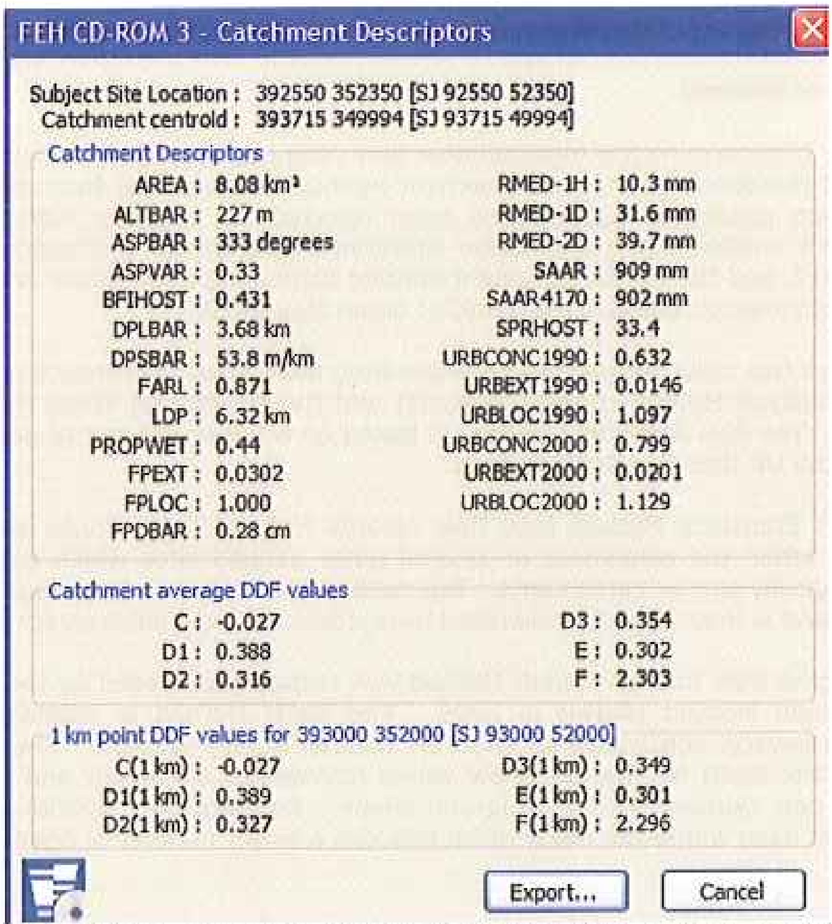


Figure 11: Catchment descriptors (Source: FEH CD-ROM Version 3)

5.1.4 $URBEXT_{2000}$ is based on a different methodology than $URBEXT_{1990}$ and therefore results in a separate set of FEH categories of urbanisation. For example, a moderately urbanised catchment will have an $URBEXT_{2000}$ value of up to 0.150 as opposed to 0.125 if using the former $URBEXT_{1990}$ value.

5.1.5 Urbanisation of the catchment since 2000 has been checked against the FEH CD-ROM values using OS mapping. The urban extent shown from the FEH CD-ROM ($URBEXT_{2000}$) is similar to the extent shown on the OS map. Therefore, as there has been no substantial development since 2000, the updating of $URBEXT_{2000}$ to 2014 using the national average model of urban growth in WINFAP-FEH Version 3 is acceptable. URBEXT has therefore increased from 0.0201 to 0.0207 and the catchment remains essentially rural.

6. ESTIMATION OF FLUVIAL FLOWS

6.1 Choice of Method

6.1.1 In order to determine the most suitable flow estimation method, the guidance outlined in the FEH Handbook and the Environment Agency's Operational Instruction entitled *Flood estimation guidelines* (2008), has been referred to, together with the EA guidance document entitled *Flood Estimation Guidelines Operational Instruction (197_08)* dated June 2012, and DEFRA/EA document entitled *Estimating flood peaks and hydrographs for small catchments: Phase 1 (SC090031)* dated May 2012.

6.1.2 There are two main approaches for estimating flood flows for catchments of this size; the FEH Statistical Method (pooled analysis) and the Revitalised Flood Hydrograph Method (ReFH). The FEH Statistical Method is based on a larger dataset of gauged flow records across the UK than the ReFH Method.

6.1.3 The FEH Statistical Method uses flow records from either a single reliable gauged site located within the catchment or several other gauged sites which are located in other hydrologically similar catchments. The method is based on a large flood event dataset in the UK and is more directly calibrated to reproduce flood frequency for UK catchments.

6.1.4 The original FEH Rainfall-Runoff Method was largely superseded by the Revitalised Flood Hydrograph Method (ReFH) in 2006. The ReFH Method is intended to update and address several constraints of the FEH Rainfall-Runoff method. The key changes are that in the ReFH Method baseflow varies throughout the event and the ReFH method uses a new (kinked) unit hydrograph shape. Furthermore, additional calibration data has been used within the ReFH which includes a larger number of flood events across the UK.

6.1.5 **Note:** In earlier guidance for small catchments below 25 km^2 the methodology outlined within the Institute of Hydrology Report 124 (IoH 124) was considered suitable, in which the mean annual flood flow QBAR is calculated. The recently published operational instruction *197_08* and science report *SC090031* discourages the use of the IoH 124 method for estimating flood flows in small catchments. The guidance recommends that FEH methods should be used in preference.

Stanley Pool

6.1.6 Stanley Pool is located approximately 430m upstream of the site and is an on-line, controlled, reservoir which will have attenuating affects on flood flows in the catchment (as denoted by a FARL value of 0.871). The relevant guidance suggests that if FARL is <0.9 and where flow records do not exist downstream of the reservoir, the QMED equation cannot be relied upon and the ReFH Method should be used.

6.1.7 Stanley Pool is a large storage reservoir built to supply water to the Caldon canal, and via that to the Trent & Mersey Canal. It is the smaller of two reservoirs which feed the summit pound of the Caldon canal, Rudyard being the larger reservoir.

6.1.8 The reservoir is under the overall control of the Canal and River Trust and it is understood from their response dated 14th August 2014 (Appendix A) that Stanley reservoir holds 610,980m^3 of water behind the dam and the surface area when full is 0.13km^2.

6.1.9 The Canal and River Trust state in their response that "*The feed to the canal is taken from the watercourse downstream of your site, via a side sluice in the watercourse* *where two large sluice can stop water flowing along the original stream. When the feed is not required or the natural flow along the watercourse is too large to send to the canal, this feed sluice is closed and the two larger sluices are opened to send all the water along the original course of the stream and under the canal itself".*

6.1.10 Additionally, it is stated in the Canal and River Trust's response that "*Once the reservoir is full, any additional water running into it is discharged over the spillway into the watercourse below the dam. Stanley reservoir is known to fill and spill quickly in storm events and this situation regularly occurs with the reservoir full and large uncontrolled flows going down the watercourse past your site. There are no actions available to mitigate this, and it is an entirely natural event. When the reservoir is spilling the feed valves are typically closed to restrict the flows in the watercourse to those that would have occurred naturally if the reservoir was not present.*

6.1.11 Catchments that include lakes and reservoirs delay and attenuate flood hydrographs, therefore it is recommended by the guidance that the critical storm duration is extended to incorporate the delay (RLAG). However, an outflow hydrograph was not available from the Canal and River Trust at the time of writing which makes the calculation of RLAG difficult.

6.1.12 Based on the above information provided by the Canal and River Trust, it is reasonable to assume a worst-case scenario whereby the reservoir is already full and additional catchment flood flows cannot be accommodated by the reservoir and spill from the reservoir into the watercourse without being diverted to the canal. This will also assume no RLAG and therefore all of the catchment flood flow would reach the site without being attenuated by the reservoir thus presenting a more conservative flood flow at the site.

6.1.13 Therefore, assuming no RLAG under these circumstances, the flood flow within the catchment and at the subject site can be calculated using the FEH Statistical Method rather than the ReFH method, as this is based on a larger dataset across the UK and uses good quality donor site data.

6.2 Improved Statistical Method

6.2.1 The original FEH Statistical Method has been improved with the release of the Science Report (SC050050/SR) entitled *Improving the FEH statistical procedures for flood frequency estimation,* carried out by the Centre for Ecology and Hydrology and published in 2008 by DEFRA and the EA.

6.2.2 As stated by the research document, the improved features include a new QMED (median annual flood) equation; an improved procedure for the formation of pooled growth curves; and a revised procedure for the use of donor catchments in the data transfer process. A new catchment descriptor which describes the floodplain extent (FPEXT) was also developed as part of the study to assist in the derivation of pooling groups.

6.2.3 The WINFAP-FEH Version 3 software incorporates all of these changes to the FEH Statistical Method and has therefore been used to assist in the flood estimation process.

6.2.4 There is no observed flow or level records available as the watercourse is ungauged at this location and the Agency has no spot gauging records. Therefore FEH Statistical Method single-site analysis is not possible. Consequently, estimation of the flood flows has been carried out using the catchment descriptor method and pooled analysis.

6.3 Estimation of QMED

6.3.1 To estimate QMED for the catchment, the catchment descriptor method has been used. This method is described in Volume 3, Chapter 13, of the FEH and has been updated in the Science Report. The method produces the mean annual flood QMED, which is the flood flow along the river that is statistically exceeded on average every other year.

6.3.2 The exercise can be done by hand using the catchment descriptors taken from the FEH CD-ROM and using the following improved QMED equation:

$$QMED = 8.3062 AREA^{0.8510} 0.1536^{\left(\frac{1000}{SAAR}\right)} FARL^{3.4451} 0.0460^{BFIHOST^2}$$

6.3.3 The QMED equation only applies to rural catchments ($URBEXT_{2000}$ <0.030) and as the catchment remains essentially rural, an urban adjustment to the QMED (rural) formula is not required.

6.3.4 The calculation using WINFAP-FEH based on catchment descriptors for the watercourse catchment gives a value for $QMED_{s,cds}$/QMED rural of 2.195 cu m/sec.

