Reconstructing the peak flow of historical flood events using a hydraulic model: The city of Bath, United Kingdom

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Abstract
Historical flood events on the River Avon in the city of Bath were reconstructed using a 1D hydraulic model. The model represents the river prior to the completion of the Bath flood defence scheme and was developed based on archived data on archival materials including historical cross sections of channel geometry, technical drawings of hydraulic structures, descriptions of floodplain extent, epigraphic markings and photographs. The model was calibrated on two historical floods for which peak flow values were available from hydrological data (March 1947 and December 1960) and the calibration was based on adjustment of Manning’s $n$ as well as the introduction of floodplains identified from historical maps and photos. Subsequently, the peak flow was assessed for a total of 16 events for which flood marks existed in the centre of the city. A composite annual maximum series of peak flow was created upstream of Bath city centre and presented a flow series consisting of data from 1866 to present day. Finally, flood frequency analysis was undertaken on the dataset, showing that the inclusion of historical floods increased the 1 in 100 year design flood by up to 30%, that is, a substantial influence.

KEYWORDS
discharge estimation, documentary sources, flood marks, flood reconstruction, historical flood, hydrograph, hydrological reconstruction, paleohydrology

1 | INTRODUCTION

Estimating the risk of future flood events is of interest to civil engineers and urban planners for the design and operation of infrastructure and hydraulic structures (e.g., bridges, dams, weirs, sluice gates). These structures are commonly designed for long-term services, and are designed to cope with a design flood event of a pre-defined return period, or expected frequency of occurrence, as estimated from statistical analysis of annual maximum series (AMS) of peak flow events recorded at flow gauging stations. The average record length of AMS series in the United Kingdom is around 40 years as the majority of the existing flow gauging stations were installed after the 1963 Water Resources Act (Lees, 1987). Consequently, considerable interpolation is often required and the estimates of design events with large return periods such as the 1 in 100 year event are highly uncertain (Kjeldsen, 2015). One possible avenue to reduce this uncertainty is to extend the record back in time by reconstructing historical events (Lumbroso, Wyncoll, Liu, & Davison, 2019; Macdonald &
Historical evidence of past events such as: water level marks (epigraphic evidence), documentary descriptions, photographs of floods from contemporary newspapers or technical publications are available in many towns and cities and there is considerable local interest in this topic (Kjeldsen et al., 2014; Wilhelm et al., 2019). However, while intuitively appealing, the use of such augmented series are not routinely included in quantitative assessments of design flood flows, mainly because of the difficulties in both deriving such series and in combining different data into a single assessment of the flood flow (Lumbroso et al., 2019). Nevertheless, if historical flood data are available, the use of such information can considerably reduce the uncertainty associated with flood frequency estimation (Prosdocimi, 2018).

Historical peak flow reconstructions have been applied to many different European rivers, but have focused primarily on the magnitude of historical flood events rather than the hydraulics of specific historical flood events and the effect of different hydraulic structures and flood protection schemes (Elleder, 2010). Herget and Meurs (2010) estimated the peak discharges of the River Rhine using historical flood water levels located in the city of Cologne, Germany. By reconstructing the river cross sections and floodplain they first derived the mean flow velocity using Manning’s equation before transforming the historical flood levels to peak discharges. Using Chow’s (1959) method, the hydraulic roughness was calculated as the average of nine different elements (surface roughness, vegetation, river irregularity, river alignment, scouring, obstructions, stage and discharge, sediment load and seasonal changes) multiplied by a channel meandering factor. The approach was validated by repeating the analysis for recently gauged floods. Nevertheless, a considerable limitation identified in this work is that for the historical period the authors considered (1342–1782), no bridges nor other hydraulic structures existed and thus these implications have not been considered in their research. The authors acknowledge this and discuss the need to evaluate the effect of such historical floods in Cologne of today taking into account the current flood protection works and measures. Elleder, Herget, Roggenkamp, and Niessen (2013) reconstructed the flood hydrograph of the 1784 event on the River Vltava in Prague, Czech Republic. The main characteristics of the event were reconstructed by combining information from different sources, including: the rate of water level rise, the duration of the flood, the flow peak, the rising and falling limbs. Once more, the effects of bridges and other hydraulic structures were not taken into account. These were only briefly discussed at the end when concluding that some of the alleviating current works in place would decrease the effect of such a flood if it were to happen today. Roggenkamp and Herget (2014) reconstructed peak discharges of historical floods following the same technique as Herget and Meurs (2010). The difference in this research is that the historical flood peak discharges were calculated at different locations along the River Ahr, Germany, offering additionally the spatial reconstruction of the flood wave.

All the aforementioned studies adopted relatively simple hydraulic procedures such as the slope area method (United States Geological Survey, 1989). This technique reconstructs the peak flows based on the static and empirical Manning’s equation and therefore does not consider the detailed hydrodynamics associated with the specific flood events, highlighting in particular the influence of hydraulic structures such as bridges and weirs. Consequently, the uncertainties associated with the reconstructed peak flow are likely to be considerable but remain largely unknown. This paper investigates the barriers and opportunities associated with the development of a more detailed hydraulic model to estimate peak flow of historical events in the city of Bath using historical sources. We discuss how a one dimensional (1D) hydraulic model enabled us to reconstruct numerically major historical floods in the city of Bath, how it was designed and model parameters selected. Historical hydrological and cross-sectional data were collected as a result of an intensive search of many sources (e.g., physical evidence in archive rooms and digital archive records), and the uncertainty associated with this data is also described. Finally, the importance of historical data, their applications and the effort required to incorporate them in modern risk assessments is discussed and the impact on design flood estimation assessed.

