

is driven by the inflow from the upper catchments, represented in the model by the observed flow at Merthyr Tydfil gauge. Inflows from the eight sub-catchments are relatively small in comparison.

The focus of the calibration has been on the general timing and peak flow in particular during Event 1, the largest of the five calibration events.

4.7.2.1 Inflow Distribution

The FF model represents the inflows from the various sub-catchments as one single inflow point per sub-catchment. This conceptual design has been modified by distributing the inflows along the main river according to the location of the main inflows. The 1:10K OS maps have been used for supporting information. Table 4.7 summarises the lateral inflows set up in ISIS to represent the inflows. This generally does not change the total computed flow at Fiddler's Elbow, but it provides a better starting point for the calibration of the hydraulic model in terms of water levels. This was reviewed during the calibration of the hydraulic model.

The inflow distribution has been kept unmodified for all events.

Table 4.7: Summary of Distribution of Lateral Inflows in ISIS

ISIS node label	Catchment	Lateral Inflow in ISIS
TAFF07639	Aberfan	Lat6
TAFFMC01941	TaffBargoed	Lat10
TAFFMC03109	TaffBargoed	Lat9
TAFF05703	TaffBargoed	Lat8
TAFF07141	Aberfan	Lat7
TAFF08606	Aberfan	Lat5
TAFF9922int	Troedyrhiw	Lat4
TAFF11048	Troedyrhiw	Lat3
TAFFMC12447	Troedyrhiw	Lat2
TAFFMC13841	Troedyrhiw	Lat1

4.7.2.2 Rainfall Adjustments

Generally it has been assumed that the derived catchment rainfall is a reasonable representation of the rainfall during the events. Changes have not been made unless there was sufficient evidence to do so.

The distribution of the catchment rainfall recorded during Event 3 shows very little rainfall in the downstream catchments. At Nant Wen Farm only 30 mm were recorded, while at Storey Arms 163 mm were recorded. Although this is not impossible, further checks have been carried out to ensure the catchment rainfall has been correctly represented.

Additional HYRAD recordings at Nant Wen and Nant Yr Ysfa during this event have been provided by the EA Wales (see Table 4.8). The HYRAD rainfall supports the suggestion that the recorded TBR rainfall does not represent the rainfall, and adjustments have been made based on the ratio of the TBR and HYRAD totals. The revised catchment rainfall totals for Event 3 are summarised in Table 4.9.

Table 4.8: Comparison of HYRAD and TBR Rainfall - 24 October 2004

Location	HYRAD	TBR
Nant Yr Ysfa	47t	53.6
Nant Wen	35.6	9.08t

Source: Environment Agency Wales

Table 4.9: Revised Rainfall – Event 3

Sub-catchment	Rainfall (mm)	
	Original	Modified
ABERFAN	30	106
BARGOED	45	157
DITCH	30	106
FIDDLERS	30	106
LLWYNON LAT	147	147
LLYWYNON	154	154
MERTHYR TOWN	152	152
NANT MORLAIS	147	147
PONTSTICILL	153	153
PONTSTICILL LAT	147	147
TAFF BARGOED	30	106
TROEDYRHIW	77	116

4.7.2.3 ReFH Catchment Parameters

Of the four parameters defining the ReFH model, modifications have been made to the maximum storage capacity (C_{max}) and the Time to peak (T_p).

Table 4.10: Adjustments to ReFH Parameters

Sub-catchment	C_{max}	T_p	Event 1		Event 2		Event 3		Event 4		Event 5	
			C_{max}	T_p	C_{max}	T_p	C_{max}	T_p	C_{max}	T_p	C_{max}	T_p
Aberfan	404	1.03	0.6	-	1.4	-	1.4	-	1.4	1.5	1.6	2
Bargoed	355	2.50	0.6	-	1.4	-	1.4	-	1.4	-	1.6	-
Ditch	420	0.61	0.6	-	1.4	-	1.4	-	1.4	-	1.6	-
Fiddlers	522	6.05	0.6	-	1.4	-	1.4	-	1.4	1.5	1.6	2
Merthyr Town	265	1.26	0.6	-	1.4	-	1.4	-	1.4	-	1.6	-
Nant Morlais	378	0.73	0.6	-	1.4	-	1.4	-	1.4	-	1.6	-
Taff Bargoed	551	2.02	0.6	-	1.4	-	1.4	-	1.4	1.5	1.6	2
Troedyrhiw	274	1.63	0.6	-	1.4	-	1.4	-	1.4	1.5	1.6	2

Source: MM

Modifications have also been made to the initial storage capacity C_{ini} .

For Event 2, and 4 the initial storage C_{ini} derived from the ReFH Design Standard Method has been reduced by 25%, and for Event 3 and 5 by 50% for all sub-catchments.

In order to derive a single set of parameters for the design model runs the parameters derived for the individual calibration events have been averaged. The design parameters are shown in Table 4.11. As there is no evidence that any of the events had either particularly reliable observed data or represents the catchments response to rainfall better than other events, equal weighting has been applied to the five events.

Table 4.11: Adjustment Factors to ReFH Parameters for Design Model Runs – Downstream Catchments

Sub-catchment	C _{max}	T _p	BR	C _{ini}
Aberfan	1.28	1.3	1	0.7
Bargoed	1.28	1	1	0.7
Ditch	1.28	1	1	0.7
Fiddlers	1.28	1.3	1	0.7
Merthyr Town	1.28	1	1	0.7
Nant Morlais	1.28	1	1	0.7
Taff Bargoed	1.28	1.3	1	0.7
Troedryhiw	1.28	1.3	1	0.7

Source: MM

4.7.2.4 Results of Calibration for Individual Events

Modifications of the ReFH model parameters have been necessary in order replicate the catchments' response to rainfall for the selected calibration events as described above. The modelled and observed peak flows are summarised in Table 4.12.

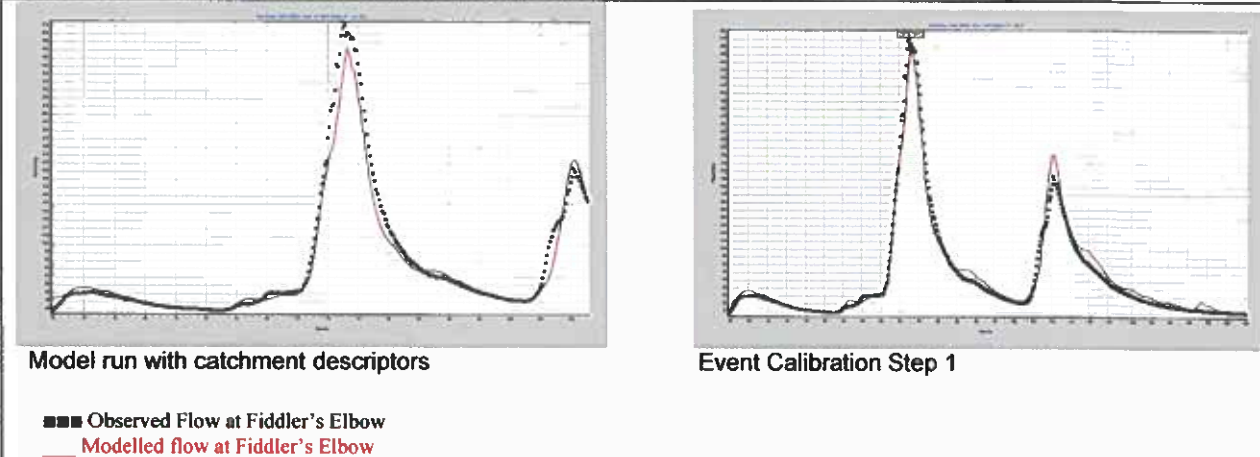
Figure 4.9 to Figure 4.13 show the results of the individually calibrated events (*'Event Calibration'*) together with a comparison of observed and modelled flow using the parameters as derived from the catchment descriptors (*'Model run with catchment descriptors'*).

Table 4.12: Comparison of Modelled and Observed Peak Flows

Table Heading Left	Observed peak flow (m ³ /s)	Modelled peak flow (m ³ /s)	Difference (Percentage of observed flow)
Event 1	364	371	+2%
Event 2	194	192	-1%
Event 3	154	149	-3%
Event 4	137	138	<1%
Event 5	259	257	<1%

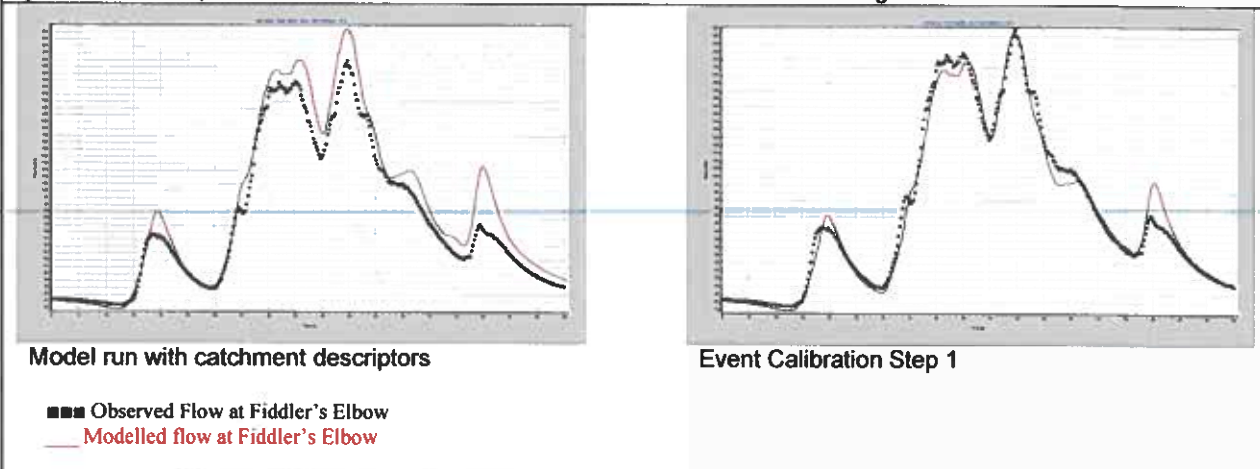
Source: MM

Figure 4.9: Comparison of Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 1



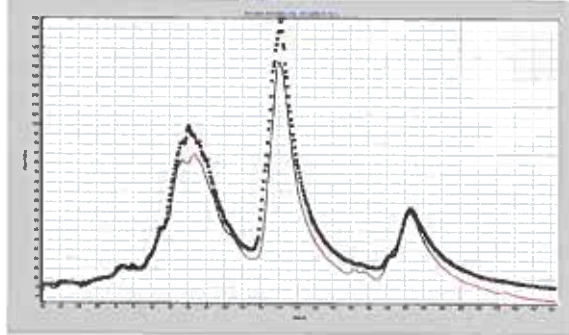
Source: MM

Figure 4.10: Comparison of Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 2

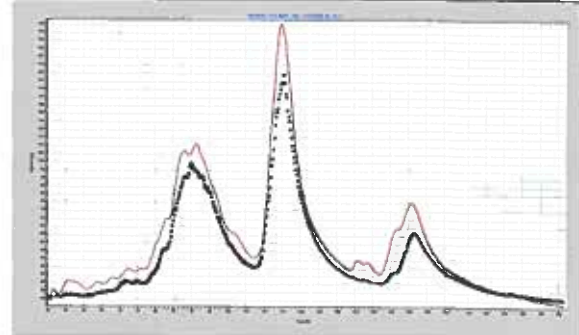


Source: MM

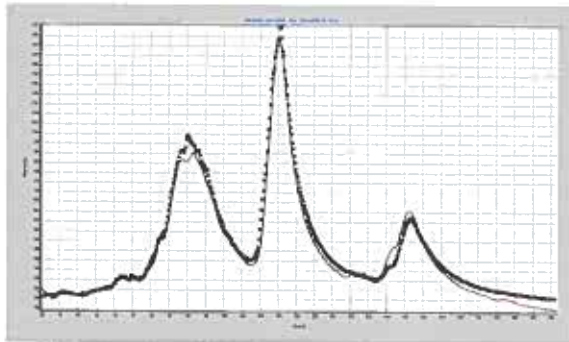
Figure 4.11: Comparison of Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 3



Model run with catchment descriptors
Original rainfall



Model run with catchment descriptors Original
Revised rainfall data



Event Calibration Step 1

■ ■ ■ Observed Flow at Fiddler's Elbow
— Modelled flow at Fiddler's Elbow

Source: MM

Figure 4.12: Comparison of Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 4

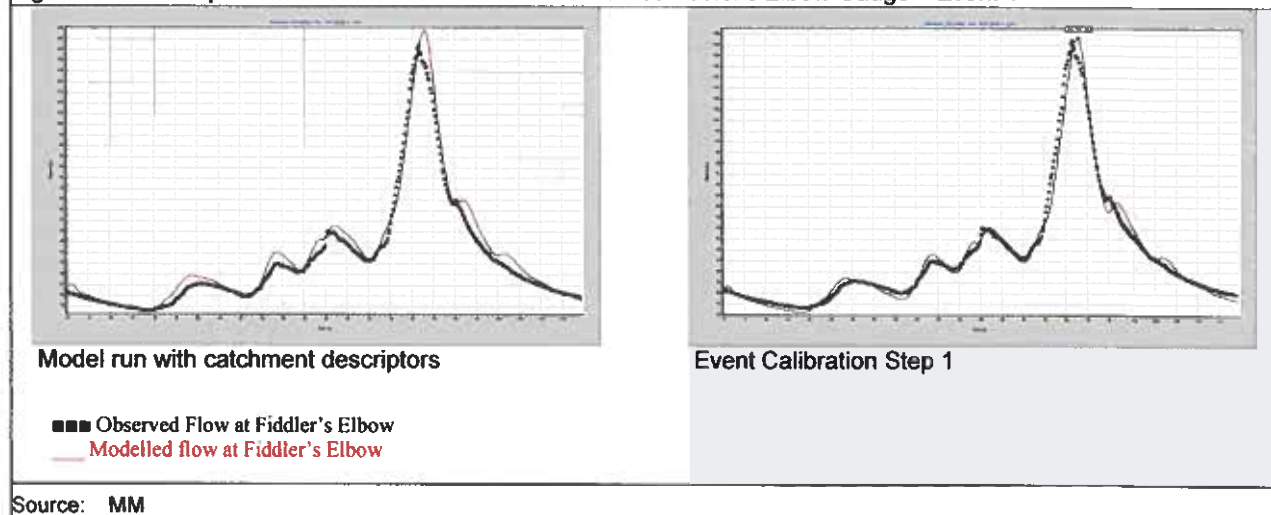
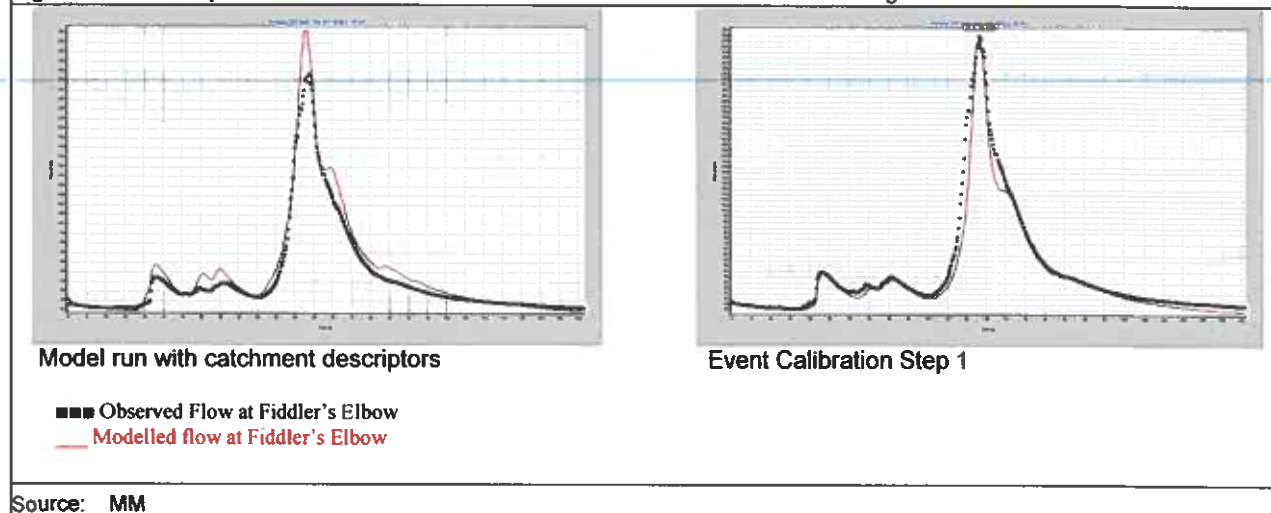


Figure 4.13: Comparison of Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 5



4.7.3 Combined Calibration Results for Individual Events

4.7.3.1 Calibration

Following the independent calibration of the upper catchments and the downstream catchments, the 1D model has been run with modelled flows from the routing model as inflow at Merthyr Tydfil replacing the observed flow at Merthyr Tydfil.

Figure 4.14 to Figure 4.18 show the comparison of calibrated modelled and observed flows at Fiddler's Elbow for the individual calibration events. Errors from both models are accumulatively reflected in the model results. Figure 4.14 to Figure 4.18 also show the results of the model runs with the final design parameter set as discussed in Chapter 6.

Table 4.13: Comparison of Modelled and Observed Peak Flows at Fiddler's Elbow

Event	Observed flow (m3/s)	Modelled flow (m3/s)
1	371	370
2	194	189
3	154	148
4	137	140
5	257	254

Source: MM

4.7.3.2 Results of Model Calibration

Following the model calibration for the five individual events, the model has been run with the derived design parameter set for all events and modelled flows at Merthyr Tydfil have been compared against the observed flows.

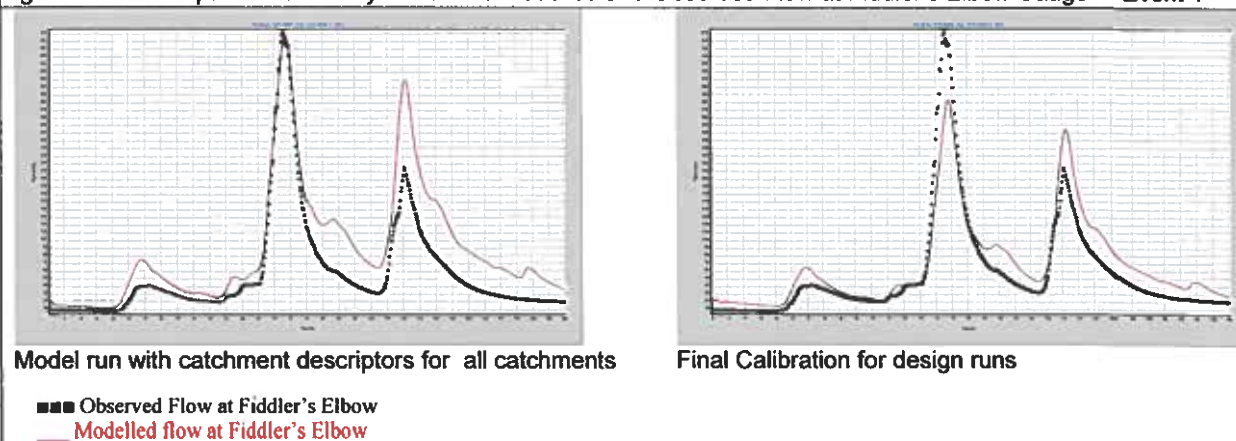
Using one single parameter set representing the average catchments' response to rainfall for a range of conditions leads to a larger variation in the difference between modelled and observed flow for the individual events. The error ranges from -24% to +23% with an average for all five events of $\pm 17\%$. Generally the resulting error in water level is less than ± 0.2 m (Event 2 to Event 5 all consistently show a difference of less than ± 0.2 m; Event 1 shows a difference of less than 0.3 m).

Figure 4.4 to Figure 4.8 show the model runs with the finalised set of design model parameter for each event ('Final Calibration for design run') together with the calibration results ('Model run with catchment descriptors for all catchments').