6.4 Revised Data Transfer Process

6.4.1 In order to make the ungauged rural estimates of $QMED_{s,cds}$ at the site more accurate, it is necessary to use flow data from a similar (rural) donor site either within the catchment, or in another catchment with similar hydrological characteristics, and where gauged information does exist for an adequate number of years. The suitability of the donor catchment will depend on how similar its catchment descriptors are to the subject catchment. For example, AREA should not differ by more than a factor of 5 and SAAR a factor of 1.1. Additional guidance is offered in the FEH Handbook.

6.4.2 A local correction or adjustment factor to the estimate of $QMED_{s,cds}$ at the subject site can then be applied. The procedure involves deriving QMED from the observed annual maximum record at a gauged site ($QMED_{g,obs}$), and also from the catchment descriptors at a gauged site ($QMED_{g,cds}$) and using the ratio of these two estimates to adjust the catchment descriptor estimate of $QMED_{s,cds}$ at the subject site.

6.4.3 The Science Report and Operational Instruction *197_08* also states that in addition to catchment similarity, the geographical proximity is important when considering the suitability of a donor site for the data transfer process, and the chosen donor should be the closest to the subject site. A new equation has therefore been developed and documented in the Science Report:

$$QMED_{s,adj} = QMED_{s,cds} \left(\frac{QMED_{g,obs}}{QMED_{g,cds}} \right)^{a_{sg}}$$

$$a_{sg} = 0.4598 \exp\left(-0.0200 d_{sg}\right) + \left(1 - 0.4598\right) \exp\left(-0.4785 d_{sg}\right)$$

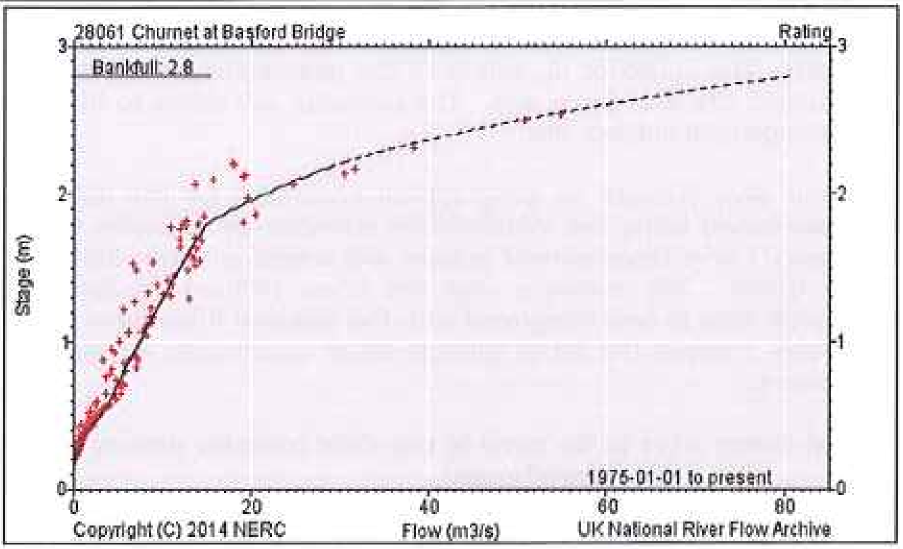
6.4.4 The subscript *s* refers to the ungauged subject site and *g* refers to the gauged donor site. The subscript *cds* refer to catchment descriptors and *obs* refers to the observed value at the donor site. The subscript d_{sg} refers to the geographical distance between the centroid of the subject site and donor site. The subscript *adj* refers to the adjusted value of QMED at the ungauged subject site.

6.4.5 A list of suitable donor sites (ranked by geographical proximity) for the data transfer process has been determined using the WINFAP-FEH software by following the *Pooled Analysis/Flood Frequency Curve Development* options and selecting *Donor Station* as the method to calculate QMED. The software uses the latest HiFlows-UK data (version 3.3.2) (Note: HiFlows-UK data is now integrated with the National River Flow Archive on the CEH website). Table 1 shows the list of suitable donor catchments as generated by the WINFAP-FEH software.

Table 1: List of potential donor sites to be used in the data transfer process for the catchment

Station	QMED donor	Centroid X	Centroid Y	Centroid distance (km)	AREA	SAAR	BFIHOST	FARL	URBEXT	Years of data	QMED AM	QMED cds
Subject site		393715	349994		8.08	909	0.431	0.871	0.021			
28061 (Churnet @ Basford Bridge)	2.058	396765	356765	7.43	136.34	976	0.442	0.927	0.029	37	28.011	33.705
28041 (Hamps @ Waterhouses)	2.276	405757	353971	12.21	36.97	1085	0.301	1	0.004	27	26.664	24.133
28018 (Dove @ Marston on Dove)	2.067	409533	349054	14.85	883.12	936	0.528	0.976	0.014	51	116.326	140.63
68005 (Dane @ Hulme Walfield)	2.274	394080	365348	15.36	149.89	1019	0.414	0.979	0.03	30	53.477	51.475
28031 (Manifold @ Ilam)	2.327	407654	356758	15.49	148.45	1099	0.455	1	0.003	44	66.727	56.18
68018 (Dane @ Congleton Park)	2.049	394491	365487	15.51	142.65	1030	0.408	0.979	0.023	57	41.679	51.1
68044 (Dane @ Hugbridge)	2.322	398633	367268	17.96	72.57	1160	0.373	0.997	0.001	19	48.634	40.03
28008 (Dove @ Rocester Weir)	2.24	412867	354827	19.75	397.97	1022	0.555	0.991	0.007	59	86.869	81.349
28002 (Blithe @ Hamstall Ridware)	1.87	402694	332292	19.85	162.11	752	0.454	0.996	0.027	15	17.52	29.419
28033 (Dove @ Hollinsclough)	1.919	404552	368179	21.13	7.93	1346	0.403	1	0	33	4.666	7.295

6.4.6 Reference to Table 1 shows that almost all suitable potential donor sites have catchment areas which are significantly higher than the subject site and significantly greater than the recommended limit as discussed in paragraph 6.4.1. Therefore, in this instance the chosen donor site should be the closest to subject site, and Station 28061, Churnet at Basford Bridge, which is ranked first in Table 1, is most acceptable in terms of its similarity and proximity to the subject catchment. The CEH website also indicates that this station is suitable for QMED.



Ref	Limb	Details	Equation	Start date	Max stage (m)	End date
1	a	Calculated	Q = 19.257 * (h -0.02457) ^ 2.550	01/01/1975	0.55	01/01/2100
1	b	Calculated	Q = 7.077 * (h + 0.04329) ^ 1.221	01/01/1975	1.79	01/01/2100
1	c	Calculated	Q = 2.405 * (h -0.05091) ^ 3.286	01/01/1975	2.22	01/01/2100
1	d	Calculated	Q = 1.916 * (h -0.1805) ^ 3.887	01/01/1975	2.8	01/01/2100

Figure 12: Rating Curve for Station 28061 (Source: CEH website NRFA data, accessed September 2014)

6.4.7 Reference to Table 1 shows that QMED for the gauged site based on observed records ($QMED_{g,obs}$) equates to 28.811 cu m/sec. QMED from catchment descriptors at the gauged site ($QMED_{g,cds}$) equates to 33.705 cu m/sec. The geographical distance between the sites (d_{sg}) equates to 7.43 km. The Science Report suggests that influence of the donor site reduces when the geographical distance between the centroids increases (typically above 75km). Therefore, by using a geographically closer donor site, there will be more of an influence on QMED at the subject site. Table 1 shows that the adjusted QMED value at the subject site, $QMED_{s,adj}$ using the new data transfer equation is 2.058 cu m/sec.

6.5 Pooled Analysis and Flood Growth Curve

6.5.1 In order to estimate a range of statistical flood return period events which will occur in the catchment, it is necessary to determine a flood growth curve and a flood frequency curve. This is done by forming a pooling group, which involves a group of gauged rural catchments across the UK which have very similar catchment characteristics such as AREA and SAAR.

6.5.2 The catchment output from the FEH CD-ROM is entered as a data file to the WINFAP-FEH software, which sorts a pooling group of similar catchments. The FEH states that the pooling group should contain 5 times as many station-years as the target return period (*5T*); however the Science Report recommends that a fixed pooling group size of at least

500 AMAX events for all required return periods should be used. The WINFAP-FEH Version 3 software incorporates the information and data gathered by the Agency's HiFlows-UK program version 3.3.2.

6.5.3 The recommended generalised logistic (GL) technique has been applied in the statistical analysis. The updated Statistical Method uses an enhanced procedure which no longer relies on pooling group ranking, but calculates separate weighting equations of the L-moment ratios within the pooling group based on record length. Weight is also applied to each catchment depending on distance in catchment space from the subject site, with more weight assigned to available "at site" data than the FEH procedure.

6.5.4 Stations that had been identified in the WINFAP-FEH software as not being suitable for pooling (as indicated by the HiFlows-UK data version 3.3.2), were removed from the pooling group and other more suitable stations added at the end of the pooling group to ensure that the total record length was at least 500 years.

Table 2: Pooling Group

Station	Distance	Years of data	QMED AM	L-CV	L-SKEW	Discordancy
27051 (Crimple @ Burn Bridge)	0.846	40	4.539	0.222	0.149	1.188
45816 (Haddeo @ Upton)	1.032	19	3.456	0.324	0.434	0.888
28033 (Dove @ Hollinsclough)	1.137	33	4.666	0.266	0.415	0.918
25019 (Leven @ Easby)	1.212	34	5.538	0.347	0.394	0.933
26802 (Gypsey Race @ Kirby Grindalythe)	1.294	13	0.109	0.261	0.199	0.562
25011 (Langdon Beck @ Langdon)	1.394	26	15.878	0.241	0.326	1.359
27010 (Hodge Beck @ Bransdale Weir)	1.464	41	9.42	0.224	0.293	0.173
44008 (Sth Winterbourne @ W'bourne Steepleton)	1.54	33	0.42	0.395	0.332	1.581
206006 (Annalong @ Recorder 1895)	1.583	48	15.33	0.189	0.052	2.301
22003 (Usway Burn @ Shillmoor)	1.66	26	19.22	0.303	0.303	0.589
71003 (Croasdale Beck @ Croasdale Flume)	1.667	35	10.9	0.206	0.331	0.689
25003 (Trout Beck @ Moor House)	1.708	39	15.164	0.176	0.291	0.764
203046 (Rathmore Burn @ Rathmore Bridge)	1.735	30	10.934	0.136	0.091	1.117
51002 (Horner Water @ West Luccombe)	1.762	31	8.354	0.382	0.326	1.741
48007 (Kennal @ Ponsanooth)	1.809	44	4.153	0.18	0.185	0.789
27032 (Hebden Beck @ Hebden)	1.843	46	4.082	0.211	0.258	0.408
Total		538				
Weighted means				0.254	0.271	

6.5.5 The WINFAP-FEH software indicates that the pooling group is strongly heterogeneous and a review of the pooling group is optional. All of the sites which are ranked are satisfactory in terms of their hydrological similarity with the subject site and the pooling group distribution provides an acceptable statistical fit. Removal or addition of extra sites was not justifiable and a representative, but heterogeneous, pooling group generally gives better flood frequency estimates, than either single site data or a pooling group that has been made homogeneous by inappropriately removing sites. The FEH also states that a significant proportion of pooling groups remain heterogeneous, even after a review and adapting a heterogeneous pooling group to make it homogeneous is not advised.