2 | THE CITY OF BATH AND THE RIVER AVON

The city of Bath is a historical UNESCO world-heritage site located in South West England and is an instructive case study in how the history, architecture and development of a city can be closely connected to the city’s relationship to its river. The River Avon (also known as the Bristol Avon) rises in South Gloucestershire and flows through the Wiltshire town of Chippenham before reaching the centre of Bath and onwards to Bristol, ultimately flowing into the Bristol Channel at Avonmouth. The Environment Agency operates a number of flow gauging stations on the river; the most central to this study is the station situated at Bathford, just upstream from Bath itself, at which point the upstream catchment area is 1,552 km². In this study we are particularly concerned about the 8 km stretch of river from Bathford
(~3 km upstream of Bath City) down to Twerton Sluices on the western side of Bath from where the river flows down to the city of Bristol.

The first known settlement in Bath (named Aquae Sulis by the Romans), was founded around 44 AD, and by 50 AD a temple and public baths had been constructed around the hot springs to exploit their healing properties. One of the earliest and most important maps of the city of Bath, the Savile Map of Bath (Figure 1), dated 1603, shows the extent of the Elizabethan city and provides the first evidence of a weir in Bath’s city centre. A medieval diagonal weir establishing a water level difference to power Monk’s Mill (Davis & Bonsall, 2006). The River Avon flowing through Bath has been used as a highway for trade since the architect John Wood the Elder (1704–1754) improved the Avon Navigation in 1727 creating weirs and locks to control the tidal river and conduct boats uphill and upstream to Bath. Ralph Allen (1693–1764), philanthropist and entrepreneur was instrumental in making the River Avon navigable as high up as Bath which brought economic prosperity to the area allowing Bath to become an inland port supporting agriculture and the burgeoning industrialisation economy (Buchanan, 1998). Bath Stone from Allen’s stone quarries up on Combe Down became one of its main exports and the river improvements enabled the transportation of Bath Stone to Bristol and beyond to be shipped all over the world and new materials such as Scandinavian deal timber to be more readily imported, together with Cornish and later Welsh slate for roofing the developing Georgian city. In the 18th century Georgian period the city was considered a spa city and there was an increase in population and development with neoclassical Palladian buildings.

Bath has always had a mixed relationship with the River Avon as from the late 19th Century the River Avon no longer served an economic purpose (Bath & North East Somerset Council, 2011). Only in recent years have new developments such as Bath Quays Waterside started promoting river living as desirable and reconnecting Bath to its river, whereas most historical buildings located riverside are facing away from the river.

**FIGURE 1** The Savile Map of 1603, historical map of the city of Bath, showing the first sighting of a weir in Bath’s city centre, a medieval diagonal weir highlighted in the map by the dashed rectangle (Manco, 1993)
2.1 Historical floods on the River Avon

As the city was always located close to the river, communities in Bath have experienced the effects of flooding since Roman times. The history of efforts to combat flooding has been documented in illustrious accounts by, for example, Buchanan (1998) and Greenhalgh (1974), citing evidence of flooding back to Roman times, though physical evidence has so far only been identified for the 19th century onwards.

Historical evidence of past flood events has been left on buildings in the city as well as documentary evidence in contemporary newspapers and technical reports. There are three locations (Figure 2) where dated historical water levels have been identified within the city: Grove Street, Norfolk buildings and Halfpenny/Widcombe Bridge. The earliest flood mark dates back to 1823 in Grove Street, but the majority of extreme floods after that have been recorded underneath Widcombe Bridge (14 marks from 1875 to 1960 seen in Figure 2). These historical flood marks predominantly predate existing flow recordings initiated in 1939 and 1969, respectively, and located further upstream at St James Bridge (1939–1968) and Bathford (1969-present), and therefore provide a unique opportunity to extend the record length in time and to include significant events.

It is important to remember that the historical flood marks represent the hydraulic conditions existing at the time of the flood. As will be discussed in the next sections, even though the river channel section between Bathford and the city centre is relatively stable and there has been minimal change over time (Bath & North East Somerset Council, 2016), considerable changes to the river geometry and hydraulic structures have been made downstream of St James Bridge since the 19th century. Therefore, hydraulic studies of these events should consider the river hydraulics as it existed during the time of the events.

2.2 The Bath flood defence scheme

Following extensive flooding in 1947 and in 1953 the then chief engineer Frank Greenhalgh initiated work on what would eventually become the present-day BFDS. Four options were initially considered, deciding that the main priorities were to increase the capacity of the river to carry flood water by removing obstacles such as bridges, dredging and streamlining the channel. The implementation of the BFDS consisted of 10 phases summarised below in Table 1 and Figure 3.

