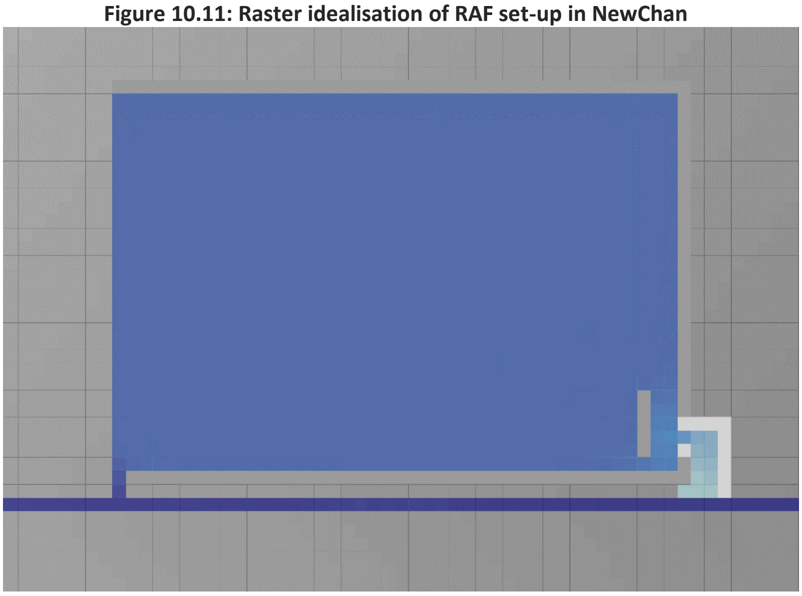


Figure 10.12: NewChan output showing full RAF during a simulation

10.5.2 Model comparisons

The virtual experiment was set-up to directly compare the hydraulic model NewChan with the Pond Network Model. A simple scenario was chosen for this analysis looking purely at impacts on river discharge. In NewChan, a 500 m reach of a simple river channel was simulated during the July 2009 storm event, which was increased by a factor of 1.75 (in order to induce out of bank flow). The storm event was simulated using river discharge alone (i.e. no rainfall was simulated over the 2D domain). The river channel was simulated both with and without the presence of RAFs (to account for channel conveyance over the short reach). In the RAF case (see Figure 10.13), the inlet heights for all the RAFs was set as 0.5 m and the potential volume of each of the RAFs was 235 m^3 (meaning a total storage of approximately 1200 m^3). The pre- and post-change hydrographs for NewChan were then compared alongside Pond Network Model output for exactly the same configuration (Figure 10.14).

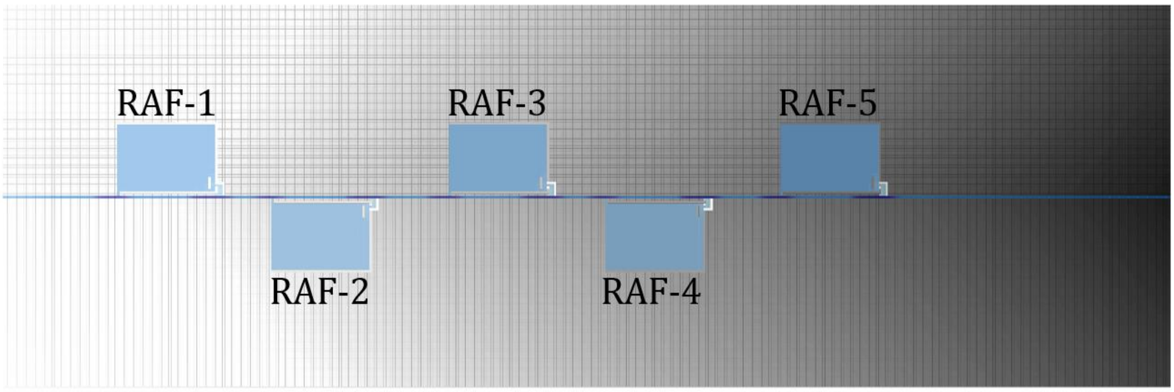


Figure 10.13: NewChan output for 5 RAFs during July 2009 storm event (x1.75)

Assessment of the output from both NewChan and the Pond Network Model reveals a greater impact upon downstream discharge in the NewChan simulation (see Figure 10.14). This is, most likely, related to the representation of channel and floodplain interactions due to the presence of RAFs in the 2D domain of NewChan (note the darker blue colour of the river channel in Figure 10.13 present at the inlet and outlet of the RAFs is a result of increasing channel friction and turbulence in the water). NewChan can represent the movement of water inside the RAFs, whereas the Pond Model completely lacks this 2D representation and simply accounts for the physical storage created by a particular RAF. For this reason, the respective representation of downstream impact differs slightly between the two model simulations. It has been noted earlier in the chapter, however, that the Pond Model is better at representing the function of individual RAFs with a more accurate outflow equation. NewChan currently relies on a modified DEM in order to generate an outflow condition. The Pond Network Model shows an instantaneous redistribution of water within the system, representing an augmented recession (a shift of water from one part of the hydrograph to another). NewChan also demonstrates an augmented recession with a reduced river discharge as a result of slowing the water at various places along the channel's reach. The output from NewChan, however, suggests that the physical interactions occurring at the inlets and outlet of the RAFs may have a greater impact upon downstream discharge than initially expected (Figure 10.14).

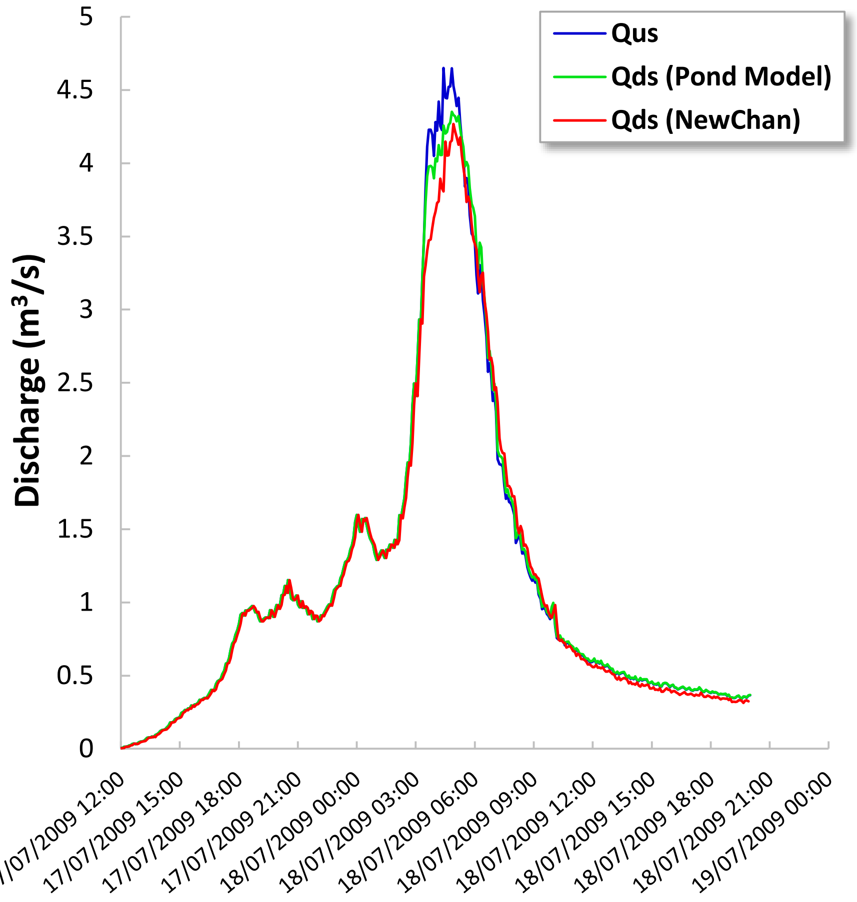


Figure 10.14: Pond Model and NewChan output showing pre- and post-change hydrographs at downstream point (5 RAFs)

The NewChan experiment was then extended to a 900 m reach with 10 RAFs and compared with the Pond Network Model using the same inflow data for the simulation (see Figure 10.15). This was done to test the impact of increasing the number of RAFs in the NewChan hydraulic model and to observe how it compared to the output from the Pond Network Model.

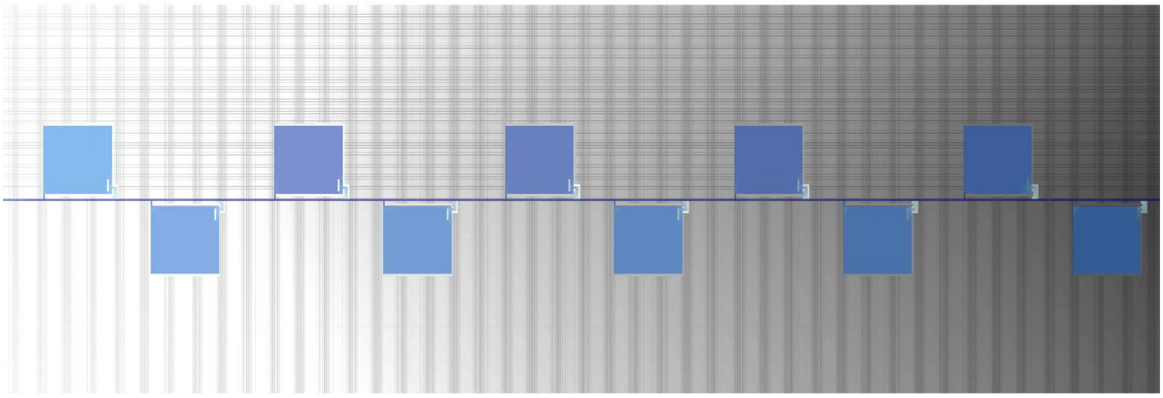


Figure 10.15: NewChan output for 10 RAFs during July 2009 storm event (x1.75)

The output of the two models reveals that, once again, the NewChan hydraulic model indicates a slightly greater impact than the Pond Network Model, which can be attributed to the 2D representation of both the channel and area within the RAFs (Figure 10.16). Output from NewChan also demonstrates a greater impact to the rising-limb of the hydrograph; shortening the duration of the peak discharge magnitude.

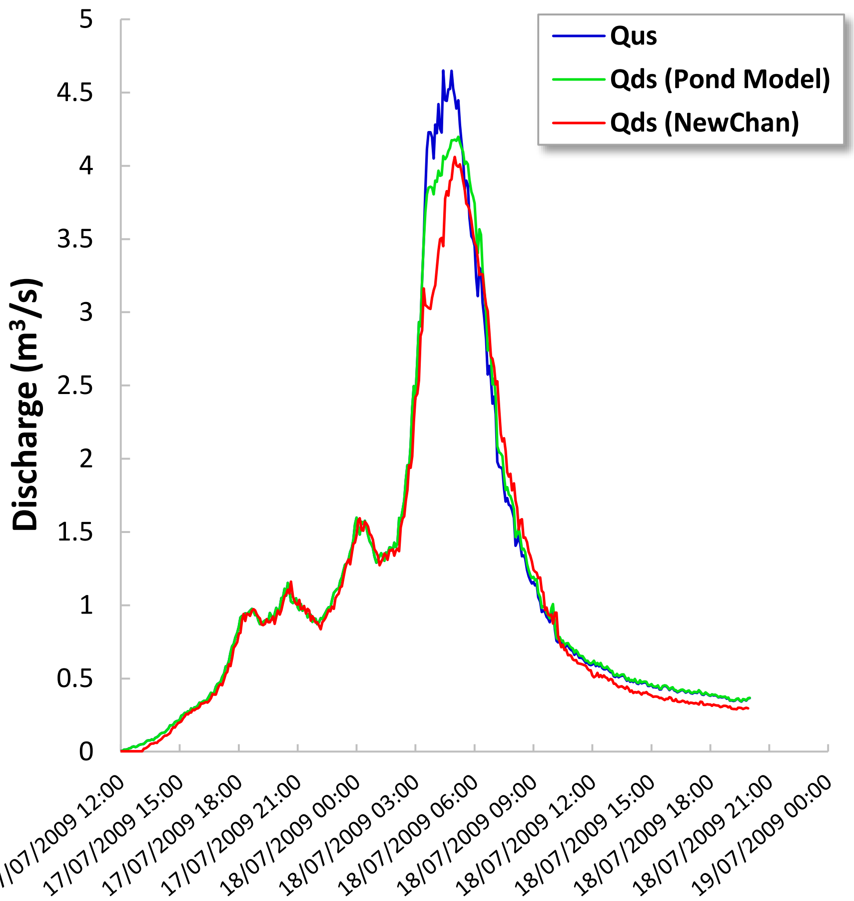


Figure 10.16: Pond Model and NewChan output showing pre- and post-change hydrographs at downstream point (10 RAFs)

The comparison between the virtual experiments using NewChan, and the representation of networks of RAFs using the Pond Model has demonstrated further evidence to the effectiveness of RAFs. It has also demonstrated the transferability of the approach in the hydraulic domain of the NewChan model.

10.6 Hydrological modelling approach (TOPCAT)

It has been demonstrated that a network of RAFs can be simulated using the hydraulic model, NewChan. Preparing simulations in hydraulic models is a time-consuming process, not only in terms of collating and formatting the input data, but also the run-time of the simulations themselves. In addition, obtaining detailed topological data and river cross-sections is financially costly and time-consuming. For these reasons, and with a view of improving the transferability of the effects of the Pond Model, it was decided to simulate the response of the Belford Burn using the simple lumped conceptual rainfall runoff model, TOPCAT (Quinn, et al., 2008). TOPCAT uses built-in storage dynamics and wetness indexes to alter (in this case, slow) the catchment response, thus representing RAFs acting in a network.

10.6.1 Background to TOPCAT

Quinn et al. (2008) present the use of a minimum information requirement (MIR) model for simulating flow and nutrient transport from agricultural systems. An MIR model is defined as the simplest, meta-model structure satisfying the modelling needs of the decision maker, whilst ensuring that the model parameters retain physical significance (Quinn, et al., 1999; Quinn, 2004). The basic assumptions of TOPMODEL (Quinn & Beven, 1993) were used to simulate subsurface hillslope flow. Simple flow attenuation assumptions were made to route and mix the flow for any catchment size (Quinn, 2004).

TOPCAT uses a soil moisture store and a subsurface flow equation from TOPMODEL (Quinn & Beven, 1993; Beven, et al., 1995), but avoids a topographic distribution function and, thus, does not allow the representation of topographically controlled variable-source areas. To combat this, TOPCAT contains a baseflow/dry-weather flow component and an overland flow component (Quinn, et al., 2008).

The TOPCAT hydrological model (Figure 10.17) uses a simple moisture root-zone store to receive inputs of observed rainfall and potential evaporation per unit time (hourly or daily) (Quinn, et al., 2008). The excess percolating flow is referred to as hydrologically effective rainfall (*HER*).

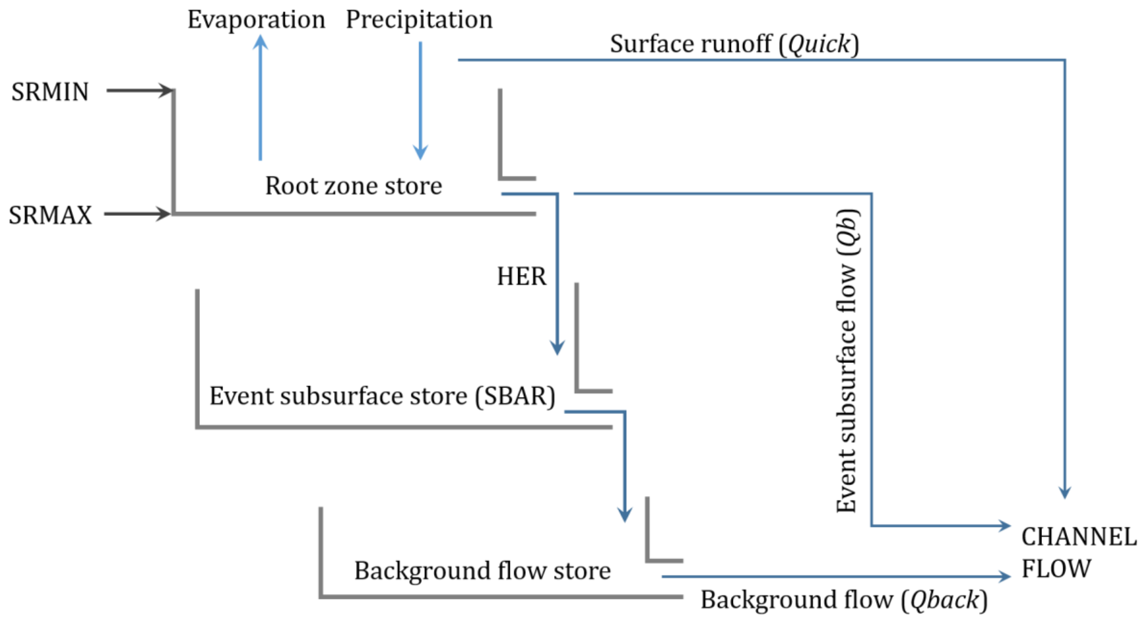


Figure 10.17: Schematic of TOPCAT model

The rate of subsurface flow leaving the event subsurface store is approximated by an exponential function taken directly from TOPMODEL (Beven & Kirkby, 1979; Quinn & Beven, 1993; Beven, et al., 1995). The current moisture status in the event subsurface store is described as *SBAR*, which is expressed as a positive soil moisture deficit value. The rate at which moisture is lost per unit time is given by:

$$Q_b(t) = exp^{(SBAR(t)/m)}$$ **Equation 10.9**

where $Q_b(t)$ is the event subsurface flow and *m* is the recession rate parameter. The recession rate parameter can either be approximated by studying recession rates in observed storm events or obtained directly from the calibration. The rate of change of storage in the catchment is primarily controlled by flow from the event subsurface store. It is assumed that the storage exhibits exponential decay, which is controlled by *m*.

Quick flow in TOPCAT is assumed to be predominantly overland flow, but it may include any very fast flow response associated with a surface flow source. As such, quick flow is always assumed to reach the channel within one time step. The *Quick* parameter determines the fraction of rainfall in one time step that is converted directly into quick surface runoff. Quick surface runoff is generated when the root zone reaches field capacity. This component of quick flow approximates large overland flow, 'wash-off,' events that are commonly observed in intense arable systems, particularly in winter. If possible, the *Quick* parameter should be calibrated to fit the sharp peaks of runoff observed in winter drainage periods. In practice, the *Quick* parameter usually lies between 0.05 and 0.3 (Quinn, et al., 2008). Hence, infiltration excess runoff is not represented explicitly, but rather as a likelihood of large winter runoff across complete fields. The component of quick flow is defined as:

$$ROQuick(t) = R(t) \times Quick$$

Equation 10.10

where *ROQuick*(*t*) is the quick flow surface runoff and *R*(*t*) is the rainfall during the time step *t*. The total discharge from the catchment is the sum of all the flow components generated within one time step:

$$Q(t) = Q_b(t) + ROQuick(t) + Qback$$

Equation 10.11

where *Q*(*t*) is the total stream flow at time step *t*. It is the mixing of these flow components that allows a sensible representation of the nutrient losses to be made at the catchment scale (Quinn, et al., 2008).