Table 4.14: Comparison of Modelled and Observed Peak Flows at Fiddler's Elbow

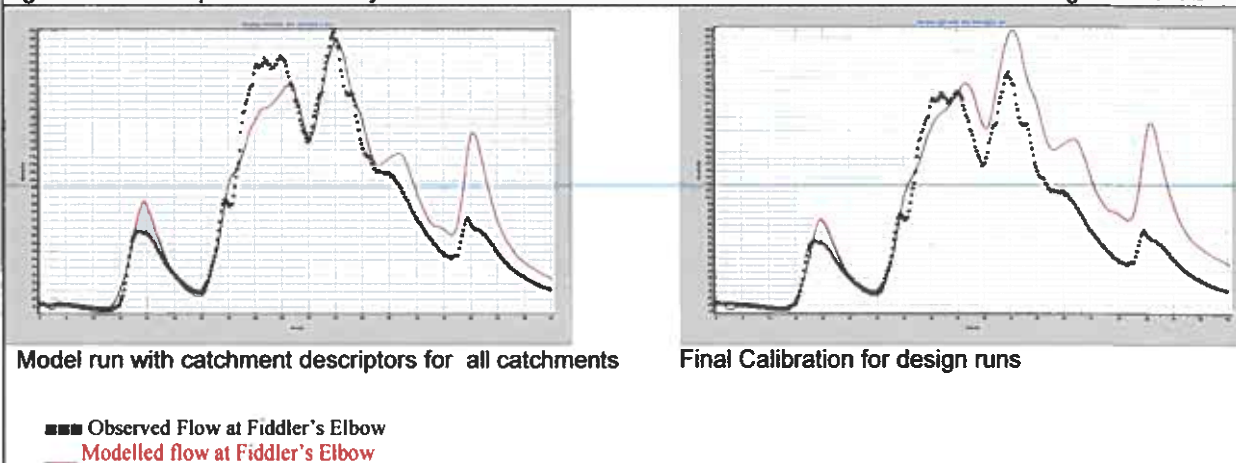
Event	Observed flow (m3/s)	Modelled flow (m3/s)	Difference (Percentage of observed flow)
1	371	283	-24%
2	194	225	16%
3	154	189	23%
4	137	122	-11%
5	257	226	-12%
Average			$\pm 17\%$

Figure 4.14: Comparison of Finally Calibrated Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 1



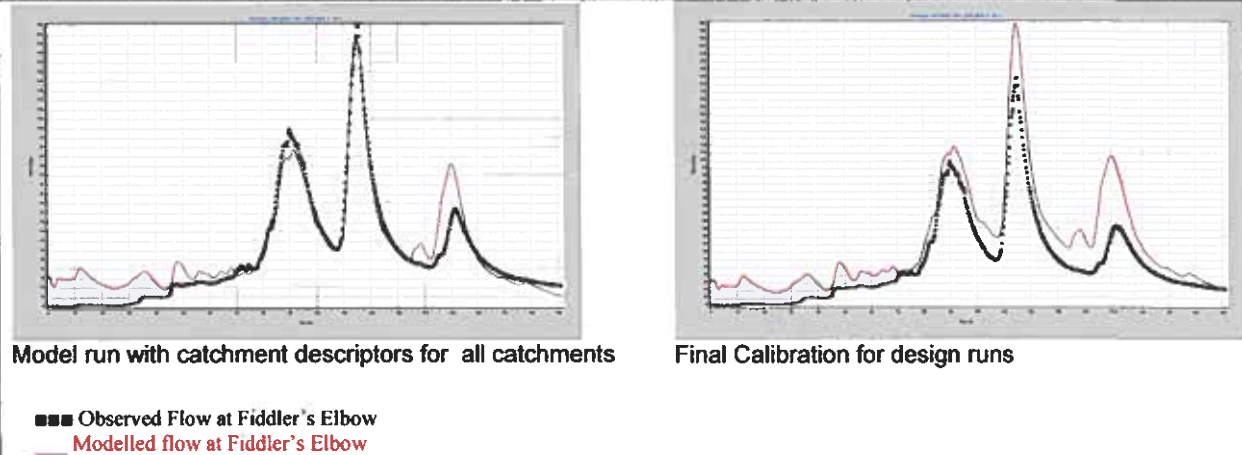
Source: MM

Figure 4.15: Comparison of Finally Calibrated Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 2



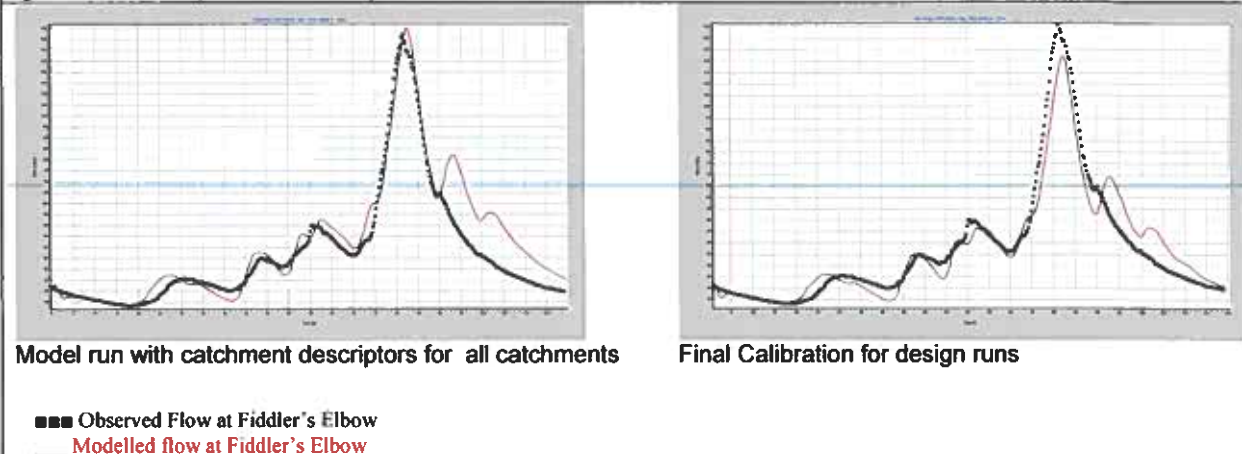
Source: MM

Figure 4.16: Comparison of Finally Calibrated Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 3



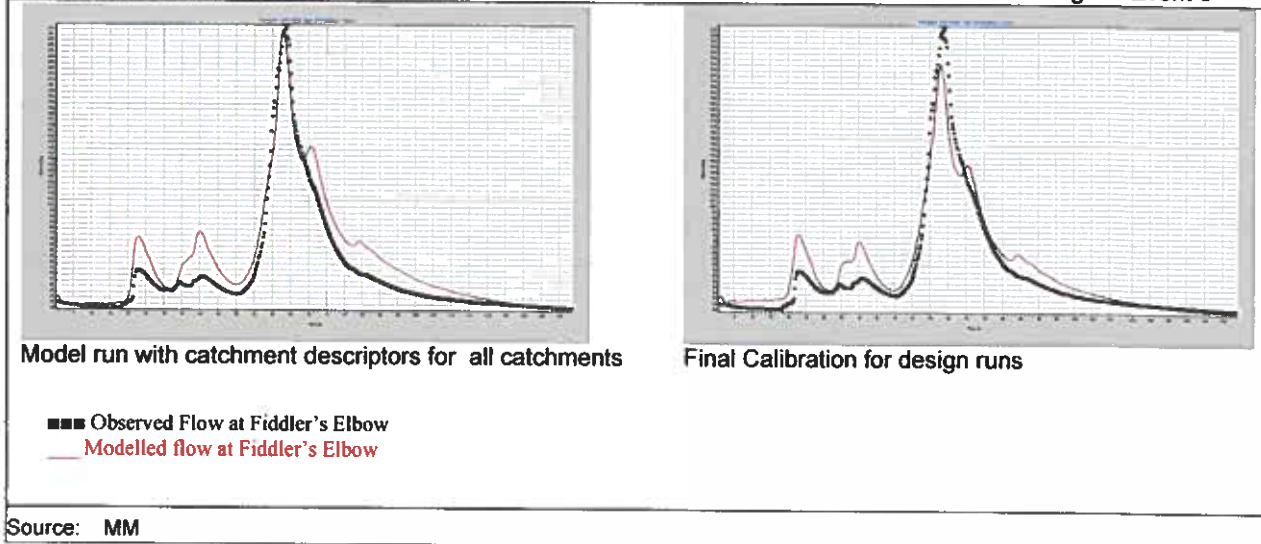
Source: MM

Figure 4.17: Comparison of Finally Calibrated Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 4



Source: MM

Figure 4.18: Comparison of Finally Calibrated Modelled and Observed Flow at Fiddler's Elbow Gauge – Event 5



4.8 Design Peak Flows

4.8.1 Introduction

This Section presents the results obtained to estimate the peak flows for 5, 10, 15, 20, , 50, 75, 100, 100+CC, 200, and 1000 year return periods for two locations on the River Taff. The first site is located at Merthyr Tydfil gauging station; the second site is located at the Fiddlers Elbow gauging station. The results will be used in the Merthyr 2D hydraulic model to derive design peak water levels. The location of these two sites is presented in Figure 4.19.

The design peak flows for the selected sites have been estimated using FEH techniques; both the methods of the statistical analysis, and the ReFH method.

Figure 4.19: Locations for Estimate of design Flows



Source: MM

4.8.2 Existing Information

Design peaks at Fiddler's Elbow for a range of return periods were derived as part of an update to the Flood Risk Mapping (FRM) Study carried out by ATKINS in 2007. This study has also been used for the CFMP of the Taff catchment carried out in 2009. A single site analysis and a pooling group analysis were carried out and the results of the single site analysis were brought forward for the determination of design peak flows up to a return period of 200 years. For the 1000 year return period the ratio between the 200 year and the 1000 year derived from the rainfall runoff method was applied to derive the 1000 year design peak flow. The estimated flood peaks are given in [1]. Table 4.15 summaries the design peak flows used for the FRM Study and the CFMP.

Table 4.15: Design Peak Flows at Fiddler's Elbow

Return Period	Flow (m ³ /s)	Methodology
2	136	Single site analysis
5	194	Single site analysis
10	232	Single site analysis
25	281	Single site analysis
50	317	Single site analysis
75	338	Single site analysis
100	354	Single site analysis

Return Period	Flow (m ³ /s)	Methodology
200	390	Single site analysis
1000 without siphons	597	Ratio of ReFH estimates of the 200 year and the 1000 year return period applied to the 200 year flow derived with the statistical method
1000 with siphons	641	

Source: Table 4.8 of [1]

The ReFH rainfall – runoff method had also been applied by ATKINS as part of the update to the Flood Risk Mapping Study. Although the single site statistical method was adopted, the findings at Fiddler's Elbow have been considered for the adjustments to the ReFH design hydrographs of the individual ungauged catchments.

Table 4.16: Comparison of Design Peak Flows using Different Methodologies by WS ATKINS

Method	100 Year	200 Year
Single Site Statistical	354	390
Rainfall runoff	363	428

Source: [1] Table 4.7 and 4.8

4.8.3 Results

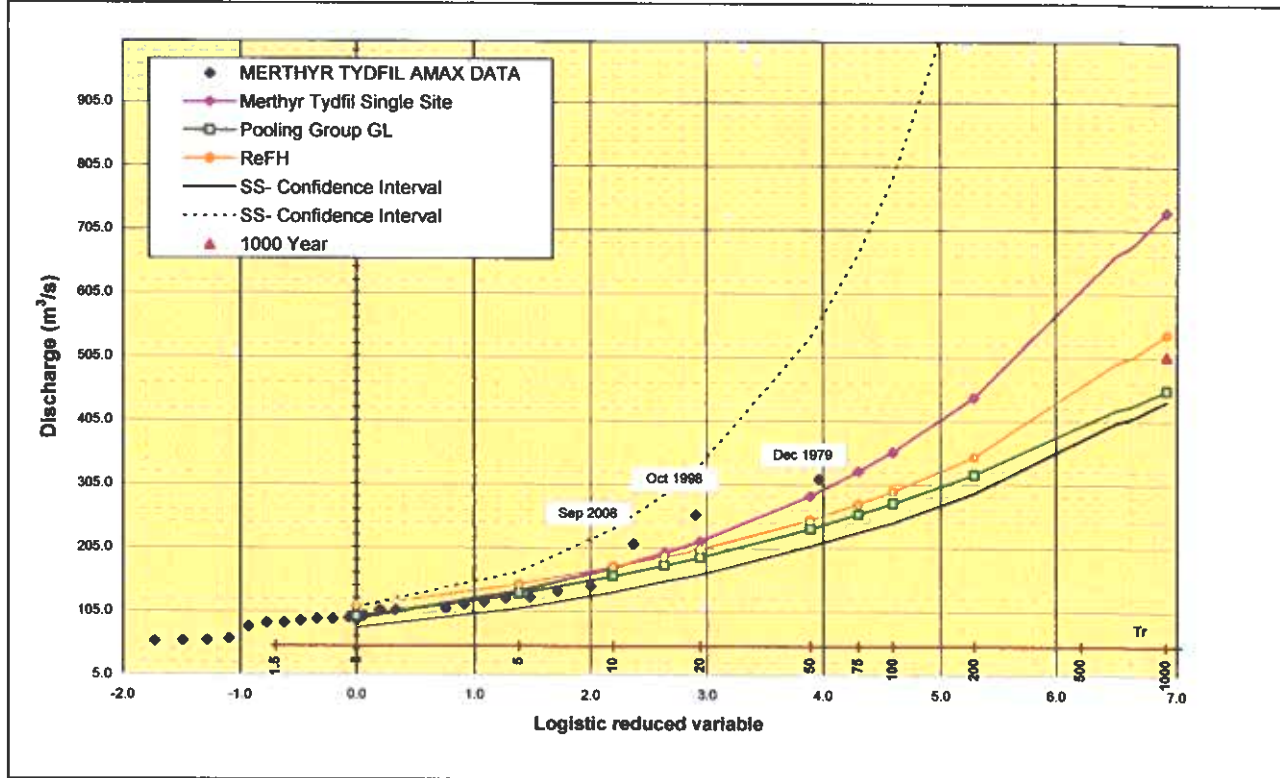
4.8.3.1 Merthyr Tydfil

The obtained growth curves and design peak flows from the single site and pooling group analysis together with the results from ReFH are presented in Table 4.17 and Figure 4.20, for specified return periods:

Table 4.17: Results of the Statistical Analysis and ReFH at Merthyr Tydfil

Return Period (Tp)	GROWTH CURVE (SS)	DESIGN PEAK FLOWS (SS) (m ³ /s)	GROWTH CURVE (PG)	DESIGN PEAK FLOWS (PG) (m ³ /s)	DESIGN PEAK FLOWS (ReFH) (m ³ /s)
2	0.98	94.28	1.00	96.31	112.45
5	1.43	137.38	1.39	133.69	147.53
10	1.80	173.21	1.67	161.07	175.77
15	2.05	197.09	1.85	178.04	191.55
20	2.24	215.73	1.98	190.71	203.72
50	2.98	286.85	2.45	235.50	249.50
75	3.38	325.33	2.68	257.90	274.04
100	3.69	355.78	2.85	274.91	293.42
200	4.59	441.72	3.32	320.13	348.27
1000	7.62	733.63	4.707	453.34	541.10

Figure 4.20: Merthyr Tydfil Design Peak Flows



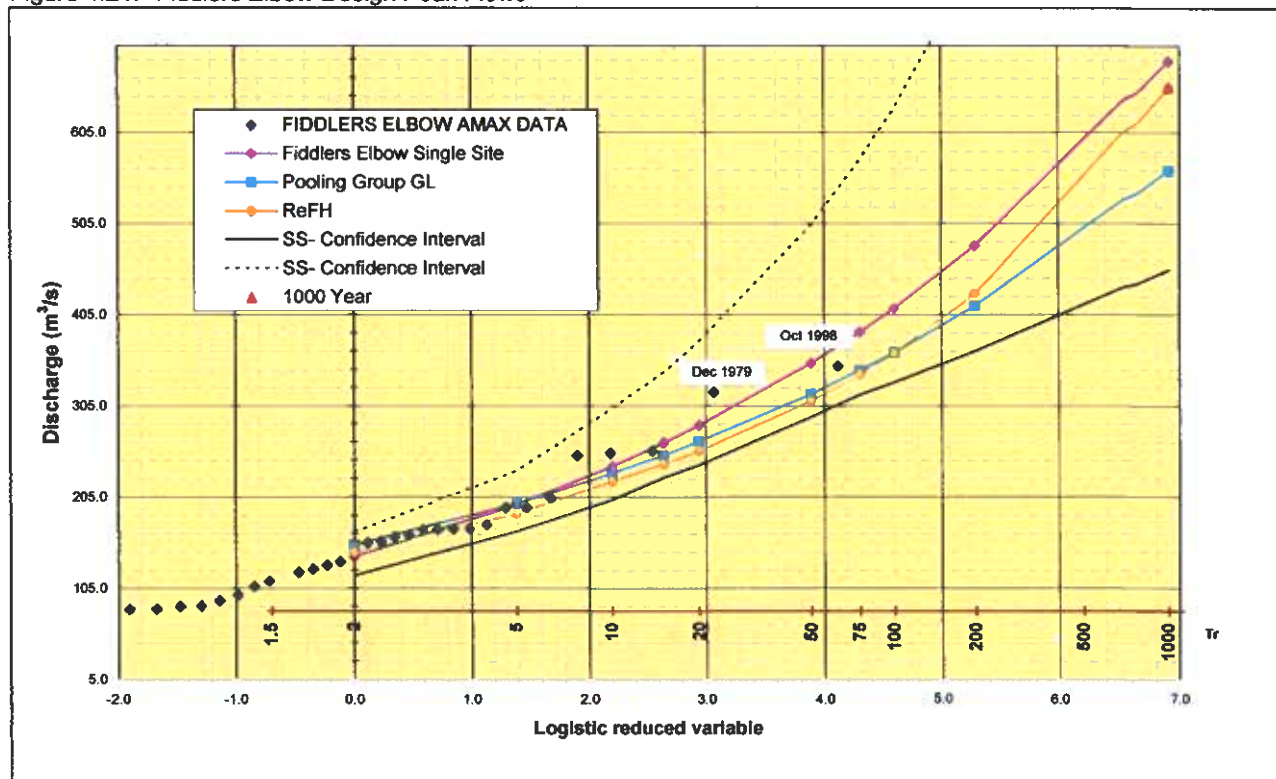
4.8.3.2 Fiddlers Elbow

The obtained growth curves and design peak flows from the single site and pooling group analysis and ReFH are presented in Table 4.18 and Figure 4.21, for specified return periods:

Table 4.18: Fiddlers Elbow Summary Results - Statistical Analysis

Return Period (Tp)	GROWTH CURVE (SS)	DESIGN PEAK FLOWS (SS)	GROWTH CURVE (PG)	DESIGN PEAK FLOWS (PG)	DESIGN PEAK FLOWS (ReFH)
2	0.91	139.55	1.00	152.68	144.68
5	1.29	196.65	1.30	198.18	187.62
10	1.56	238.49	1.51	231.01	221.98
15	1.73	264.37	1.64	251.01	240.95
20	1.86	283.68	1.74	265.97	255.54
50	2.31	351.93	2.08	318.04	310.33
75	2.53	386.13	2.25	343.84	339.63
100	2.70	411.94	2.38	363.38	362.71
200	3.15	480.95	2.72	414.68	427.78
1000	4.48	683.56	3.69	563.09	653.73

Figure 4.21: Fiddlers Elbow Design Peak Flows



4.8.3.3 Discussion

Merthyr Tydfil

The design peak flows obtained at Merthyr Tydfil using the three methods (single site, pooling group and ReFH) are very similar for return periods up to 20 years. This is expected given the length of the annual maximum series (30 years). For return periods higher than 20 years the single site analysis curve is the steepest of the three, due to the flood events of December 1979, October 1998 and September 2008. The magnitude of the December 1979 and October 1998 events need to be treated with caution since they have been estimated by extrapolation. Furthermore, the December 1979 event destroyed the gauging station, although it is recognised as the highest event in the catchment, its magnitude is uncertain.