Institute of Hydrology - Flood Peaks Database
Printed : 3 September 2014
Station : 999200 (gb 392550 352350 (sj 92550 52350))

Growth Curve Fittings

Standardised by median

Pooled L-moments

L-CV: 0.254
L-skewness: 0.271

Fitted parameters

	Location	Scale	Shape	Bound
GL	1.000	0.252	-0.271	0.069

Return periods

	GL
2	1.000
5	1.424
10	1.758
20	2.136
50	2.742
100	3.303
200	3.976
500	5.082
1000	6.119

Figure 13: Flood Growth Curve Fittings

6.6 Flood Frequency Curve

6.6.1 The WINFAP-FEH software allows the user to generate a flood frequency curve for the specified return period based on the adjusted $QMED_{s,adj}$ value and growth curve fittings established during the pooling group stage and statistical analysis. The results can be seen on Figure 14.

Institute of Hydrology - Flood Peaks Database
Printed : 3 September 2014
Station : 999200 (gb 392550 352350 (sj 92550 52350))

Fittings for FFC

Standardised by median

Return periods

	GL
2	2.058
5	2.931
10	3.617
20	4.397
50	5.642
100	6.797
200	8.183
500	10.458
1000	12.594

Figure 14: Flood Frequency Curve Fittings (cu m/sec)

6.6.2 Applying 20% to the flows to accommodate the expected climate change effect over the next 100 years, as recommended by the Environment Agency and NPPF, the resultant flood flows can be seen in Table 3.

Table 3: Flood Flows (cu m/sec)

Flood Frequency	Q20	Q100	Q1000
Flood Flow	4.397	6.797	12.594
Flood Flow including climate change	5.276	8.156	15.113

6.7 Hybrid Method

6.7.1 Having determined that the FEH Statistical Method is preferred for estimating flood flows, a flow hydrograph is required for input into the hydraulic model, with a peak flow that matches the corresponding flood frequency estimate.

6.7.2 It is common to generate such a hydrograph using the ReFH Method, then scale it to match the FEH statistical flood flow estimates.

6.7.3 The catchment descriptors were imported into Version 11.5 of the InfoWorks modelling software. The appropriate flood return period, storm duration and data interval was set, as discussed below, to enable appropriate flows to be estimated.

6.7.4 The model parameters for the ReFH Method (time-to-peak, baseflow, and standard percentage runoff) should ideally be based on actual flood event data comprising rainfall and flow records rather than catchment descriptors alone. However, due to the lack of available rainfall and flow data for the catchment, the catchment descriptor method and ReFH design standards has been adopted in this instance based on the relevant technical guidance.

6.7.5 The critical storm duration was calculated as 4.980 hours from the time-to-peak (T_p) from catchment descriptors (2.609 hours) using the equation provided in Volume 4 of FEH:

$D = T_p\ (1 + SAAR/1000)$

Where:
D is the critical storm duration
T_p is the time-to-peak
SAAR is the standard average annual rainfall

6.7.6 In addition to the storm duration it is necessary to select an appropriate data interval. According to the FEH handbook (Volume 4) a data interval of 10-20% of the time-to-peak (T_p) is usually suitable so that the design flood hydrograph is well defined. A data interval of 1 hour was selected as a convenient and appropriate value which produced a smooth hydrograph.

6.7.7 The ReFH requires the user to have a design storm duration divided by the data interval which is an odd integer to ensure the use of an odd number of rainfall blocks in the storm profile. Therefore the design storm duration was rounded to 5 hours which is the nearest odd integer.

6.7.8 A 75% winter storm profile was used as the catchment is rural (N.B. urban catchments are defined as those with URBEXT >0.125 in the ReFH Method).

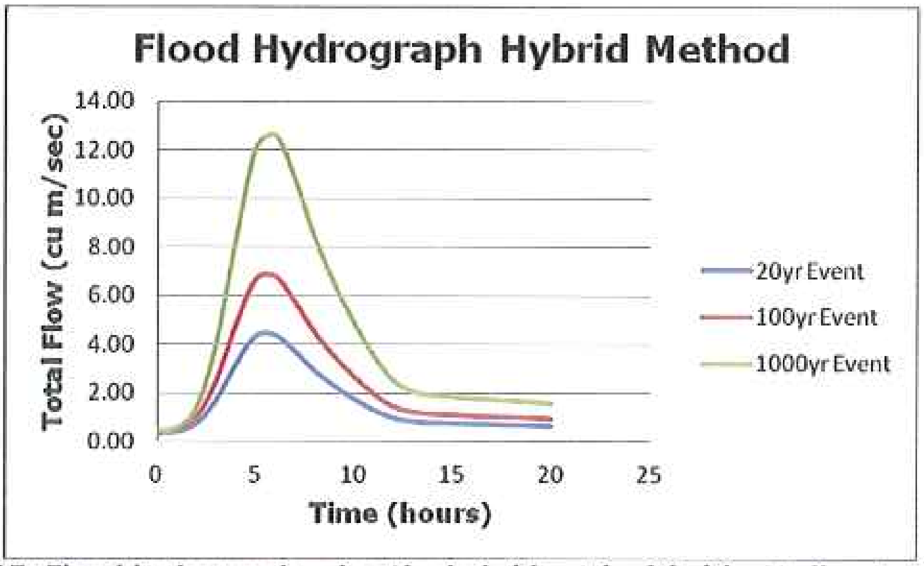


Figure 15: Flood hydrograph using the hybrid method (without climate change)

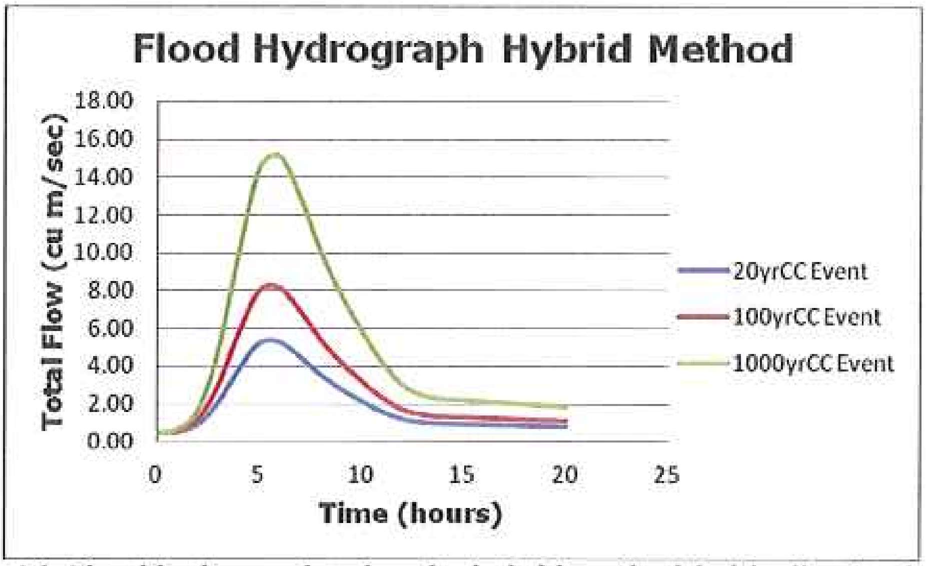


Figure 16: Flood hydrograph using the hybrid method (with climate change)

7. HYDRAULIC ANALYSIS

7.1 Introduction

7.1.1 A site specific assessment of the probability and consequences of the site flooding from the watercourse has been undertaken using well established hydraulic modelling and flood mapping techniques. The Agency's guidance document entitled *Fluvial Design Guide (2009)* has been consulted.

7.2 InfoWorks Model Development

7.2.1 One-dimensional (1D) unsteady hydrodynamic modelling of the watercourse and the study area was undertaken using the hydraulic modelling package InfoWorks RS Version 11.5. This software package combines the advanced ISIS Flow simulation engine and GIS functionality within a single environment.

7.2.2 The topographical survey was imported into MapInfo GIS software and a ground model was generated which allowed the interpolation of ground levels between available elevation points. Filtered LIDAR survey data was used to supplement the ground model in areas outside of the site boundary and therefore not covered by the topographical survey. The combined ground model (Figure 17) was then exported in a suitable format which could be read by the InfoWorks software. The final ground model as it appears in the InfoWorks model is shown on Figure 18.