10.6.2 Calibration

The model was calibrated to simulate the discharge at a point in the River using rainfall as an input parameter. The input data spanned over an 8-month period to include the July 2009, November 2009 and March 2010 rainfall events, though the model was calibrated to best represent the winter season – due to the March 2010 event being the highest magnitude on record. Figure 10.18 shows the calibration of TOPCAT using the March 2010 storm event (the other events are included in Appendix A.3). Here, rainfall is routed through the model without the transfer function (i.e. not representing additional storage).

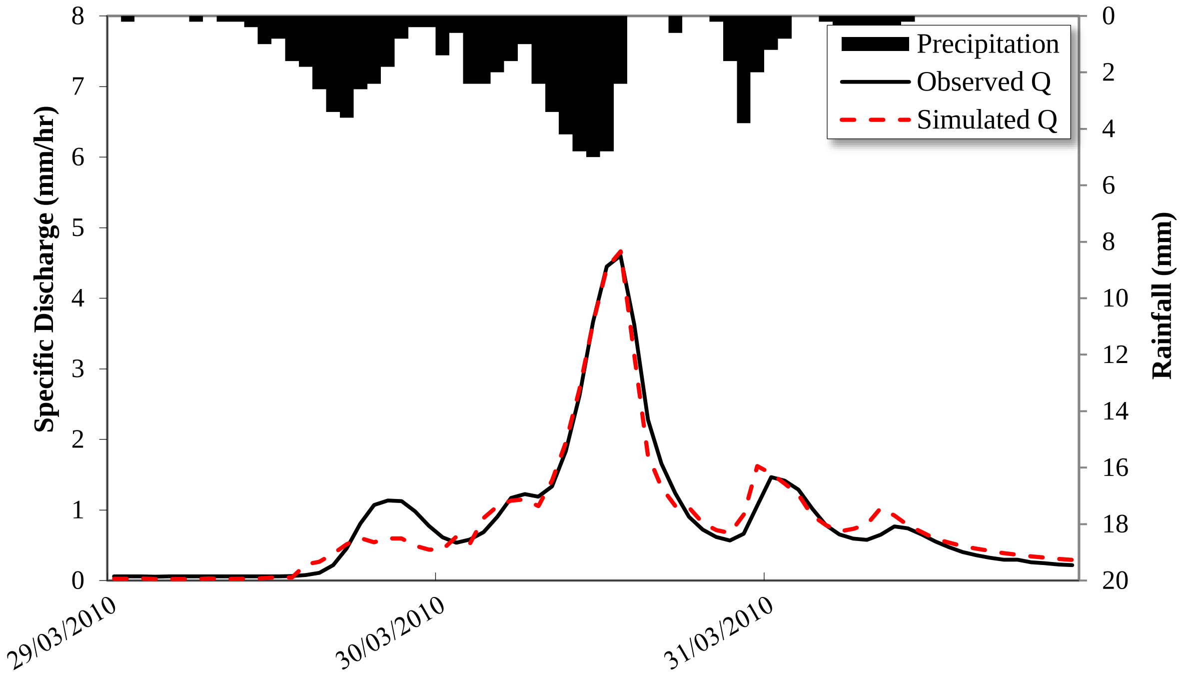


Figure 10.18: TOPCAT calibration to R4 River gauge during March 2010 storm

10.6.3 Emulating the Pond Network Model

It is possible to modify the hydrological parameters in the model so that simulation emulates the output of the Pond Network Model (see Chapter 9 – 9.6). In order to achieve this, a unit hydrograph (UH) approach has been applied, which assumes that 100% of runoff reaches the outfall in 1-timestep (see blue precipitation and discharge in Figure 10.19). A linear transfer function is then applied based on the delay produced by 35 RAFs in the Pond Model (2-3 hours). The precipitation is divided by 3 and evenly distributed over 3-timesteps, which alters the shape of the hydrograph at the outfall (see red precipitation and discharge in Figure 10.19).

To represent the UH approach in TOPCAT, the recession rate parameter of the model is modified to slow the flow rate and secondly, a channel routing function simulates the observed time delay. In essence a simple catchment runoff hydrological model can reflect the increase of storage/attenuation. However, this assumes the RAF network analyses are a fair representation of the catchment scale impact.

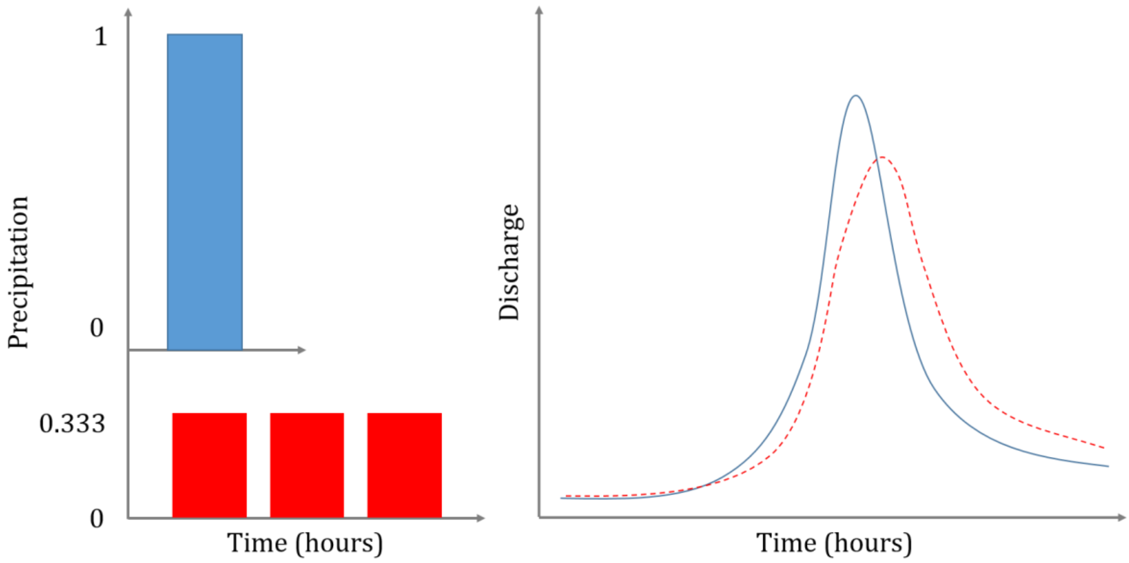


Figure 10.19: Example linear transfer function used in TOPCAT for Belford

The goodness of fit of the hydrological model to the observed flow and a range of RAF impacts can be assessed. Figure 10.20 shows the calibrated model and the observed data and the simulations for the RAF implementation, i.e. emulating the Pond Network Model results.

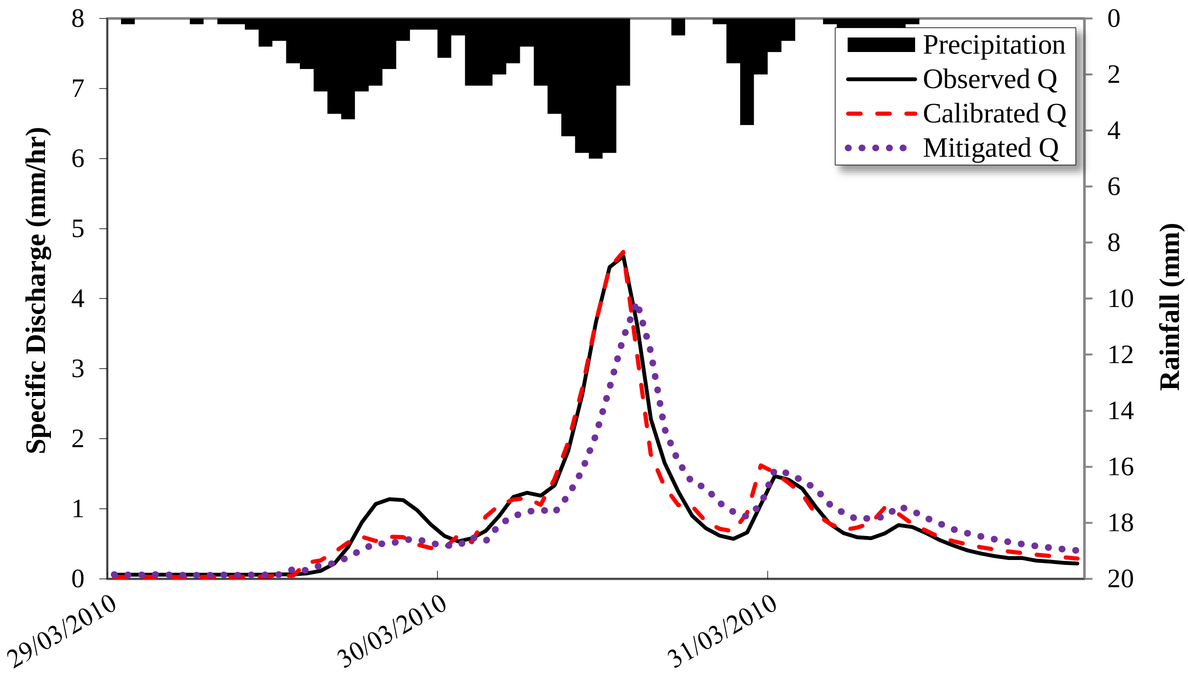


Figure 10.20: TOPCAT simulation – emulating the results from the Pond Model (m = 8.6, SRMAX = 3)

Based on the outcomes of the above simulations (and others shown in Appendix A.3), it is concluded that simple hydrological models can emulate the RAF network impact. The UH approach is, essentially, a MIR version of the Pond Model (for representing networks of RAFs). Tools such as TOPCAT have the potential to be used as a tool for small catchment scale impact studies (<10km^2), although it must be recognised that there is uncertainty in predictions. The model can be viewed as a tool with which to learn about the catchment and the hydrological effects of the interventions; as more field data become available the model can be further tested and refined. TOPCAT can be run for any observed event or design storm or for any similar catchment. The model in principal can suggest the impact of having more or less RAFs constructed (by increasing the storage function, m). It can point toward some basic assessment of cost to achieve a desired peak flow reductions, whilst allowing for uncertainty in the method (Quinn, et al., 2013).

Clearly this hydrological modelling is at an early phase and will need many storms and numerous catchment studies in order to improve the method. There is need to correlate the type, location and density of the RAFs with the likely impact on the overall runoff regime. Independent testing of flood event data on more instrumented RAFs is needed. More work is needed to reconcile hydraulic simulations and hydrological approximations. A more detailed version of a hydrological model may then be possible, which allows multiple RAF objects to be added to real catchments and the impact downstream to be quickly tested.

10.7 Summary

This Chapter has described the application of the hydraulic model, NewChan, and the simple lumped conceptual rainfall runoff model, TOPCAT, for emulating the results of the Pond Model simulations for Belford. The purpose of this has been to generate a set of transferrable tools for simulations on similar catchments. NewChan has demonstrated that precise designs of RAFs can be embedded into a DEM for a particular catchment and, using a recorded or design storm hydrograph, an estimate of downstream reduction in river level and discharge can be produced. TOPCAT has shown that without detailed topological information, but the requirement for precipitation and flow data, an estimate in the reduction in peak discharge can be demonstrated in catchments of a similar size to Belford (<10 km^2). The UH technique shown in the TOPCAT methodology would, however, require a certain degree of sensitivity analysis prior to application on other sites (based on catchment response to precipitation).

Difficulties were found during the calibration stage of NewChan's application on the Belford catchment due mainly to sensitivities found in the DEM. These problems could potentially be solved with increased survey data for the river and the application of a full 2D hydraulic model. This would improve the representation of the river channel at the inlets, but would require greater computational run-time.

11. Conclusions and recommendations

11.1 Research summary

This study has focussed on the application of a Natural Flood Management (NFM) scheme to reduce the risk of flooding in the village of Belford, Northumberland. The research has presented the data-driven and modelling-based approaches, which have demonstrated the benefits of the application of a NFM scheme (in the form of RAFs). Bespoke models using surveyed measurements have been compared to observations as well as hydraulic and hydrological models to indicate a level of transferability of techniques to similar catchment studies.

"For communities, such as Belford, this sort of approach is going to give real solutions, real benefits; at a fraction of the cost" Phil Welton, EA (From Belford official video).

Following all the chapters of this research project, the question remains: ***Does the RAF approach work?***

Although a substantial amount of qualitative and quantitative evidence has been gathered, there is no simple recipe for the construction of a RAF. This will depend on local factors, including land owner/farmer preferences, local terrain and available budget. This study has found that a range of features can be used, depending on location within the catchment, and contributing upstream area. Offline RAFs typically occupy an area of 500m^2, with an average cost of £3K. RAFs are designed to target fast runoff pathways or peak river flow, and should fill and empty in a timely fashion (<10-hours). Whilst each RAF installed within a catchment performs a small amount of runoff storage/attenuation, it is the collective network of RAFs that aim to provide downstream flood mitigation. Results from this study have demonstrated that a significant network of RAFs are required to significantly impact large flood events in Belford. The estimated cost of such a scheme is between £70-100K. For a hypothetical network providing 19,250m^3 of storage, the peak flow reduction is estimated between 15-30%. The effectiveness of RAFs, in terms of reduction in peak flow, is related to the return period of the event. It is therefore recommended that additional RAFs be constructed, if costs allow, in order to reduce flood hazard further. It is concluded that the RAF approach, as implemented in Belford, is suited to similar small rapid response catchments.

"At a time when local government is going to be strapped for cash, for many years to come; this is, quite honestly, a breakthrough. It's cheap, it's effective and it works!" Cllr Geoff O'Connell (From Belford official video).

11.2 Conclusions

The main aim of this PhD project has been to quantify the mitigation effects of low-cost on-farm flood storage (in the form of RAFs), and; to assess the viability of a network of these features at the on-farm and small-catchment scale. This has been done using analytical techniques, of hydrometric recordings, and through modelling techniques both adapted and created for the study to demonstrate transferability of the RAF approach to similar catchments. The conclusions related to the research objectives presented in Chapter 1 are outlined below.

- **Literature Review:** The literature review described the current level of knowledge regarding NFM techniques in field and modelling studies. Natural flood management (NFM) encompasses a range of methods in which to modify hydrological processes at a range of scales. The key function of NFM is to link hydrological processes and take an holistic approach to their management. Under this wider description RAFs have been classified as being a spatially diffuse approach that work close to the source of runoff generation. The question arose: How can these benefits be simulated and measured? This identified a clear gap in the understanding of NFM techniques on growing catchment scale. Previous studies had used monitoring for the plot-scale or modelling for the larger-scale, but had not attempted both, and; very few studies had looked at a combination of mitigation methods. Given these challenges and gaps in knowledge, this study aimed to quantify the effects of the non-traditional flood defence scheme being installed in the Belford catchment using a data-driven approach.

- **Catchment Description:** A desk study; drawing from previous studies in the Belford catchment was undertaken. Aware that Halcrow reached their conclusions based on widely accepted forms of analysis and modelling techniques (FEH); it was decided to adopt a more novel, analytical approach, which would be evidence-based.

- **Catchment Data:** A multi-scale, nested hydrometric network was installed in the Belford catchment (monitoring data for a five-year period) with further gauges added to monitor the growing network of RAFs. The purpose of this was, originally, to detect changes in catchment response as a result of mitigation in the form of RAFs. Unfortunately, as this was primarily a flood alleviation scheme, there was a need to progress with the installation of RAFs before enough baseline data could be gathered; meaning there was no pre-mitigation data for use in analysis.

- **Results and hydrological data analysis:** The multi-scale nested hydrometric network was used to assess the hydrology of the catchment and determine the types of storm that cause flooding in Belford. It has been identified that the Belford catchment is susceptible to flooding from short duration, intense periods of rainfall that occur within multiday events. The fast catchment response, achieved when soil capacity is exceeded, highlighted that fast runoff sources and areas of high hydrological connectivity must be mitigated on land or in the river network to reduce peak discharge at the catchment outlet. The solution, therefore, potentially lies targeted management of runoff and its sources throughout the catchment.

- **Analysing Belford Storms:** The analysis of storm hydrographs from different gauging stations has indicated that the Belford catchment has a greater response to rainfall at the top of the catchment, where the soil is shallow and the slopes are steep. Over long-lasting events or following periods of prolonged, low magnitude rainfall the soil capacity is exceeded and the catchment shows a more uniform response to rainfall at all of its gauging locations. Observations gathered by the monitoring network demonstrated the misleading nature of the FEH approach for Belford – specifically in terms of return period.

- **Mitigation methods in Belford:** The function of Runoff Attenuation Features (RAFs) has been described, and the rationale behind the use of RAFs within the Belford catchment has been introduced. The rationale identifies that there is the potential to drastically reduce peak discharge in the river and overland by targeting runoff sources through effective timing of mitigation and suggests that if all rainfall/runoff, that lies above a specified threshold, can be temporarily stored; flood hazard can be reduced.

- **Pond forensic analysis:** RAFs, located in the Belford catchment, have been monitored using pressure transducers (measuring on a five-minute interval) as part of the wider catchment monitoring regime. Surveys using GPS devices have allowed the generation of stage-volume look-up tables, which have played a key role in the analysis of RAFs during storm events. An analytical technique was developed – utilizing the observed data from within the RAFs and from nearby river gauging stations – to demonstrate the impact of individual RAFs upon downstream discharge. This method has been able to detect percentage decreases (>5%) in discharge downstream of RAFs during observed short duration, low-medium magnitude events (for offline diversion ponds). An evolved methodology was also able to infer the impact of overland flow interception RAFs during a storm event – demonstrating a 50% decrease in discharge magnitude in the form of local surface runoff.

- **The Pond Model:** The analytical method allowed the development of the bespoke modelling tool, the Pond Model, which was used to simulate changes to existing RAFs, the impacts of new RAFs and to identify how many RAFs (storage) would be required to mitigate against the highest magnitude historical events and design storms at Belford village. It was validated using observed data from historical storm events and comparisons to the analytical method described above. Simulations of RAF-0 during the large September 2008 storm identify that the RAF can reduce peak flow in the river, adjacent, by 15% (see Appendix A1). It has been shown that a network of 35 RAFs (550 m^3 capacity, with an inlet height of 0.45 m) could potentially reduce discharge at the catchment outlet by between 18% (for the 1:100 year winter design storm) and 30% for the high magnitude historical events observed during the monitoring period. The pond model was also used to estimate the impact of a network of overland flow interception RAFs over a 1 km^2 experimental reach. It suggested that 4 RAFs, in sequence, could decrease peak overland flow by approximately 40% at the end of the 1 km^2 reach, and that strategic placement of these features throughout the catchment had the potential to significantly change runoff response at the catchment outlet.