The scheme was designed to defend the city against the worst flood on record which at the time was considered to be the 1882 flood, estimated to have a peak flow of 12,950 ft³/s (367 m³/s) (Greenhalgh, 1974). However, in December 1960 Bath experienced an even larger flood, peaking at an estimated flow rate of 424 m³/s (this is the estimated value as the flood exceeded the capacity of the monitoring system at 398.5 m³/s). The economic impact (£1.4 million in damages) of the fourth and fifth December 1960 flood acted as a catalyst for the BFDS. The works began in 1964 and finished in 1974 with the successful completion of Phase X at a total cost at the time of £3 million.

**FIGURE 2** Map of flood marks in Bath City centre (left) and photo of flood marks at Halfpenny/Widcombe Bridge wall (right)
3 | RECONSTRUCTION USING NUMERICAL MODELLING

Implementation of the BFDS included a number of radical changes to the hydraulic structure of the river in Bath, notably increasing conveyance by streamlining cross-sections, removal of bridges and redesigning weirs. A detailed 1D hydraulic model was therefore developed using available historical information from multiple sources, representing the River Avon in Bath covering the 150 years of flood marks before the implementation of the BFDS, that is, pre 1960. The numerical analysis was undertaken using the commercially available Flood Modeller river modelling system (Jacobs, 2020). This section defines the governing equations of the model and the input model parameters, and presents the selected methodology for the peak flood flow reconstruction using the available flood marks.

3.1 | 1D hydraulic river model

The one-dimensional 1D Flood Modeller engine is a commercially available river modelling system solving the conservation of continuity and momentum equations simultaneously via the Saint-Venant Equations which can be expressed as follows (Radecki-Pawlik, Pagliara, & Hradecky, 2017; Saint-Venant, 1871):

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + gA(So - Sf) = 0$$

where $Q$ is the flow rate (m$^3$/s), $A$ is the flow cross-sectional area (m$^2$), $x$ is the distance in the direction of the flow (m), $So$ is the bed slope (m/m) and $Sf$ is the friction slope (m/m).

TABLE 1 Description of Phases I–X of the Bath flood defence scheme

| Phase | Description |
|-------|-------------|
| I     | Replacement of Old Bridge with two single span bridges for road and pedestrian traffic. |
| II    | Realignment, resection and regradement of river section between Horseshoe Bend and Twerton Gates. |
| III   | Improvement to the discharge capacity of the main arch in Newbridge. |
| IV    | Replacement of the two weirs at Twerton and replacement by a twin automatic sluice barrage. |
| V     | Channel realignment and resection; trapezoidal channel sections with stone toe protection, channel sections with one slope bank and one vertical bank in concrete and rectangular channel sections with steel sheet piling to both banks. |
| VI    | Pulteney Weir to be remodelled and provided with a new sluice gate, Pulteney Bridge foundations to be protected and regarding of channel ceases. |
| VII   | Diversion of services wherever they occur. |
| VIII  | One bank to have vertical masonry faced concrete wall and the other bank to be sloping earth with stone toe protection. |
| IX    | Dredging along length from Twerton to U/S limit. |

FIGURE 3 The Bath flood defence scheme: Map of the River Avon highlighting the 10 different phases of the scheme (Phase I–Phase X)
The equations are solved using a finite difference solver and an implicit operator (CH2MHILL, 2012). It is a widely used tool for modelling flood hydraulics in the United Kingdom (Ahmadian, Falconer, & Wicks, 2018) including the ability to incorporate different hydraulic structures (e.g., weirs, spills, arch bridges, etc.) in the simulations (CH2MHILL, 2012).

3.2 | Model parameters

Reconstruction of the historical flood events using a numerical model requires reconstruction of the hydraulic conditions of the river at the time of the floods. The following section details how this information was extracted for the city of Bath from a combination of historical and modern data sources for the purpose of providing credible values of the input parameters for the 1D hydraulic model, including: (a) cross sections of the river channel, (b) the distance between these sections, (c) the geometry of the hydraulic structures, (d) the energy slope, (e) the roughness of the river bed measured using the Manning’s coefficient and (f) the inflow hydrograph (boundary condition).

3.2.1 | Inflow hydrograph, Q

The upstream boundary of the hydraulic model was at St James’ Bridge, located 3.3 km downstream from Bathford. The gauging station at St James was operational between 1939 and 1968, thus capturing major events in 1947 and 1960, but replaced in 1969 by the gauging station at Bathford which has remained operational to this day. The inflow hydrograph for all historical flood events was modelled by scaling the 1 in 100 year design hydrograph for the River Avon developed by Reed (1988) using flow data both from Bathford and St James’ gauge stations. For the 1960 flood, the most recent significant pre-BFDS flood on the Avon, a microfilm version of the water levels recorded during the flood was located in the physical archives of the National River Flow Archive (NRFA), in Wallingford (Figure 4) and used in the model. The microfilm was manually scanned and digitised to create the 1960 inflow hydrograph using an existing rating curve for St James Bridge. Figure 5 shows a comparison of the digitised data for the 1960 flood and the 1 in 100 year design hydrograph at Bathford adopted from Reed (1988). Generally, there appears to be a good agreement between the shape of the two events. Initial model runs suggested that the shape (mainly the width) of the hydrograph did not change considerably when routed from Bathford to St James past the intervening catchment and floodplains and thus the same shaped hydrograph was used for all model simulations.