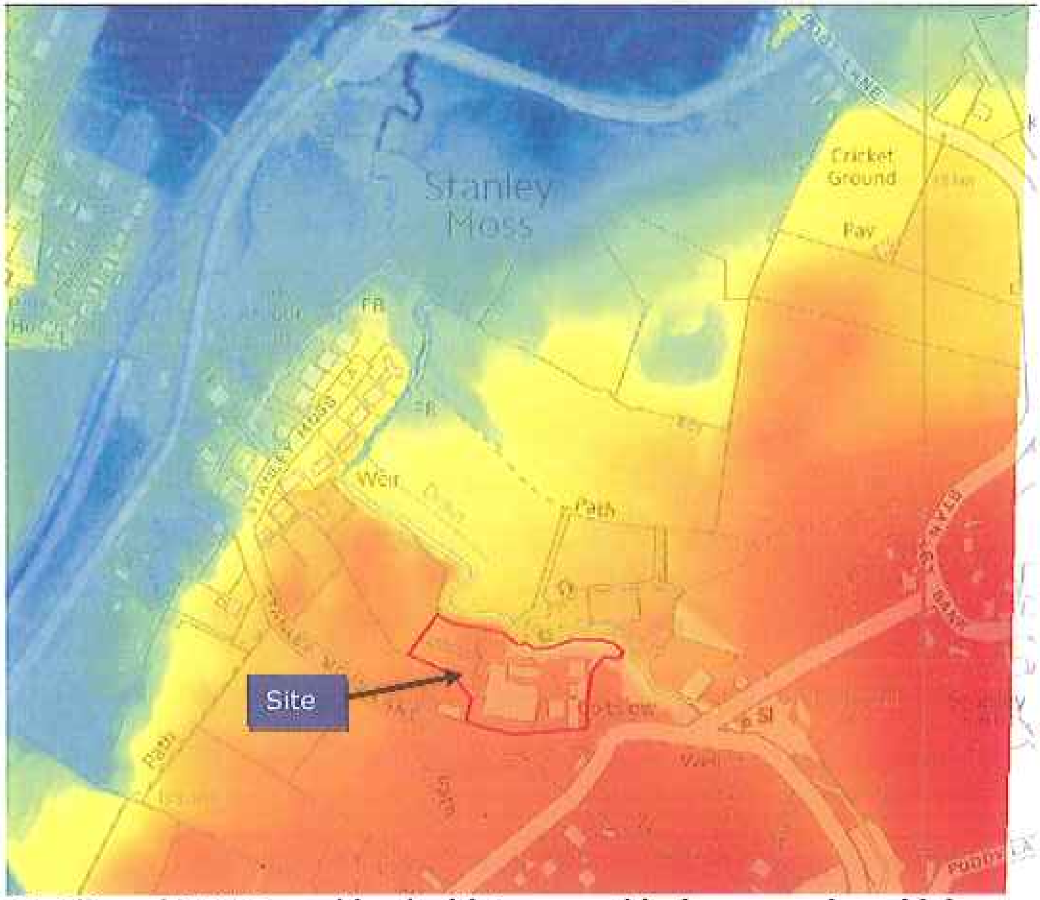


Figure 17: Filtered LIDAR combined with topographical survey where higher ground is represented by red and orange colours

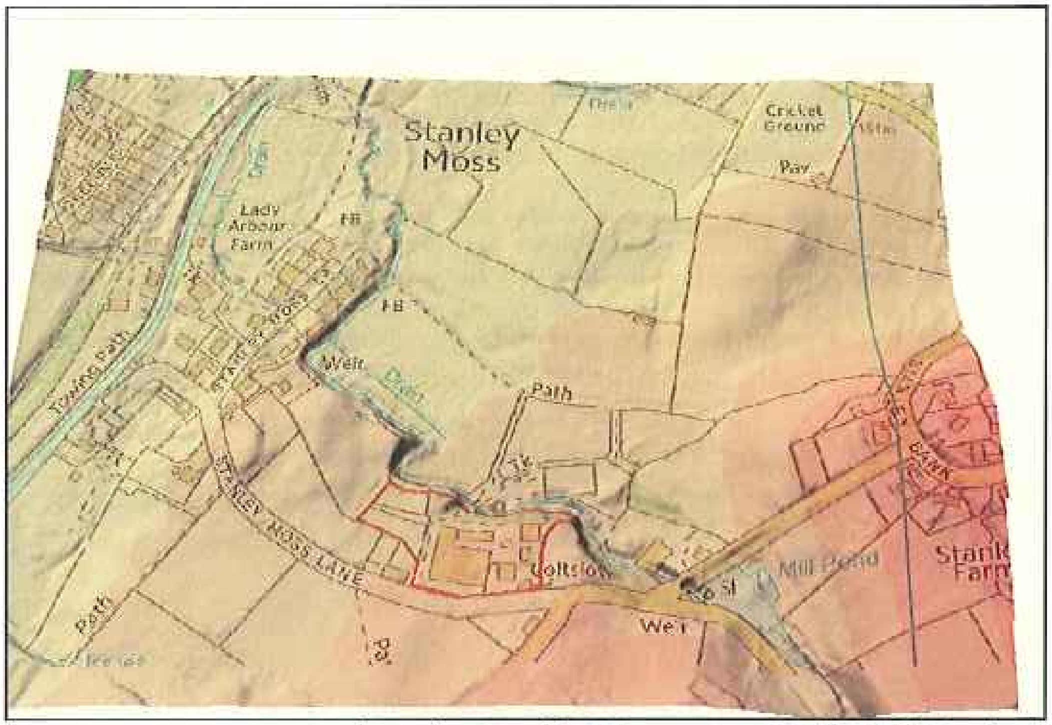


Figure 18: 3D representation of DTM with OS as presented in InfoWorks RS

7.2.3 Figure 19 shows that by forming a ground model which includes the topographical survey information, a more accurate and representative ground model can be generated in contrast to LIDAR alone.

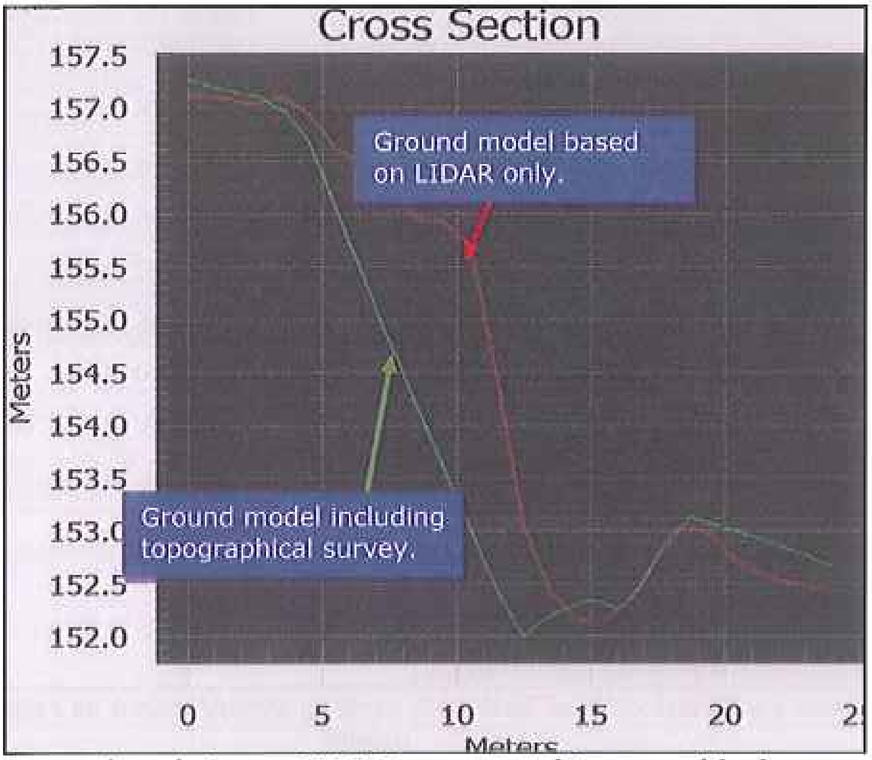


Figure 19: Comparison between LIDAR survey and topographical survey across the site when creating a ground model

7.2.4 The bridge crossing (Figure 5) which provides access from the site across the watercourse was included in the model by using an Arch Bridge unit. The rectangular shape of the bridge opening was modelled by specifying one value as the soffit height and spring height. The geometry of this structure, such as invert, sill, and soffit level was extracted from the topographical survey and site observations.

7.2.5 As the Arch Bridge unit does not model the potential overtopping of floodwater across the road deck, a Spill unit was applied perpendicular to the bridge and levels derived from the ground model. Levels were manually modified to include the raised stone wall as shown on Figure 6 as this would influence the ability of floodwater to overtop the bridge deck (although flows are permitted to outflank the bridge where the stone wall ends).