- **Hydraulic and Hydrological modelling and experimentation:** RAF networks have been simulated using both hydraulic and hydrological models to demonstrate transferability to other catchments and other studies. The NewChan hydraulic model was used to simulate RAFs 1-3 in the Belford catchment, and showed a closeness of fit to both the observed levels in the RAFs and the Pond Model predictions. This approach proved difficult, due to the nature of editing Belford's DEM. It was decided to create a simple DEM in order to provide an independent comparison to the results from the Pond Network model. Finally, the Pond Model has been emulated using the simple lumped conceptual rainfall runoff model, TOPCAT. Instead of topographic data, this model used observed rainfall and runoff alongside potential evaporation to calibrate the existing runoff regime with the model parameters. A unit hydrograph technique was then applied to represent physical storage and a change in time-to-peak as a result of mitigation within the catchment. TOPCAT has shown that an estimate in the reduction in peak discharge can be demonstrated in catchments of a similar size to Belford (<10 km^2).

11.3 Recommendations and future work

There are several ways in which the work presented in this study can be taken forward:

- Continued long-term monitoring of RAFs of varying types. This should be continued in the Belford catchment, but also initiated in others – preferably considering a range of spatial scales. Evidence from this project has already begun informing stakeholders as to the effectiveness of certain RAFs, but further analysis could help identify the best forms of mitigation within a catchment.
- Further work should be pursued regarding simulation and testing of networks of RAFs in the hydraulic environment. The RAF surveys (obtained during this project) should be used as a starting point for the development of a full catchment-scale hydraulic model. This will require a lot of work and additional surveys to ensure the domain is as close as possible to the actual topography of the Belford catchment. The domain could be formatted to represent the Belford RAF network at several stages throughout the project to experiment with the addition of RAFs over the course of the study – as well as demonstrating a full before and after mitigation comparison. Data from this study would be incredibly valuable to planners thinking of adopting similar methods in other catchments.
- There is a desire to actively manage catchments to provide a host of ecosystem services. This is driven by recent legislation, including the Water Framework Directive and the Nitrates Directive. The Defra funded Demonstration Test Catchment (DTC, http://www.demonstratingcatchmentmanagement.net/) initiative is exploring ways of reducing diffuse pollution at the catchment scale, while ensuring the delivery of sustainable food production. The approach taken by DTC has similarities with the philosophy underpinning the use of RAFs for flooding. Hence there is a potential to harmonise and co-fund interventions aimed at reducing flood risk and improving water

quality. This has, to-date, not been exploited. Closer cooperation and funding of ecosystems services could encourage wider uptake of the RAF approach.

Bibliography

Abbott, M. B. (1999). Introducing Hydroinformatics. *Journal of Hydroinformatics*, 3-19.

Alias, N. A., Liang, Q., & Kesserwani, G. (2011). A Godunov-type scheme for modelling 1D channel flow with varying width and topography. *Computers and Fluids, 46*, 88-93.

Allen, R. G., Pereira, L. S., Raes, D., & Smith, M. (1998). Crop evapotranspiration-guidelines for computing crop water requirements. *Rome, Italy, FAO - Food and Agriculture Organization of the United Nations*.

Ambroise, B. (1988). Interactions eaux souterraines-eaux de surface dans le bassin du Ringelbach Soultzeren (Haute Vosges, France): Role hydrologique des surfaces saturees. In P. Dahblom, & G. Lindh, *Proceedings of the International Symposium on Interaction Between Groundwater and Surface Water* (pp. May 30 - June 3). Ystad, Sweden.

Amoros, C., & Bornette, G. (2002). Connectivity and biocomplexity in waterbodies of riverine floodplains. *Freshwater Biology, 47*, 761-776.

Anderson, M. P., & Woessner, W. W. (1992). *Applied Groundwater Modeling: Simulation of Flow and Advection.* San Diego: Academic Press, Inc.

Ball, T. (2008). Management approaches to floodplain restoration and stakeholder engagement in the UK: a survey. *Ecohydrology and Hydrobiology, 8*(2-4), 273-280.

Band, L. (1986). Topographic partition of watersheds with digital elevation models. *Water Resources Research, 22*(1), 15-24.

Barber, N. J. (2013). *Sediment, nutrient and runoff management in rural catchments.* PhD Thesis, School of Civil Engineering and Geosciences, Newcastle University.

Barber, N. J., & Quinn, P. F. (2012). Mitigating diffuse water pollution from agriculture using soft-engineered runoff attenuation features. *Area, 44*(4), 454-462.

Bates, P. D., & De Roo, A. P. (2000). "A simple raster-based model for flood inundation simulation". *Journal of Hydrology, 236*, 54-77.

Bates, P. D., & De Roo, A. P. (2000). "A simple raster-based model for flood inundation simulation". *Journal of Hydrology, 236*, 54-77.

Bates, P. D., Anderson, M. G., Baird, L., Walling, D. E., & Simm, D. (1992). "Modelling floodplain flow with a two-dimensional finite element scheme". *Earth Surface Processes and Landforms, 17*, 575-588.

Bates, P. D., Stewart, M. D., Siggers, G. B., Smith, C. N., Hervouet, J. M., & Sellin, R. H. (1998). "Internal and external validation of a two-dimensional finite element code for river flood simulation". *Proc. Instn Civ Engrs Wat., Marit. & Energy. 130*, pp. 127-141. Bristol: ICE.

Beven, K. (1979). On the generalized kinematic routing model. *Water Resources Research, 15*, 1238-1242.

Beven, K. J., & Kirkby, M. J. (1979). A physically-based, variable contributing area model of basin hydrology. *Hydrological Sciences Research, 24*, 43-69.

Beven, K., Gilman, K., & Newson, M. (1979). Flow and flow routing in upland channel networks. *Hydrological Sciences*, 24 (3), 369-379.

Beven, K., Lamb, R., Quinn, P., Romanowicz, R., & Freer, J. (1995). TOPMODEL. In V. P. Singh (Ed.), *Computer Models of Watershed Hydrology* (pp. 627-668). Highlands Ranch, CO: Water Resources Publications.

Bloschl, G. (2001). Scaling in hydrology. *Hydrological Processes, 15*, 709-711.

Bloschl, G., & Sivapalan, M. (1995). Scale issues in hydrological modelling: A review. *Hydrological Processes, 9*, 251-290.

Boardman, J. (1995, July). Damage to property by runoff from agricultural land, South Downs, southern England, 1976-93. *The Geographical Journal, 161*(2), 177-191.

Boardman, J., Ligneau, L., de Roo, A., & Vandaele, K. (1994). Flooding of property by runoff from agricultural land in northwestern Europe. *Geomorphology, 10*, 183-196.

Bradford, S. F., & Sanders, B. F. (2002). "Finite-Volume Model for Shallow-Water Flooding of Arbitrary Topography". *Journal of Hydraulic Engineering, 128*(3), 289-298.

Brath, A., Montanari, A., & Moretti, G. (2006). Assessing the effect on flood frequency of land use change via hydrological simulation (with uncertainty). *J. Hydrol.*(324), 141-153.

Bronstert, A., Niehoff, D., & Burger, G. (2002). Effects of climate and land-use change on storm runoff generation: present knowledge and modelling capabilities. *Hydrological Processes, 16*, 509-529.

Bruneau, P., Gascuel-Odoux, C., Robin, P., Merot, P., & Beven, K. J. (1995). Sensitivity to space and time resolution of a hydrological model using digital elevation data. *Hydrological Processes, 9*, 69-81.

Burt, T. P., & Heathwaite, A. L. (1996). The hydrology of the Slapton catchments. *Field Studies*(8), 543-557.

Causon, D. M., Mingham, C. G., & Ingram, D. M. (1999). "Advances in Calculation Methods for Supercritical Flow in Spillway Chanells". *Journal of Hydraulic Engineering, 125*(10), 308-316.

Chow, V. T. (1959). *Open Channel Hydraulics* (International Edition ed.). Singapore: McGraw-Hill Book Company.

Collier, C. G. (2007). Flash flood forecasting: What are the limits of predictability? *Quarterly Journal of the Royal Meteorological Society, 133*, 3-23.

Connell, J. H. (1978). Diversity in tropical rainforests and coral reefs. *Science*, 199, 1302-1310.

CRUE. (2008). *CRUE Research Report No. 1-6: Efficiency of non-structural flood mitigation measures: "room for the river" and "retaining water in the landscape".* Prepared by the Joint Project Consortium, 2008: Available at http://www.crue-eranet.net (accessed 20 October 2010).

Cunge, J. A., Holly Jr, F. M., & Verwey, A. (1976). *"Practical Aspects of Computational River Hydraulics".* London: Pitman.

Dale, M. (2005). Impact of climate change on UK flooding and future predictions. *Institution of Civil Engineers. Proceedings of Water Management, 158*, 135-140.

Dankers, R., & Feyen, L. (2008). Climate change impact on flood hazard in Europe: An assessment based on high resolution climate simulations. *Journal of Geophysical Research, 113*, 1-17.

Dawson, R. J., Ball, T., Werritty, J., Werritty, A., Hall, J., & Roche, N. (2011). Assessing the effectiveness of non-structural flood management measures in the Thames Estuary under conditions of socio-economic and environmental change. *Global Environmental Change, 21*, 628-646.

de Roo, A. P., Gouweleeuw, B., Thielen, J., Bartholmes, J., Bongioannini-Cerlini, P., Todini, E., . . . Sprokkereef, E. (2003). Development of a European flood forecasting system. *Intl. J. River Basin Management, 1*(1), 49-59.

Defra. (2005). *Making space for water, taking forward a new government strategy for flood and coastal risk management in England, First response.*

Defra. (2008a). *The Government's Response to Sir Michael Pitt's Review of the 2007 Floods.* London: Defra Publications.

Defra. (2008b). *Future Water: The government's water strategy for England.*

Defra/EA. (2005). *Revitalisation of the FSR/FEH rainfall runoff method.* Department for Environment, Food and Rural Affairs.

Dooge, J. C. (1973). *Linear theory of hydrologic systems.* Tech. Bull. 1468, Agric. Res. Service, Agric., US Dept., Washington, 327pp.

Dudley, S. J., Fischenich, J. C., & Abt, S. R. (1998, October). Effect of woody debris entrapment on flow resistance. *Journal of the American Water Resources Association, 34*(5), 1189-1197.

Environment Agency. (2007). *Review of the 2007 summer floods.* Bristol: Environment Agency.

Environment Agency. (2009). *Flooding in England: A National Assessment of Flood Risk.* Bristol: Environment Agency.

Environment Agency. (2010). *Working with Natural Processes to Manage Flood and Coastal Erosion Risk.* Bristol: Environment Agency.

Environment Agency. (2011). *Guidance on maintaining the rapid response catchment register and creating action plans, Operational Guidance Note 1131_08.*

Evans, E. P., Ashley, R. M., Hall, J., Penning-Roswell, E., Sayers, P., Thorne, C., & Watkinson, W. (2004). *Foresight; Future Flooding. Scietific Summary.* Volume II: Managing future risks. London: Office of Science and Technology.

Evans, R., & Boardman, J. (2003). Curtailment of muddy floods in the Sompting catchments, South Downs, West Sussex, southern England. *Soil use and management, 19*(3), 223-231.

Evrard, O., Vandaele, K., van Wesemael, B., & Bielders, C. L. (2008). A grassed waterway and earthen dams to control muddy floods from a cultivated catchment of the Belgian loess belt. *Geomorphology, 100*, 419-428.

Ewen, J. (2000). Moving packet model for variably saturated flow. *Water Resources Research*, 36, 2587-2594.

Ewen, J., & Birkinshaw, S. J. (2007). Lumped hysteretic model for subsurface stormflow developed using downward approach. *Hydrol. Process.*(21), 1496-1505.

Ewen, J., Geris, J., O'Donnell, G., Mayes, W., & O'Connell, E. (2010). *Multiscale Experimentation, Monitoring and Analysis of Long-Term Land Use Changes and Flood Risk - EA project SC060092SCaMP: Final Science Report.* Newcastle upon Tyne: Newcastle University in collaberation with Imperial College London and Lancaster University.

Ewen, J., O'Donnell, G., Burton, A., & O'Connell, P. (2006). Errors and uncertainty in physically-based rainfall-runoff modelling of catchment change effects. *Journal of Hydrology*(330), 641-650.

Ewen, J., Parkin, G., & O'Connell, P. E. (2000). SHETRAN: Distributed river basin flow and transport modelling system. *Journal of Hydrologic Engineering, 5*(3), 250-258.

Fread, D. L. (1984). Chapter 14. In M. G. Anderson, & T. P. Burt, *"Hydrological Forecasting".* Chichester: Wiley.

Frei, C., Scholl, R., Fukutome, S., Schmidli, J., & Vidale, P. L. (2006). Future change of precipitation extremes in Europe: Intercomparison of scenarios from regional climate models. *J. Geophys. Res., 111*, D06105.

Gee, D. M., Anderson, M. G., & Baird, L. (1990). "Large scale floodplain modelling". *Earth Surface Processes and Landforms, 15*(6), 512-523.

Gippel, C. J. (1995). Environmental Hydraulics of Large Woody Debris in Streams and Rivers. *Journal of Environmental Engineering*, 388-395.

Gledhill, J. (2007). Britania rules the waves but Canute got his feet wet: flooding risk. *Environmental Law and Management, 19*, 224-226.

Golding, B., Clark, P., & May, B. (2005). The Boscastle flood: Meteorological analysis of the conditions leading to flooding on 16 August 2004. *Weather, 60*, 230-235.

Grubb, M., Koch, M., Munson, A., Sullivan, F., & Thomson, K. (1993). *The Earth Summit Agreements: A Guide and Assessment.* London: Earthscan Publications.

Gyssels, G., Poesen, J., Nachtergaele, J., & Govers, G. (2002). The impact of sowing density of small grains on rill and ephemeral gully erosion in concentrated flow zones. *Soil & Tillage Research, 64*(3-4), 189-201.

Hager, W. H. (1987). Lateral outflow over side weirs. *J. Hydraul, Eng., 113*(4), 491-504.

Halcrow. (2007). *Environment agency North East region Belford burn, Belford: prefeasibility study.* Edinburgh: Halcrow.

Hand, H. W., Fox, N. I., & Collier, C. G. (2004). A study of twentieth-century extreme rainfall events in the United Kingdom with applications for forecasting. *Meteorological Applications, 11*, 15-31.

Heathwaite, A. L., & Burt, T. P. (1991). Predicting the effect of land use on stream water quality in the UK. In N. E. Peters, & D. E. Walling (Eds.), *Sediment and Stream Water Quality in a Changing Environment* (pp. 209-219). IAHS Wallingford, UK: IAHS Publication 203.

Heathwaite, A. L., Burt, T. P., & Trudgill, S. T. (1990). Land-use controls on sediment production in a lowland catchment, south-west England. In J. Boardman, I. D. Foster, & J. A. Dearing, *Soil Erosion on Agricultural Land.* John Wiley and Sons Ltd.

Heathwaite, A. L., Quinn, P. F., & Hewett, C. J. (2005). Modelling and managing critical source areas of diffuse pollution from agricultural land using flow connectivity simulation. *Journal of Hydrology, 304*, 446-461.

Herschy, R. W. (1995). *Streamflow Measurement* (Second Edition ed.). London: New York: E and FN Spon.

Hewitson, B. C., & Crane, R. G. (1996). Climate Downscaling: techniques and application. *Climate Res*(7), 85-95.

Hollis, G. E. (1975). The effect of urbanisation on flood of different recurrence interval. *Water Resour. Res*(11), 431-435.

Hooda, P. S., Edwards, A. C., Anderson, H. A., & Miller, A. (2000). A review of water quality concerns in livestock farming areas. *The Science of the Total Environment, 250*, 143-167.

Horrit, M. S. (2000). "Calibration of a two-dimensional finite element flood flow model using satellite radar imagery". *Water Resources Research, 17*(4), 727-746.

Horritt, M. S., & Bates, P. D. (2002). "Evaluation of 1D and 2D numerical models for predicting river flood inundation". *Journal of Hydrology, 268*, 87-99.

Hsu, C. C., Tang, C. J., Lee, W. J., & Shieh, M. Y. (2002). Subcritical 90 degree equal-width open-channel dividing flow. *J. Hydraul. Eng., 128*(7), 716-720.

Hunter, N. M., Bates, P. D., Neelz, S., Pender, G., Villanueva, I., Wright, N. G., . . . Mason, D. C. (2007). "Benchmarking 2D hydraulic models for urban flooding". *Water Management, 161*, 13-30.

IH. (1999). *Flood Estimation Handbook.* Walingford, UK: Institute of Hydrology.

Jarvis, R. A., Bendelow, V. C., Bradley, R. I., Carroll, D. M., Furness, R. R., Kilgour, I. N., & King, S. J. (1984). *Soils and their use in northern england* (Bulletin no. 10 ed.). Soil Survey of England and Wales.

Jasper, K., Gurtz, J., & Herbert, L. (2002). Advanced Flood Forecasting in Alpine Watersheds by Coupling Meteorological Observations and Forecasts with a Distributed Hydrological Model. *J. Hydrology, 267*(1-2), 40-52.

Johnson, A. K., Shrubsole, D., & Merrin, M. (1996). Integrated Catchment Management in northern Australia. *Land Use Policy, 13*(4), 303-316.

Jonkman, S. N. (2005). "Global Perspectives on Loss of Human Life Caused by Floods". *Natural Hazards, 34*, 151-175.

Kaneki, M. (2001). "Minimization of flood disasters in urban areas". *Prevention Against Natural Disasters*.

Kerr, G., & Nisbet, T. R. (1996). *The Restoration of Floodplain Woodlands in Lowland Britain: A Scoping Study and Recommendations for Research.* Bristol: Environment Agency.

Kesserwani, G., Vasquez, J., Riviere, N., Liang, Q., Travin, G., & Mose, R. (2010). New Approach for Predicting Flow Bifurcation at Right-Angled Open-Channel Junction. *Journal of Hydraulic Engineering, 136*(9), 662- 668.

Klemes, V. (1983). Conceptualisation and scale in hydrology. *Journal of Hydrology, 65*, 1-23.

Kopmann, R., & Markofsky, M. (2000). "Three-dimensional water quality modelling with TELEMAC-3D". *Hydrological Processes, 14*, 2279-2292.

Krzysztofowicz, R. (1995). Recent advances associated with flood forecast and warning systems. US National Report to IUGG, 1991-1994. *Rev. Geophys, 33*(Part 2, Suppl. S), 1139-1147.

Kundzewicz, Z. W. (1999). Flood protection - sustainability issues. *Hydrolic Sciences Journal*, 44(4), 559-571.

Lamb, R. (1996). Distributed hydrological prediction using generalised TOPMODEL concepts. *Ph. D. thesis, Lancaster University, Lancaster, UK*.

Lamb, R., & Beven, K. (1997). Using interactive recession curve analysis to specify a general catchment storage model. *Hydrology and Earth System Sciences, 1*, 101-113.

Lane, S. N. (1995). "Within-reach spatial patterns of process and channel adjustment". (E. J. Hickin, Ed.) *Journal of River Geomorphology*, 105-130.