The December 1979 flow of 313 m³/s would have a return period just under 200 years when considering the pooling group curve, and a return period around 50 years when considering the single site curve, this big difference highlights the uncertainty in the estimation of flow magnitude.

The pooling group curve is the flattest of the three. The pooling group is homogeneous; hence the growth curves of the gauging stations considered does not differ significantly between them. This shows that catchments similar to Merthyr Tydfil have not recorded floods as severe as the December 1979 event. This fact emphasizes the uncertainty in the magnitude of this flood event.

The 1000 year design peak flow obtained from rainfall data (ReFH) has a value between the single site value and the pooling group value.

Fiddlers Elbow

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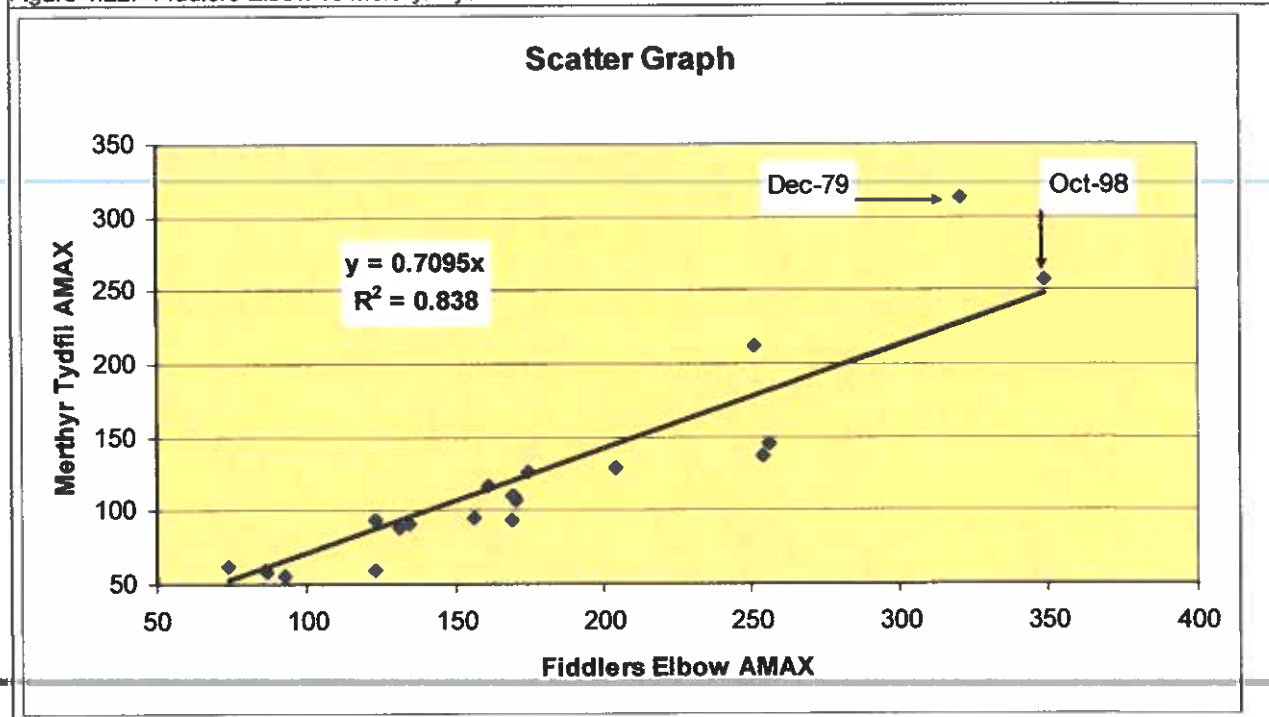
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The design peak flows obtained at Fiddlers Elbow using the three methods (single site, pooling group and ReFH) are very similar for return periods up to 15 years. This is expected given the length of the annual maximum series (35 years). Although there is a difference in design flows for return periods higher than 20 years between the three methods, the difference is not above what is expected. The single site curve is the highest of the three, but does not differ greatly from the pooling group curve. The greatest flood recorded at Fiddlers Elbow is the October 1998 event rather than the December 1979 event as it is at Merthyr Tydfil.

The October 1998 flow shows an increase of 26% from Merthyr Tydfil (258) to Fiddlers Elbow (350). The December 1979 flow shows an increase of 2% from Merthyr Tydfil (313) to Fiddlers Elbow (321). Although it is expected that the greater the event the more attenuation can occur, the apparent attenuation in flow for this event is considered unrealistic.

A relationship between the Fiddlers Elbow and Merthyr Tydfil annual maximum flow data has been derived where the dates coincide, and it is presented in Figure 4.22. The October 1998 event follows very well the derived relationship, while the December 1979 is far away. Following the derived relationship, the December 1979 flow at Merthyr Tydfil is 227 m³/s. This flow has a return period in the region of 50 years, according with design flows derived by the pooling group method. The return period derived for this event on the River Taff at Tongwynlais is in the region of 75 years, according to the Cardiff FAS hydrology report².

Figure 4.22: Fiddlers Elbow vs Merthyr Tydfil Amax flows



² Cardiff Flood Protection Scheme. Note on the Hydrological Reassessment of the 100 year Design Flood in view of the flood of 27 December 1979. Welsh Water Authority. Sir William Halcrow and Partners, March 1980

The Fiddlers Elbow pooling group is homogeneous; hence the growth curves of the gauging stations considered does not differ highly among them. The similarity of the single site and the pooling group curves shows a similar behaviour in all the gauging stations included in the analysis, and a more consistent set of data than that from Merthyr Tydfil.

The 1000 year design peak flow obtained from rainfall data (ReFH) is a value between the single site value and the pooling group value.

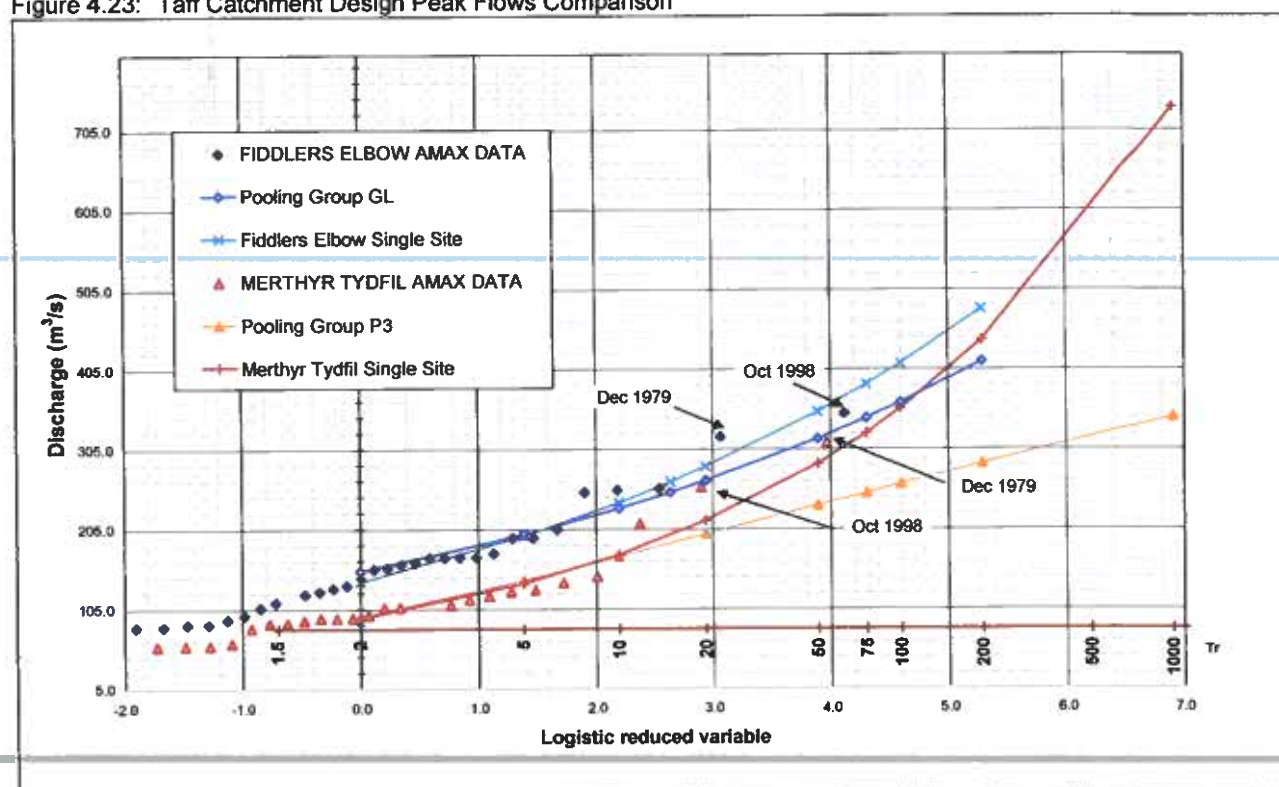
Catchment Analysis

The single site and pooling group analysis for the two sites has been compared in magnitude. The results are presented in Figure 4.23.

The Merthyr Tydfil single site analysis shows a very different behaviour to any other growth curve analysis. Design peak flows for this site considering a single site growth curve produce flows much greater than downstream at Fiddlers Elbow.

Design Flows obtained from pooling group growth curves are more robust and consistent catchment wide.

Figure 4.23: Taff Catchment Design Peak Flows Comparison



4.8.4 Peak Flows for Modelling

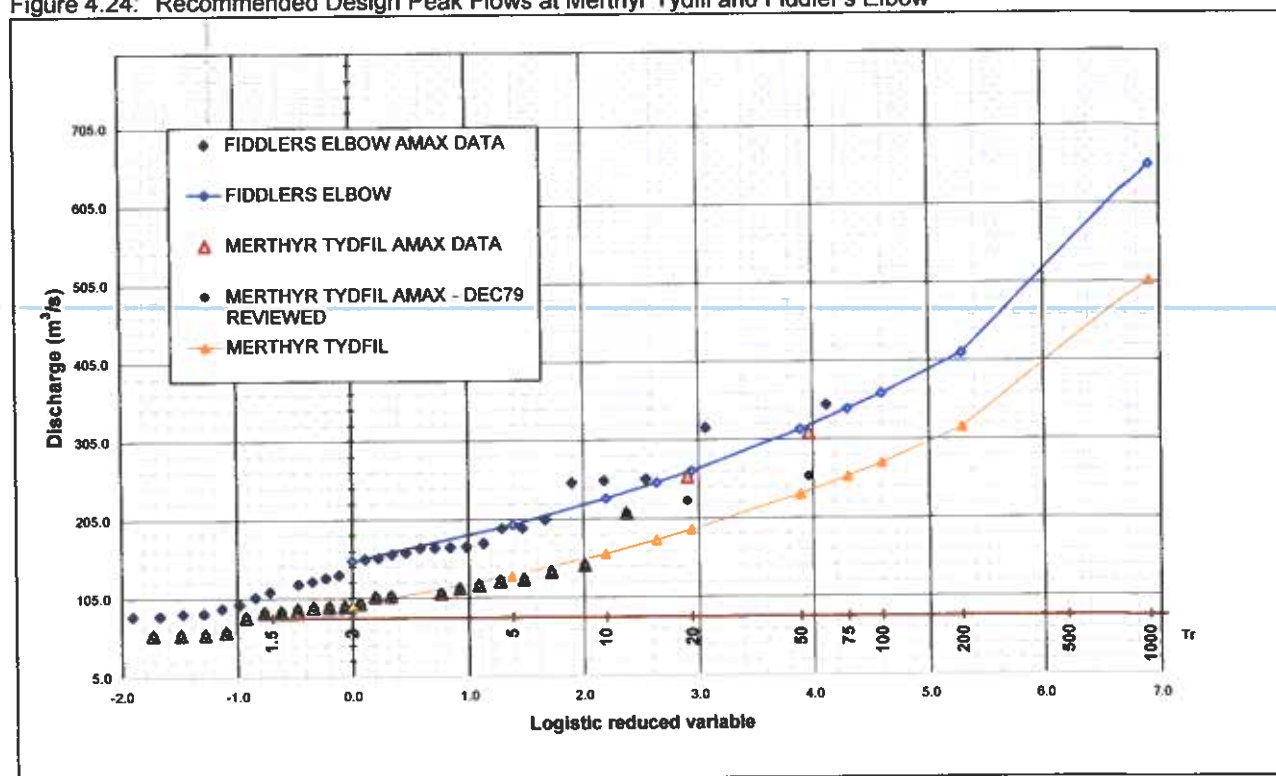
It is recommended to obtain the design peak flows for both Merthyr Tydfil and Fiddlers Elbow from the pooling group analysis for return periods of 2 to 200 years. The 1000 year design flow is recommended to be that obtained from the ReFH methodology and factored as described above.

The design flows are summarised in and presented in Table 4.19.

Table 4.19: Recommended Design Peak Flows

Return Period (years)	MERTHYR TYDFIL	FIDDLERS ELBOW
	Design peak flow (m ³ /s)	
2	96	153
5	134	198
10	161	231
15	178	251
20	191	266
50	236	318
75	258	344
100	275	363
200	320	415
1000	507	655

Figure 4.24: Recommended Design Peak Flows at Merthyr Tydfil and Fiddler's Elbow



4.9 Design hydrographs

4.9.1 ReFH Model parameters

Based on the calibration of the ReFH models for five individual events described in 4.7.1 and 4.7.2 a ReFH model has been derived for each of the twelve sub-catchments which represents the average catchment response to rainfall. These models have been used in conjunction with the estimated design peak flows to derive a set of design inflow hydrographs for the required return periods.

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Modifications made to the ReFH model parameters derived using the catchment descriptors are summarised in Table 4.20.

Table 4.20: Adjustment Factors to ReFH Parameters for Design Model Runs

Sub-catchment	C _{max}	T _p	BR	C _{ini}
Pontsticill	1	1	0.8	1
PontsticillLat	1	1	0.8	1
Llwynon	1	1	0.8	1
LlwynonLat	1	1	0.8	1
Aberfan	1.28	1.3	1	0.7
Bargoed	1.28	1	1	0.7
Ditch	1.28	1	1	0.7
Fiddlers	1.28	1.3	1	0.7
Merthyr Town	1.28	1	1	0.7
Nant Morlais	1.28	1	1	0.7
Taff Bargoed	1.28	1.3	1	0.7
Troedyrhiw	1.28	1.3	1	0.7

Source: MM

4.9.2 Critical Storm Duration

Two methods have been applied to derive the critical storm duration for the design model runs:

- the ISIS optimising facility and
- the estimation based on catchment descriptor ($D = T_p(1 + (SAAR/1000))$).

The application of the two methods results in two significantly different storm durations as demonstrated in Figure 4.25 which compares the flow hydrographs of the 100 year design model run at Merthyr Tydfil. Note that the hydrographs have been scaled to match the estimated design peak flow of 261 m³/s at Merthyr Tydfil.

The application of the optimised storm duration of 15.25 hours results in a much more voluminous hydrograph than the estimated critical storm duration of approximately 5 hrs for the four catchments upstream of Merthyr Tydfil.

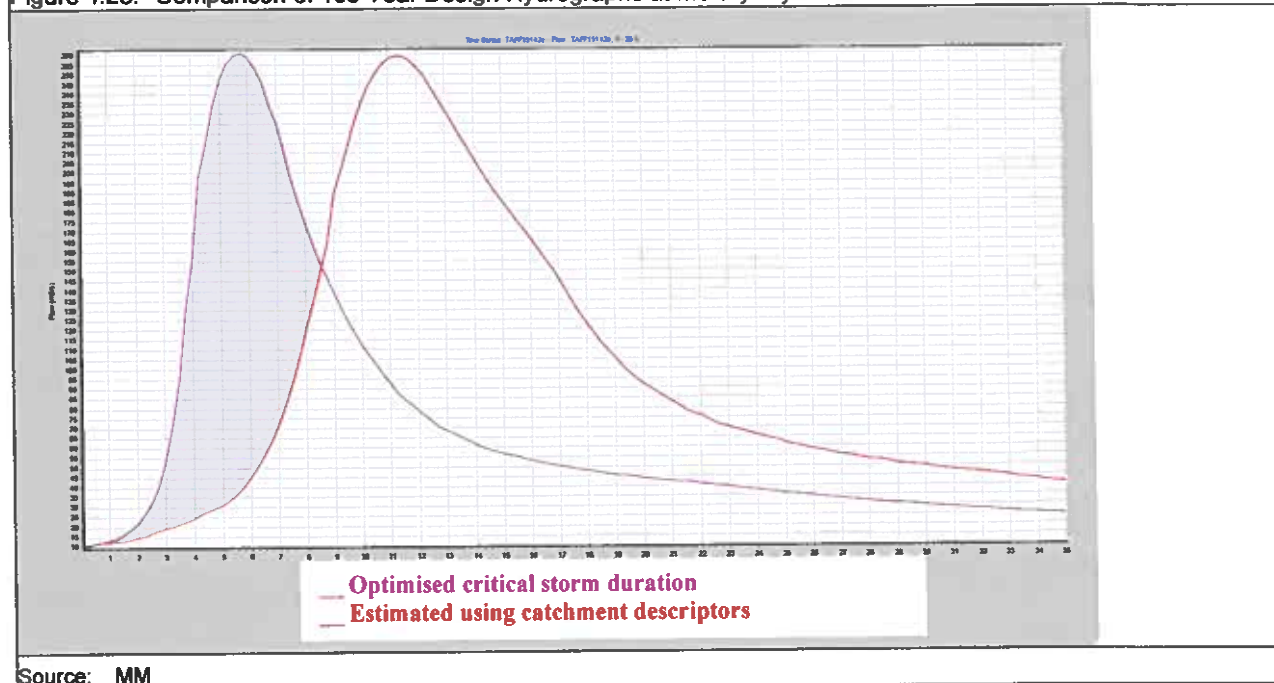
The results of the optimising procedure carried out to the maximum flow at Merthyr Tydfil and at Fiddlers Elbow respectively are shown in Table 4.21

The critical storm duration for the four upper catchments is 15.25 hours and 16.75 for the catchments downstream of Merthyr Tydfil.

The optimised critical storm duration for the catchments downstream of Merthyr Tydfil is consistent with the findings of the update to the Flood Risk Mapping (FRM) Study carried out by WS ATKINS in 2007. There a duration of 15.75 hours was applied.

For the design model runs the critical storm duration of 15.75 hrs has been applied.

Figure 4.25: Comparison of 100 Year Design Hydrographs at Merthyr Tydfil for Different Critical Storm Durations



Source: MM

Table 4.21: Results of Optimised Critical Storm Duration

Storm Duration	Flow at Merthyr Tydfil (m³/s)	Storm Duration	Flow at Fiddler's Elbow (m³/s)
15.25	405.34	16.75	376.90
15.75	404.72	16.25	376.58
14.25	404.63	17.25	376.47
16.25	404.30	15.25	374.32
14.75	403.85	19.25	368.61
13.25	403.67	11.75	350.27
18.25	402.31	25.25	316.13
11.25	401.40	5.25	243.20
21.75	398.25	16.75	376.90
30.25	385.54	16.25	376.58
2.25	208.86	17.25	376.47

4.10 Design Model Runs

4.10.1 5 to 200 Year

The upstream routing model has been run with the derived design model parameters shown in Table 4.20 for the required return periods. The ReFH design hydrographs have been scaled to match the design peak flow at Merthyr Tydfil estimated using the statistical method.