3.2.2 | Energy slope, $S$

As in many previous hydraulic studies of historical floods (e.g., Herget & Meurs, 2010; Roggenkamp & Herget, 2014) the energy slope is assumed to be the slope of the water surface rather than the slope of the energy line. In the historical period of interest predating the BFDS, no notable changes have taken place in the river (Bath & North East Somerset Council, 2011) and therefore the modern value of 0.00010 m/m was applied in the numerical model (Environment Agency and Black & Veatch, 2013).
3.2.3 | Manning’s roughness coefficient, $n$

Estimating a river’s hydraulic roughness is challenging (Herget & Meurs, 2010) as multiple factors affect the flow in natural channels. Using the spot flow gauging data available from the NRFA for the Bathford station (providing hydrometric data upstream of the city of Bath) between 1970 and 2018, Manning’s $n$ was estimated. All values were found to be within the range 0.01–0.05. A sensitivity analysis was conducted as part of the calibration process and resulted in two Manning’s $n$ values for the channel bed, 0.035 and 0.045 representative of natural channels with irregular side slopes (Chow, 1959). The floodplains were assigned a Manning’s $n$ of 0.09 to simulate a low flow rate.

3.2.4 | Distance between cross sections, $x$

The distance between cross sections was calculated from an existing engineering drawing of a longitudinal section between Netham (Bristol) and Bathampton (upstream of Bath) dated 1954 (see Figure 6). The individual cross sections were found in the Environment Agency’s Digital Archives in their office in Bridgwater among another 1,112 scanned drawings relevant to the BFDS. A total of 233 locations were recorded on the longitudinal section, measuring the right and left bank levels, the bed level and their distance from Avonmouth in feet (see Figure 6). In 54 of these recorded locations full cross sections were drawn but cross sections C.S. 155 to C.S. 232 are of interest as they represent the river geometry of the stretch of river of interest to this study between Twerton weir (downstream of Bath’s city centre) and Bathampton (upstream of Bath’s city centre, close to the Bathford gauge station). All sections were taken in 1934 and were replotted after 1954 to Newlyn Datums.

3.2.5 | River cross sections and hydraulic structures

The 25 cross sections used for the design of the numerical model pre-BFDS are identified as C.S. 155 to C.S. 232 which represent the 8 km stretch of river channel between Bathampton and Twerton. Drawings of the river cross-sections were discovered in the Environment Agency’s Digital Archives at the local office in Bridgwater. All filenames were in date formats from their scanned date, for example, “2011-12-013” and there was no accompanying index, thus each individual cross section needed to be separated individually. Each river cross section drawing was manually digitised, and all dimensions converted from inches to metres before being added to the numerical model. Using historical photographs (Bath in Time, 2019a, 2019b, 2019c, 2019d, 2019e) and the Environment Agency’s historic flood map GIS layer (Environment Agency, 2020) showing the

![Figure 5](image1.png)

**Figure 5** One in 100 year flood hydrograph for the River Avon at Bathford (Reed, 1988) and compared with the 1882, 1894 and 1960 floods scaled to peak flow values based on Reed (1988)

![Figure 6](image2.png)

**Figure 6** 1954 Engineering drawing of a longitudinal section between Netham (Bristol) and Bathampton (upstream of Bath) dated 1954 showing 223 individually recorded locations. The drawing was found in the Environment Agency’s Digital Archives, Bridgwater
maximum extent of individual Recorded Flood Outlines based on records from 1946, the digitised cross sections were then extended from both sides to incorporate the effect of the inundated floodplains around the river.

### 3.2.6 Hydraulic structures

The two main hydraulic structures in the numerical model are bridges and weirs. In the case where an arch bridge’s soffit level was too low and thus created blockage of the water, a numerical spill was introduced in parallel to the bridge in order to calculate the remaining flow from the overtopping. During Phases I (Figures 7 and 8) and VII (Figures 9 and 10) of the BFDS implementation, Old Bridge (C.S. 192) was replaced with two single span bridges for road and pedestrian traffic; the current-day Churchill Bridge. At the same time, Pulteney Weir (C.S. 206) was remodelled, and a new sluice gate was installed. Figure 7 shows the 5-arch Old Bridge on the left and the replaced single span Churchill Bridge on the right. Figure 9 shows the old diagonal weir at Pulteney on the left and the remodelled horseshoe weir on the right.

Figures 8 and 10 show the cross section C.S. 192 at Old Bridge and C.S. 206 at Pulteney Weir respectively.

### 3.3 Methodology for the reconstruction of historical floods

The hydraulic conditions of the River Avon prior to the BFDS were re-constructed in the Flood Modeller system using the model parameters described above. A detailed schematic of the setup is shown in Figures 11 and 12.