Lane, S. N., Biron, P. A., Bradbrook, K. F., Butler, J. B., Chandler, J. H., Crowell, M. D., . . . Roy, A. G. (1998). "Integrated three-dimensional measurement of river channel topography and flow using acoustic doppler velocimetry". *Earth Surfaces Processes and Landforms, 23*, 1247-1267.

Lane, S. N., Brookes, C. J., Hardy, R. J., Holden, J., James, T. D., Kirkby, M. J., . . . Yu, D. (2003). Land management, flooding and environmental risk: New approaches to a very old question. *Proceedings of the Annual CIWEM Conference.* Harrogate.

Law, S. W., & Reynolds, A. J. (1966). Dividing flow in an open channel. *J. Hydr. Div., 92*(2), 207-231.

Leedal, D., Neal, J., Beven, K., Young, P., & Bates, P. (2010). Visualization approaches for communicating real-time flood forecasting level and inundation information. *Journal of Flood Risk Management*(3), 140-150.

Leonard, J., Mietton, M., Najib, H., & Gourbesville, P. (2000). Rating curve modelling with Manning's equation to manage instability and improve extrapolation. *Hydrological Sciences - Journal des Sciences Hydrologiques, 45*, 739-750.

Letts, J. (2012). *Rural Sustainable Drainage Systems.* Bristol, UK: Environment Agency.

Li, R.-M., Simons, D. B., & Stevens, M. A. (1975, April). Nonlinear Kinematic Wave Approximation for Water Routing. *Water Resources Research, 11*(2), 245-252.

Liang, D., Falconer, R. A., & Lin, B. (2007). Linking one- and two-dimensional models for free surface flows. *Water Management, 160*(3), 145-151.

Liang, Q. (2008). "Simulation of Shallow Flows in Nonuniform Open Channels". *Journal of Fluids Engineering, 130*(011205).

Liang, Q., Du, G., Hall, J. W., & Borthwick, A. G. (2008). "Flood Inundation Modelling with an Adaptive Quadtree Grid Shallow Water Equation Solver". *Journal of Hydraulic Engineering, 134*(11), 1603-1610.

Maitre, V., Cosandey, A. -C., Desagher, E., & Parriaux, A. (2003). Effectiveness of groundwater nitrate removal in a river riparian area: the importance of hydrological conditions. *Journal of Hydrology*, 278(1-4), 76-93.

Mander, U., Kuusemets, V., Lohmus, K., & Mauring, T. (1997). Efficiency and dimensioning of riparian buffer zones in agricultural catchments. *Ecological Engineering*, 8(4), 299-324.

Marks, K., & Bates, P. (2000). Integration of high-resolution topographic data with floodplain flow models. *Hydrolog. Process*(14), 2109-2122.

Marriot, M. J., Featherstone, R. E., & Nalluri, C. (2009). *Nalluri & Featherstone's civil engineering hydraulics: essential theory with worked examples* (Vol. 5). Chichester, West Sussex, United Kingdom: John Wiley & Sons Ltd.

Metz, B. (2001). *Climate Change 2001: Mitigation, (Contribution of Working Group III to the IPCC Third Assessment Report).* Cambridge: Cambridge University Press.

Milly, P. C., Betancourt, J., Falkenmark, M., Hirsch, R. M., Kundzewicz, Z. W., Lettenmaier, D. P., & Stouffer, R. J. (2008). Stationarity is dead: Whither water management? *Science, 319*, 573-574.

Moore, R. J. (2007). The PDM rainfall-runoff model. *Hydrology and Earth System Sciences, 11*(1), 483-499.

Morris, D. G., & Heerdegen, R. G. (1988). Automatically derived catchment boundary and channel networks and their hydrological applications. *Geomorphology, 1*, 131-141.

Nelder, J. A., & Mead, R. (1965). A simplex method for function minimization. *Computer Journal, 7*, 308-313.

Newson, M. D. (1975). *Flooding and Flood Hazard in the United Kingdom.* London: Oxford University Press.

Newson, M. D. (1997). *Land, Water and Development: River Basin Systems and their Sustainable Management.* London: Routledge.

Nicholson, A. R., Quinn, P. F., O'Donnell, G. M., Wilkinson, M. E., & Liang, Q. (2012b). Development of a modelling tool to assess the impact of storage ponds as a form of natural flood management in small catchments. Dundee: Proceedings of BHS 11th National Symposium, 2012, Hydrology for a Changing World; University of Dundee.

Nicholson, A. R., Wilkinson, M. E., O'Donnell, G. M., & Quinn, P. F. (2012a). Runoff Attenuation Features: A sustainable flood mitigation strategy in the Belford Catchment, UK. *Area, 44*(4), 463-469. Retrieved from http://onlinelibrary.wiley.com/doi/10.1111/j.1475-4762.2012.01099.x/full

Niehoff, D., Fritsch, U., & Bronstert, A. (2002). Land-use impacts on storm runoff generation: scenarios of land-use change and simulation of hydrological response in a meso-scale catchment in SW-Germany. *J. Hydrol.*(267), 80-93.

O'Connell, E., Ewen, J., O'Donnell, G., & Quinn, P. (2007). Is there a link between land-use management and flooding? *Hydrology & Earth Systems Sciences*, 96-107.

O'Connell, P. E., Beven, K. J., Carney, J. N., Clements, R. O., Ewen, J., Fowler, H., . . . Shepherd, M. (2005). *Review of Impacts of rural land use and management on flood generation.Part A: Impact study report.* London: Department of Environment, Food and Rural Affairs.

O'Donnell, G. M., Ewen, J., & O'Connell, P. E. (2011). Sensitivity maps for impacts of land management on an extreme flood in the Hodder catchment, UK. *Physics and Chemistry of the Earth, Parts A/B/C, 36*(11), 630-637.

O'Donnell, G., Ewen, J., & O'Connell, E. (2008). *Hydroinformatic information tracking for flood impact assessment.* Newcastle upon Tyne: School of Civil Engineering and Geosciences, Newcastle University.

Palmer, M. (2012). *Agricultural fine sediment: Sources, pathways and mitigation.* PhD Thesis, School of Civil Engineering and Geosciences, Newcastle University.

Palmer, T. N., & Raisanen, J. (2002). Quantifying the Risk of Extreme Seasonal Events in a Changing Climate. *Nature*(415), 512-514.

Pappenberger, F., Beven, K. J., Hunter, N. M., Bates, P. D., Gouweleeuw, B. T., Thielen, J., & de Roo, A. P. (2005). Cascading model uncertainty from medium range weather forecasts (10 days) through a rainfall-runoff model to flood inundation predictions within the European Flood Forecasting System (EFFS). *Hydrology and Earth System Sciences, 9*(4), 381-393.

Parliamentary Offices of Science and Technology. (2011, December). *Natural Flood Management.* Retrieved January 27, 2012, from Parliament: http://www.parliament.uk/briefing-papers/POST-PN-396

Parrott, A., Brooks, W., Harmar, O., & Pygott, K. (2009). Role of rural land use management in flood and coastal risk management. *Journal of Flood Risk Management, 2*(4), 272-284.

Patric, J. H., & Reinhart, K. G. (1971). Hydrologic effects of deforesting two mountain watersheds in West Virginia. *Water Resour. Res.*(7), 1182-1188.

Pitt, M. (2007). *"Learning Lessons from the 2007 Floods".* London: Cabinet Office.

Ponce, V. M. (1989). *Engineering Hydrology: Principles and Practices.* Cambridge University Press/Prentice Hall.

POST. (2011). *Natural Flood Management POSTNOTE 396.* London, UK: Parliamentary Offices of Science and Technology. Retrieved from http://www.parliament.uk/briefing-papers/POST-PN-396

Posthumus, H., Hewett, C. J., Morris, J., & Quinn, P. F. (2008). Agricultural land use and flood risk management: Engaging with stakeholders in North Yorkshire. *Agricultural Water Management, 95*, 787-798.

Press, W. H., Teukolsky, S. A., Vetterling, W. T., & Flannery, B. P. (1992). *Numerical recipes for FORTRAN - The art of scientific computing.* (Second ed.). Cambridge: Cambridge University Press.

Quinn, P. F. (2004). Scale appropriate modelling: representing cause and effect relationships in nitrate pollution at the catchment scale for the purpose of catchment scale planning. *Journal of Hydrology, 291*, 197-217.

Quinn, P. F., & Beven, K. (1993). Spatial and temporal predictions of soil-moisture dynamics, runoff, variable source areas and evapotranspiration for Plynlimon, Mid-Wales. *Hydrol. Processes, 7*(4), 425-448.

Quinn, P. F., Anthony, S., & Lord, E. (1999). Basin scale nitrate modelling using a minimum information requirement approach. In S. Trudgill, D. Walling, & B. Webb, *Water Quality: Processes and Policy* (pp. 101-117). Chichester: Wiley.

Quinn, P. F., Beven, K. J., & Lamb, R. (1997). The ln(α/tanβ) index: how to calculate it and how to use it within the TOPMODEL framework. In K. J. Beven, *Distributed Hydrological Modelling: Applications of the TOPMODEL concept* (pp. 31-52). Chichester: Wiley.

Quinn, P. F., Hewett, C. J., Jonczyk, J., & Glenis, V. (2007). *The proactive approach to Farm Integrated Runoff Management (FIRM) plans: Flood storage on farms.* Newcastle: Newcastle University.

Quinn, P. F., Hewett, C., & Dayawansa, N. (2008). TOPCAT-NP: a minimum information requirement model for simulation of flow and nutrient transport from agricultural sources. *Hydrological Processes, 22*, 2565-2580. doi:10.1002/hyp.6855

Quinn, P., Beven, K., Chevallier, P., & Planchon, O. (1991). The prediction of hillslope flow paths for distributed hydrological modelling using digital terrain models. *Hydrological Processes, 5*, 59-79.

Quinn, P., O'Donnell, G., Nicholson, A., Wilkinson, M., Owen, G., Jonczyk, J., Barber, N., Hardwick, M., & Davies, G. (2013). *Potential use of Runoff Attenuation Features in small rural catchments for flood mitigation: Evidence from Belford, Powburn and Hepscott.* Joint Newcastle University, Royal Haskoning and Environment Agency Report. Retrieved from http://research.ncl.ac.uk/proactive/belford/newcastlenfmrafreport/reportpdf/June%20NFM%20RAF%20Report.pdf

Rajaratnam, N., & Pattabiramiah, K. P. (1960). A new method to predict flow in a branch channel. *Irrig. Power.*

Ramamurthy, A. S., Qu, J., & Vo, D. (2007). Numerical and experimental study of dividing open-channel flows. *J. Hydraul. Eng.*, 1135-1144.

Robarts, R. (1998). Factors controlling primary production in hypertrophic lakes (Hartbeespoort Dam, South Africa). *Journal of Plankton Research*, 6, 91-105.

Samuels, P. G. (1990). "Cross section location in one-dimensional models". *International Conference on River Flood Hydraulics* (pp. 339-350). Chichester: Wiley.

Shaw, E. M., Beven, K. J., Chappell, N. A., & Lamb, R. (2011). *Hydrology in Practice* (Fourth ed.). Abingdon: Spon Press.

Shields, F. D., & Gippel, C. J. (1995, April). Prediction of effects of woody debris removal on flow resistance. *Journal of Hydraulic Engineering, 121*(4), 341-354.

Snyder, F. F. (1939). A concept of runoff phenomina. *Eos Trans. AGU, 20*, 725-738.

Sprokkereef, E. (2001). Extension of the Flood Forecasting Model FloRIJN. *NCR publication 12-2001, ISSN no. 1568234X*.

Suh, S. -W. (2006). A hybrid model to particle tracking Eularian-Lagrangian models in the simulation of coastal dispersion. *Environmental Modelling and Software*, 21, 234-242.

Tang, X. N., Knight, D. W., & Samuels, P. G. (2001). Variable parameter Muskingum-Cunge method for flood routing in a compound channel. *Journal of Hydraulic Research, 37*, 591-564.

Thomas, H., & Nisbet, T. R. (2007). An assessment of the impact of floodplain woodland on flood flows. *Water and Environment Journal, 21*, 114-126.

Thomas, T. G., & Williams, J. J. (1995). "Large eddy simulation of turbulent flow in an asymetric compound open-channel". *Journal of Hydraulic Research, 33*, 27-41.

Toebes, C., & Strang, D. D. (1996). On recession curves, 1. Recession equations. *J. Hydrol. New Zealand, 3*(2), 2-15.

Van der Helm, P. P. (1987). Bodemerosie en wateroverlast; een milieuprobleem in het Zuid-Limburgse heuvelland. *MSc Thesis, Dept. of Physical Geography, University of Utrecht*.

Verstraeten, G., & Poesen, J. (1999). The nature of small-scale flooding, muddy floods and retention pond sedimentation in central Belgium. *Geomorphology, 29*, 275-292.

Wheater, H. (2006). Flood hazard and management: a UK perspective. *Phil. Trans. R. Soc. Lond. A., 365*, 2135-2145.

Wheater, H., Reynolds, B., Mcintyre, N., Marshall, M., Jackson, B., Frogbrook, Z., Soloway, I., Francis, O. & Chell, J. (2008). *Impacts of upland land management on flood risk: Multi-scale modelling methodology and results from the pontbren experiment.* FRMRC Research Report UR 16.

Wilby, R. L., & Keenan, R. (2012). Adapting to flood risk under climate change. *Progress in Physical Geography, 36*(3), 348-378.

Wilkinson, M. E., & Quinn, P. F. (2010). Belford Catchment Proactive Flood Solutions: A toolkit for managing runoff in the rural landscape. *Proceedings of the SAC and SEPA Biennial Conference.* Edinburgh.

Wilkinson, M. E., Quinn, P. F., & Welton, P. (2010a). Runoff management during the September 2008 floods in the Belford catchment, Northumberland. *Journal of Flood Risk Management, 3*(4), 285-295.

Wilkinson, M. E., Quinn, P. F., Benson, I., & Welton, P. (2010b). Runoff Management: Mitigation measures for disconnecting flow pathways in the Belford Burn catchment to reduce flood risk. Newcastle upon Tyne: Proceedings of the British Hydrological Society International Symposium 2010.

Wilkinson, M., Quinn, P., Jonczyk, J., Nicholson, A., Owen, G., & Barber, N. (2011). Evaluating multipurpose soft engineered mitigation measures in the Belford Burn catchment, Northumberland, UK. Dublin: Catchment Science 2011. Retrieved November 15, 2011, from http://www.jorireland.com/events/CatchmentScience2011/documents/Invited_speaker_abstracts2.pdf

Williams, J. R., Dyke, P. T., Fuchs, W. W., Benson, V. W., Rice, O. W., & Taylor, E. D. (1990). *EPIC - Erosion/Productivity Impact Calculator: 2 User Manual.* (A. N. Sharpley, & J. R. Williams, Eds.) U.S. Department of Agriculture Technical Bulletin No. 1768.

Woods-Ballard, B., Kellagher, R., Martin, P., Jefferies, C., Bray, R., & Shaffer, P. (2007). *The SUDS manual.* London: CIRIA.

Yang, D., Koike, T., & Tanizawa, H. (2004). Application of a distributed hydrological model and weather radar observations for flood management in the upper Tone River of Japan. *Hydrological Processes, 18*, 3119-3132. doi:10.1002/hyp.5752

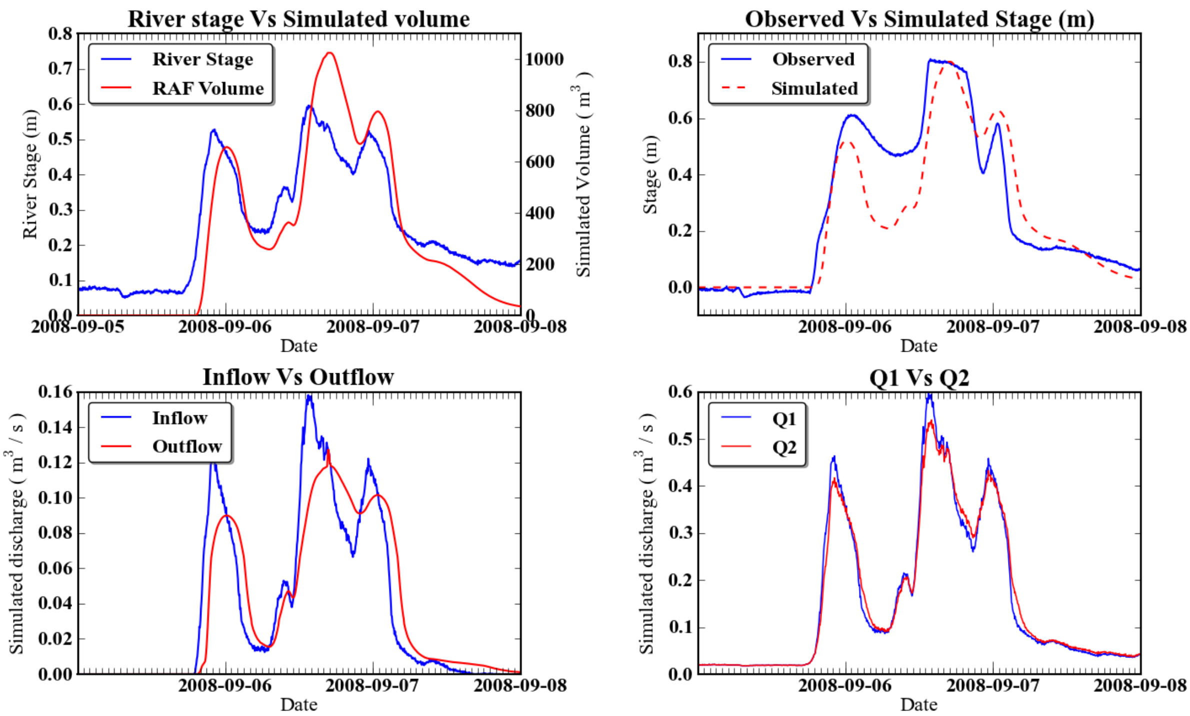
Yen, B. C. (1986). "Rainfall-runoff Process on Urban Catchments and Its Modeling". In C. Maksimovic, & M. Radojkovic, *Urban Dainage Modelling* (pp. 3-26). Dubrovik, Yugoslavia: Pergamon Press Ltd.

Younis, B. A. (1996). "Progress in turbulence modelling for open channel flows". In M. G. Anderson, D. E. Walling, & P. D. Bates, *Floodplain Processes* (pp. 299-332). Chichester: Wiley.

Zalewski, M. (2002). Ecohydrology - the use of ecological and hydrological processes for sustainable management of water resources. *Hydrological Sciences Journal*, 47(5), 823-832.