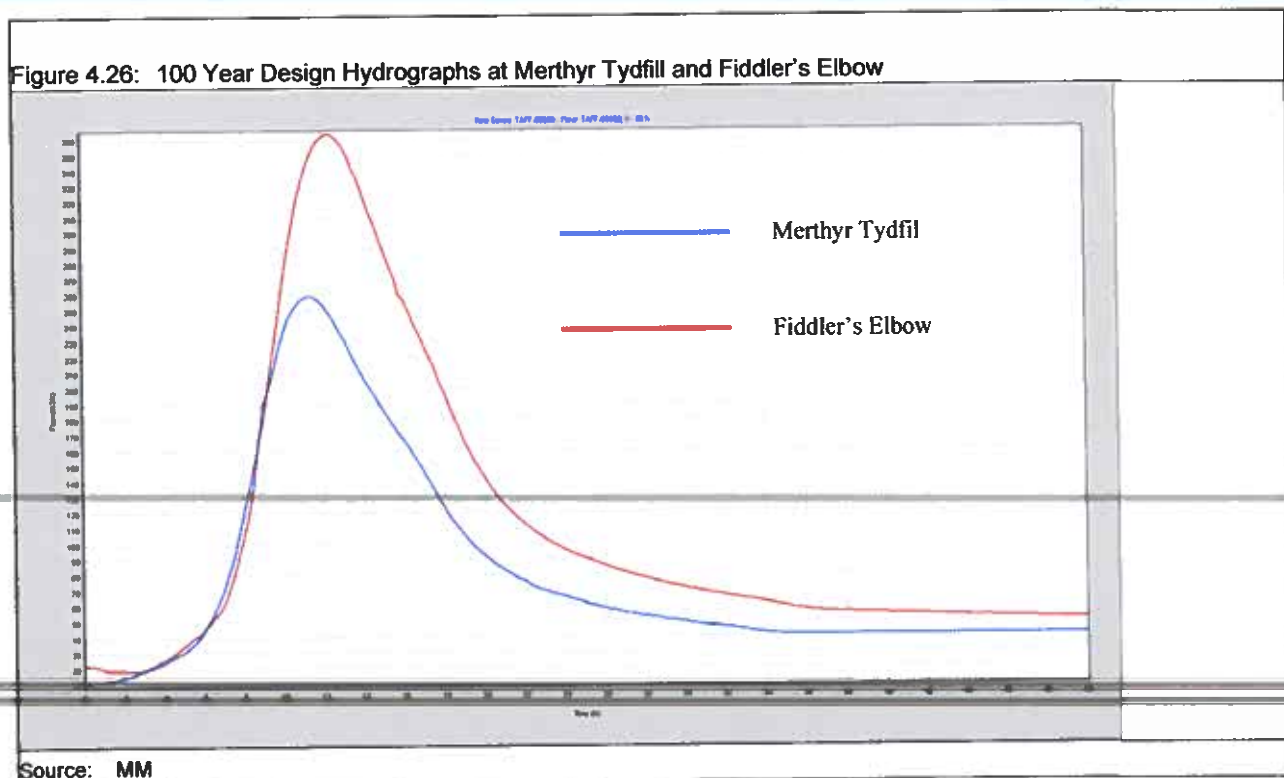
The resulting hydrographs at Merthyr have been initially fed into the unmodified 1D model and a similar factoring of the inflows has been carried out to match the target design peak flows at Fiddler's Elbow. The purpose of the initial design model runs was to provide information on the range of required inflows to support the development of the 1D model.

Once the 1D model and the 1D-2D model were finalised, the design runs have been repeated and adjustments have been made to ensure the design peak flows at Fiddler's Elbow are matched. A summary is given in Chapter 6.

Table 4.22: Initial Scaling Factors of the ReFH inflows for Design Model Runs

Return Period (Tp)	MERTHYR TYDFIL			FIDDLERS ELBOW		
	Target flow (m ³ /s)	Scaling factor u/s ReFH models	Design peak flow (m ³ /s)	Target flow (m ³ /s)	Scaling factor d/s ReFH models	Design peak flow (m ³ /s)
5	134	0.78	136	198	1.05	197
10	161	0.75	162	231	1.00	231
15	178	0.73	178	251	1.00	253
20	191	0.72	190	266	0.9	266
50	236	0.7	237	318	0.85	320
75	258	0.69	260	344	0.8	346
100	275	0.68	276	363	0.75	363
200	320	0.66	323	415	0.65	413

Figure 4.26 shows the derived 100 year design flow hydrographs at Merthyr Tydfil and Fiddler's Elbow.



4.10.2 1000 Year

There are three siphons located at the outlet of the Llwynon reservoir which are not active during low return periods. As the flood volume increases the siphons become active, thereby increasing the outflow from the reservoir. It was estimated that the siphons become active when flows exceed approximately a 200 year return period (EA Wales).

Previous studies [1] estimated an increased outflow from the reservoir of $44 \text{ m}^3/\text{s}$ for a 1000 year design flood.

In order to take the increase in flow into account the routing model has first been run with the siphons removed. The flow from the ReFH boundaries have been scaled to match the target design peak flow at Merthyr Tydfil as described above.

The model with the same boundary conditions (1000 year scaled design hydrographs) has then been run with the siphons added back into the model. Figure 4.27 shows the comparison of the 1000 year design flood hydrographs routed to Merthyr Tydfil. The flow has increased by $40 \text{ m}^3/\text{s}$.

The 1000 year flow taking the increased flow from the siphons into account has been used for flood risk mapping.

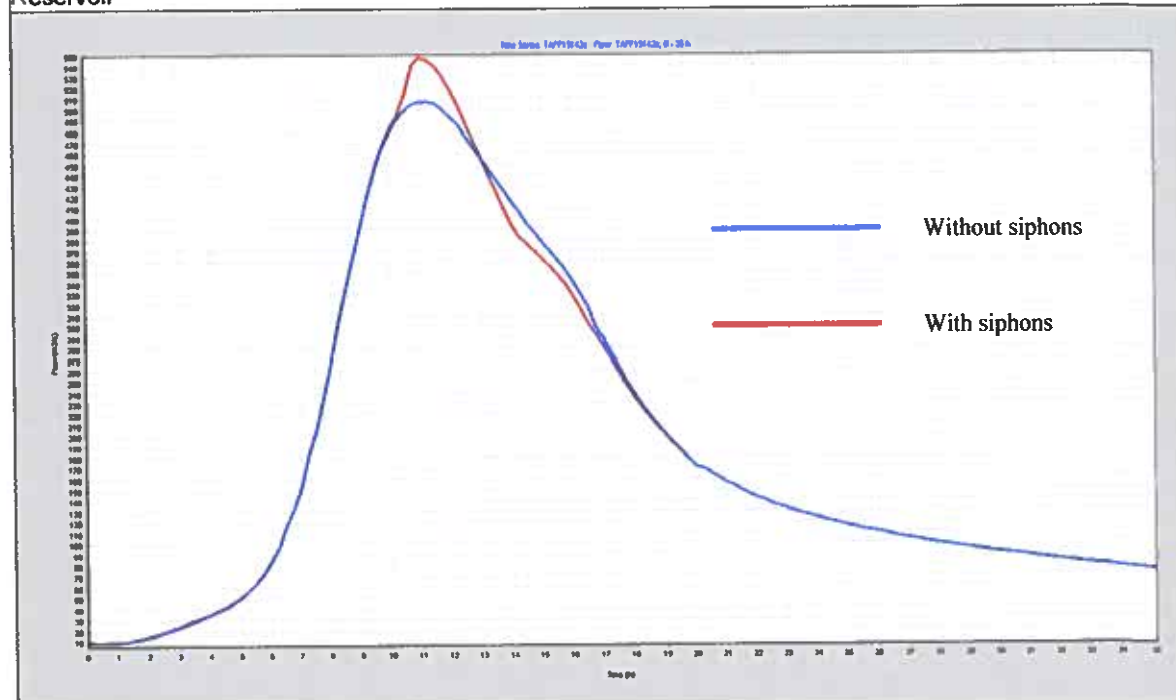
Table 4.23: 1000 Year Design Model Results

Return Period (Tp)	MERTHYR TYDFIL		FIDDLERS ELBOW	
	Target flow (m ³ /s)	Modelled flow (m ³ /s)	Target flow (m ³ /s)	Modelled flow (m ³ /s)
Without siphons	507	507	655	656
With siphons	-	547	-	677

1) without siphons at Llwynon reservoir, model run to derive scaling factor for run with siphons

2) with siphons at Llwynon reservoir

Figure 4.27: Comparison of 1000 Year Design Model Run at Merthyr Tydfil with and without Siphons at Llwynon Reservoir



Source: MM

5. Hydraulic Modelling

5.1 Method Statement

In order to develop and calibrate the hydrodynamic model required for this study we have carried out the following steps which are described in this section:

1. Assessment of initial calibration of existing model (FF model) in terms of water levels.
2. Further development of the 1D model from Merthyr Tydfil to Fiddler's Elbow.
3. Calibration of 1D model from Merthyr Tydfil to Fiddler's Elbow for selected events.
4. Development of 1D-2D linked model from Troedyrhiw to Merthyr Vale.
5. Calibration of 1D-2D linked model.

Observed inflows at the upstream end of the model at Merthyr Tydfil have been used for the calibration of the hydrodynamic model in order to assist the identification of errors related to shortcomings in the hydrodynamic model.

Issues identified in the process of developing and calibrating the model are summarised in Chapter 8.

5.2 Data

Various information representing features impacting on the water levels such as river cross sections, bank levels, dimension of structures, ground levels has been used to develop the hydrodynamic model. Details are given in Chapter 3 and in the sections below.

The following data and information has been used for the calibration of the model:

- Observed flow at Merthyr Tydfil,
- Observed water levels at Merthyr Tydfil, Merthyr Town Weir, Troedyrhiw and Aberfan,
- Flood photographs Event 5,
- Various information on flooding during Event 5.

5.3 Software

The following software has been used for the modelling of the River Taff:

- ISIS 3.1.0.65,
- ISIS-TUFLOW 2009-07-AE-iSP,
- MapInfo Professional 10.0 and ArcGIS 9.3.

5.4 Model Development

5.4.1 1D Model Merthyr Tydfil to Fiddler's Elbow

The changes made to the FF model are described in this Section. The following issues have been identified and addressed if possible as part of the model development for the purpose of flood risk mapping:

- Replacement and modification of cross sections,
- ~~Modelling of the drain through Troedyrhiw,~~
- Modelling of the fishpass at Merthyr Town Weir,
- Extension of river sections onto the floodplain to accommodate 1000 year flood,

- Identification of defences and development of defended and undefended model.

5.4.1.1 Cross Sections

Routing sections

The FF model comprises a number of routing sections (*VPMC Cross Section (MUSK-XSEC)*) which allow the modelling of the flow but which do not provide water levels.

For the purpose of flood forecasting routing sections are suitable in locations where the lack of detail does not have an impact on the accuracy of the predicted water levels. They are generally used to reduce model instabilities and to reduce the model run time.

For the purpose of flood mapping routing sections are not suitable as they do not provide water levels for the mapping of the flood extent.

The routing sections in the 1D model from Merthyr Tydfil to Fiddler's Elbow are summarised in Table 5.1.

Initially attempts were made to replace the routing sections with more densely spaced river sections where available. In the 1km reach downstream of Merthyr Road Business Park, routing section TAFFMC12447 was replaced with a number of existing cross sections. However, this caused significant model instabilities and it was agreed with the EA Wales not to follow this up.

Consequently areas where routing sections have been used have to be excluded from flood mapping.

Table 5.1: Summary of ISIS Routing Sections Unsuitable for Mapping

ISIS Section	Location	Distance to next section (m)	Comment
TAFFMC12447	Downstream of Merthyr Road Business Park	1162	Left in the model
TAFFMC13335	Downstream of dismantled railway crossing	757	Left in the model
TAFFMC13841	From A4102 crossing downstream	506	Left in the model
TAFFMC14291	Downstream Merthyr Town Weir	450	Left in the model

Source: MM

Added and modified sections

In order to improve accuracy of the predicted water levels a number of interpolated sections upstream of Merthyr Town Weir have been replaced with survey cross sections. The added sections are summarised in Table 5.2.

The cross section immediately upstream of Merthyr Town Weir has been changed in accordance with the survey drawings.

In March 2010 cross sections of a recent survey were made available. These have been incorporated into the 1D model. It should be noted that due to the study approaching completion, the calibration of the 1D

model has not been repeated. However, the calibration of the 1D-2D linked model has taken these updates into account.

Table 5.2: Summary of Added and Modified Cross Sections

Added ISIS Section	Origin	Comment
TAFF14468		Interpolated section replaced
TAFF14421		
TAFF14432		
TAFF14310		Updated
TAFF12447 - TAFF11285		Initially used to replace routing sections but agreed to leave them unchanged due to stability issues
TAFF06749	FCA model	FF model section replaced
TAFF06669		
TAFF06573		
TAFF06552		
TAFF06270		
TAFF06177		
TAFF06092		
TAFF06000		
TAFF06625		Added to FF model
TAFF06585		
TAFF06500		
TAFF06425		
TAFF06330		
TAF06225		
TAFF06193		

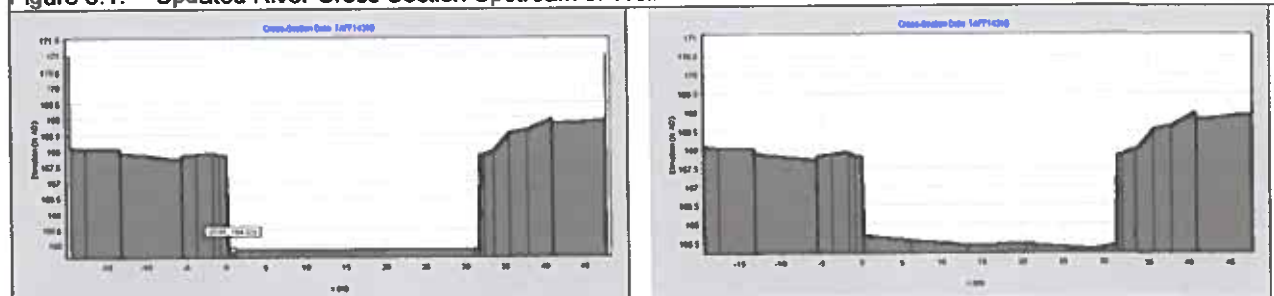
5.4.1.2 Merthyr Town Weir

At Merthyr Town weir a fishpass was built in 2007 which also led to the lowering of the weir crest to compensate for the increased flood risk caused by the obstruction of flow by the new structure. Although all calibration events for which data is available at this location fall into the period prior to the building of the fishpass, the model needs to be updated for the design model runs.

Based on the survey drawings made available the fishpass has been modelled. A simplified approach has been taken, modelling the complex structure (see Table 5.3) as a second weir. The characteristics of Town weir have been updated together with the cross section immediately upstream of the weir (see Figure 5.1). The weir characteristics prior to the building of the fishpass and as built together with the characteristics used in the original FF model are summarised in Table 5.3.

Figure 5.2 shows a picture of the fishpass and the ISIS model schematic of the updates.

Figure 5.1: Updated River Cross Section Upstream of Weir



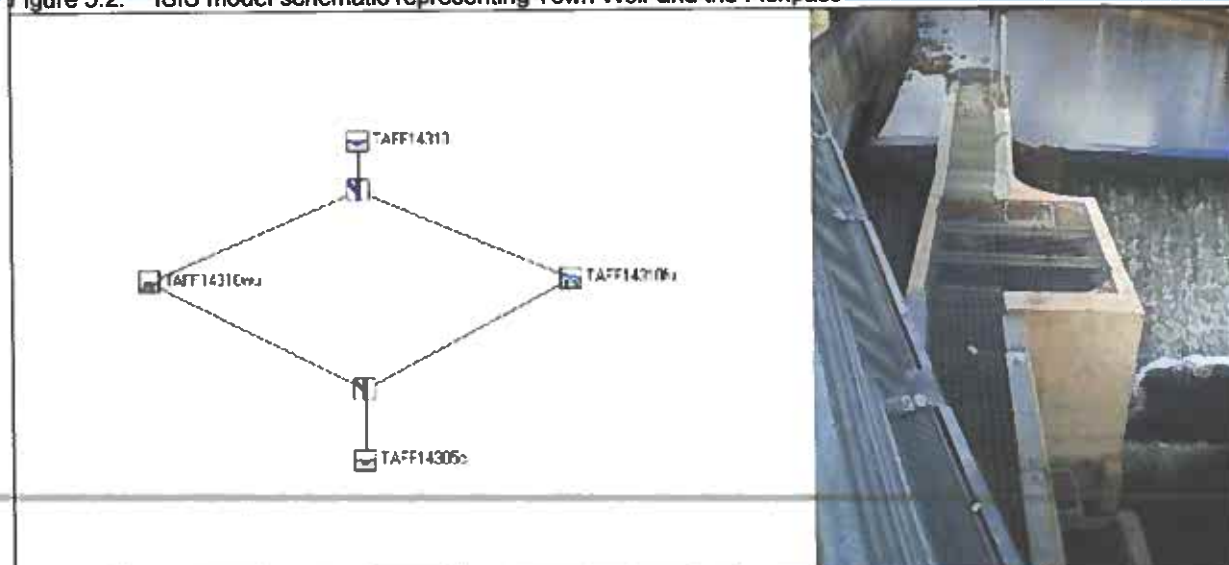
Source: MM

Table 5.3: Summary of Updates at Merthyr Town Weir in ISIS

	FF Model	Prior to building of fishpass	Post building of fishpass
Crest level (mAOD)	165.830	165.8	165.69
P1 (m)	1	0.471	0.361
P2 (m)	4.1	3.61	3.5
Breadth	31.17	31.5	29.54
Length (m)	1	8.76	8.76
Weir coefficient	1	1.5	1.5

Source: MM

Figure 5.2: ISIS model schematic representing Town Weir and the Fishpass



Source: MM

5.4.1.3 Weir at Aberfan

In March 2010 the EA Wales identified an error in the FF model which had previously not been noticed. The weir at Aberfan (upstream of Nixonville Bridge) had been represented twice in the model, once as a

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spill (TAFF07384) and once as a weir (TAFF07334). Both use the correct dimensions of the weir. It was decided to remove the spill (TAFF07384). It should be noted that due to the study approaching completion, the calibration of the 1D model has not been repeated. However, the calibration of the 1D-2D linked model has taken this correction into account.

5.4.1.4 Modelling of Ditch

There is a drain flowing through Troedyrhiw which had caused some flooding in the past. It was agreed that it should be modelled in more detail than was the case in the FF model.

Contributing catchment

The catchment contributing to the inflow into the ditch extends further upstream beyond the natural boundary of the catchment (named *Ditch*). Flows from the catchment upstream of the *Ditch* catchment (named *US_Ditch*) and a small additional area further upstream are linked to the *Ditch* catchment via a culvert.

The areas contributing flow are shown in Figure 5.3.

ReFH models have been set up for the *Ditch* catchment and the *US_Ditch* catchment. The area of the *US_Ditch* catchment has been increased to take the additional area into account. The catchment area of the *Troedyrhiw* catchment has been reduced accordingly.

Storage area

The EA Wales has carried out a JFLOW model run to support the understanding of the flood dynamics in this area. The result is shown in Figure 5.4 together with some annotations. The results show that during flood conditions the amount of water entering the *Ditch* catchment is limited and water is stored in an area north-west of the Industrial Park. The EA Wales also confirmed that the culvert capacity is limited and water is stored further upstream. As the storage area reaches its capacity flow spills towards the River Taff.

The results of the JFLOW model runs and the DTM have been used to identify the storage area and to derive the characteristics needed to model it in ISIS. The EA Wales also provided an area elevation relationship which has been modified for low elevations to ensure consistency between the flows into the storage and outflow through the culvert.

A reservoir unit has been used in ISIS to represent the storage area.

Culvert linking the storage area to ditch channel

The *Ditch* catchment is linked to the upstream contributing areas via a culvert. Some survey information on the dimensions of the culvert has been provided which has been used to model the culvert. A small ISIS model has been set up representing the culvert and a couple of cross sections which has been run for inflows representing the range of event and design inflows. The Q-H relationship has then been derived from the model which has been used to represent the culvert as Q-H control in the ISIS model of the *Ditch*.