The first phase of the project consisted of a sensitivity analysis on Manning’s $n$ from which the two different hydraulic models were built to provide the upper and lower threshold envelopes of our results taking into account the uncertainty in calibration values. The first model was calibrated using as input the microfilm version of the water levels recorded during the 1960 flood (Figure 4). The second model was calibrated using the maximum recorded water level at St James during the 1947 flood. For the 1947 flood, the 1 in 100 year inflow hydrograph, as defined by Reed (1988), was specified at St James and was scaled to match the measured...
peak discharge therefore establishing the magnitude of the historical flood. The two calibrations resulted in values of Manning’s $n$ of 0.035 and 0.045 respectively. During the simulation, the hydraulic model routes the flood waves down the river system, calculating both the flow and water level at each of the specified cross-sections (Figure 12). As shown schematically on Figure 13, using the specified inflow hydrographs and the historical water marks at Widcombe Bridge, Manning’s $n$ was adjusted to match the peak historical water marks directly allowing for a ±5 cm margin of error.

Once the two models were calibrated, the second modelling phase comprised of adjusting the peak flow of a typical inflow hydrograph shaped as the 1 in 100 year design event, as specified in Section 3.2, until an agreement between simulated and observed water levels was obtained under Widcombe Bridge. Three major flood events were first selected as validation and then the remaining historical flood events were modelled accordingly. The outcome of this analysis was an upper and lower set of historical peak flow values at St James representing the results obtained using the two different Manning’s $n$ values.

Finally, the calculated historical peak flows at St James station were translated to equivalent flow at Bathford (dashed blue square in Figure 11) in order to extend the available data series. This was achieved by extending the hydraulic model upstream to Bathford (C.S. 232) and following the same methodology described above. The hydrographs were adapted to incorporate a
higher peak flow and were adjusted until the same agreement was found between simulated and observed water levels was under Widcombe Bridge. This outcome was also in the form of an upper and lower set of historical peak flow values at Bathford.

4 | RESULTS

Following the three-step procedure outlined above, hydrographs for all historical flood events were derived and plotted in Figure 13 and the historical peak flows at St James station were calculated and translated further upstream to Bathford. Table 2 summarises the results obtained for the historical flood events, including the historical water levels on Widcombe Bridge wall, and a comparison with previous estimates of peak flow for these events. The calculated peak discharge at St James is then presented for the two different Manning’s roughness numbers ($n = 0.035$ and $n = 0.045$), and then extended to Bathford within the same threshold envelope.

A total of 16 historical floods were compiled for which water levels have been recorded below the Halfpenny/Widcombe Bridge between January 1866 and December 1960. The five largest historical flood events for which estimates of peak flow have been reported in previous studies are the floods of October 1882, 13th and 15th November 1894, March 1947 and December 1960 (Greenhalgh, 1974; Reed, 1988). As previously discussed, the hydraulic 1D models were calibrated using the March 1947 and December 1960 events, and therefore the three remaining events of October 1882 and November 1894 events were used for validation. Given the uncertainty in the previous peak flow estimates it was assumed in the analysis that the peak flow values of these historical flood events were not known exactly. However, the previous peak flow estimates for the floods of October 1882 (366.7 m$^3$/s), 13th November 1894 (341.6 m$^3$/s) and 15th November 1894 (375.0 m$^3$/s) were found to be within the proposed envelope threshold (353.00–429.04, 340.85–416.75 and 371.54–406.94 m$^3$/s respectively). To observe that the modelled peak flows are in close agreement with the previous estimates, that were also used in the design of the BFDS, gives confidence in the hydraulic model and credibility to the results.

There is no period of overlap between recordings at St James gauge station and the newer Bathford gauging station. When running the extended hydraulic model from
Bathford to St James, the hydrograph shape was not significantly affected by the floodplains upstream of the city centre which contradicts one of the conclusions of Reed (1988). While this could be considered a limitation and further work might be required in investigating the effect of the different hydrograph widths in this case, for the aim of this research, the use of a 1D hydraulic model was able to provide a considerably accurate representation of the different historical flood events matching the peak flows and historical water levels and provided further insight into the hydraulic modelling of such events.

In the final phase of this research the peak flow values of the historical floods were estimated at the Bathford gauging station. Thus, the methodology described in Section 3.3 was effectively applied into augmenting the existing AMS of peak flow recorded at Bathford gauging station prior to its start date in 1969. Figure 14 shows the AMS of peak flow data at Bathford combined with the estimated historical flood events, including the upper and lower envelope thresholds. All the translated results from St James to the equivalent peak flows at Bathford (red dots) showed that all flood...
TABLE 2  Peak flow reconstruction with range between lower and upper envelope threshold, dependent on Manning's $n$ value