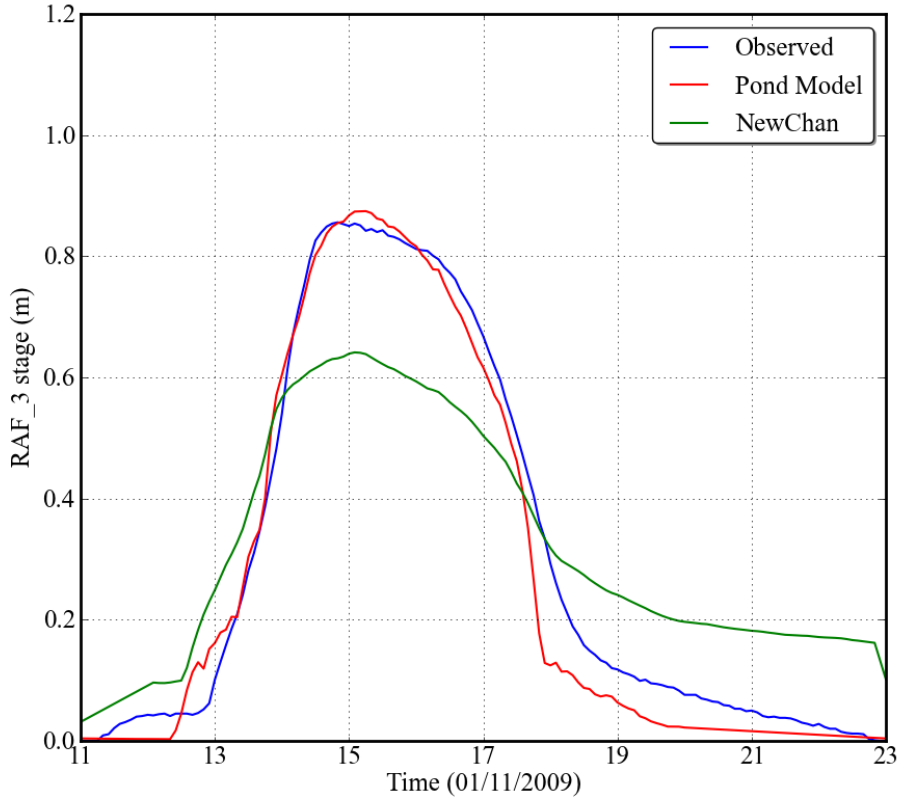
Appendix

A.1 Pond Model output

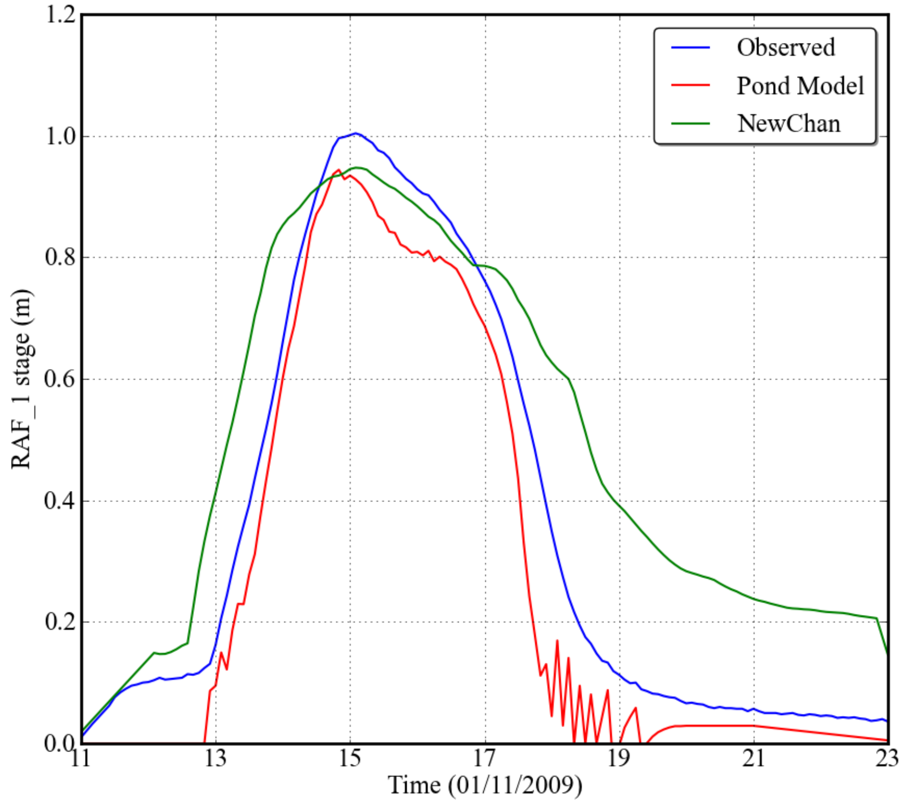


Above: Pond model indicates over 15% reduction in flow from the upstream gauging station (R1) during the September 2008 storm event as a result of RAF-0 (Pilot Pond).

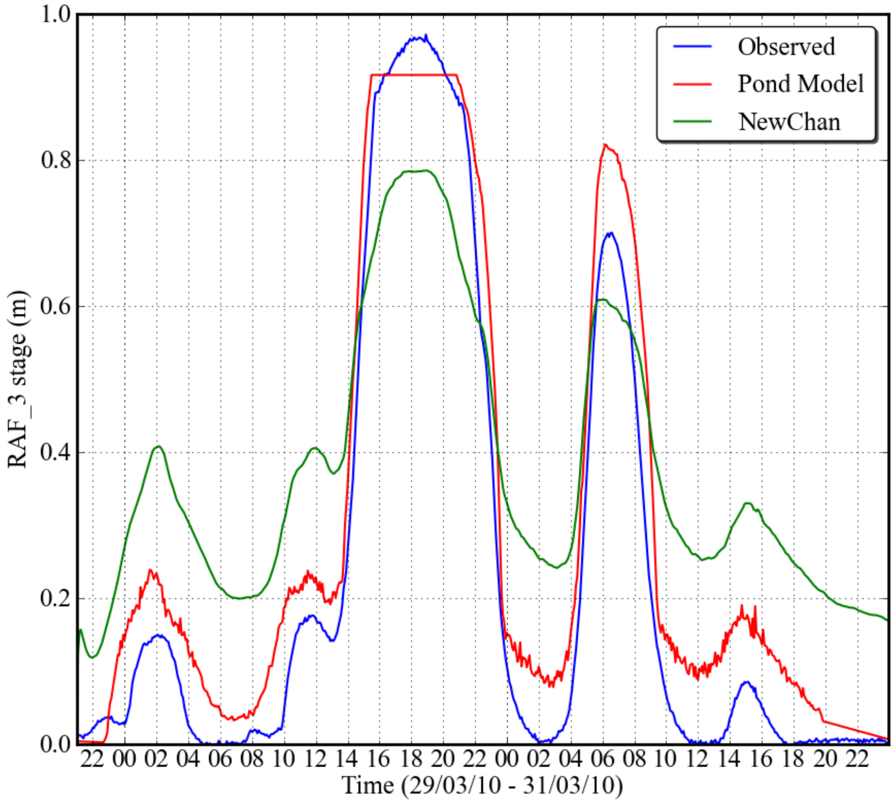
A.2 NewChan output and comparisons



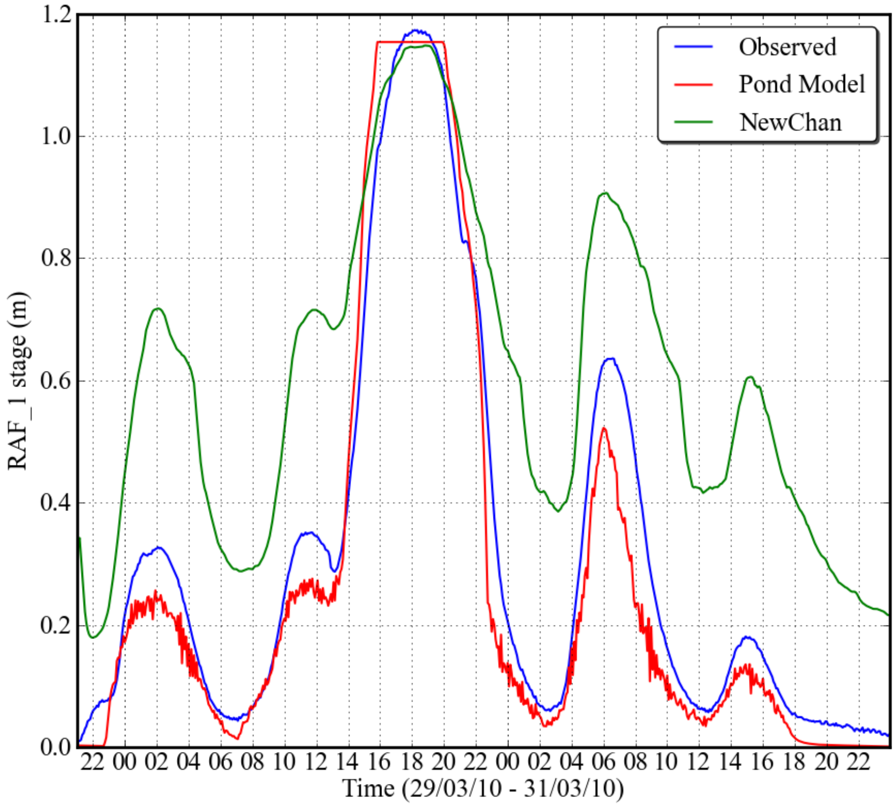
RAF-3 Comparison for November 2009 storm event



RAF-1 Comparison for November 2009 storm event

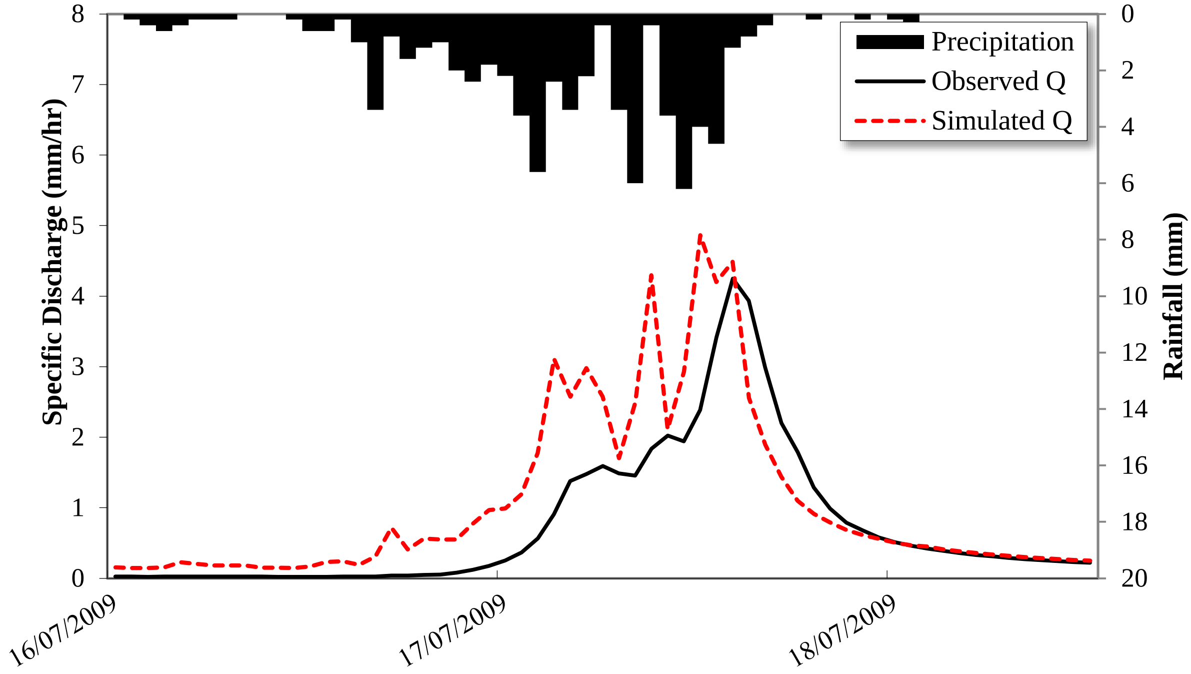


RAF-3 Comparison for March 2010 storm event

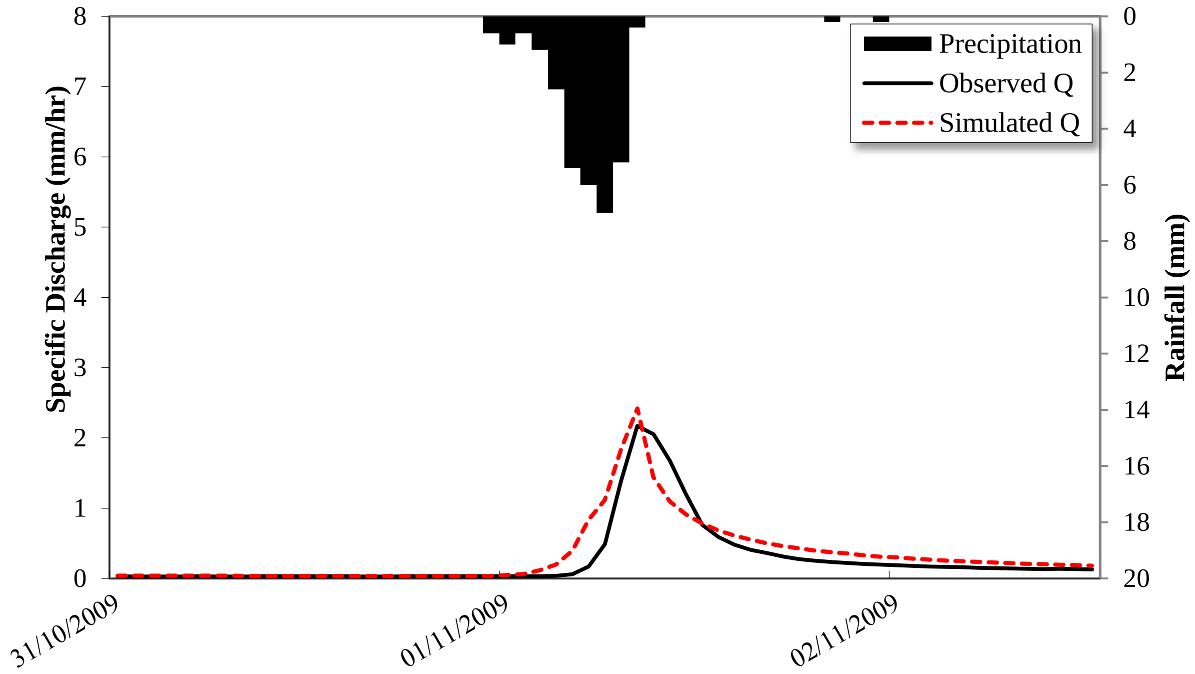


RAF-1 Comparison for March 2010 storm event

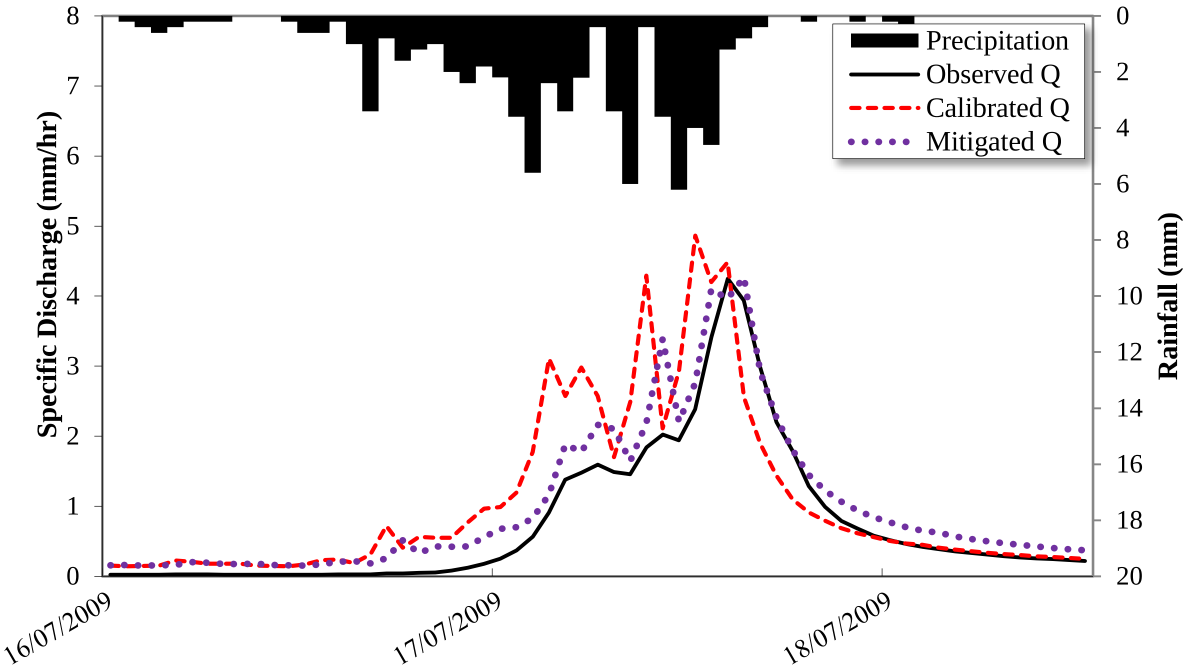
A.3 TOPCAT output



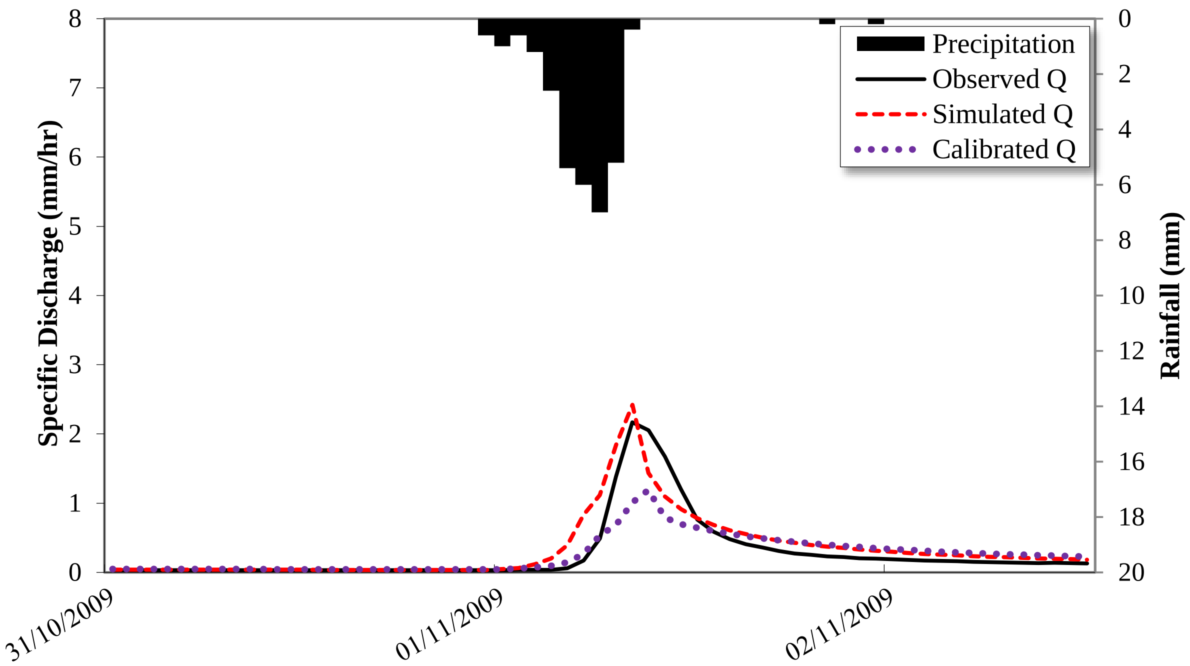
Above: TOPCAT output at R4 gauging station during July 2009 storm event. Note the faster response from the model compared to the observed data. This can be explained by the fact that the model was calibrated to best fit the winter storms.