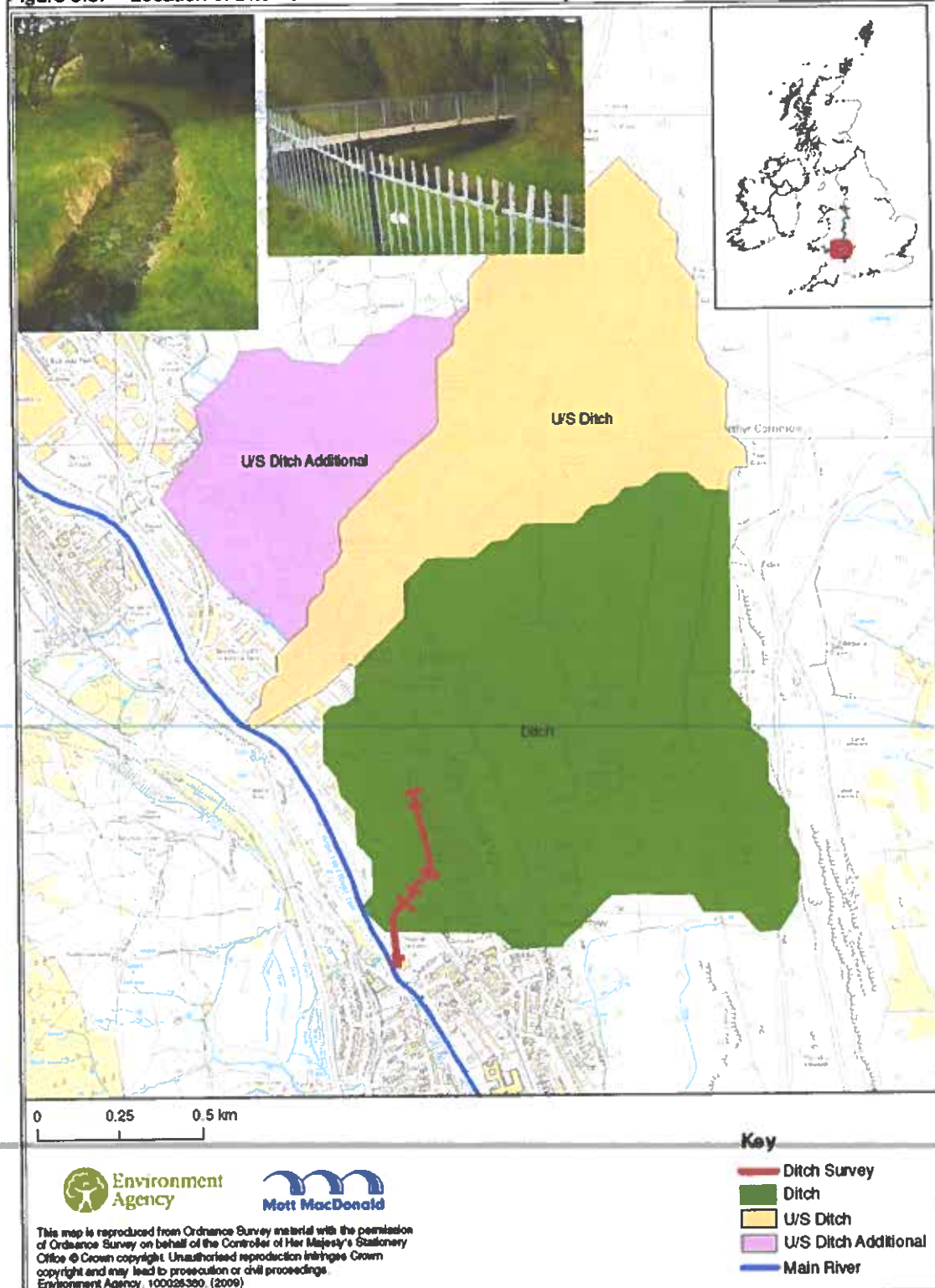
~~There is some uncertainty involved in the modelling of the culvert as the exact dimensions were not available. Therefore sensitivity tests have been carried out varying the diameter of the culvert. The~~

diameter has been varied from 1 m to 2 m. A diameter of 1.5 m has been chosen as the calculated flow that seems to best represent the estimated flow (Agreed with EA Wales.)

Ditch channel and structures

Channel and structures downstream of the culvert have been modelled using survey information provided by the EA Wales. Spills have been added to consider flow over the deck of the bridges. No changes have otherwise been made to consider floodplain flow as the model will be linked to the 2D domain.

Figure 5.3: Location of Ditch Channel and Location of Survey



Source: MM

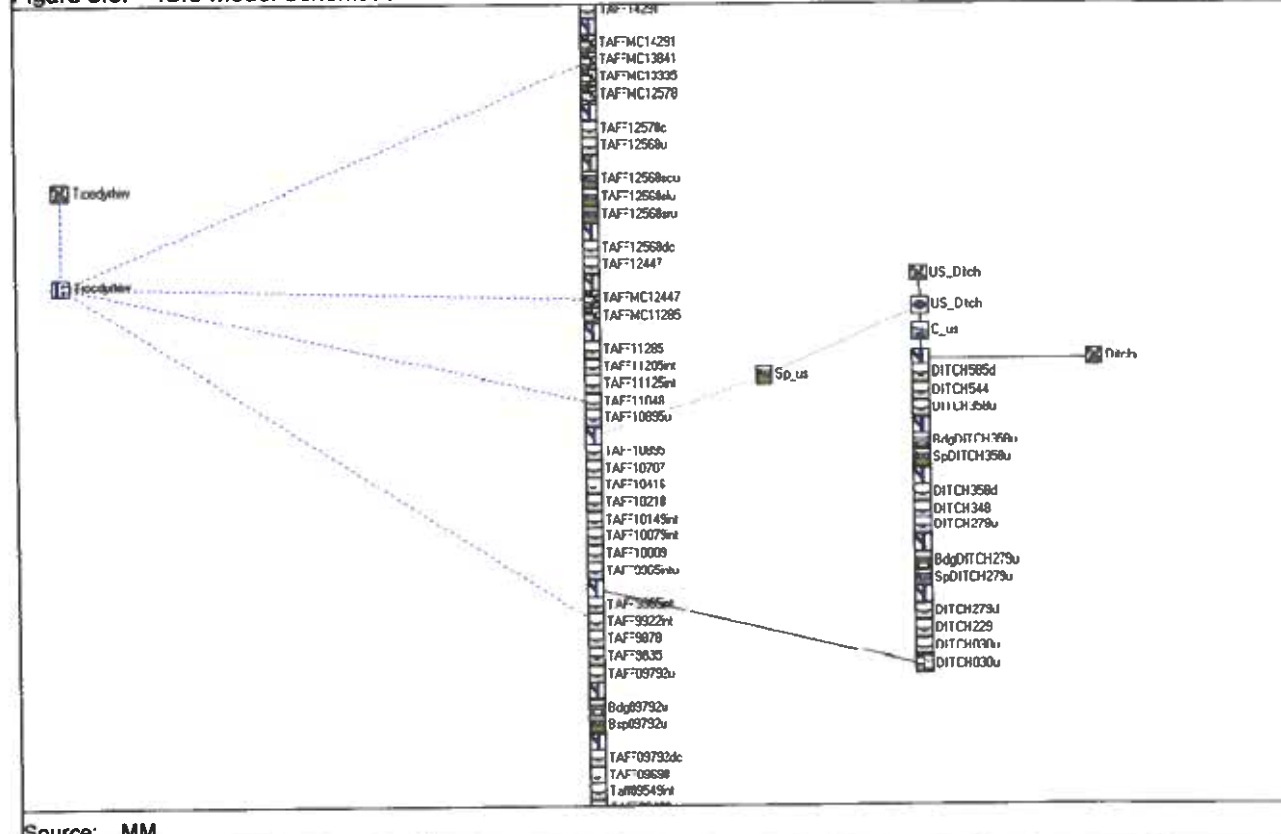
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[illegible]

Source: MM

Figure 5.5: ISIS Model Schematic of Ditch Channel



5.4.1.5 Representation of Floodplain

The river sections as set up in the FF model have been revised to accommodate flows of the higher return periods, in particular flows of the 1000 year design flood. The following steps have been undertaken:

- Geo-referencing of cross sections based on provided and revised gxy file,
- Removal of in-built glass walls,
- Extension of cross sections,
- Adding floodplain spills at structures such as weirs and bridges as appropriate.

Based on the gxy file, survey locations, the ISIS chainages and the location of structures on the 1:10K OS map the cross sections have been geo-referenced as accurately as possible.

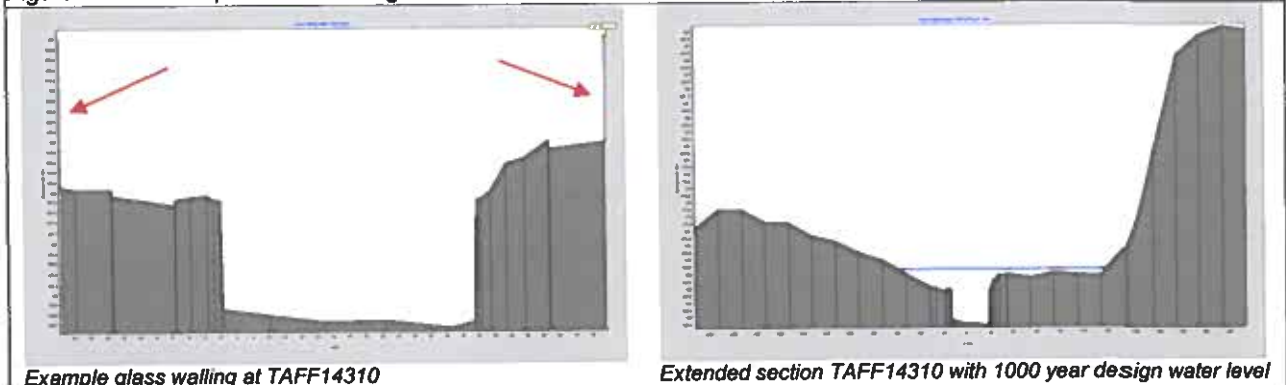
The geo-referenced cross sections have been extended onto the floodplain using the facilities provided in ISIS Mapper.

Figure 5.6 shows an example of in-built glass walling at cross section TAF-14310. Any glass walling has been removed and the cross sections have been extended onto the floodplain. The extended section together with the 1000 year peak water level is shown in the same Figure 5.6.

The extended sections have been used for the mapping of the flood extents in the 1D modelling reach.

It should be noted that no changes to the representation of the floodplain have been made in the reach which will be linked to the 2D model.

Figure 5.6: Example Glass Walling and Extension of Cross Section at TAFF14310



Source: MM

5.4.1.6 Defences

There are no defences in the 1D modelling reach.

5.4.2 1D-2D Linked Model

The 2D floodplain model was constructed using TUFLOW build 2009-07-AE-isp. TUFLOW is a computer program for simulating depth-averaged, two and one-dimensional free-surface flows such as those which occur from floods and tides. TUFLOW is specifically orientated towards establishing flow patterns in coastal waters, estuaries, rivers, floodplains and urban areas where the flow patterns are essentially 2D or 3D in nature, and cannot be represented or would be awkward to represent using a 1D network model. The TUFLOW computational engine computes flow paths by dividing the floodplain into a grid of individual cells. The flow of water between cells is then computed repeatedly at regular time-steps by solving the 2D shallow water equations to estimate the flood spread and flow. As each cell contains information on level, flood flows are routed in any direction and will naturally tend to follow the local topography. These models are therefore suited to cope with morphological changes in the floodplain.

5.4.2.1 Development of DTM

The Digital Terrain Model (DTM) used in this study was constructed using LiDAR data provided by the Environment Agency. The LiDAR used has a 2 m grid size; with a vertical accuracy of ± 0.15 m. LiDAR provides good coverage of the study area.

The DTM was enhanced using z-lines to represent bank heights, roads and railways as well as the flood defences at Nixonville, Aberfan and Troedyrhiw as detailed in section 5.4.2.5.

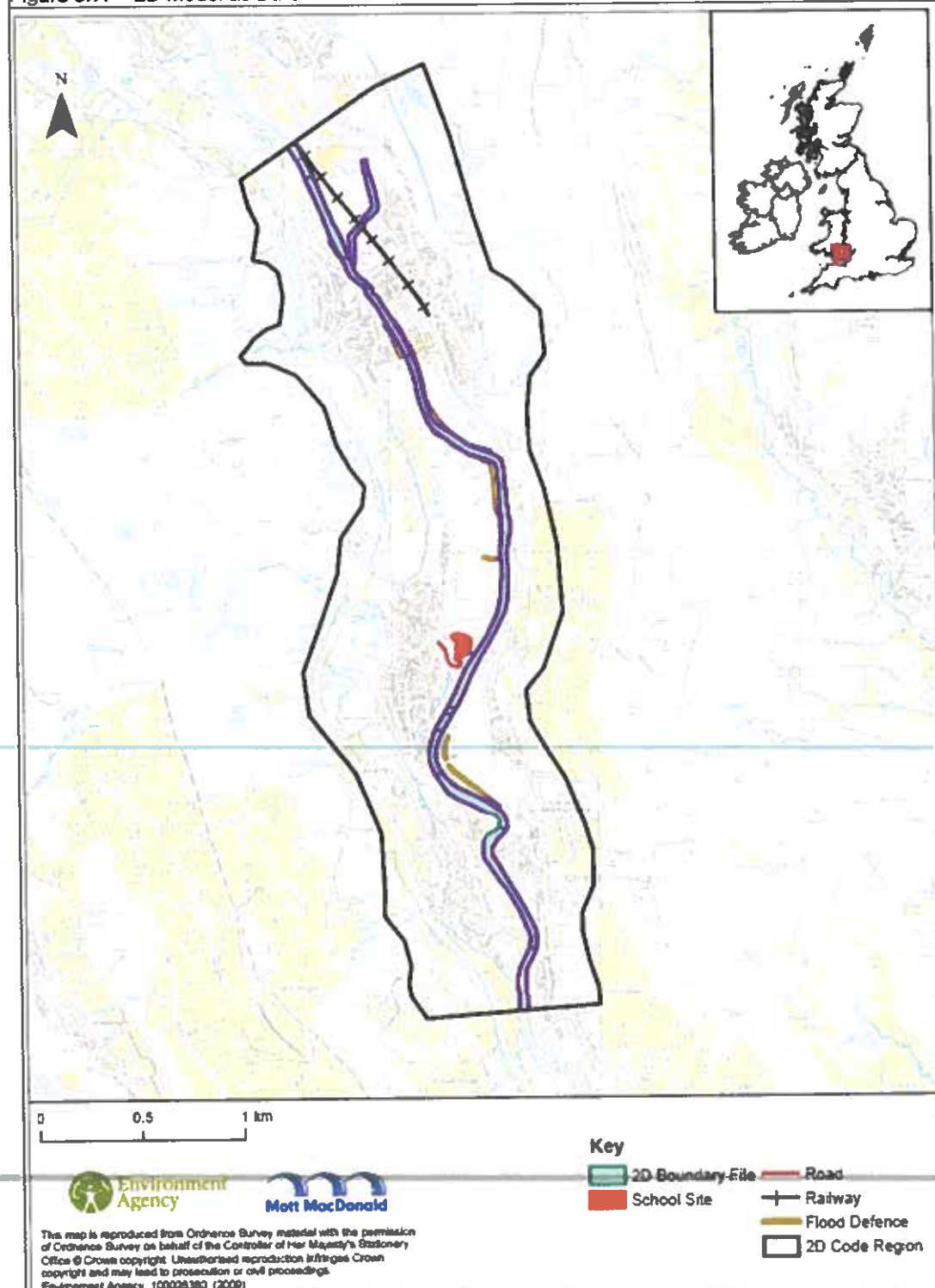
A MapInfo z file was used to represent the ground raising at the school site at Aberfan based on as-built drawings.

5.4.2.2 Development of 2D Model Grid

A 5m grid size was appropriate for this study, as it is recommended that the grid size is at least half the width of the watercourse. A 1 second timestep was used for the 2D model, and a 1 second timestep for the ISIS 1D model, with the exception of the 1 in 1000 year model where a 0.25 second timestep was used for both for the 2D and 1D ISIS models to aid stability.

One model boundary was derived for this study. Figure 5.7 shows the location of the model boundary (code region) together with the defences and the boundary of the 1D domain.

Figure 5.7: 2D Model as Built



P:\Cambridge\Demeter - Deedalus\NEM\Projects\266381 Merthyr 2D Modelling\3 GIS\3.3 Report Figures\Figure 5-16 Flood Defence

Source: Mott MacDonald

5.4.2.3 Hazard Rating

The model brief requires the hazard rating to be calculated for design events. The Flood Hazard Rating has been calculated in accordance with the *Supplementary Note on Flood Hazard ratings and Thresholds for Development Planning and Control Purpose - Clarification of the Table 13.1 of FD2320/TR2 and Figure 3.2 of FD2321/TR1* (Defra, 2008). The Flood Hazard has been calculated as:

$$HR = d \times (v + 0.5) + DF$$

where, HR is hazard rating; d is depth of flooding (m); v is the velocity of floodwaters (m/s) and DF is debris factor. DF can be determined by depth alone (conservative) or depth and land use, as shown in Table 5.4. For this study the conservative approach was used to determine DF.

Table 5.4: Debris Factor

Depths	Pasture/Arable	Woodland	Urban	Conservative
0 to 0.25m	0	0	0	0.5
0.25 to 0.75m	0	0.5	1	1
D > 0.75m and/or V > 2.0 m/s	0.5	1	1	1

Critical values of Flood Hazard are shown in Table 5.5. These are used to categorise the hazard in terms of danger to people.

Table 5.5: Hazard Rating

Hazard Rating	Degree of Flood Hazard	Description
< 0.75	Low	Caution "Flood zone with shallow flowing water or deep standing water"
0.75 to 1.25	Moderate	Dangerous for some (e.g. children) "Danger: Flood zone with deep or fast flowing water"
1.25 to 2.0	Significant	Dangerous for most people "Danger: flood zone with deep fast flowing water"
> 2.0	Extreme	Dangerous for all "Extreme danger: flood zone with deep fast flowing water"

5.4.2.4 Floodplain Roughness

Mastermap tiles were provided by the Environment Agency to be used to derive the floodplain roughness. Polygons for each land use were derived from the Mastermap tiles; these were then read into the 2D model with a Manning's n value assigned to each land use type. Table 5.6 details the Manning's n roughness coefficient used for each land use type in the 2D domain. A default value of 0.1 was set for the 2D domain.

Table 5.6: Floodplain Manning's Roughness Values

Mastermap Descriptive Group	Mastermap Feature Code	Manning's n value
Building	10021	0.30
General Surface	10053/10054/10056	0.02
Glasshouse	10062	0.30
Inland Water	10089	0.035

Mastermap Descriptive Group	Mastermap Feature Code	Manning's n value
Landform	10093/10096/10099	0.04
Natural Environment	1011	0.10
Path	10119/10123	0.02
Rail	10167	0.04
Road or Track	10172	0.02
Structure	10185	0.03
Unclassified	10217	0.04

5.4.2.5 Defences

There are five defences that have been modelled in the 2D domain as shown in Figure 5.1.6. Section 5.6.2 details how these defences have been modelled in the 2D domain.

5.5 The Model Calibration and Verification

The calibration of the hydraulic model has been carried out using observed inflows at Merthyr Tydfil, the upstream end of the model together with individually calibrated ReFH models representing inflows from the various sub-catchments.