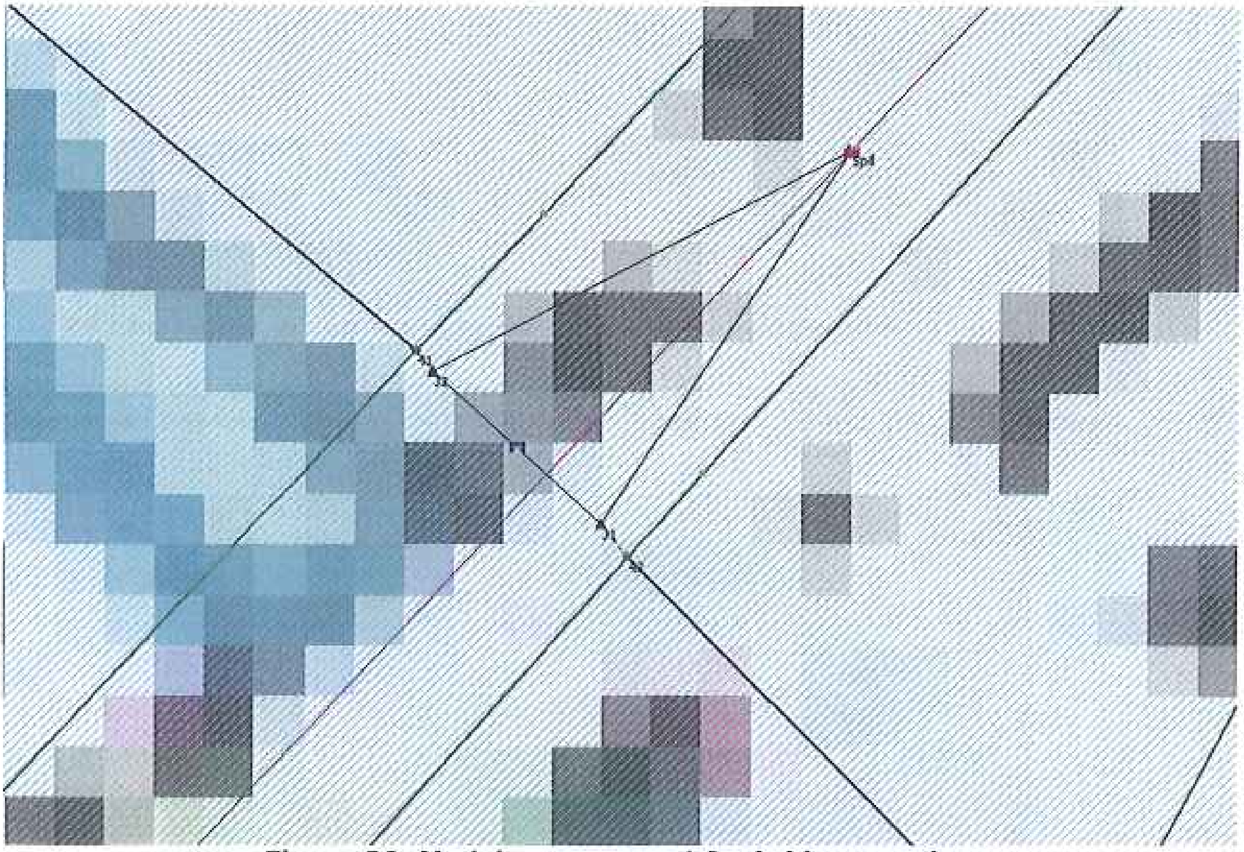


Figure 20: Model arrangement for bridge crossing

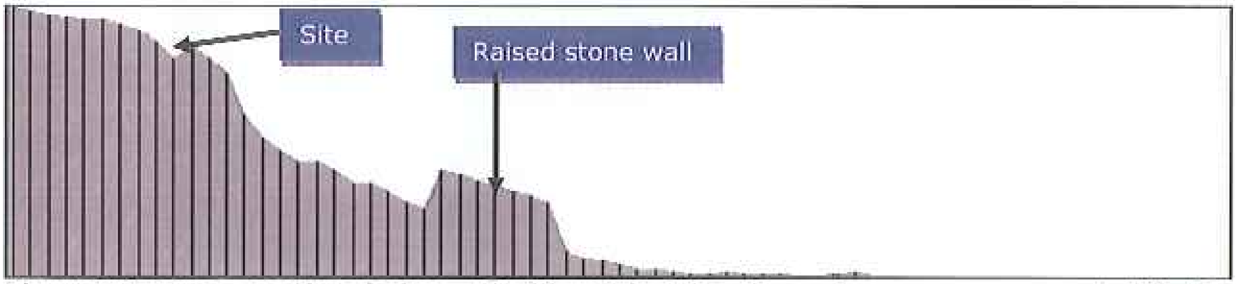


Figure 21: Cross sectional view of Spill unit looking downstream as represented in the model

7.3 Surface Roughness

7.3.1 Surface roughness varies across the study area as a result of different land uses, such as grassland, urban areas and channel vegetation. To ensure an accurate representation of the impact of different surface roughness values on the flood flows, information from the OS map and site observations was used. The anticipated roughness values were checked with the CES Roughness Advisor created by Wallingford Software and resultant Manning's "n" values were entered for each cross section.

	Zone Name	Com...	Type	Unit Roughness	Lower	Upper
	Bed		Bed	0.018	0.015	0.022
	Grass short		Floodplain	0.021	0.018	0.024
	Urban		Floodplain	0.02	0.018	0.022
	Grass high		Floodplain	0.08	0.07	0.09
	Trees		Floodplain	0.065	0.05	0.1

Figure 22: Manning's "n" roughness values derived from the CES Roughness Advisor

7.3.2 The watercourse channel is generally free from vegetation (Figure 3) however, despite the CES Roughness Advisor suggesting a channel roughness of 0.018 on Figure 22, a channel roughness of 0.035 has been used in the model instead of that shown on Figure 22 to consider possible vegetation growth during the summer months, or fallen bank vegetation.

7.4 Model Boundary Conditions

7.4.1 The following flood event scenarios have been modelled to allow the extent of the fluvial floodplain across the site to be determined and appraised in terms of NPPF:

1. 20yr event (present day Flood Zone 3b)
2. 20yr plus climate change event (future Flood Zone 3b)
3. 100yr event (present day Flood Zone 3a)
4. 100yr plus climate change event (future Flood Zone 3a)
5. 1000yr event (present day Flood Zone 2)
6. 1000yr plus climate change event (future Flood Zone 2)

Upstream Boundary

7.4.2 Having determined that the FEH Statistical Method is preferred for estimating flood flows, a flow hydrograph is required for input into the hydraulic model, with a peak flow that matches the corresponding flood frequency estimate.

7.4.3 It is common to generate a hydrograph using the ReFH Method, then scale it to match the statistical flow estimate as discussed in Section 6.7. This hydrograph then forms the upstream inflow boundary condition. It was ensured that the hydrograph parameters, shape, duration, data interval and results for each return period determined in Section 6.7 were reproduced in the InfoWorks RS software.

7.4.4 The upstream model boundary was positioned downstream of Stanley Bridge Road in order to represent a worst-case scenario by ignoring any upstream flow restriction caused by the bridge.

Downstream Boundary

7.4.5 For the downstream boundary, the InfoWorks software allows the user to define a Normal/Critical Depth downstream boundary which generates a flow-head relationship based on the downstream slope (i.e. 1 in 50).

7.4.6 As discussed between the Environment Agency and the Client, the downstream extent of the model was limited to the point at which the watercourse flows under the Canal, after which the Environment Agency's flood model is understood to begin. Figure 23 shows the final model schematic as it appears in the InfoWorks software.

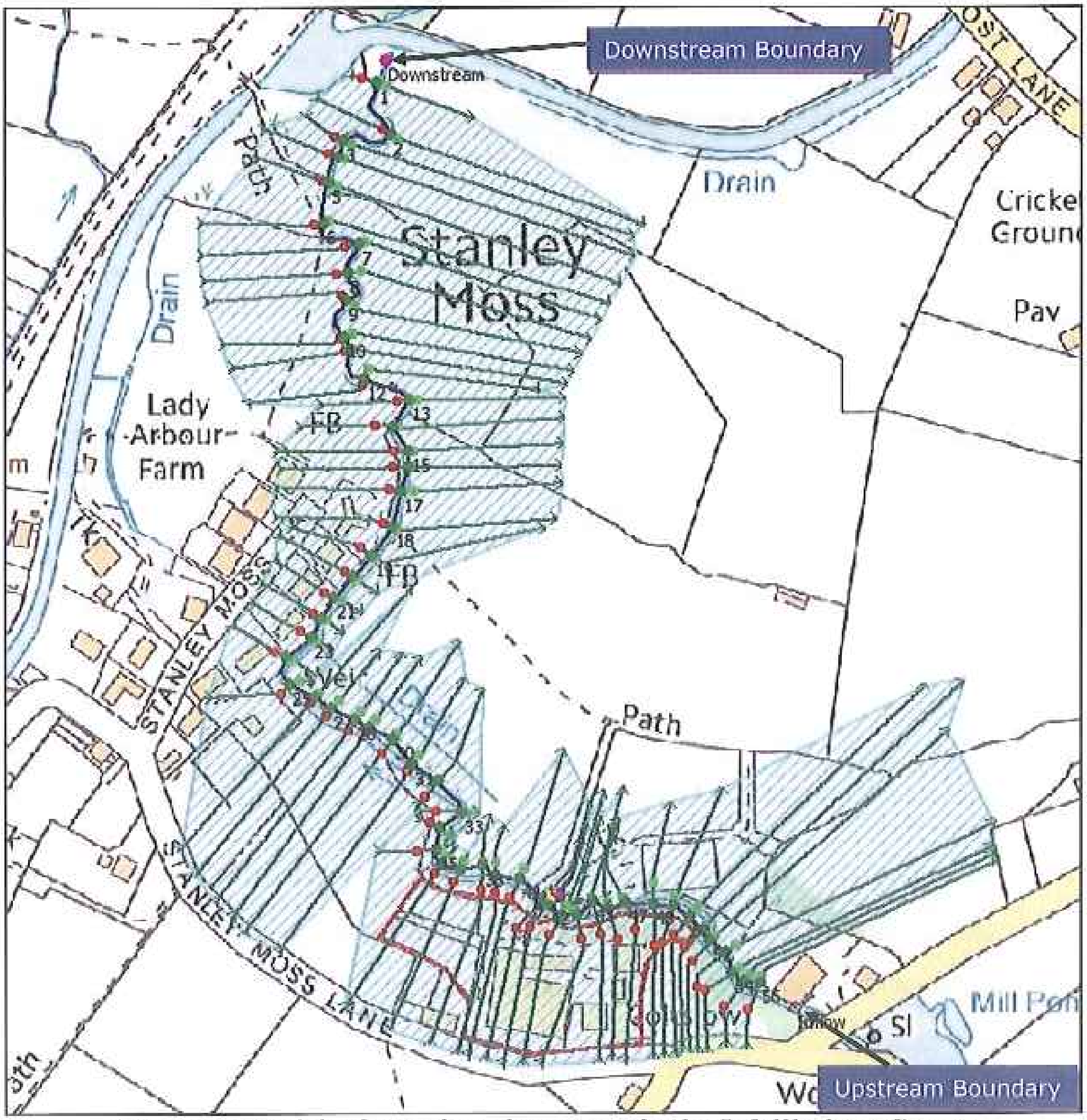


Figure 23: Model schematic as it appears in the InfoWorks software

7.5 Results

7.5.1 The model was initially run to consider the worst-case climate change 1 in 1000 year event, as this would allow the identification of any model instabilities and errors and the opportunity to correct them.

7.5.2 There were modelling instabilities and convergence issues when the model was run with right bank extended cross sections. This was a result of ground levels north of the watercourse being generally low-lying, flat and dropping below the bed level of the watercourse further north away from the watercourse. Therefore, cross sections were subsequently shortened to improve model stability as shown on Figure 23, and a minimum timestep of 0.1 seconds was used. Lastly, the model was run with a hydrological start time of 1 hour in order to prevent the channel from running dry at the beginning of the event.