Above: TOPCAT output at R4 gauging station during November 2009 storm event.



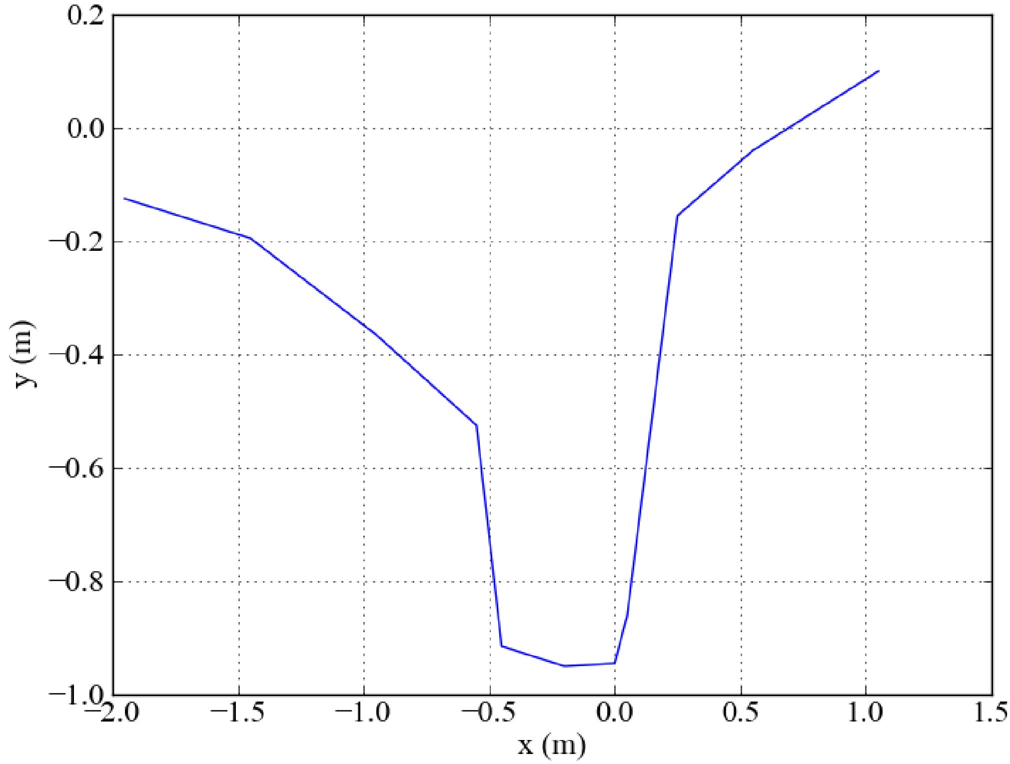
Above: TOPCAT output at R4 gauging station during July 2009 storm event with the mitigated Q included.

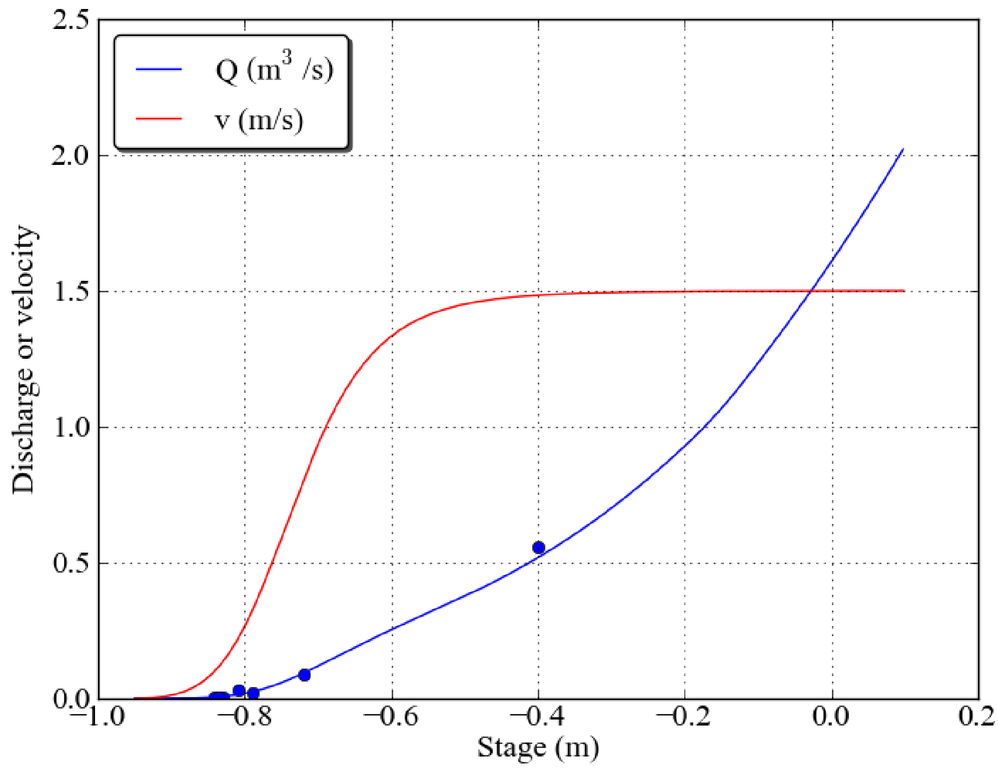


Above: TOPCAT output at R4 gauging station during November 2009 storm event with the mitigated Q included.

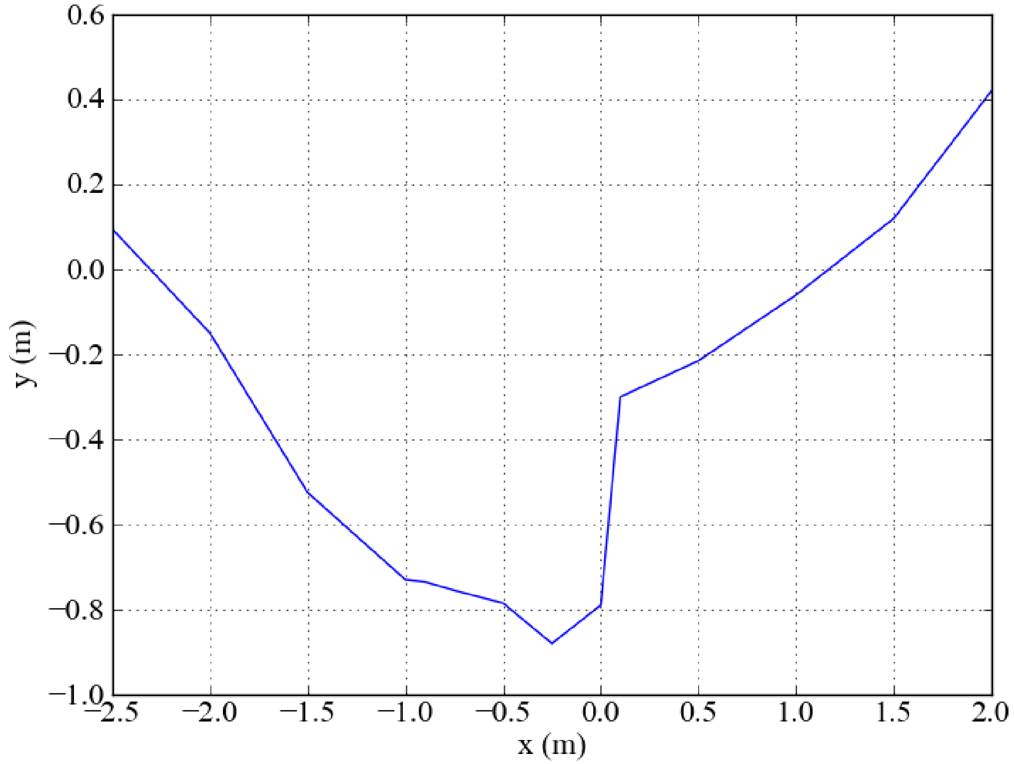
B.1 Cross sectional data and rating curves for all gauging stations

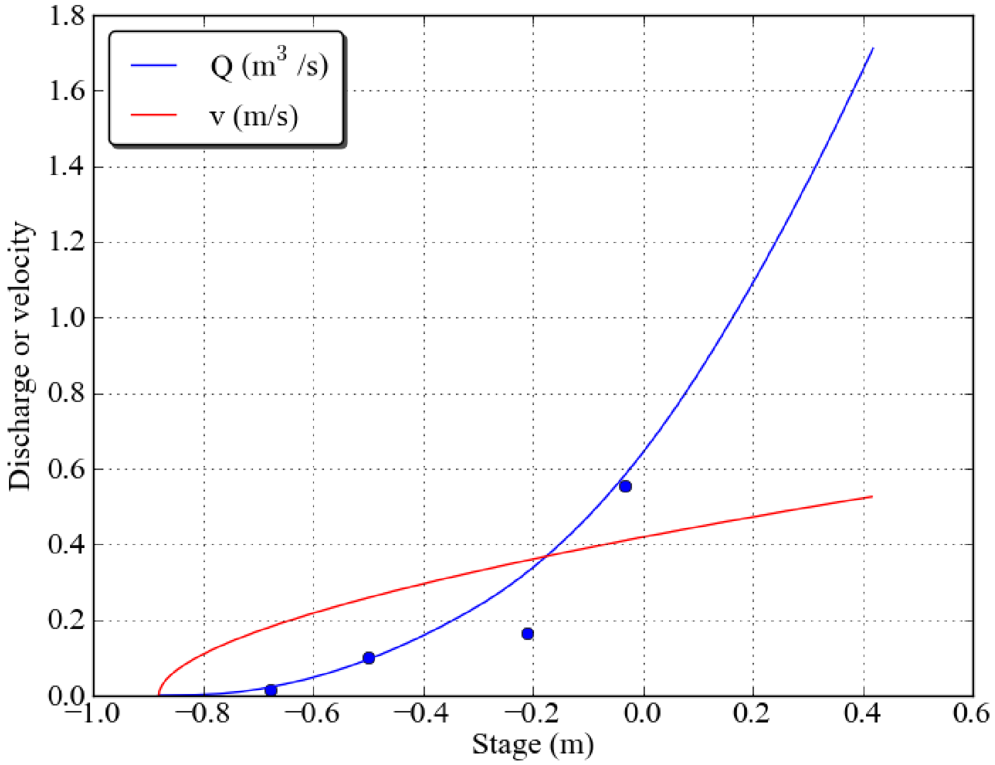
R1 cross section and rating curve:



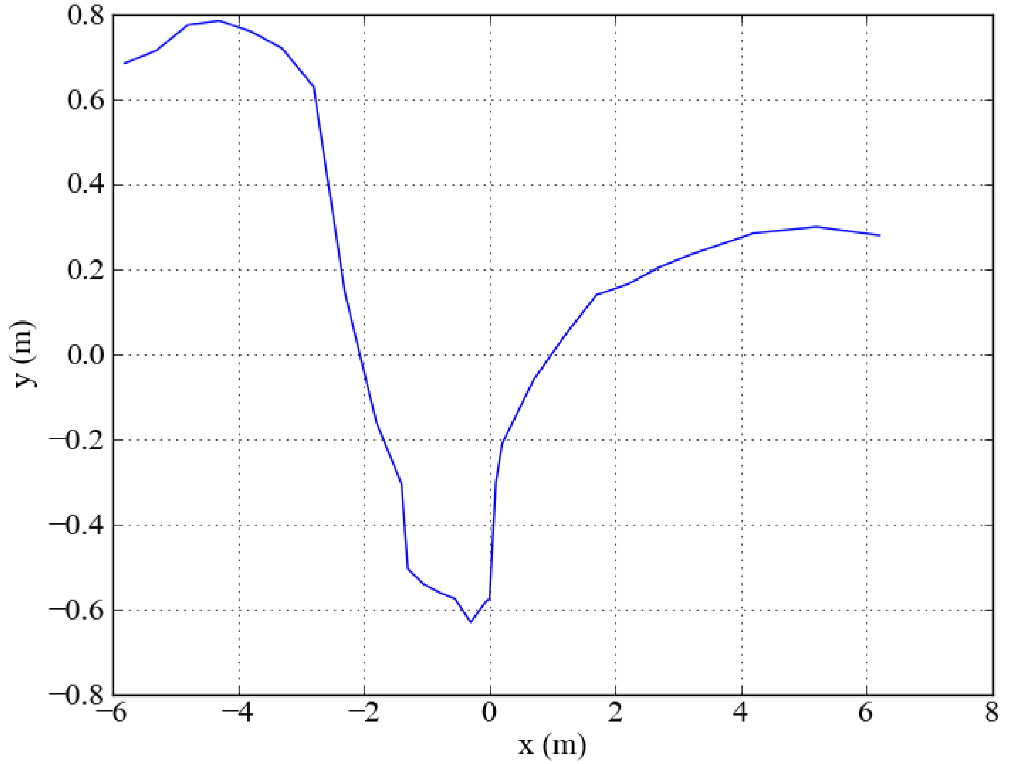


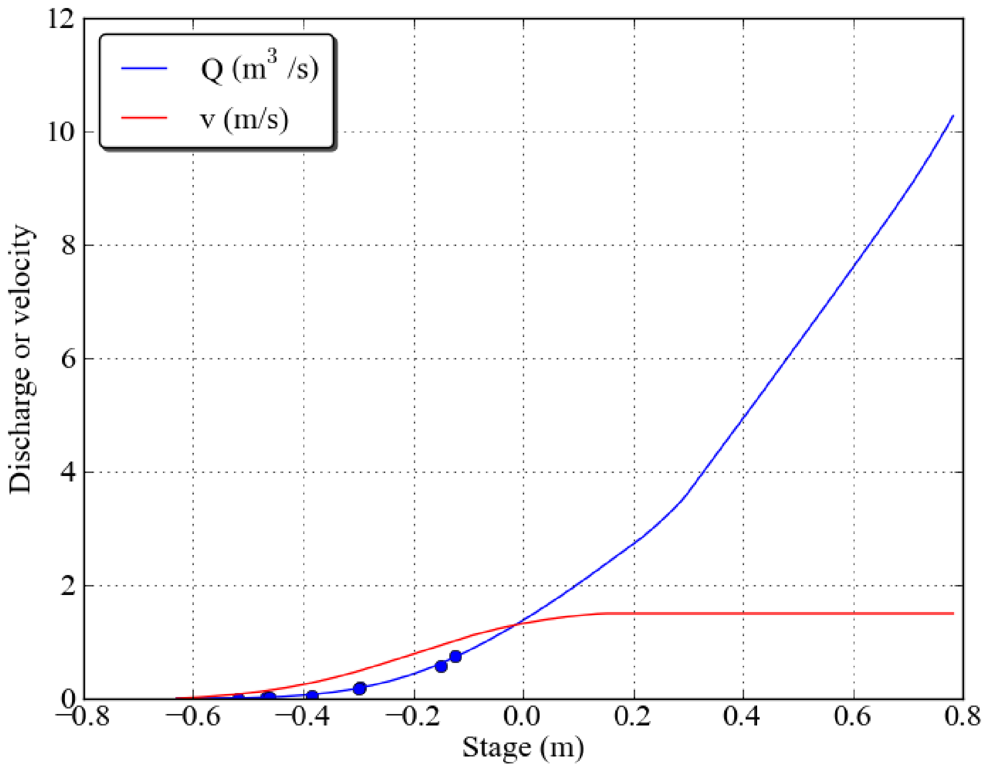
R2 cross section and rating curve:



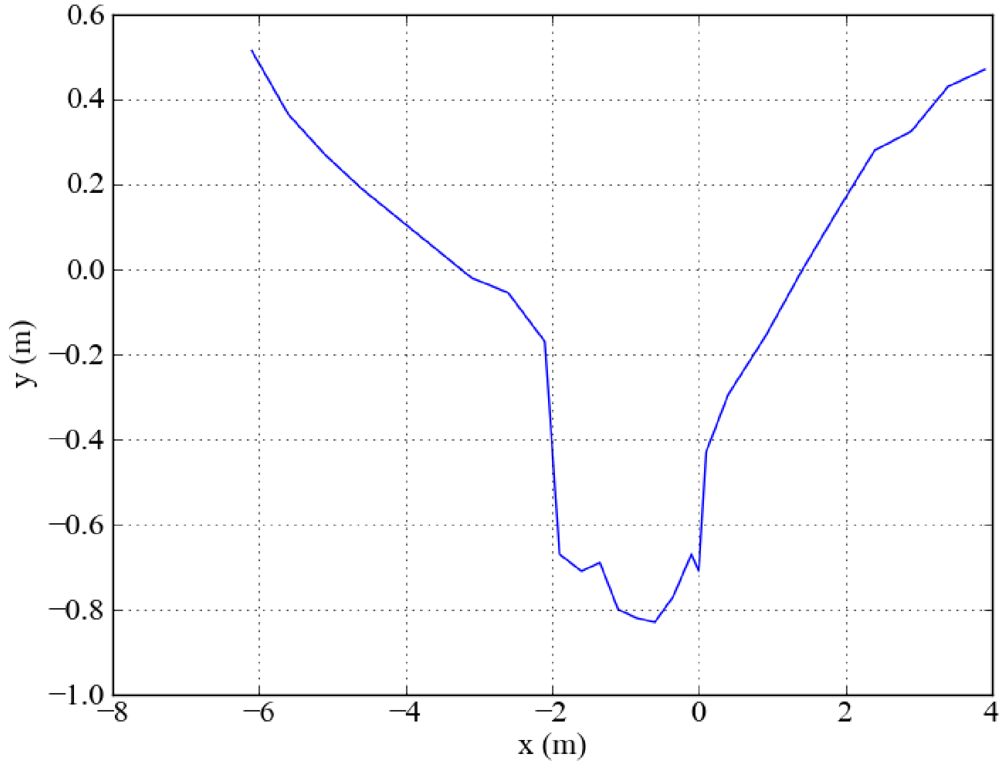


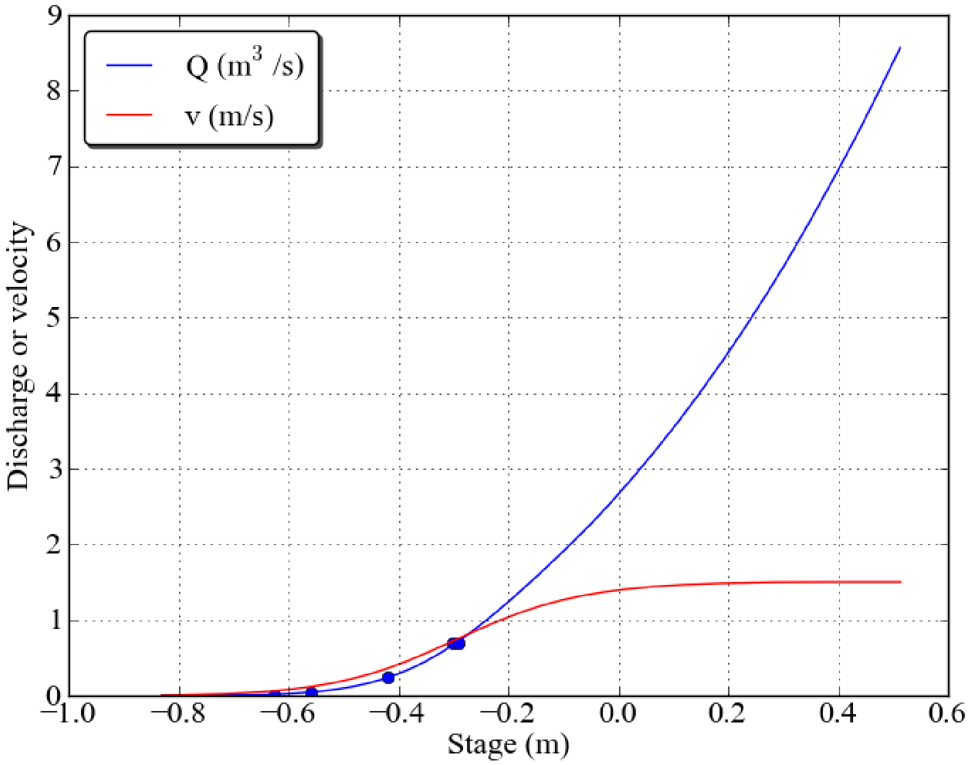
R3 cross section and rating curve:





R4 cross section and rating curve:





B.2 Calculation of Net Radiation from the Boulmer weather station

Extraterrestrial radiation for hourly or shorter periods (R_a):

$$R_a = \frac{12(60)}{\pi}.G_{sc}.d_r.\left[(\omega_2 - \omega_1).sin(\varphi).sin(\delta) + cos(\varphi).cos(\delta).\left(sin(\omega_2) - sin(\omega_1)\right)\right]$$ **Equation B.1**

where R_a is extraterrestrial radiation in the hour (or shorter) period [$MJm^{-2}hour^{-1}$], G_{sc} is the solar constant = 0.0820 [$MJm^{-2}min^{-1}$], d_r is inverse relative distance Earth-Sun (Equation B.2), δ is solar declination [rad] (Equation B.3), ϕ is latitude [rad] (Equation B.4), ω_1 and ω_2 represent the solar time angle at the beginning and end of the period, respectively (Equations B.5 and B.6).

$$d_r = 1 + 0.033.cos\left(\frac{2\pi}{365}.J\right)$$ **Equation B.2**

$$\delta = 0.409.sin\left(\frac{2\pi}{365}.J - 1.39\right)$$ **Equation B.3**

where J is the number of days in the year between 1 (1 January) and 365 or 366 (31 December).

$$\varphi\ (in\ radians)\ = \frac{\pi}{180} \times latitude\ (in\ decimal\ degrees)$$ **Equation B.4**

The solar time angles at the beginning and end of the period are given by:

$$\omega_1 = \omega - \frac{\pi t_1}{24}$$ **Equation B.5**

$$\omega_2 = \omega + \frac{\pi t_1}{24}$$ **Equation B.6**

where ω is the solar time angle at the midpoint of the hourly or shorter period [rad], and t_1 is the length of the calculation period [hour]: i.e. 1 for hourly period or 0.5 for a 30-minute period.

The solar time angle at the midpoint of the period is:

$$\omega = \frac{\pi}{12}[(t + 0.06667(l_z - l_m) + S_c) - 12]$$ **Equation B.7**

where t is the standard clock time at the midpoint of the calculation period [hour] (e.g. for a period between 14:00 and 15:00 hours, t = 14.5), L_z is longitude of the centre of the local time zone [degrees west of Greenwich] (L_z = 0° for Greenwich), L_m is longitude of the measurement site [degrees west of Greenwich], and S_c is the seasonal correction for solar time [hour] given by:

$$S_c = 0.1645.sin(2b) - 0.1255.cos(b) - 0.025.sin(b)$$ **Equation B.8**

$$b = \frac{2\pi(J-81)}{364}$$ **Equation B.9**

Extraterrestrial radiation for daily periods (R_a):

$$R_a = \frac{24(60)}{\pi}.G_{sc}.d_r.[\omega_s.sin(\varphi).sin(\delta) + cos(\varphi).cos(\delta).sin(\omega_s)]$$ **Equation B.10**

The above equation calculates R_a for daily periods. This removes the need to calculate ω_1, ω_2, ω, S_c and b.

Daylight hours (N):

$$N = \frac{24\omega_s}{\pi}$$ **Equation B.11**

where ω_s is the sunset hour angle in radians given by:

$$\omega_s = arccos[-tan(\varphi).tan(\delta)]$$ **Equation B.12**

Solar radiation (R_s):

$$R_s = \left(a_s + b_s\frac{n}{N}\right).R_a$$ **Equation B.13**

where R_s is solar or shortwave radiation [$MJm^{-2}day^{-1}$], n is actual duration of sunshine [hour], N maximum possible duration of sunshine or daylight hours [hour] (Equation B.11), n/N is relative sunshine duration [-], a_s is the regression constant, expressing the fraction of extraterrestrial radiation reaching the earth on overcast days (n = 0), $a_s + b_s$ is the fraction of extraterrestrial radiation reaching the earth on clear days (n = N) (the values a_s = 0.25 and b_s = 0.50 are recommended.

Clear-sky solar radiation (R_{so}):

The calculation of the clear-sky solar radiation R_{so}, when n = N, is required for computing net longwave radiation. The following equation is used for calculating clear-sky solar radiation at near sea level:

$$R_{so} = (a_s + b_s). R_a \quad \textbf{Equation B.14}$$

where R_{so} is clear-sky solar radiation [$MJm^{-2}day^{-1}$].

Net solar or net shortwave radiation (R_{ns}):

The net shortwave radiation resulting from the balance between incoming and reflected solar radiation is given by:

$$R_{ns} = (1 - \alpha). R_s \quad \textbf{Equation B.15}$$

where R_{ns} is the net solar or shortwave radiation [$MJm^{-2}day^{-1}$], and α is the albedo or canopy reflection coefficient, which is 0.23 for the hypothetical reference crop [dimensionless].

Net longwave radiation (R_{nl}):

The rate of longwave energy emission is proportional to the absolute temperature of the surface raised to the forth power. This relation is expressed quantitatively by the Stefan-Boltzmann law. The net energy flux leaving the earth's surface is, however, less than the emitted and given by the Stefan-Boltzmann law due to the absorption and downward radiation from the sky. Water vapour, clouds, carbon dioxide and dust are absorbers and emitters of longwave radiation. Their concentrations should be known when assessing the net outgoing flux. As humidity and cloudiness play an important role, the Stefan-Boltzmann law is corrected by these two factors when estimating the net outgoing flux of longwave radiation. It is thereby assumed that the concentrations of the other absorbers are constant:

$$R_{nl} = \sigma \left[\frac{T_{max,K}{}^4 + T_{min,K}{}^4}{2}\right] \left(0.34 - 0.14\sqrt{e_a}\right) \left(1.35 \frac{R_s}{R_{so}} - 0.35\right)$$ **Equation B.16**

where R_{nl} is net longwave radiation [$MJm^{-2}day^{-1}$], σ is the Stefan-Boltzmann constant [$4.903x10^{-9}MJK^{-4}m^{-2}day^{-1}$], $T_{max,K}$ and $T_{min,K}$ are the maximum and minimum absolute temperatures during the 24-hour period, respectively [K = °C + 273.16], e_a is the actual vapour pressure [kPa], and R_s/R_{so} is the relative shortwave radiation (limited to ≤ 1.0).

An average of the maximum air temperature to the forth power and the minimum air temperature to the forth power is commonly used in the Stefan-Boltzmann equation for 24-hour time steps. The term (0.34 - 0.14$\sqrt{e_a}$) expresses the correction for air humidity, and will be smaller if the humidity increases. The effect of cloudiness is expressed by (1.35 R_s/R_{so} – 0.35). The term becomes smaller if the cloudiness increases and hence R_s decreases. The smaller the correction terms, the smaller the net outgoing flux of longwave radiation.

Net radiation (R_n):

The net radiation (R_n) is the difference between the incoming net shortwave radiation (R_{ns}) and the outgoing net longwave radiation (R_{nl}):

$$R_n = R_{ns} - R_{nl}$$ **Equation B.17**

B.3 Calculation of potential evaporation

Consistent with the FAO report, the following calculations use *kPa* for units of pressure:

$$\gamma = 0.665 \times 10^{-4} p_b$$ **Equation B.18**

where γ ($kPaK^{-1}$) is the psychrometric constant;

$$\mathrm{T}' = \mathrm{T_a} + 237.3$$ **Equation B.19**

$$\mathrm{e^0} = 0.6108\exp\left(\frac{17.27\mathrm{T_a}}{\mathrm{T}'}\right)$$ **Equation B.20**

where e^0 (kPa) is the saturated vapour pressure;

$$\Delta = \frac{4098\mathrm{e^0}}{\mathrm{T}'^2}$$ **Equation B.21**

where Δ ($kPaK^{-1}$) is the slope of the saturation vapour pressure curve;

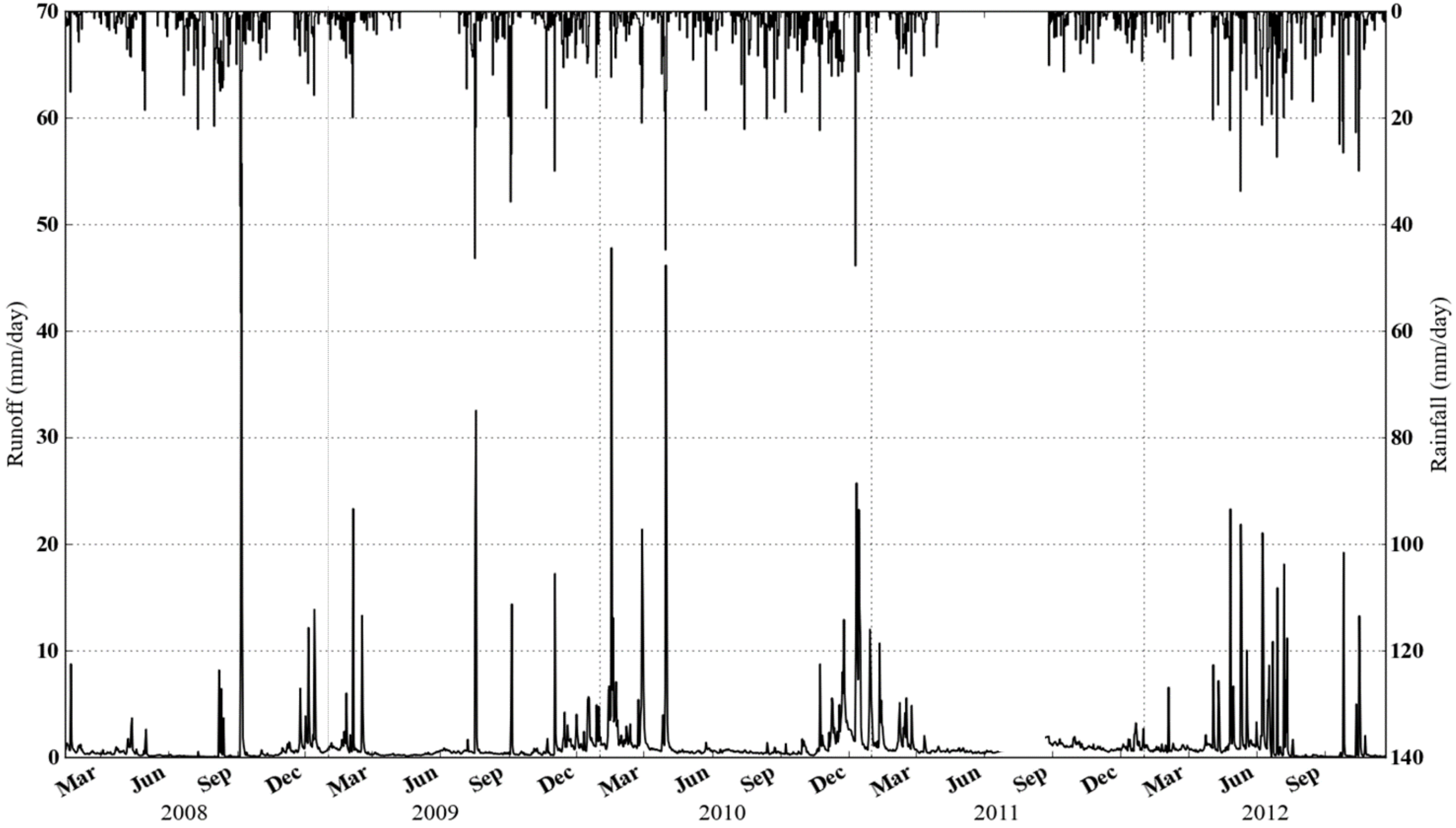
$$r_g = \begin{cases} 0.1\mathrm{r_n} & \text{daytime} \\ 0.5\mathrm{r_n} & \text{night time} \end{cases}$$ **Equation B.22**

where r_g (W/m^2) is the soil heat flux; finally giving:

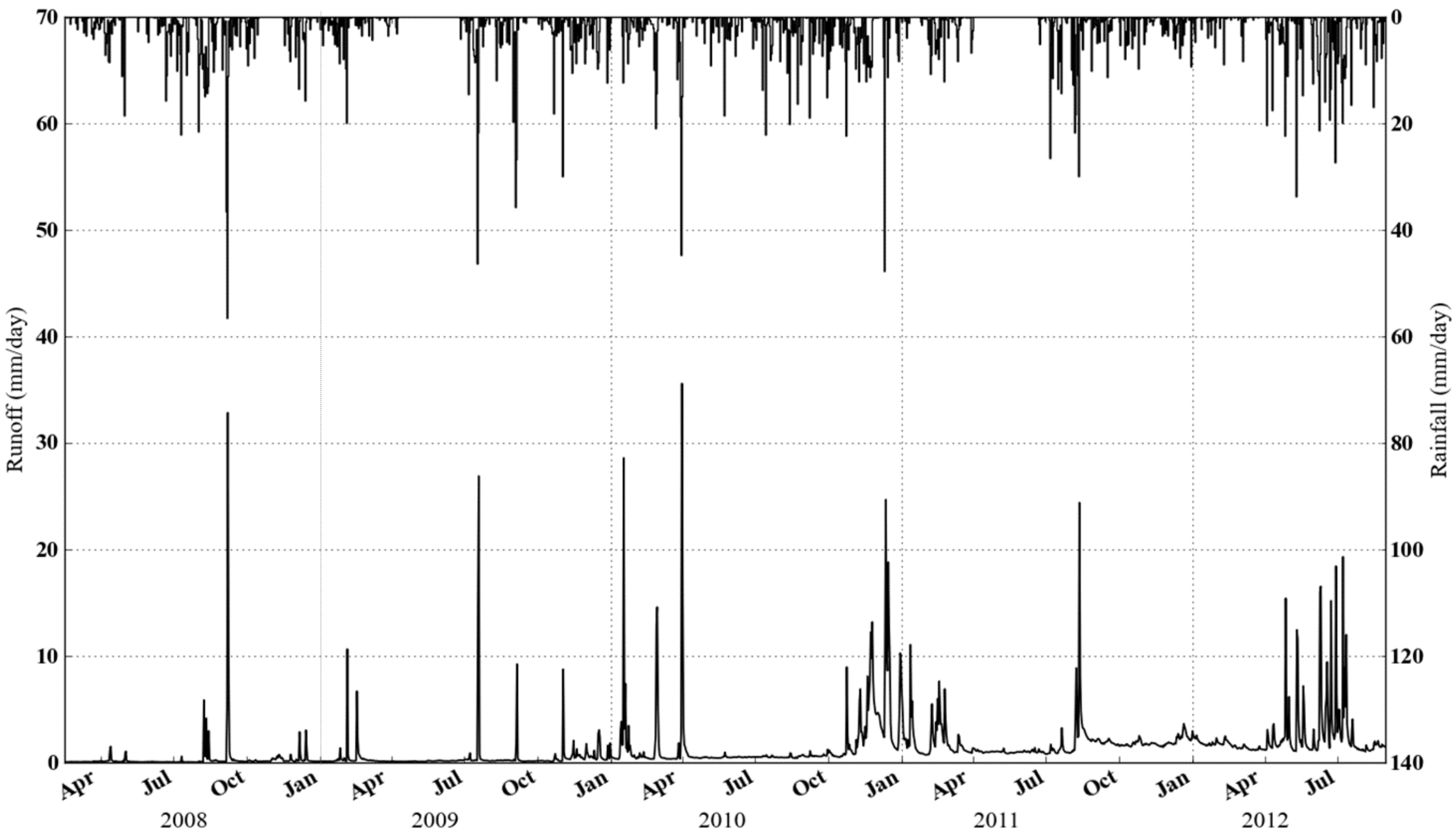
$$\mathrm{e} = \frac{0.408\Delta\left[0.0036\left(\mathrm{r_n} - \mathrm{r_g}\right)\right] + \gamma \frac{37}{\mathrm{T_a} + 273}\mathrm{ue^0}(1 - 0.01\mathrm{h})}{\Delta + \gamma(1 + 0.34\mathrm{u})}$$ **Equation B.23**

where e ($mm\ h^{-1}$) is the potential evaporation rate.