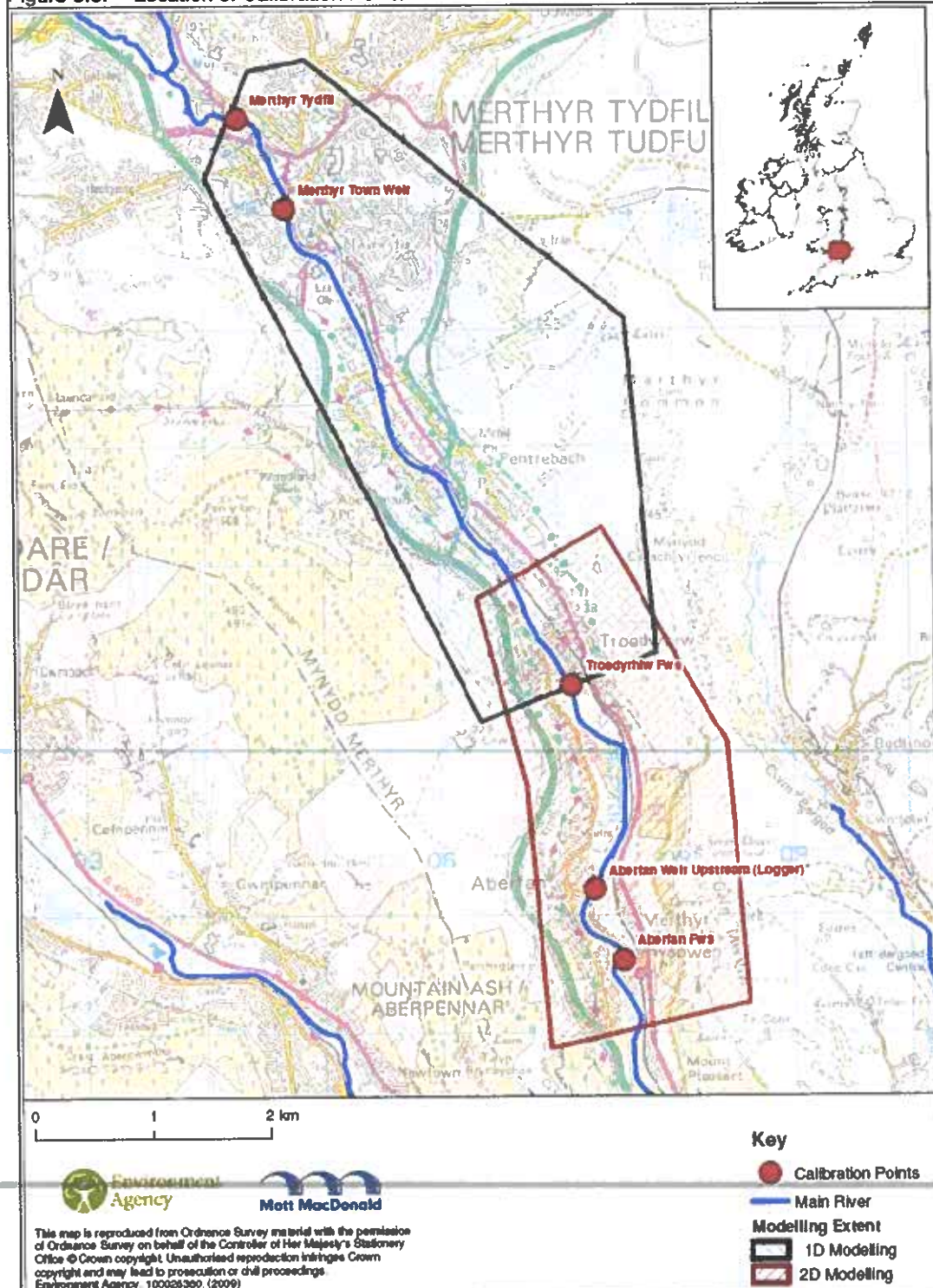
There are four gauges recording water levels in the modelling reach, three are located in the reach covered by the 1D ISIS model and one in the 1D-2D modelling reach. The locations are shown in Figure 5.8. Table 5.7 summarises the calibration points together with their respective ISIS cross section nodes.

The EA Wales also made photographs of the September 2008 flood (Calibration Event 5) available to assist the model calibration.

Table 5.7: Summary of Calibration Points

Water Level Gauge	ISIS cross section node	Comment
Merthyr Tydfil	TAFF15142c	Represented in ISIS by a Q-H unit
Merthyr Town Weir	TAFF14310	
Troedyrhiw	TAFF09400dc	
Aberfan Weir us	TAFF07348	No event data
Aberfan FWS	TAFF06571	(assumption: d/s of bridge)

Figure 5.8: Location of Calibration Points



5.5.1 1D Model

5.5.1.1 Introduction

Figure 5.11 to Figure 5.15 show the results of the calibration of the hydraulic model starting with the initial calibration which represents a comparison of observed and modelled water levels following the hydrological model calibration and prior to the development of the hydrodynamic model. Based on the initial calibration results changes have then been made to the model and further developments have been undertaken as described in Section 5.4.1 above. Modelled and observed water levels at the four calibration points have then been compared and analysed and subsequently changes have been made to the model. The final calibration results are also shown in Figure 5.11 to Figure 5.15 (*'Final Calibration'*) together with the initial results (*'Initial Calibration'*).

The results of the model calibration at Aberfan, located in the 1D-2D modelling reach, presented in this section are initial results and will be discussed in more detail in Section 5.5.2.

5.5.1.2 Peak Water Levels at Key Locations

Table 5.8: Comparison of Observed and Modelled Peak Water Levels at Key Locations

Event	Merthyr Town weir		Troedyrhiw		Aberfan	
	Observed peak water level (mAOD)	Modelled peak water level (mAOD)	Observed peak water level (mAOD)	Modelled peak water level (mAOD)	Observed peak water level (mAOD)	Modelled peak water level (mAOD)
1	No observed data available					
2	166.81	167.23	135.21	135.37	124.01	124.14
3	166.76	167.04	134.96	135.00	123.52	123.78
4	-	-	134.83	134.88	123.98	123.69
5			136	135.99	124.97	124.65

Source: MM

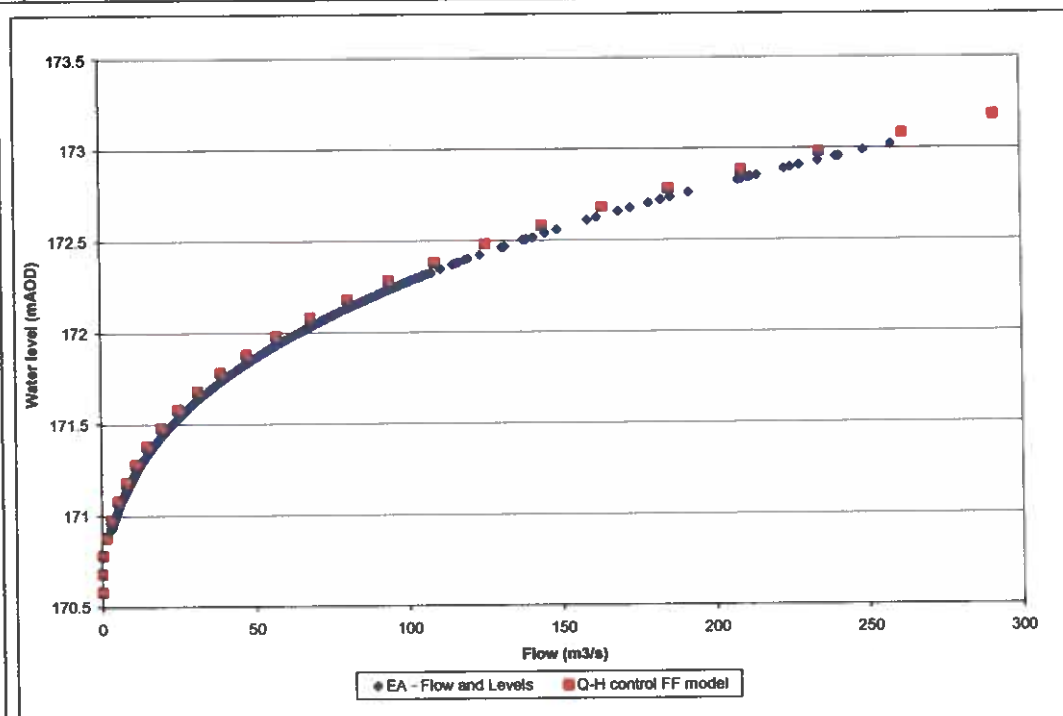
5.5.1.3 Merthyr Tydfil

The FF model originally represented the gauging structure at Merthyr Tydfil as Q-H control. Observed flows at Merthyr have been used at the upstream end of the model at Merthyr Tydfil for the purpose of the calibration of the hydraulic model. Three problems have been identified:

- There is a discrepancy (although small) between the EA rating and the Q-H control applied in the FF model which leads to a slight overestimation of water levels.
- The Q-H control is unsuitable for flows of high return periods as the Q-H control does not accommodate for the impact of backwater from rising water levels downstream of the weir.
- The existing rating is based on extrapolation of flow over the gauging structure and spot gaugings which might not correctly represent the level – flow relationship once the flood waters have been spilled onto the floodplain. Therefore modelling the gauging structure and the floodplain at the gauge is likely to result in a different rating and consequently a discrepancy between observed and modelled water levels. This work is currently being undertaken by Edenvale Young Associates and it is recommended to update the model according to the findings of their study.

Figure 5.11 shows a comparison of the EA rating and the Q-H control. The difference in water levels does not exceed 0.05 m. The discrepancy is mirrored in the difference between observed and modelled water level for the calibration events shown in Figure 5.11 to Figure 5.15.

Figure 5.9: Comparison on of EA Rating and Q-H Control in FF Model at Merthyr Tydfil Gauge



Source: MM

5.5.1.4 Merthyr Town Weir

Observed water levels at Town Weir are only available for two of the five calibration events. Both events initially showed a significant overestimation in water levels upstream of the weir.

Changes to weir, the upstream cross section and the representation of the flow which does not go over the structure have been made. The fishpass has been added to represent the actual conditions.

Following the changes the difference between the modelled and observed water levels has improved although the computed levels are still up to 0.4 m higher than the recorded water levels. Despite all efforts a match between the modelled and observed levels could not be achieved. Therefore model results and also recorded levels at this location should be treated with caution.

It should be noted that at the time of the final submission of this document in September 2010 a rating review has been carried out based on recent survey. This information could not be used for this study. It is recommended to investigate whether the hydrology needs to be revised in the context of the changes to the flow calculations.

5.5.1.5 Troedyrhiw

The modelled and observed water levels at Troedyrhiw generally match well. The difference in peak water levels ranges from 0.16 m (Event 2) to -0.01 m (Event5).

5.5.1.6 Aberfan

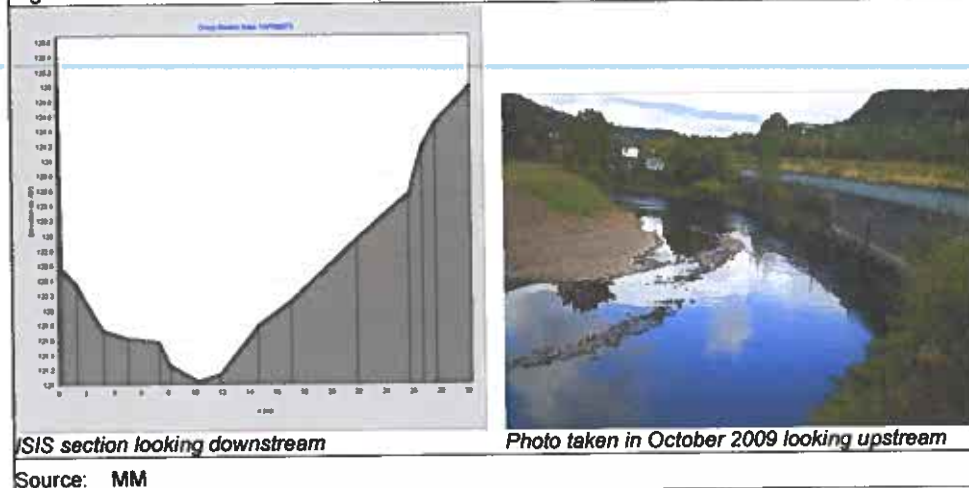
Water levels at the Aberfan gauge are affected by the accumulation of shoals in the area upstream and downstream of the footbridge. It is likely that the discrepancy between modelled and observed water levels which ranges from 0.13 m (Event 2) to 0.32 m (Event5) is partly caused by the changes in bed level and subsequent variation in recorded water levels which are not fully reflected by the model.

However, the cross section upstream of the footbridge (see Figure 5.10) appears to be based on survey taken at a time when some shoaling material had accumulated (note in Figure 5.10 that right bank concrete wall is not visible as it is covered by shoaling material nor was the wall modelled in ISIS).

As the shoaling material is regularly removed by the EA Wales (for details see Section 2.2.5) any model representation of the channel in this area is likely to give erroneous water levels.

A sensitivity analysis has been carried out (see Section 6.1.4 and 6.2.2.4) to assess the error related to the changes in profile at this particular location.

Figure 5.10: ISIS Cross Section upstream of the Footbridge at Aberfan



5.5.1.7 Other Modifications

Following the model development described in Section 5.4.1 above, an independent internal model review has been carried out which resulted in a number of recommendations which are summarised below in Table 5.9 together with comments on the action taken.

Generally the subsequent changes had no significant impact on the calibration of water levels at the key locations described above. However, these improved the stability of the model and water levels in areas where improvements have been made should now be modelled more accurately.

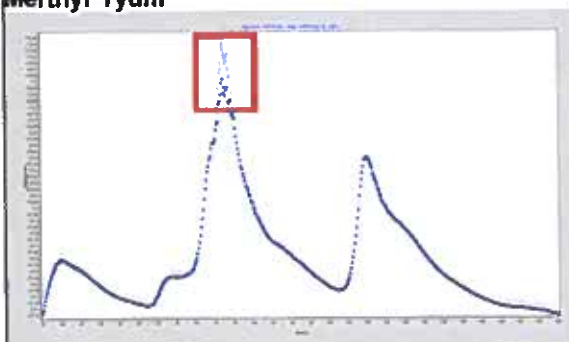
Table 5.9: Summary of Independent Internal Model Review and Actions Taken

Modeller's comments	Checker's notes
High Priority	
corrected	1. DITCH manning's n is 0.35 and should be 0.035
corrected	2. QH control C _{us} has decreasing flow with increasing stage at high water level.
spills representing right and left bank added	3. TAFF14310wu and fish pass may need a bypass spill for the 1:100 event.
spills representing right and left bank added	4. TAFF07334w needs a bypass spill for 1:100
also u/s drop of 1.5 m (spill), to be discussed	5. TAFF07334w weir crest level is 2m above upstream river bed level. Is this right?
Manning's n values at TAFF15142 corrected (no justification for low floodplain values)	6. TAFF15142c and TAFF15142c_c have out-of-bank Manning's n = 0.022 and in-bank n = 0.035. Check out-of-bank Manning's n.
Ditch000 spill modified (area as u/s section, bed level raised by 100mm)	7. DITCH000 spill has much smaller flow area than the upstream section.
no changes, see photos provide at 14310 (Town Weir, loss is high)	8. There are high head losses at TAFF14310, TAFF12568, TAFF07335, TAFF00966c during the 1:100 yr event. Check losses are appropriate.
Low priority	
split, coefficients adjusted	9. Bank flow and weir flow at TAFF12568u may be better split over two spills with different weir coefficients.
No action taken	10. TAFF00966c, TAFF07348 and TAFF08817 are inline spills with weir coefficient 1.7. Comments should be added to justify high coefficient. Particularly for the spills that do not drown out at high stage. It may also be better to separate them into parallel spills with different weir coefficients to represent in-bank and out-of-bank flow paths.
corrected	11. Bridge Bdg14888u has FLAT arch setting but is not flat.
No action taken	12. Weir TAFF14310wu should include comment about Cv = 1.5
No action taken	13. Weir TAFF14310fu should include comment about Cd = 1.44. Modular ratio should be checked since it might be expected that a fish pass would drown earlier than a normal weir. RNWEIR units are more stable than a WEIR unit and are usually to be preferred.
Corrected	14. TAFF07348 glass walls during 1:100.
corrected	15. TAFF07347c has a flat bed which will be unstable at low flow.
No action taken	16. TAFF07263, TAFF07029 bridge section extends past the abutments of bridge. This can produce high blockage ratios warnings and affect the backwater calculations.

Figure 5.11: Comparison of Modelled and Observed Water Levels – Event 1

■ ■ ■ Observed water level
— Modelled water level
— Modelled water level

Merthyr Tydfil



No further changes

Initial Calibration

Merthyr Town Weir

No data

Troedyrhiw

No data

Aberfan FWS

No data

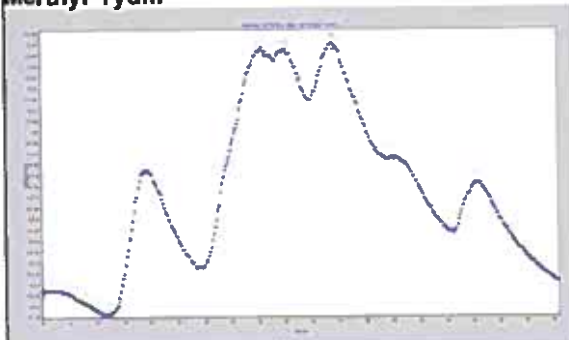
Source: MM

Figure 5.12: Comparison of Modelled and Observed Water Levels – Event 2

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

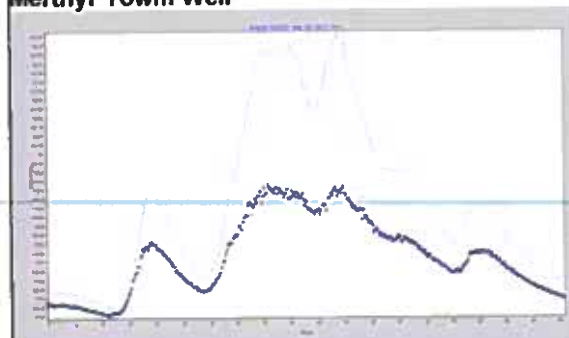
Merthyr Tydfil



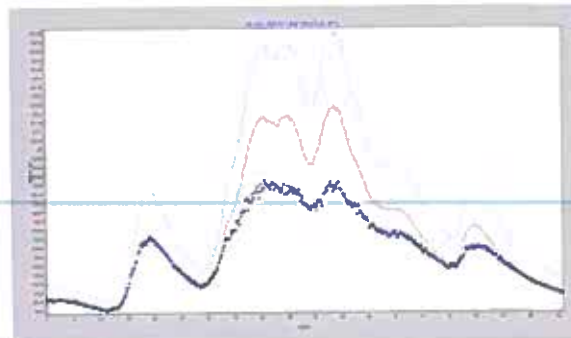
No further changes

Initial Calibration

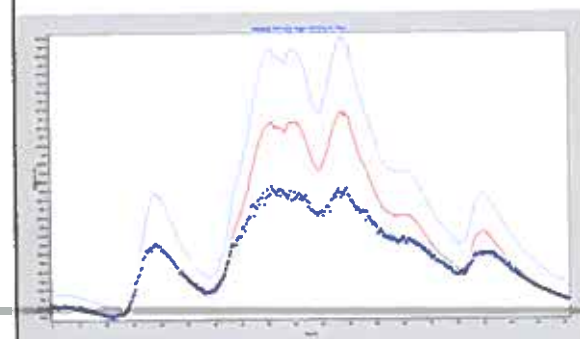
Merthyr Town Weir



Initial Calibration



Model Development



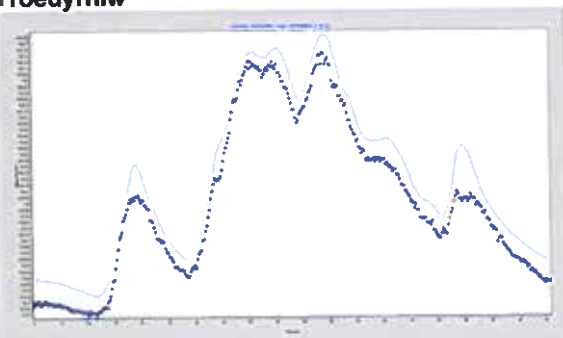
Final Calibration

Figure 5.12: Comparison of Modelled and Observed Water Levels – Event 2

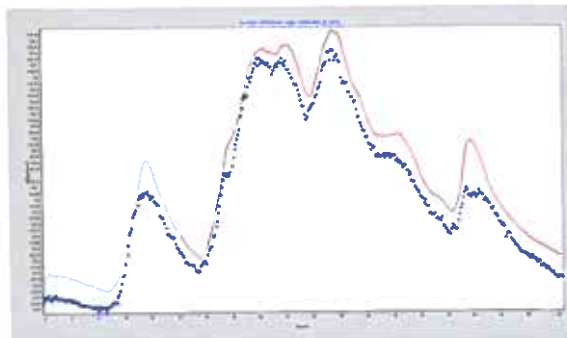
■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