| Flood event (year [month]) | Historical water levels on Widcombe Wall (m) | Previous peak flow estimates (m$^3$/s) | 1947 calibration (Manning's $n$ 0.045) | 1960 calibration (Manning's $n$ 0.035) | Extended data series (m$^3$/s) |
|----------------------------|--------------------------------------------|--------------------------------------|-------------------------------------|-------------------------------------|-------------------------------|
|                            |                                            |                                      | Average                             | Average                             | Bathampton (Manning's $n$ 0.045) | Bathampton (Manning's $n$ 0.035) |
| 1866 (Jan)                 | 18.92                                      | 206.0                                | 208                                 | 227                                 | 245                           | 214                           | 256                           |
| 1867 (Mar)                 | 19.18                                      | 228.0                                | 232                                 | 254                                 | 275                           | 238                           | 283                           |
| 1875 (Jul)                 | 18.09                                      | 218.0                                | 221                                 | 241                                 | 260                           | 228                           | 270                           |
| 1882 (Oct)                 | 20.51                                      | 366.7                                | 353                                 | 391                                 | 429                           | 368                           | 439                           |
| 1888 (Nov)                 | 18.89                                      |                                      | 203                                 | 222                                 | 241                           | 210                           | 248                           |
| 1889 (Mar)                 | 19.55                                      | 264.0                                | 265                                 | 291                                 | 318                           | 273                           | 327                           |
| 1894 (Nov 13th)            | 20.39                                      | 341.6                                | 341                                 | 379                                 | 417                           | 359                           | 425                           |
| 1894 (Nov 15th)            | 20.64                                      | 375.0                                | 372                                 | 407                                 | 442                           | 386                           | 455                           |
| 1897 (Feb)                 | 18.41                                      |                                      | 150                                 | 163                                 | 177                           | 151                           | 185                           |
| 1900 (Dec)                 | 19.96                                      | 302.0                                | 300                                 | 331                                 | 363                           | 308                           | 372                           |
| 1900 (Feb)                 | 19.30                                      | 239.0                                | 243                                 | 266                                 | 289                           | 250                           | 296                           |
| 1903 (Jun)                 | 18.70                                      |                                      | 189                                 | 204                                 | 219                           | 190                           | 227                           |
| 1925 (Jan)                 | 19.46                                      | 255.0                                | 258                                 | 283                                 | 308                           | 265                           | 314                           |
| 1947 (Mar)                 | 20.51                                      | 287.7                                | 288                                 | 316                                 | 345                           | 296                           | 354                           |
| 1960 (Dec)                 | 20.48                                      | 424.4                                | 352                                 | 388                                 | 424                           | 361                           | 433                           |

FIGURE 14  Gauged annual maximum series (AMS) of peak flow at Bathford station (1969-present) and AMS at St James station (1939–1968) (Reed, 1988). The AMS at Bathford is therefore extended using the values from St James' station and the modelled historical floods with upper and lower envelope threshold for the River Avon at Bath, United Kingdom.
events had larger peak flows at Bathford than St James. This could be explained by the effect the floodplains have on the flow and reduce the peak by the time it arrives at St James.

5 | DISCUSSION

The benefits accrued from the use of historical data are demonstrated here through flood frequency analysis, focusing on the design flood with a return period of 1 in 100 years and the associated SD, following the procedure developed by Macdonald, Kjeldsen, Prosdocimi, and Sangster (2013). A generalised logistic (GLO) distribution is fitted to an AMS of peak flows using a maximum likelihood method that can accommodate series with and without the inclusion of data on historical flood events.

Table 3 summarises the outcome of the flood frequency analysis conducted on three combinations of AMS of peak flow derived in this study:

1. The contemporary AMS of peak flows recorded at the gauging station located at Bathford (gauge 53,018) consisting of 47 events (1970–2016).
2. The combined AMS of peak flows from gauge 53,018 and the adjusted series from St James (gauge 53,003); a combined record of 77 events covering the period 1940–2016.
3. The combined records (gauges 53,018 and 53,003) as well as historical flood events from 14 water years derived from the result in Table 2.

For the historical floods, the estimates of peak flow are assumed known with values corresponding to the average of the estimates for different values of Manning’s n were used. Finally, a perception threshold of 150 m$^3$/s was adopted (see Macdonald et al., 2013 for details).

Comparing the outcome of the flood frequency analysis on the three different subsets of data reported in Table 3 it is noticeable how relatively low the 1 in 100 year flood ($x_{100}$) is when based on only the most recent events (302 m$^3$/s) and that inclusion of events from a more distant past results in an increase between 20 and 30% (385 m$^3$/s). This is a substantial increase with potential important implications for hydraulic design and spatial planning. This increase is reflecting that the contemporary record from 1970 onwards (gauging station 53,018 only) does not contain any events of a magnitude comparable to the historical floods recorded in 1960 and earlier. Therefore, extrapolating from this dataset only will result in design floods that are relatively low when compared to the magnitude of the major historical events highlighted in Table 2. In fact, the impact of the large historical events on the resulting design flood estimates highlights the importance of including historical floods into contemporary design whenever possible to ensure that the design flood estimates reflect the true risk as much as possible. Interestingly, the SD increases when considering the combined record from the two gauges 53,018 and 53,003 (second row in Table 3) compared to using only data from the modern gauge 53,018; from 39 to 49 m$^3$/s. Under normal circumstances, an extension of the record would be expected to result in a reduction of the SD. However, in this case the change in the GLO distribution forced by the inclusion of the large events appear to have an even stronger influence. Adding the additional 14 historical events into the analysis (last row in Table 3) further reduces the uncertainty as expected. The reduction is moderate, reflecting that the combined record is already quite long (77 events).