Climate change 1 in 1000 year event

7.5.3 The results show that due to the elevated position of the site and surrounding area, floodwater does not reach a level high enough to inundate the site during the climate change 1 in 1000 year event as illustrated on Figures 24, 25 and 26. Table 4 shows that the maximum flood level adjacent to the site is 153.709m AOD at cross section 48, which is up to 5.79m AOD lower than the left top bank level and site. It is also noted on Figure 28 that flood flow is sufficiently accommodated by the stone bridge without overtopping of the bridge deck.

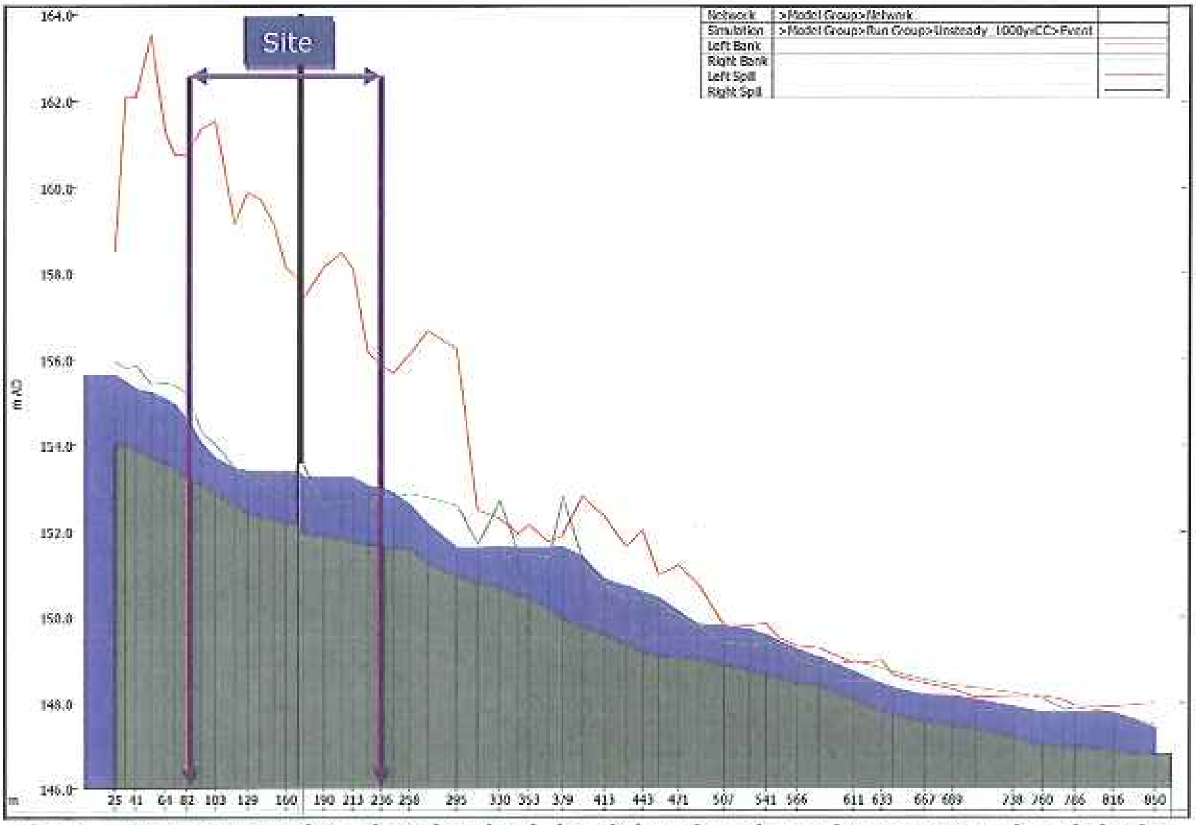


Figure 24: Long section showing bed, bank level and maximum water level during climate change 1 in 1000 year event

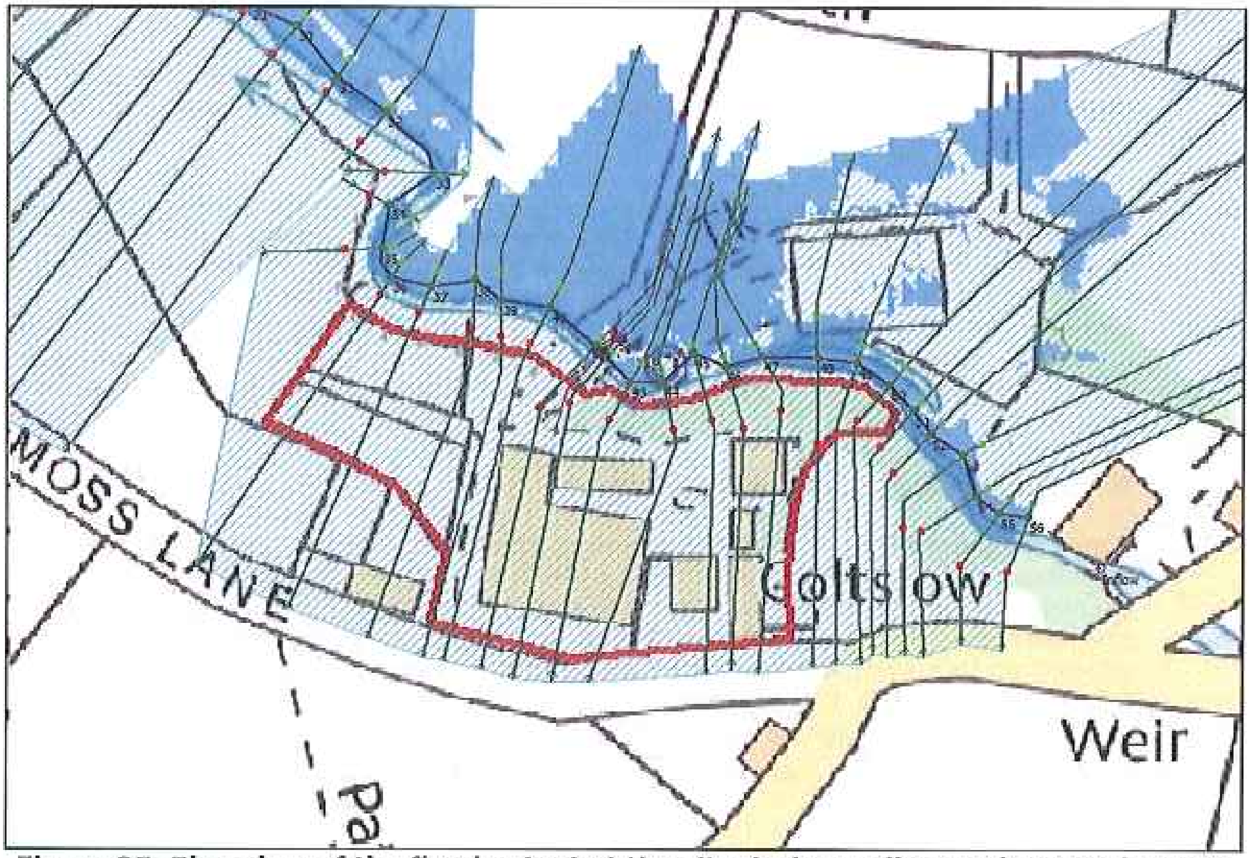


Figure 25: Plan view of the flood extent at the site during a climate change 1 in 1000 year event

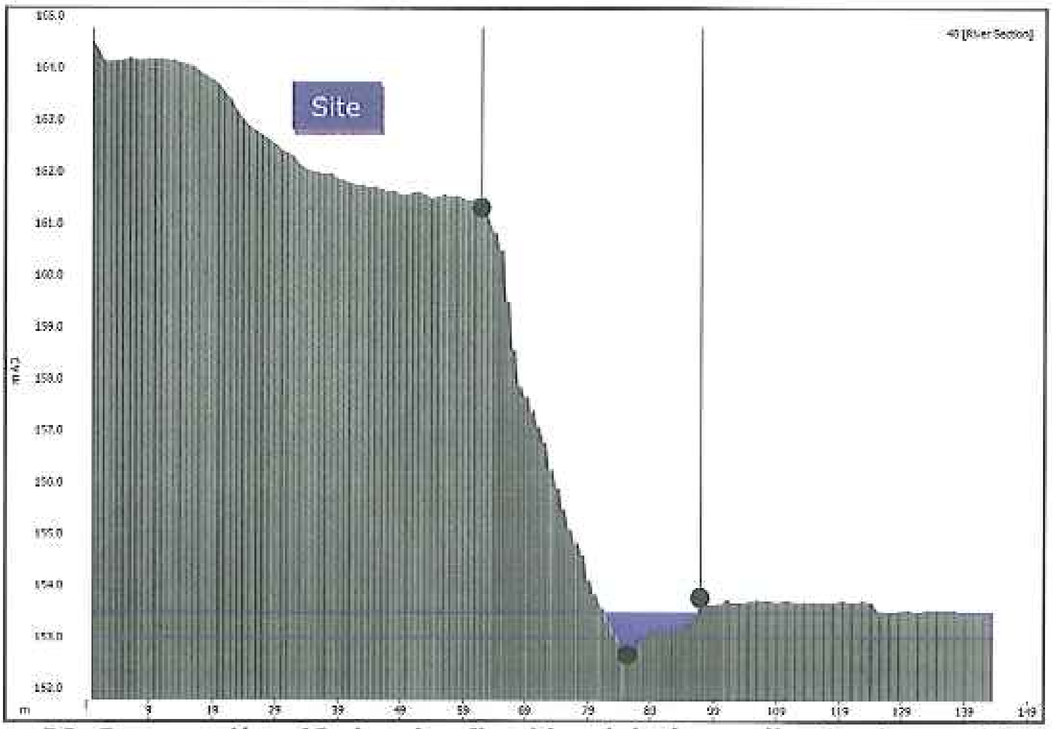


Figure 26: Cross section 48 showing flood level during a climate change 1 in 1000 year event