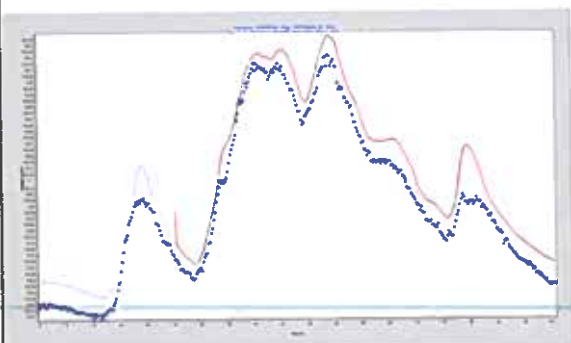
Troedyrhiw



Initial Calibration

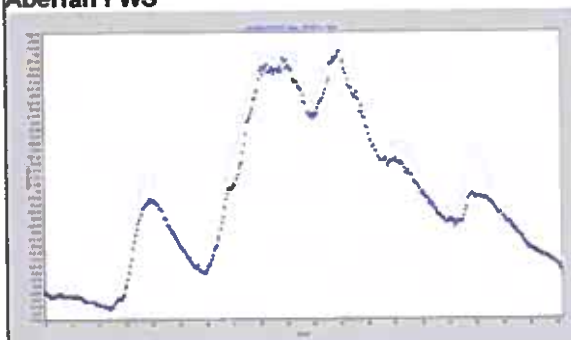


Model Development

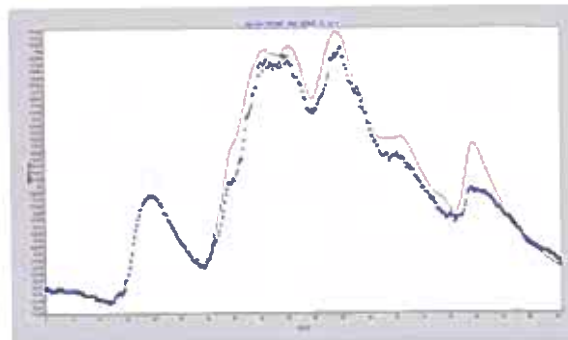


Final Calibration

Aberfan FWS



Initial Calibration

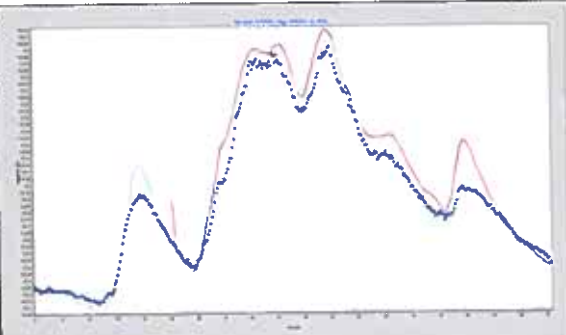


Model Development

Figure 5.12: Comparison of Modelled and Observed Water Levels – Event 2

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration



Final Calibration

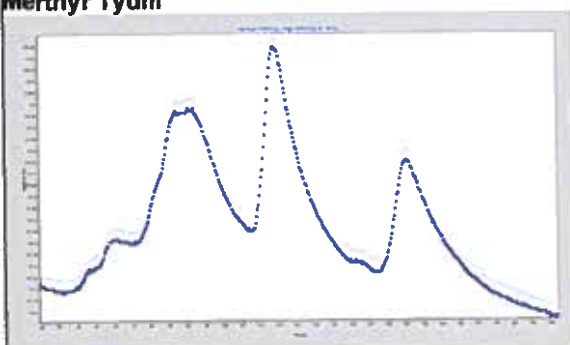
Source: MM

Figure 5.13: Comparison of Modelled and Observed Water Levels – Event 3

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

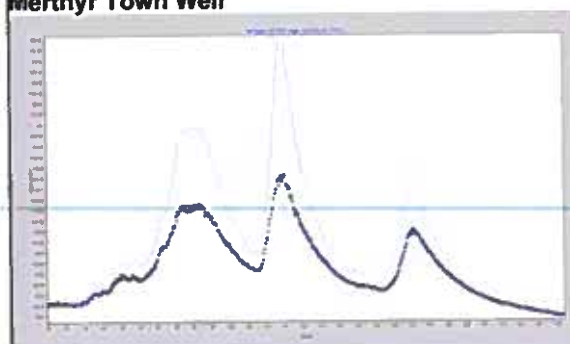
Merthyr Tydfil



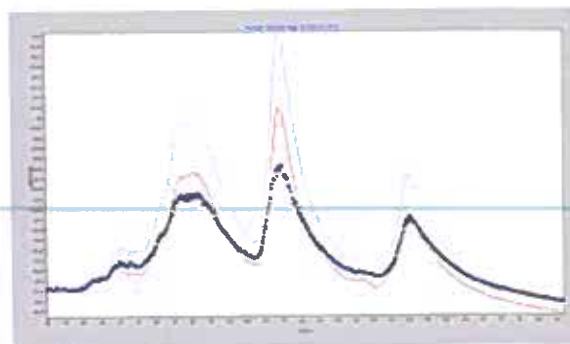
No changes

Initial Calibration

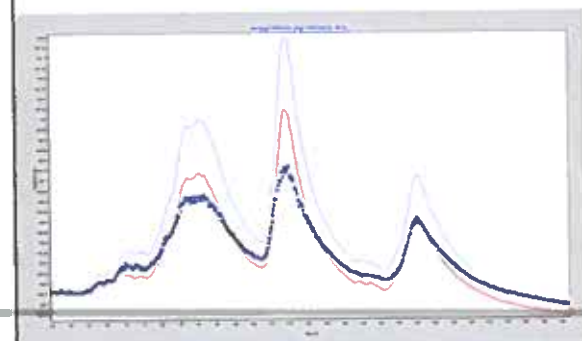
Merthyr Town Weir



Initial Calibration



Model Development



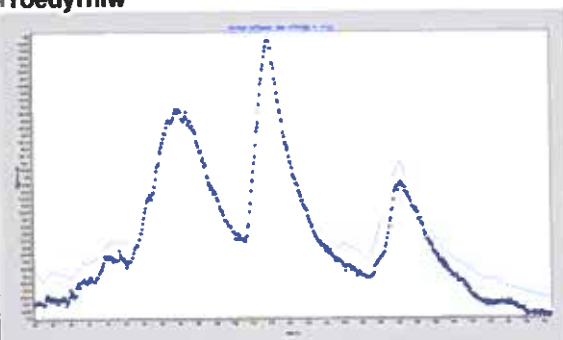
Final Calibration

Figure 5.13: Comparison of Modelled and Observed Water Levels – Event 3

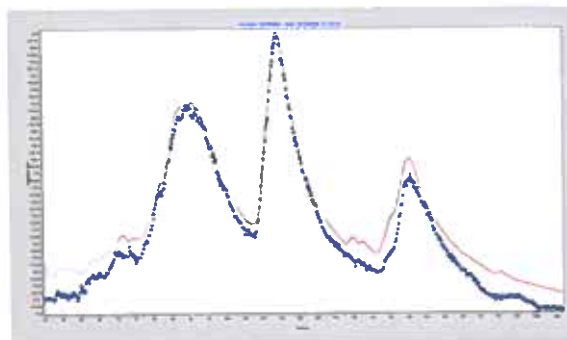
■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

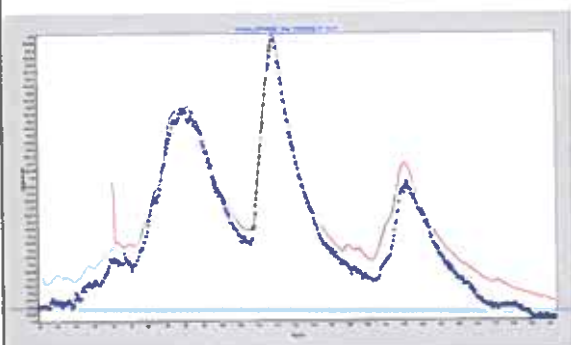
Troedyrhiw



Initial Calibration

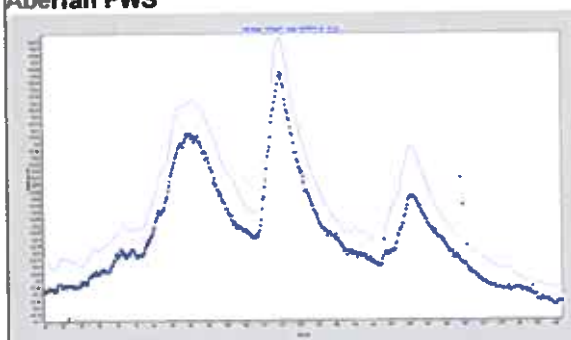


Model Development

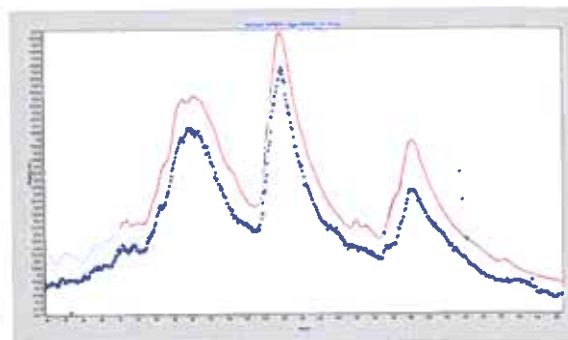


Final Calibration

Aberfan FWS



Initial Calibration

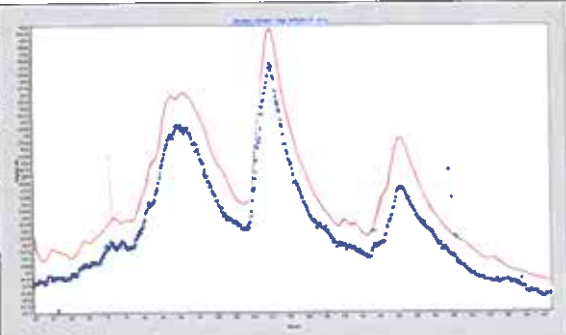


Model Development

Figure 5.13: Comparison of Modelled and Observed Water Levels – Event 3

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration



Final Calibration

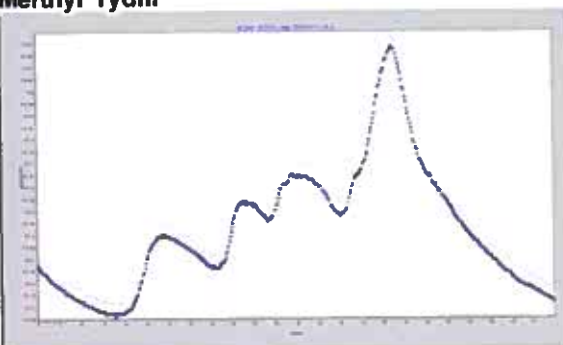
Source: MM

Figure 5.14: Comparison of Modelled and Observed Water Levels – Event 4

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

Merthyr Tydfil

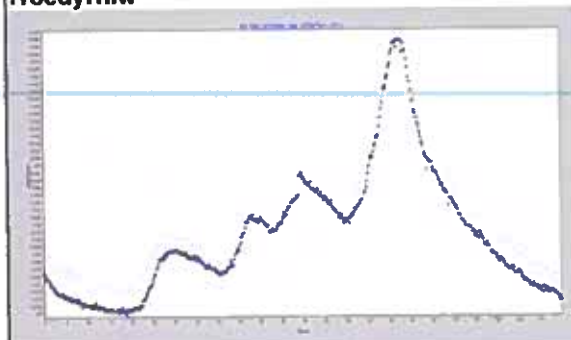


No changes

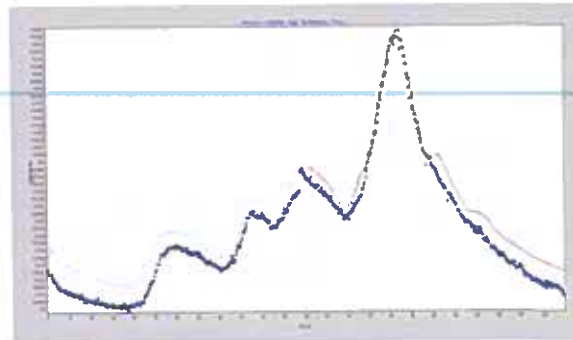
Initial Calibration

Merthyr Town Weir
No data

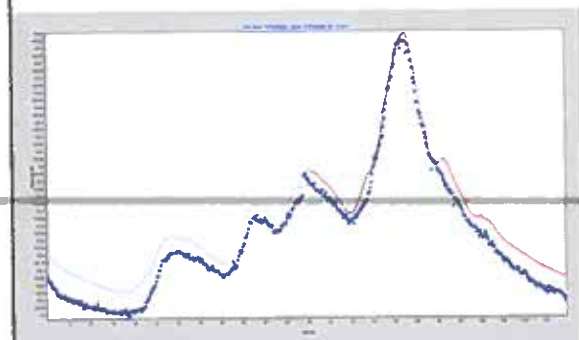
Troedyrhiw



Initial Calibration



Model Development



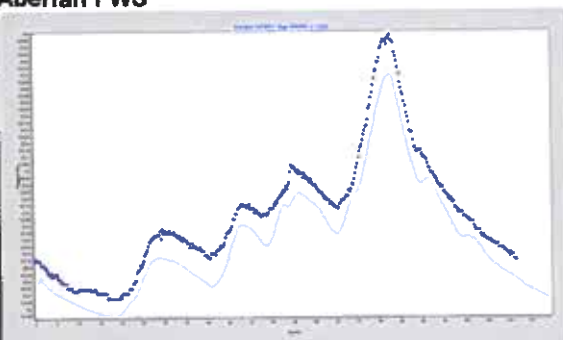
Final Calibration

Figure 5.14: Comparison of Modelled and Observed Water Levels – Event 4

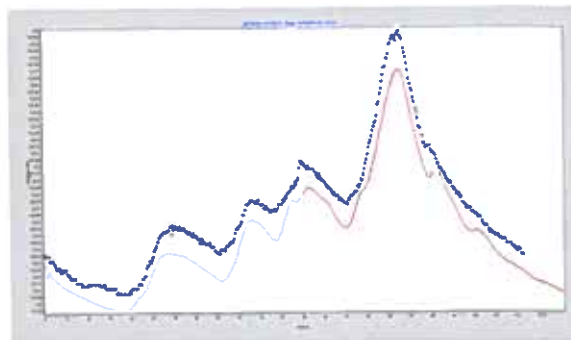
■ ■ ■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

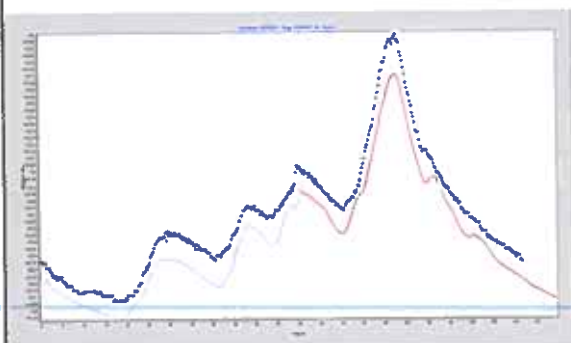
Aberfan FWS



Initial Calibration



Model Development



Final Calibration

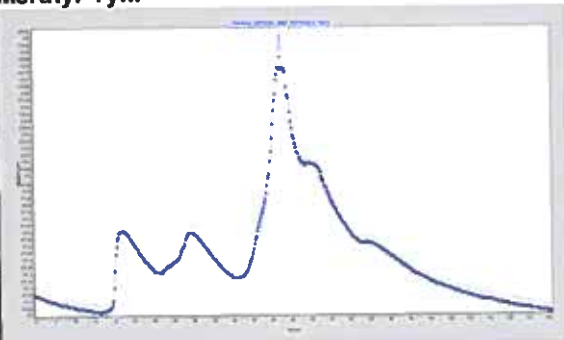
Source: MM

Figure 5.15: Comparison of Modelled and Observed Water Levels – Event 5

■ Observed water level
— Modelled water level
— Modelled water level

Note: start time and run time have been varied during calibration

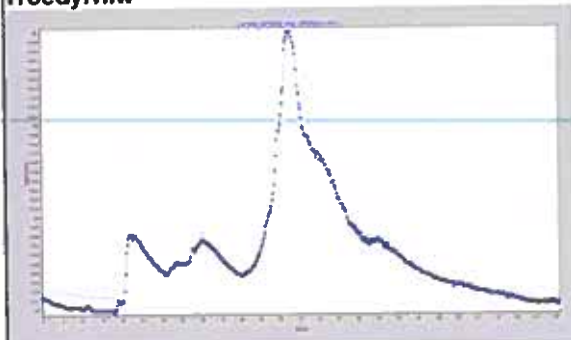
Merthyr Tyfil



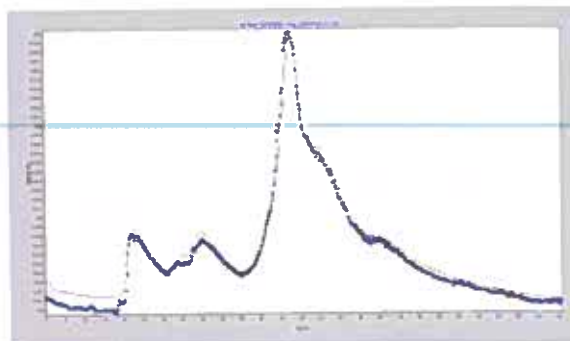
Merthyr Town Weir

No data

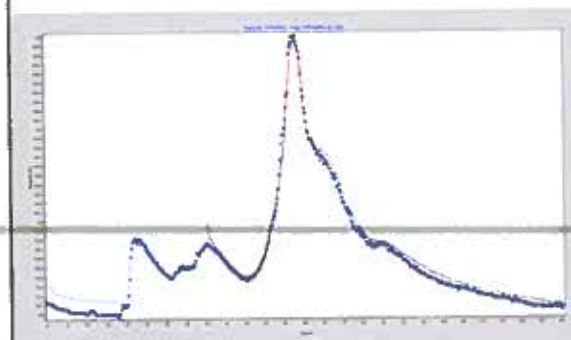
Troedyrhiw



Initial Calibration

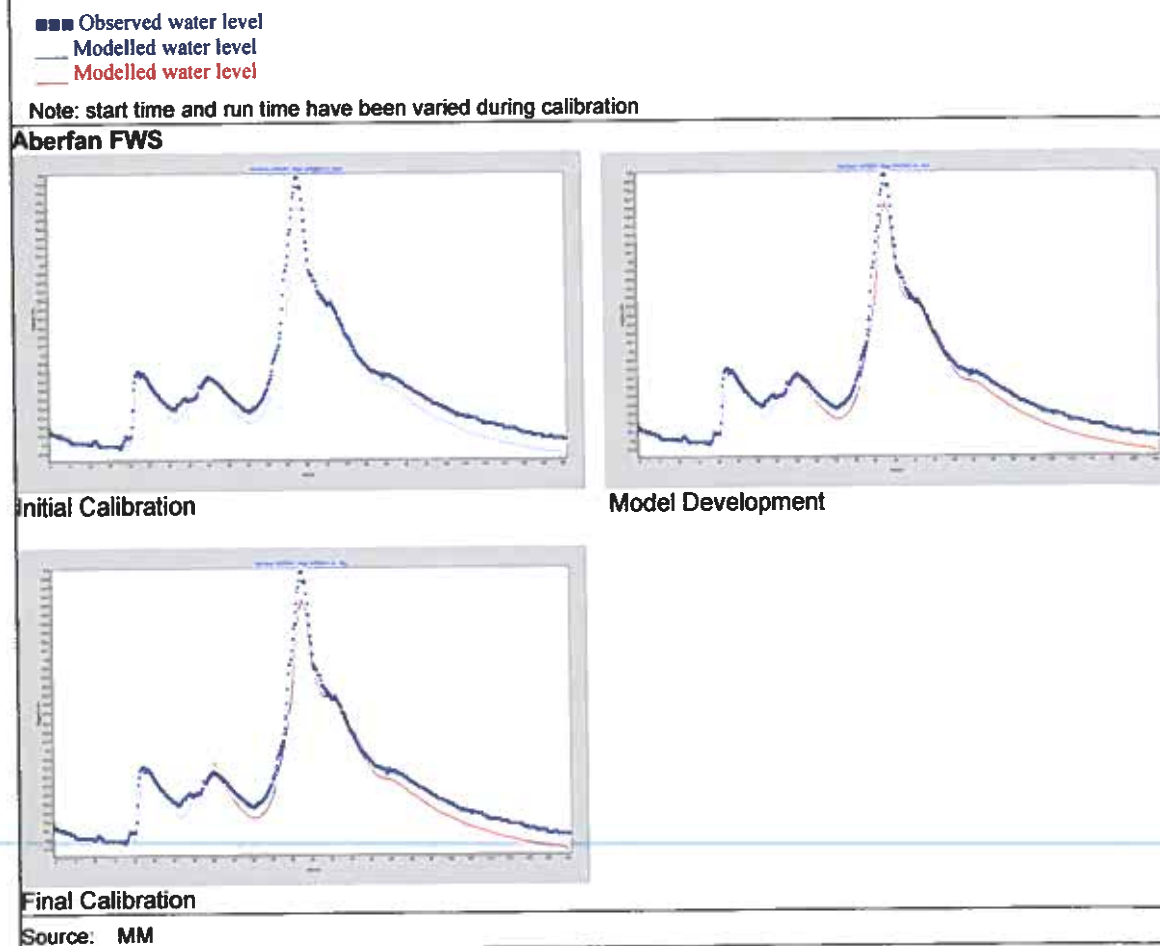


Model Development



Final Calibration

Figure 5.15: Comparison of Modelled and Observed Water Levels – Event 5



5.5.2 1D-2D Model

For the calibration of the 1D-2D linked model the event in September 2008 (Event 5) became the key event as, in addition to the recorded water levels at Treodyrhiw and Abefan, photographs taken during the event together with records taken by EA staff on site provided valuable information not available for other events. Event 5 was modelled in the 1D-2D linked model. To calibrate the linked model, modelled water levels in the channel at Aberfan and Treodyrhiw were compared with observed water levels. Photographs taken during the 2008 flood event were also compared with modelled extents and depths.