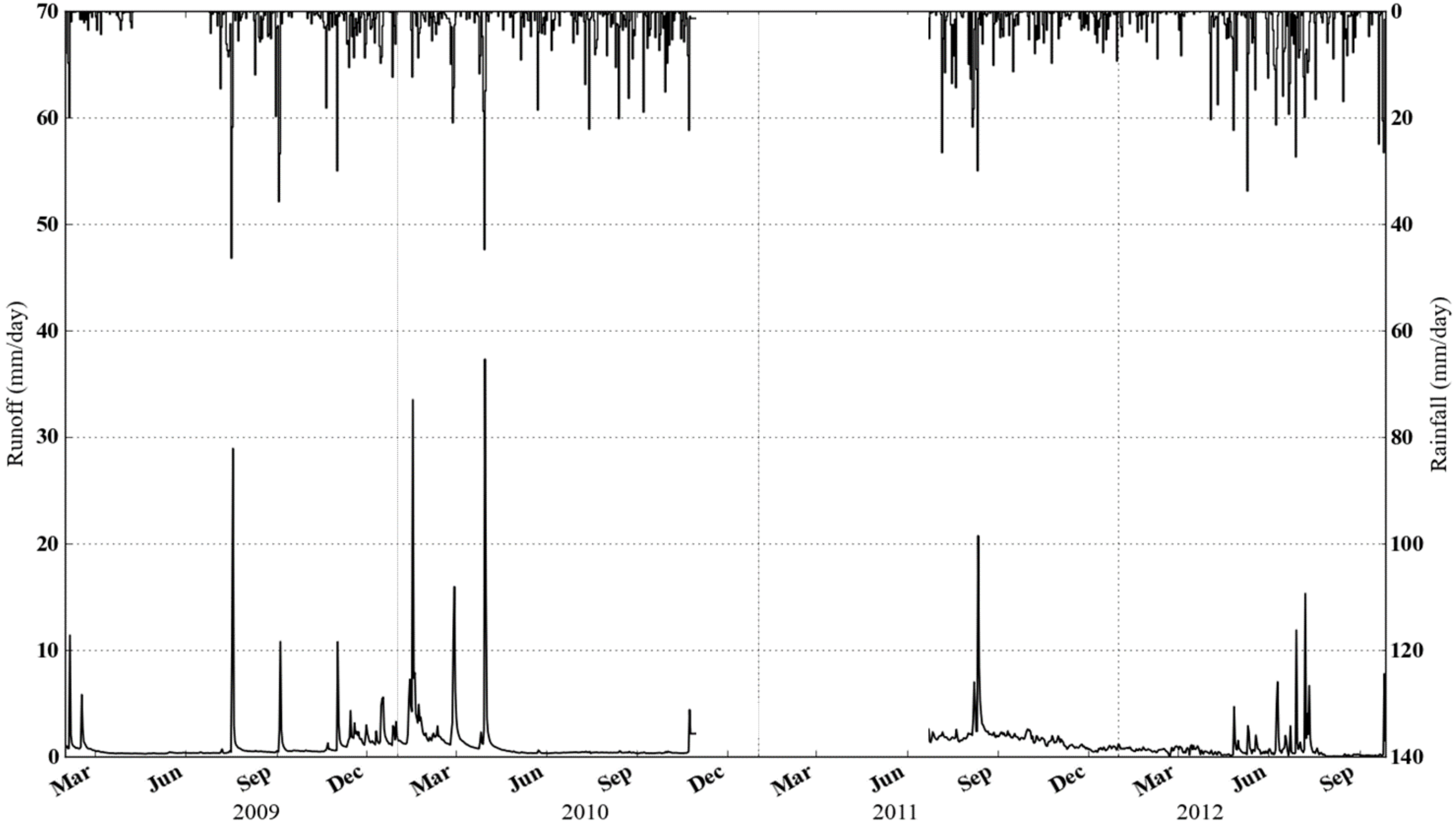
B.4 Rainfall-runoff data from gauging stations in Belford Catchment



Belford Burn rainfall-runoff from 2008-2012 (R1) – Note gap in data in August 2011 (due to broken diver)



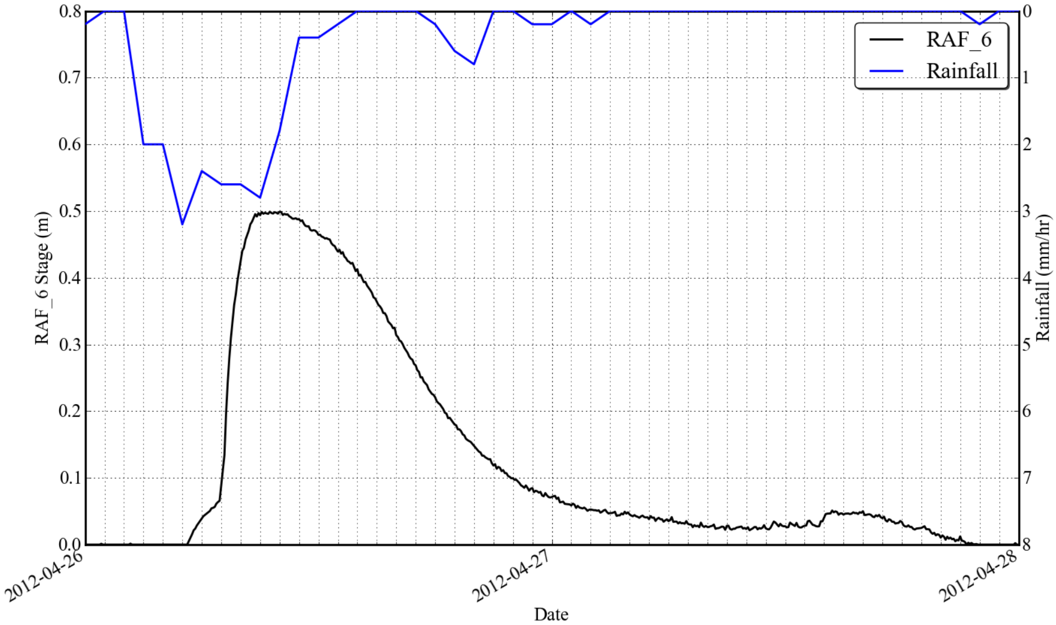
Belford Burn rainfall-runoff from 2008-2012 (R2)



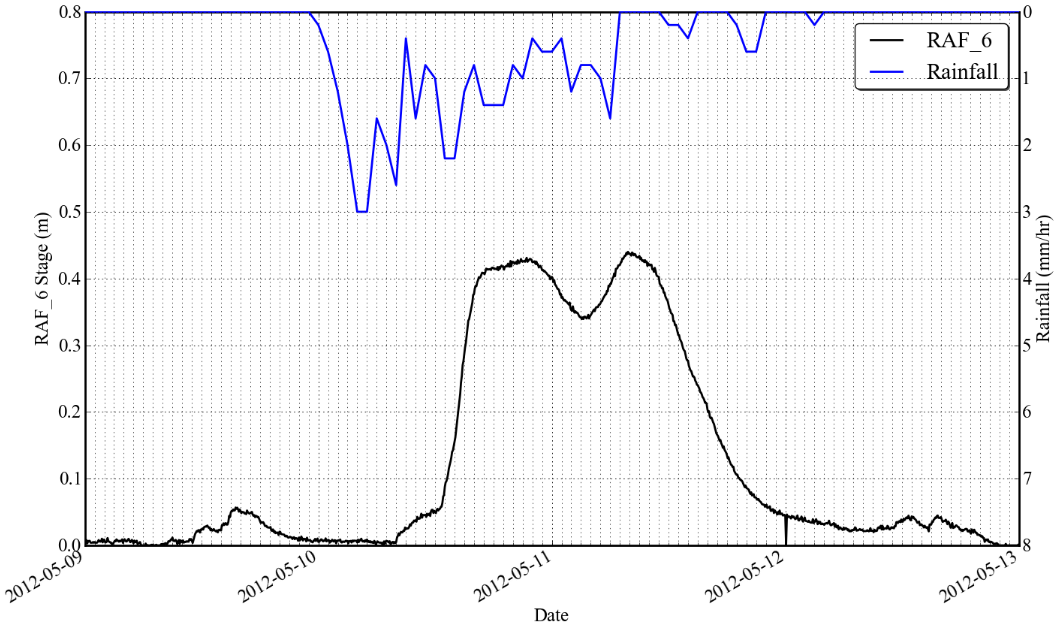
Belford Burn rainfall-runoff from 2008-2012 (R4) – Note considerable gap in data in 2011 (due to broken diver)

C.1 Observations from Belford RAFs

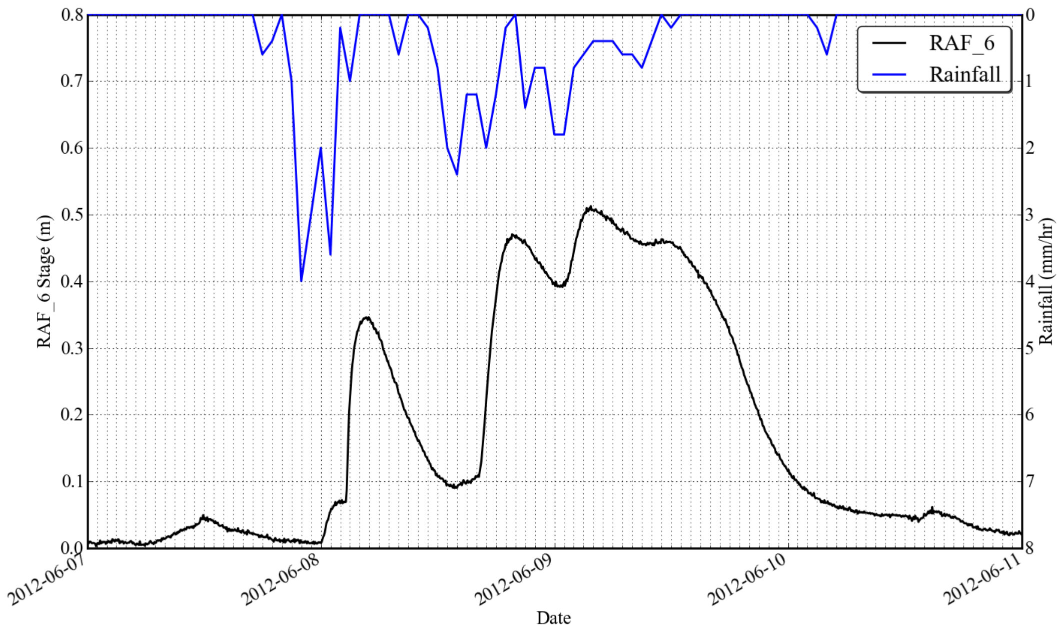
C.1.1 RAF-6



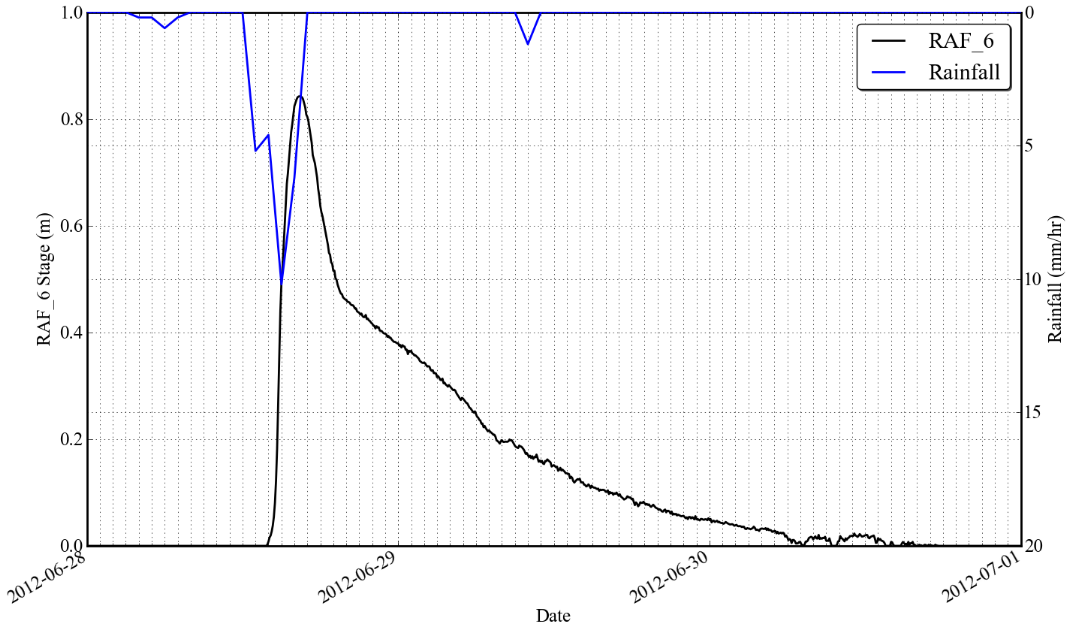
RAF-6 during April 2012 event



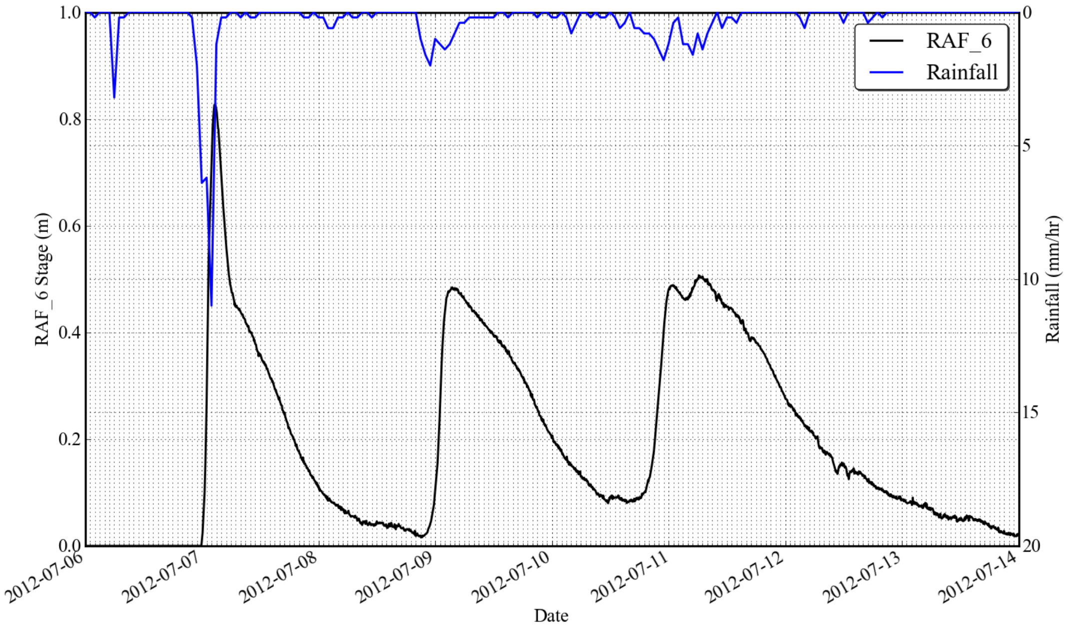
RAF-6 during May 2012 event



RAF-6 during early-June 2012 event

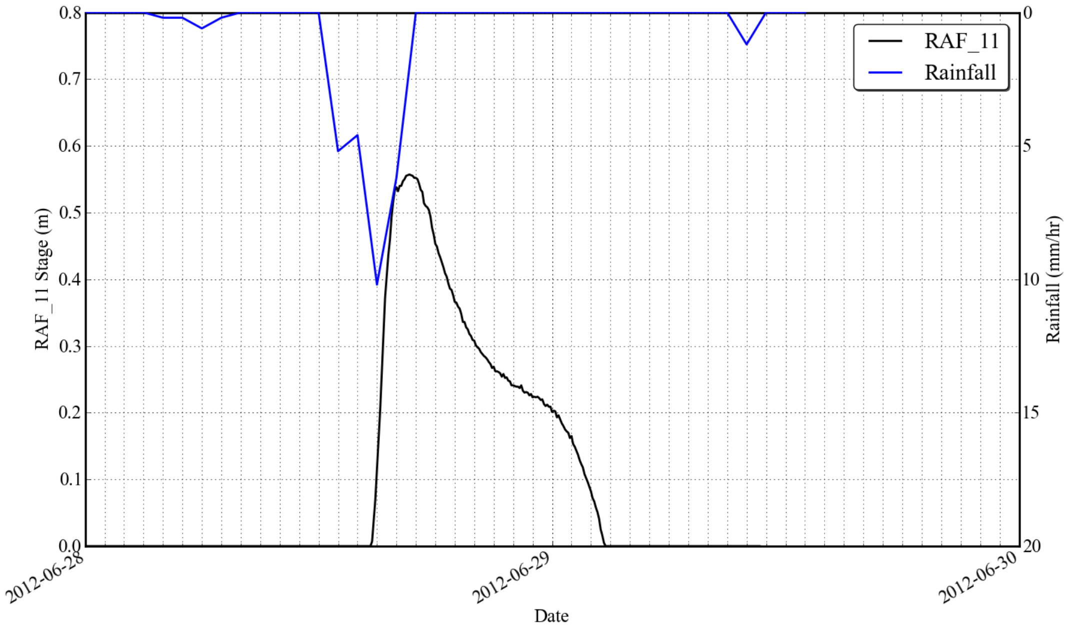


RAF-6 during late-June 2012 event

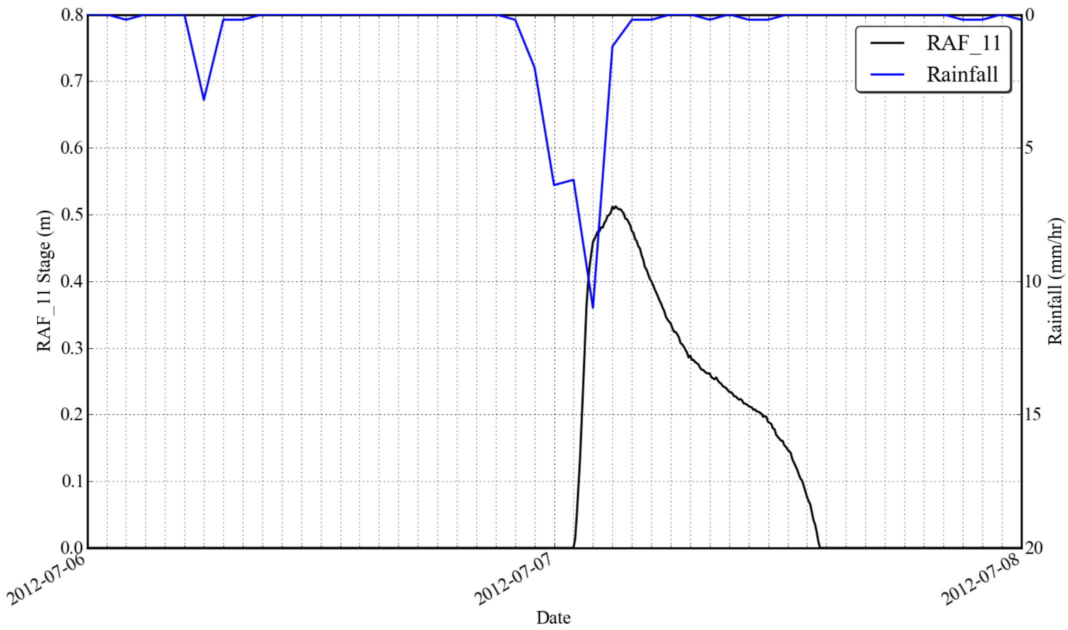


RAF-6 during three close events in July 2012

C.1.2 RAF-11



RAF-11 during late-June 2012 event



RAF-11 during July 2012 event