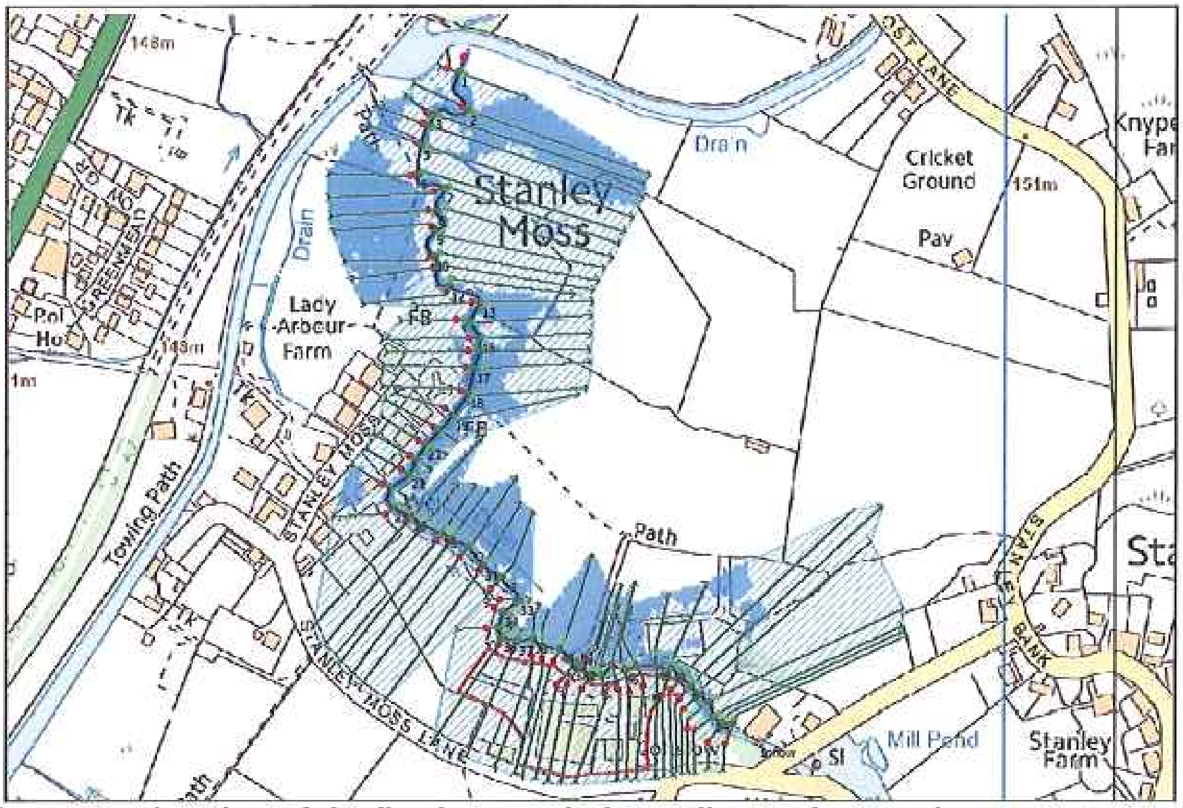


Figure 27: Plan view of the flood extent during a climate change 1 in 1000 year event

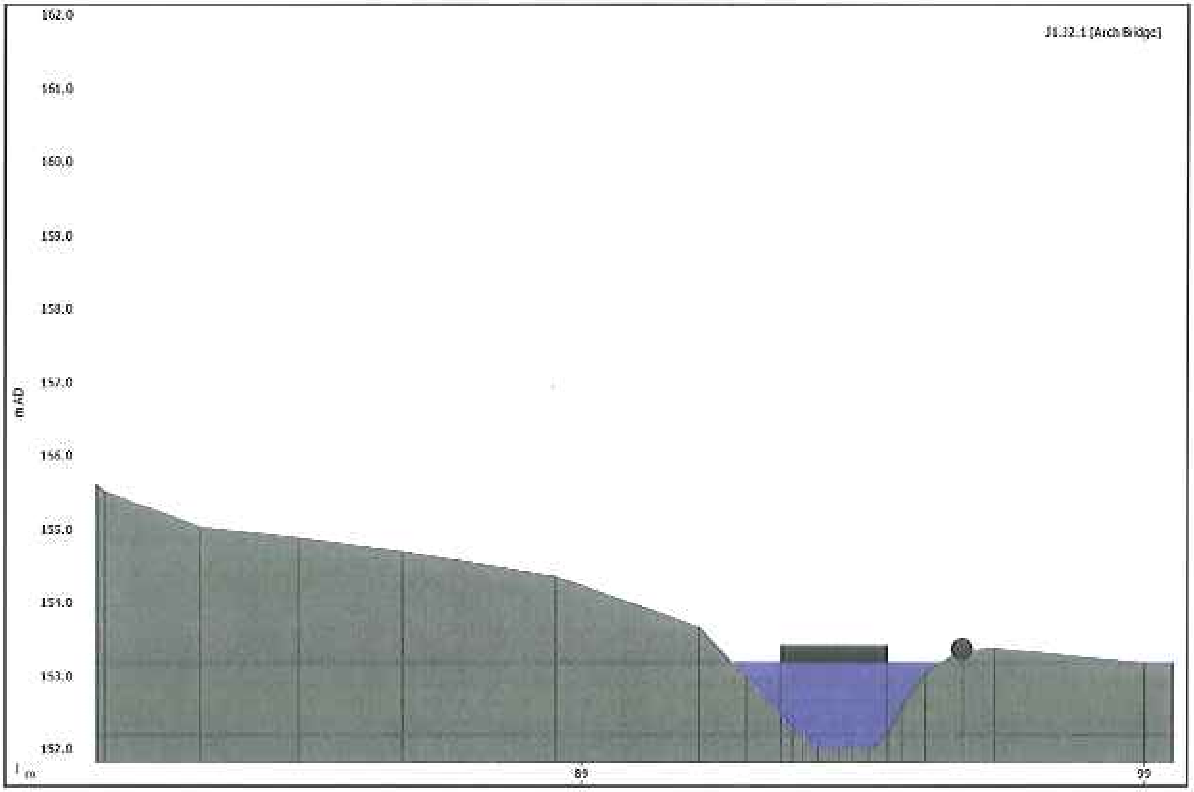


Figure 28: Cross section results for stone bridge showing flood level below the soffit of the bridge opening

Table 4: Results for the climate change 1 in 1000 year event (site results shown in red)

Cross Section	Results - 1000yrCC Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
56	15.113	155.643	2.136
55	15.113	155.484	2.912
54	15.112	155.325	2.207
53	15.112	155.23	1.835
52	15.111	155.093	2.556
51	15.111	154.97	2.053
50	15.11	154.607	3.582
49	15.11	154.07	3.429
48	15.108	153.709	2.4
47	15.107	153.476	1.172
46	15.106	153.419	1.685
45	15.102	153.403	1.021
44	15.1	153.383	1.283
43	15.098	153.4	1.648
42	15.095	153.379	3.328
41	15.095	153.242	1.535
40	15.086	153.254	0.224
39	15.083	153.238	0.562
38	15.082	153.243	0.432
37	15.08	153.068	1.724
36	15.08	153.008	1.736
35	15.08	152.882	2.89
34	15.079	152.631	2.904
33	15.078	152.153	3.816
32	15.074	151.626	1.143
31	15.074	151.625	1.199
30	15.072	151.631	0.774
29	15.068	151.625	1.669
28	15.069	151.626	1.239
27	15.067	151.626	1.647
26	15.069	151.632	0.168
25	15.073	151.385	2.904
24	15.075	150.885	2.788
23	15.077	150.722	1.345
22	15.078	150.574	2.161
21	15.079	150.448	2.077
20	15.079	150.121	3.189
19	15.08	149.828	1.268
18	15.081	149.797	0.949
17	15.083	149.706	1.265
16	15.082	149.576	1.886
15	15.081	149.44	1.868
14	15.08	149.24	1.484
13	15.08	149.067	1.324
12	15.08	148.724	2.023
11	15.081	148.431	1.505
10	15.08	148.352	1.393
9	15.08	148.189	0.977
8	15.079	148.148	0.749
7	15.078	148.094	0.848
6	15.074	147.921	1.468
5	15.072	147.779	1.113
4	15.065	147.773	1.296
3	15.063	147.778	0.607
2	15.062	147.776	0.473
1	15.061	147.439	2.312

Other Modelled Events

7.5.4 Despite the model showing no flooding across the site during the climate change 1 in 1000 year event, the other flood events outlined in paragraph 7.4.1 were modelled for consistency. The tabulated results are only shown hereafter as corresponding flood levels are lower than those derived for the climate change 1 in 1000 year event and would therefore not subsequently result in flooding of the site.

Table 5: Results relevant to the site location for remaining flood events

	Results - 20yr		
Cross Section	Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
50	4.396	154.35	3.626
49	4.396	153.785	2.585
48	4.396	153.499	1.454
47	4.395	153.344	0.678
46	4.396	153.241	1.706
45	4.396	153.136	1.014
44	4.395	153.078	1.274
43	4.395	153.053	1.421
42	4.394	153.03	2.752
41	4.395	152.876	1.559
40	4.393	152.883	0.389
39	4.394	152.879	0.688
38	4.392	152.88	0.203
37	4.392	152.843	0.903
36	4.392	152.825	0.761
	Results - 20yrCC		
Cross Section	Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
50	5.275	154.404	3.63
49	5.275	153.82	2.754
48	5.275	153.524	1.576
47	5.275	153.355	0.753
46	5.275	153.259	1.669
45	5.275	153.164	1.004
44	5.274	153.113	1.278
43	5.274	153.11	1.504
42	5.273	153.091	2.75
41	5.273	152.915	1.561
40	5.273	152.924	0.357
39	5.272	152.919	0.51
38	5.271	152.921	0.228
37	5.271	152.875	0.922
36	5.271	152.856	0.835
	Results - 100yr		
Cross Section	Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
50	6.793	154.461	3.665
49	6.793	153.877	3.001
48	6.794	153.562	1.76
47	6.794	153.374	0.864
46	6.794	153.289	1.669
45	6.794	153.211	1.009
44	6.793	153.158	1.28
43	6.793	153.171	1.48
42	6.793	153.153	2.76
41	6.793	152.968	1.564
40	6.797	152.977	0.371
39	6.805	152.969	0.532
38	6.809	152.972	0.275
37	6.819	152.906	1.04
36	6.829	152.879	1.032
	Results - 100yrCC		
Cross Section	Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
50	8.155	154.499	3.627
49	8.155	153.92	3.157
48	8.155	153.593	1.901
47	8.154	153.392	0.935
46	8.154	153.312	1.674
45	8.154	153.241	1.007
44	8.153	153.199	1.279
43	8.153	153.215	1.573
42	8.15	153.197	2.916
41	8.15	153.013	1.547
40	8.15	153.023	0.275
39	8.15	153.014	0.454
38	8.147	153.017	0.306
37	8.146	152.932	1.172
36	8.146	152.896	1.174
	Results - 1000yr		
Cross Section	Max Flow (m3/s)	Max Stage (m AD)	Max Velocity (m/s)
50	12.591	154.574	3.624
49	12.592	154.029	3.414
48	12.595	153.673	2.283
47	12.601	153.444	1.121
46	12.593	153.384	1.673
45	12.601	153.343	1.016
44	12.573	153.32	1.277
43	12.59	153.337	1.555
42	12.6	153.317	2.823
41	12.6	153.161	1.583
40	12.515	153.175	0.293
39	12.718	153.161	0.521
38	12.695	153.164	0.397
37	12.539	153.025	1.541
36	12.542	152.97	1.564