This study developed a 1D hydraulic model representing the stretch of the River Avon through the city of Bath and used the model to assess the peak flow of historical flood events in the city based on flood marks. The model was constructed using data collected from a variety of sources, including engineering drawings, technical reports, water level charts, and physical markings of historical water levels at key locations in the city. Identification and translation of this material into a unified format required for developing a 1D hydraulic model was a major undertaking, at times relying on serendipity. If studies of historical flood, such as this one, are to be conducted for other locations, it is recommended that a central repository of this information is created to avoid

| AMS dataset       | Number of events | GLO parameters | 1 in 100 year event | SD       |
|-------------------|------------------|----------------|---------------------|----------|
|                   |                  | Location $\mu$ | Scale $\alpha$     | Shape $\kappa$ | $x_{100}$ (m$^3$/s) | $SD(x_{100})$ (m$^3$/s) |
| 53,018            | 47               | 168.64         | 22.82               | -0.10     | 302              | 39                  |
| 53,018 + 53,003   | 77               | 158.77         | 29.25               | -0.21     | 385              | 49                  |
| 53,018 + 53,003 + historical | 91   | 140.24         | 25.64               | -0.26     | 370              | 45                  |

Abbreviations: AMS, annual maximum series; GLO, generalised logistic.
future duplication of significant efforts to collate the required information. There is a certain urgency implied as the type of information required may be considered of little or no importance to current management, and therefore not deemed a priority for preservation efforts. However, as demonstrated above (and by other researchers) the inclusion of historical flood data can have a dramatic effect on the outcome of a flood frequency analysis (e.g., Engeland, Aano, Steffensen, Støren, & Paasche, 2020; Longfield et al., 2019; MacDonald et al., 2013; Prosdocimi, 2018; Wilhelm et al., 2019).

The hydraulic model was calibrated on two flood events occurring in 1947 and 1960, both prior to the construction of the BFDS. The calibration was based primarily on adjusting the value of Manning’s $n$ of the main channel, but also an assessment of floodplain inundation based on evidence from historical photographs. The calibration resulted in a range of credible values of Manning’s $n$ ranging from 0.035 to 0.045. The validation on three further historical events (1882 and 13th and 15th November 1894) where previous peak flow estimates exit considered an envelope of peak flow estimates by repeating the simulations using the different values of the Manning’s $n$. In all three cases, the previous peak flow estimates fall within the envelope. While this should not be considered a formal uncertainty analysis, it provides some confidence that the historical model is a credible representation of the main flood hydraulics of the river before the construction of the BFDS. Further research could consider in more detail the uncertainty involved in estimating values of Manning’s $n$ based on historical flood events.

Interestingly, even though the 1882 flood was considered to be one of the worst flood in the history of the city, and the BFDS was designed to defend against such peak flows, this research calculated that the 1894 flood of November 15th to have had a higher peak flow.

6 | CONCLUSIONS

This paper shows that the approach to the historical peak discharge calculation presented can be successfully applied and was effectively used to augment an already existing AMS of peak flow. This potentially allows an assessment of long-term trend or shifts in flood risk of the city and paves new avenues towards the inclusion of historical peak flows in future quantitative flood risk assessments. Specifically, the study concludes that:

1. It is possible to use historical information collated from a variety of historical sources to create a credible hydraulic model representing the hydraulic conditions of the River Avon prior to the implementation of the BFDS.
2. Using only the Manning’s roughness $n$ as a calibration parameter, the hydraulic model was found to provide a range of possible peak flow estimates of historical floods.
3. A flood frequency analysis of annual maximum peak flow data, and including the historical floods derived in this study, demonstrated that design floods on the river might be 20–30% higher than estimates based purely on the most recently available record.

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DATA AVAILABILITY STATEMENT

Data sharing is not applicable to this article as no new data were created or analyzed in this study.

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REFERENCES

Ahmadian, R., Falconer, R. A., & Wicks, J. (2018). Benchmarking 1-D/2-D linked inundation models. Journal of Flood Risk Management, 11, S314–S328. https://doi.org/10.1111/jfr3.12208

Bath in Time. (2019a). Retrieved from https://www.bathintime.co.uk

Bath in Time. (2019b). Pulteney Weir and Bridge 3 January 1956. Retrieved from https://www.bathintime.co.uk/pulteney-weir-and-bridge-3-january-1956-50571.html

Bath in Time. (2019c). The first vehicles over Churchill Bridge 1 October 1965. Retrieved from https://www.bathintime.co.uk/the-first-vehicles-over-churchill-bridge-1-october-1965-50595.html

Bath in Time. (2019d). The last days of the Old Bridge Bath 9 October 1964. Retrieved from https://www.bathintime.co.uk/the-last-days-of-the-old-bridge-bath-9-october-1964-50589.html

Bath in Time. (2019e). The New Weir on the River Avon and Pulteney Bridge Bath 1972. Retrieved from https://www.bathintime.co.uk/the-new-weir-on-the-river-avon-and-pulteney-bridge-bath-1972-17831.html

Bath & North East Somerset Council. (2011). Bath Avon River Economy. Retrieved from https://democracy.bathnes.gov.uk/
Bath history

Buchanan, R. A. (1998). Bath & North East Somerset Council (BATHNES). (2016). Bath

Elleder, L. (2010). Reconstruction of the 1784 flood hydrograph for

Davis, G., & Bonsall, P. (2006).