7.6 Sensitivity Analysis

7.6.1 Chapter 7 of the Agency's guidance document entitled *Fluvial Design Guide (2009)*, suggests that the model should be tested for sensitivity by adjusting key parameters such as the channel roughness values, downstream slope and flow rate.

7.6.2 In order to determine whether the model is sensitive when considering a particular parameter, each sensitivity test was carried out individually and as a separate model run. The sensitivity analysis has been carried out for the climate change 1 in 1000 year event within the watercourse as this will represent a worst-case scenario.

7.6.3 The channel Manning's roughness has been increased by 20% (i.e. from an already precautionary 0.035, to 0.042 in order to consider a higher density of channel vegetation).

7.6.4 The gradient of the downstream boundary slope has also been made shallower by 20% (i.e. from 1:50 to 1:60).

7.6.5 To model a 95% blockage of the bridge opening caused by lack of maintenance, debris or vegetation growth, a Blockage unit was placed before the Arch Bridge unit in the model and the blockage proportion set at 0.95.

7.6.6 The results in **Table** 6 show that when compared to the above modelling results, flood levels are variable but overall marginally higher when considering an increase in channel roughness.

7.6.7 Table 7 shows that there is a negligible increase in flood levels at the site when considering a shallower downstream slope which is to be expected as the downstream boundary is sufficiently downstream of the site.

7.6.8 It can be concluded that the model is overall not significantly sensitive to an increase in channel roughness or shallower slope and neither causes a significant increase in the flood extent.

7.6.9 Table 8 shows that when introducing a 95% blockage to the opening of the bridge, the upstream flood levels increase by up to 1.31m. The results show that despite this, the site remains sufficiently above the flood level of 154.689m AOD and therefore under these conditions the site would remain unaffected.

Table 6: Results comparison for increased "n" during climate change 1 in 1000 year event (site results shown in red)

Channel Manning's n = 0.042				Original Results			
Node	Max Stage (m AD)	Max Velocity (m/s)	Node	Max Stage (m AD)	Max Velocity (m/s)	Stage Difference (m)	
56	155.697	1.987	56	155.643	2.136	0.054	
55	155.526	2.673	55	155.484	2.912	0.042	
54	155.352	2.126	54	155.325	2.207	0.027	
53	155.253	1.76	53	155.23	1.835	0.023	
52	155.104	2.516	52	155.093	2.556	0.011	
51	154.975	2.017	51	154.97	2.053	0.005	
50	154.61	3.511	50	154.607	3.587	0.003	
49	154.095	2.999	49	154.07	3.429	0.025	
48	153.73	2.232	48	153.709	2.4	0.021	
47	153.479	1.162	47	153.476	1.172	0.003	
46	153.421	1.596	46	153.419	1.685	0.002	
45	153.406	0.989	45	153.403	1.021	0.003	
44	153.387	1.228	44	153.383	1.283	0.004	
43	153.403	1.546	43	153.4	1.648	0.003	
42	153.383	3.188	42	153.379	3.328	0.004	
41	153.261	1.498	41	153.242	1.535	0.019	
40	153.272	0.219	40	153.254	0.224	0.018	
39	153.257	0.55	39	153.238	0.562	0.019	
38	153.262	0.424	38	153.243	0.432	0.019	
37	153.092	1.662	37	153.068	1.724	0.024	
36	153.03	1.673	36	153.008	1.736	0.022	
35	152.908	2.53	35	152.882	2.89	0.026	
34	152.686	2.668	34	152.631	2.904	0.055	
33	152.215	3.415	33	152.153	3.816	0.062	
32	151.671	1.029	32	151.626	1.143	0.045	
31	151.669	1.142	31	151.625	1.199	0.044	
30	151.675	0.752	30	151.631	0.774	0.044	
29	151.669	1.625	29	151.625	1.669	0.044	
28	151.67	1.041	28	151.626	1.239	0.044	
27	151.67	1.359	27	151.626	1.647	0.044	
26	151.675	0.162	26	151.632	0.168	0.043	
25	151.417	2.642	25	151.385	2.904	0.032	
24	150.922	2.397	24	150.885	2.788	0.037	
23	150.697	1.22	23	150.722	1.345	-0.025	
22	150.628	1.906	22	150.574	2.161	0.054	
21	150.495	1.818	21	150.448	2.077	0.047	
20	150.152	2.827	20	150.121	3.189	0.031	
19	149.844	1.155	19	149.828	1.268	0.016	
18	149.825	0.891	18	149.797	0.949	0.028	
17	149.726	1.177	17	149.706	1.265	0.02	
16	149.598	1.641	16	149.576	1.886	0.022	
15	149.461	1.696	15	149.44	1.868	0.021	
14	149.249	1.41	14	149.24	1.484	0.009	
13	149.075	1.267	13	149.067	1.324	0.008	
12	148.731	1.961	12	148.724	2.023	0.007	
11	148.433	1.488	11	148.431	1.505	0.002	
10	148.355	1.37	10	148.352	1.393	0.003	
9	148.193	0.959	9	148.189	0.977	0.004	
8	148.151	0.739	8	148.148	0.749	0.003	
7	148.093	0.849	7	148.094	0.848	-0.001	
6	147.93	1.315	6	147.921	1.468	0.009	
5	147.807	0.992	5	147.779	1.113	0.028	
4	147.81	1.098	4	147.773	1.296	0.037	
3	147.812	0.542	3	147.778	0.607	0.034	
2	147.81	0.431	2	147.776	0.473	0.034	
1	147.474	2.058	1	147.439	2.312	0.035	

Table 7: Results comparison for increased "slope" during climate change 1 in 1000 year event (site results shown in red)

Channel slope = 1:60				Original Results		
Node	Max Stage (m AD)	Max Velocity (m/s)	Node	Max Stage (m AD)	Max Velocity (m/s)	Stage Difference (m)
56	155.643	2.136	56	155.643	2.136	0
55	155.484	2.913	55	155.484	2.912	0
54	155.325	2.207	54	155.325	2.207	0
53	155.23	1.835	53	155.23	1.835	0
52	155.093	2.556	52	155.093	2.556	0
51	154.97	2.053	51	154.97	2.053	0
50	154.607	3.582	50	154.607	3.582	0
49	154.07	3.381	49	154.07	3.429	0
48	153.709	2.4	48	153.709	2.4	0
47	153.476	1.173	47	153.476	1.172	0
46	153.419	1.693	46	153.419	1.685	0
45	153.403	1.021	45	153.403	1.021	0
44	153.383	1.283	44	153.383	1.283	0
43	153.4	1.648	43	153.4	1.648	0
42	153.379	3.328	42	153.379	3.328	0
41	153.242	1.535	41	153.242	1.535	0
40	153.254	0.224	40	153.254	0.224	0
39	153.238	0.562	39	153.238	0.562	0
38	153.243	0.433	38	153.243	0.432	0
37	153.068	1.724	37	153.068	1.724	0
36	153.008	1.736	36	153.008	1.736	0
35	152.882	2.89	35	152.882	2.89	0
34	152.631	2.904	34	152.631	2.904	0
33	152.153	3.816	33	152.153	3.816	0
32	151.626	1.143	32	151.626	1.143	0
31	151.625	1.143	31	151.625	1.199	0
30	151.631	0.773	30	151.631	0.774	0
29	151.625	1.669	29	151.625	1.669	0
28	151.626	1.155	28	151.626	1.239	0
27	151.626	1.647	27	151.626	1.647	0
26	151.632	0.168	26	151.632	0.168	0
25	151.385	2.911	25	151.385	2.904	0
24	150.911	2.785	24	150.885	2.788	0.026
23	150.716	1.345	23	150.722	1.345	-0.006
22	150.574	2.361	22	150.574	2.161	0
21	150.448	2.194	21	150.448	2.077	0
20	150.121	3.266	20	150.121	3.189	0
19	149.828	1.261	19	149.828	1.268	0
18	149.797	0.949	18	149.797	0.949	0
17	149.706	1.265	17	149.706	1.265	0
16	149.576	1.886	16	149.576	1.886	0
15	149.44	1.867	15	149.44	1.868	0
14	149.24	1.483	14	149.24	1.484	0
13	149.067	1.324	13	149.067	1.324	0
12	148.724	2.023	12	148.724	2.023	0
11	148.431	1.505	11	148.431	1.505	0
10	148.352	1.392	10	148.352	1.393	0
9	148.189	0.977	9	148.189	0.977	0
8	148.149	0.747	8	148.148	0.749	0.001
7	148.095	0.841	7	148.094	0.848	0.001
6	147.919	1.47	6	147.921	1.468	-0.002
5	147.769	1.163	5	147.779	1.113	-0.01
4	147.73	1.376	4	147.773	1.296	-0.043
3	147.74	0.62	3	147.778	0.607	-0.038
2	147.736	0.471	2	147.776	0.473	-0.04
1	147.466	2.117	1	147.439	2.312	0.027