Chow, V. T. (1959).

CH2MHILL. (2012). ISIS technical summary. Retrieved from

Elleder, L., Herget, J., Roggenkamp, T., & Niessen, A. (2013). Historc floods in the city of Prague—A reconstruction of peak discharges for 1481-1825 based on documentary sources. Hydrology Research, 44, 202–214.

Engeland, K., Aano, A., Steffensen, I., Steren, E., & Paasche, Ø. (2020). New flood frequency estimates for the largest river in Norway based on the combination of short and long time series. Hydrology and Earth System Sciences, 24(11), 5595–5619.

Environment Agency. (2020). Historic flood map. Retrieved from Environment Agency, https://data.gov.uk/dataset/76292bec-7d8b-43e8-9c98-02734fd89c81/historic-flood-map

Environment Agency, & Black & Veatch. (2013). Bath flood modelling study: Hydraulic modelling technical note. Retrieved from https://www.bathnes.gov.uk/sites/default/files/sitedocuments/Planning-and-Building-Control/Planning-Policy/Evidence-Base/Flood-Risk/bath_frm_project_technical_note.pdf

Greenhalgh, F. (1974). Bath flood protection scheme, Bristol: Wessex Water Authority.

Herget, J., & Meurs, H. (2010). Reconstructing peak discharges for historic flood levels in the city of Cologne, Germany. Global and Planetary Change, 70, 108–116.

Jacobs. (2020). Flood modeller. Retrieved from https://www.floodmodeller.com/

Kjeldsen, T. R. (2015). How reliable are design flood estimates in the UK? Journal of Flood Risk Management, 8(3), 237–246.

Kjeldsen, T. R., Macdonald, N., Lang, M., Mediero, L., Albuquerque, T., Bogdanowicz, E., … Gül, G. O. (2014). Documentary evidence of past floods in Europe and their utility in flood frequency estimation. Journal of Hydrology, 517, 963–973.

Longfield, S. A., Faulkner, D., Kjeldsen, T. R., Macklin, M. G., Jones, A. F., Foulds, S. A., … Griffiths, H. M. (2019). Incorporating sedimentological data in UK flood frequency estimation. Journal of Flood Risk Management, 12(1), 1–19.

Lees, M. L. (1987). Inland water surveying in the United Kingdom. In 1985 yearbook, hydrological data UK series (pp. 35–48). Wallingford, England: Institute of Hydrology.

Lumbroso, D. M., Wyncoll, D. P., Liu, Y., & Davison, M. (2019). The challenges of including historical events using Bayesian methods to improve flood flow estimates in the United Kingdom: A practitioner’s point of view. Journal of Flood Risk Management, 12, 1–14.

Macdonald, N., & Black, A. R. (2010). Reassessment of flood frequency using historical information for the River Ouse at York, UK (1200–2000). Hydrological Sciences Journal, 55(7), 1152–1162.

Macdonald, N., Kjeldsen, T. R., Prosdocimi, I., & Sangster, H. (2013). Reassessing flood frequency for the Sussex Ouse, Lewes: The inclusion of historical flood information since AD 1650. Natural Hazards and Earth System Sciences, 14(10), 2817–2828.

Manco, J. (1993). Henry Savile’s map of Bath. Somerset Archaeology and Natural History, 136, 127–139.

Prosdocimi, I. (2018). German tanks and historical records: The estimation of the time coverage of ungaged extreme events. Stochastic Environmental Research and Risk Assessment, 32, 607–622.

Radecki-Pawlik, A., Pagliara, S., & Hradecky, J. (2017). Open channel hydraulics, river hydraulic structures and fluvial geomorphology: For engineers, geomorphologists and physical geographers. Boca Raton: CRC Press.

Reed, D. W. (1988). Design flood hydrographs for the Avon at Bathford. Retrieved from http://nora.nerc.ac.uk/id/eprint/14226/1/N014226CR.pdf

Roggenkamp, T., & Herget, J. (2014). Reconstructing the peak discharge of historic floods of the river Ahr, Germany. Erdkunde, 68(1), 49–59.

Saint-Venant, A. J. C. B. (1871). Théorie du mouvement non permanent des eaux, avec application aux crues des rivières et a l’introduction de marées dans leurs lits. Comptes Rendus de l’Académie des Sciences, 73, 147–154 237–240.

United States Geological Survey (1989). Measurement of peak discharge by the slope-area method. In T. Darlymple & M. A. Benson (Eds.), Applications of hydraulics. Reston: USGS.

Wilhelm, B., Ballesteros Cánovas, J. A., Macdonald, N., Toonen, W. H., Baker, V., Barriendos, M., … Glaser, R. (2019). Interpreting historical, botanical, and geological evidence to aid preparations for future floods. Wiley Interdisciplinary Reviews: Water, 6(1), e1318.

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