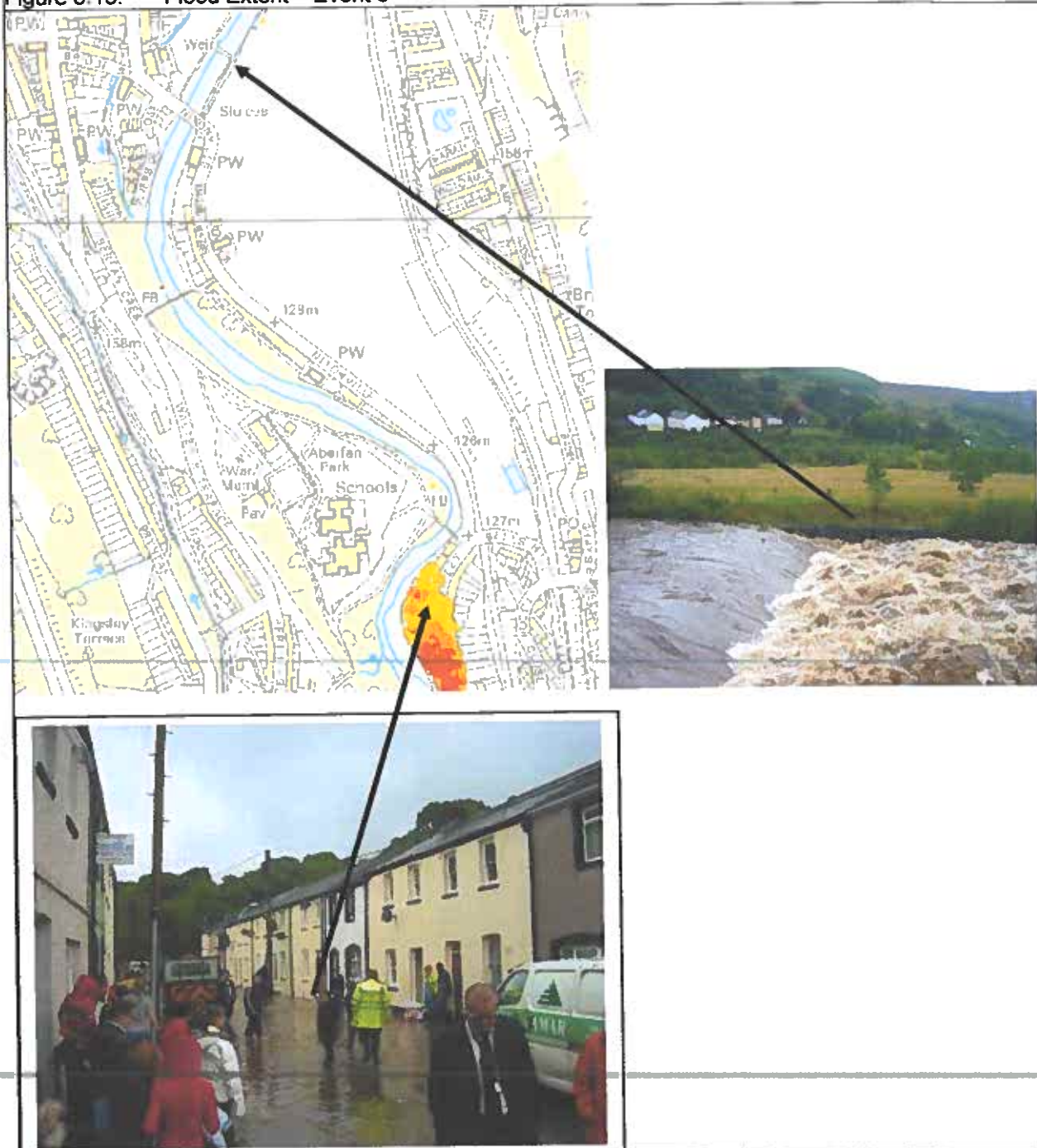
During calibration the following changes were made to the model:

- Modification of the z-line representing the River Taff bank heights.
- Removal of a duplicated weir in the 1D model
- Manning's n to a value of 0.3 in downstream of cross section TAFF08606 in the 1D model.

Figure 5.16 shows the flood depth modelled for Event 5 together with photographs at key locations. The photograph taken at the weir upstream of Nixonville Road Bridge indicates that the floodplain remained dry

during the event. This is replicated by the model. Flooding occurred at Crescent Street which is also shown in the model results.

Figure 5.16: Flood Extent – Event 5



Source: Photos EA, Figure MM

5.6 Development of Undefended Model

5.6.1 1D Modelling Reach

There are no defences in the 1D modelling reach.

5.6.2 1D-2D Modelling Reach

There are five defences that have been modelled in the 2D domain. These are listed below and shown in Figure 5.17.

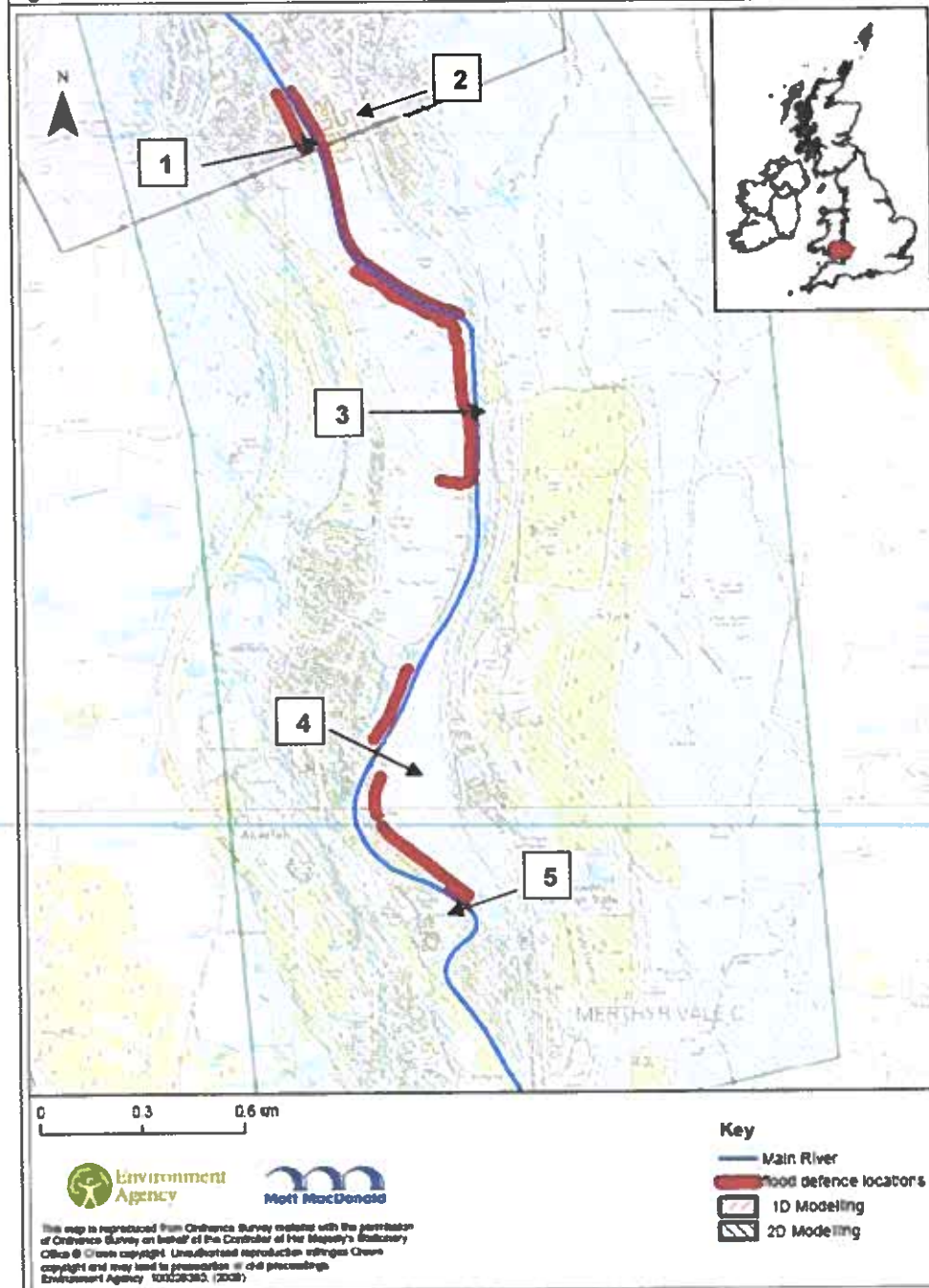
- 1 – Left bank at Troedyrhiw school
- 2 – Right bank at Troedyrhiw school
- 3 – Right bank upstream of Aberfan school
- 4 – Right bank downstream of Aberfan school
- 5 – Along Nixonville.

Table 5.10 details how these defences have been modelled in the 2D domain.

Table 5.10: Defences Considered in the 2D Domain

Defence	Defended Model	Undefended Model
1	Included in "2d_zln_banks_Taff_009.MIF"	"2d_zln_troed_LB_undefended.MIF" reduces point elevation in "2d_zln_banks_Taff_009.MIF" along this reach of the river.
2	Added as z-line "2d_zln_troed.MIF" which contains the elevation the grid should be raised to.	"2d_zln_troed.MIF" removed from .tgc. "2d_zln_troed_RB_undefended.MIF" reduces the defence elevation in the model grid.
3	Added as z-line "2d_zln_Aberfan.MIF" which contains the elevation the grid should be raised to.	Remove "2d_zln_Aberfan.MIF" from .tgc. "2d_zln_Aberfan_US_undefended.MIF" reduces the defence elevation in the model grid.
4	Included in "2d_zln_banks_Taff_009.MIF"	"2d_zln_Aberfan_DS_undefended.MIF" reduces point elevation in "2d_zln_banks_Taff_009.MIF" along this reach of the river.
5	Added as z-line "2d_zln_NixonvilleWalls.MIF" which details the elevation the grid in the 2D domain should be raised by.	"2d_zln_NixonvilleWalls.MIF" removed.

Figure 5.17: Location of Flood Defences



Source: MM

6. Model Runs and Results

6.1 Design Model Runs

6.1.1 Summary Design Model Parameters

Table 6.1: Adjustment Factors to ReFH Parameters for Design Model Runs

Sub-catchment	C _{max}	T _p	BR	C _{ini}
Pontsticill	1	1	0.8	1
PontsticillLat	1	1	0.8	1
Llwynon	1	1	0.8	1
LlwynonLat	1	1	0.8	1
Aberfan	1.28	1.3	1	0.7
Bargoed	1.28	1	1	0.7
Ditch	1.28	1	1	0.7
Fiddlers	1.28	1.3	1	0.7
Merthyr Town	1.28	1	1	0.7
Nant Morlais	1.28	1	1	0.7
Taff Bargoed	1.28	1.3	1	0.7
Troedyrhiw	1.28	1.3	1	0.7

Source: MM

The critical storm duration is 15.75 hrs.

6.1.2 Design Model Runs

Table 6.2: Final Scaling Factors of the ReFH inflows for Design Model Runs

Return Period (T _p)	MERTHYR ³⁾ TYDFIL			FIDDLERS ELBOW		
	Target flow (m ³ /s)	Scaling factor u/s ReFH models	Design peak flow (m ³ /s)	Target flow (m ³ /s)	Scaling factor d/s ReFH models	Design peak flow (m ³ /s)
5	134	0.78	136	198	1.05	197
10	161	0.75	162	231	1.00	231
15	178	0.73	178	251	1.00	254
20	191	0.72	190	266	0.90	263
50	236	0.7	237	318	0.85	321
75	258	0.69	260	344	0.8	343
100	275	0.68	276	363	0.8	363
200	320	0.66	323	415	0.7	413
1000 (WOS) ¹⁾	507	0.692	507	655	0.77	651
1000 (WS) ²⁾	-	0.692	547	-	0.77	558

1) without siphons at Llwynon reservoir, model run to derive scaling factor for run with siphons

2) with siphons at Llwynon reservoir

These scaling factors have been initially derived using the 1D model and have then been finalised based on the results of the 1D-2D model.

6.1.3 Discussion of Errors

The hydrological and the hydraulic model have been calibrated independently where possible to assist the identification of the sources of errors.

The hydrological calibration has initially been carried out for individual events and parameters have been derived on an event basis. Average design model parameters have subsequently been derived from these event parameters and the model results re-assessed. Flows have been assessed at two key locations: Merthyr Tydfil and Fiddler's Elbow. The average error in modelled peak flow at Fiddlers Elbow is $\pm 17\%$.

The hydraulic model calibration has been carried out using observed inflows where available (at Merthyr Tydfil) and modelled inflows as calibrated for the individual calibration events. The error in terms of modelled peak water levels varies locally and between the events. It can be as high as +0.35m (at Merthyr Town Weir) and as little as +0.06m (at Troedryhiw).

The design peak flows have been estimated using the Statistical Method and therefore there are uncertainties attached, in particular for the higher return periods.

For the design model runs, Merthyr Tydfil and Fiddler's Elbow have been used as control points. Inflows from the sub-catchments have been scaled to match the target design flows at these control points thereby minimising the error in design water levels.

6.1.4 Sensitivity Tests

Tests have been carried out to assess the sensitivity of the model results to changes in the assumed conditions such as the seasonal variation in floodplain roughness or the blockage of structures due to debris in the river which has not been considered in the model. The sensitivity tests are based on the 100 year design model run. The test runs carried out are listed below:

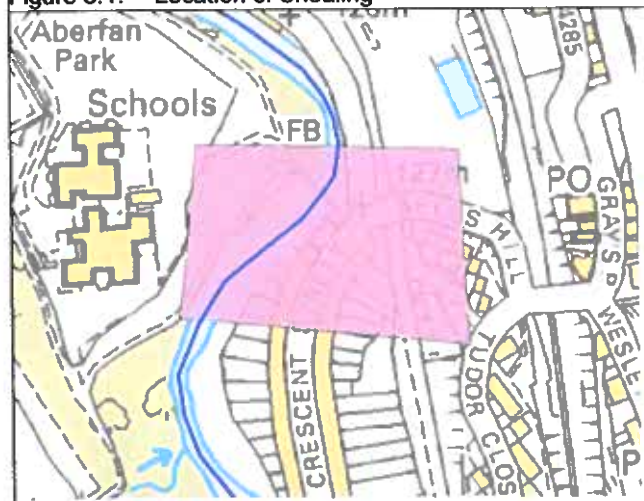
- 50% blockage at structures on the Ditch,
- Floodplain roughness set at 0.1,
- Height of buildings in the 2D domain increased to 0.15 m,
- De-shoaled channel at Aberfan,
- Additional shoaling at Aberfan.

In the vicinity of Crescent Street in Merthyr Vale shoaling occurs in an area upstream and downstream of the footbridge shown in Figure 6.1. The EA Wales regularly removes the accumulated material upstream of the bridge. It was agreed that investigation of the impact of accumulated material blocking the river bed should be carried out jointly for the two areas affected.

The de-shoaling scenario made use of the bank survey carried out for this study by the EA in 2009, which measured the height of the toe of the wall on the right bank which was invisible when MM went on a site visit in September 2009 (see photograph in Figure 6.1).

The worst case shoaling scenario made use of the most recent survey section with some added shoaling.

Figure 6.1: Location of Shoaling



30/09/2009 -- Looking downstream from footbridge

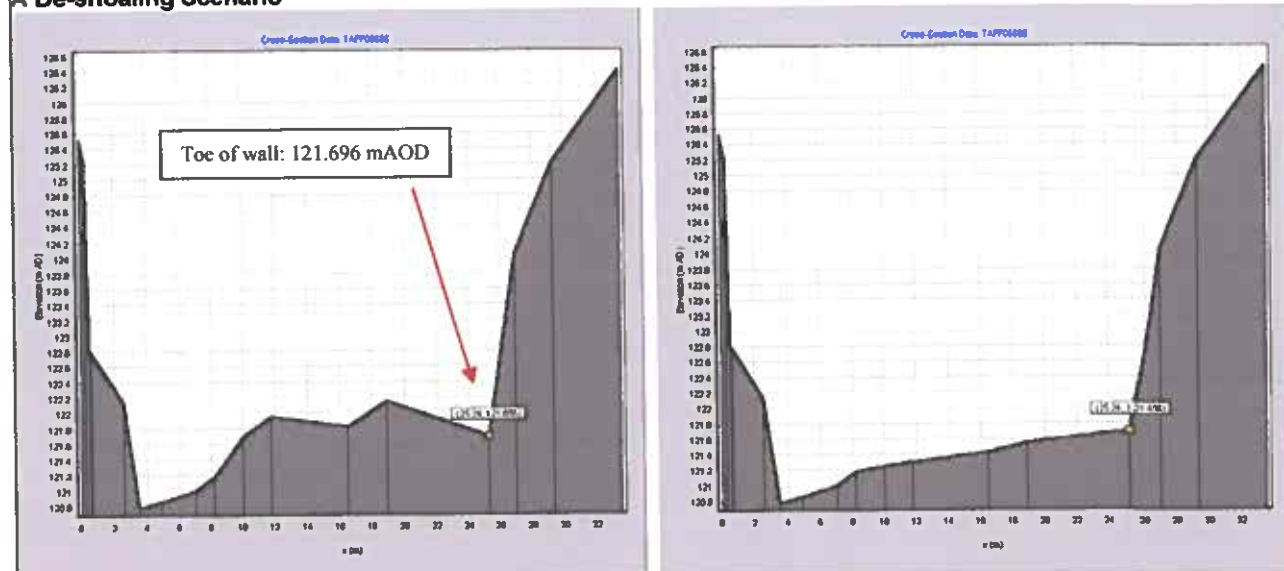


30/09/2009 -- Looking upstream from footbridge

Source: MM

Figure 6.2: Cross Section Modifications at TAFF06585 for Shoaling Scenarios

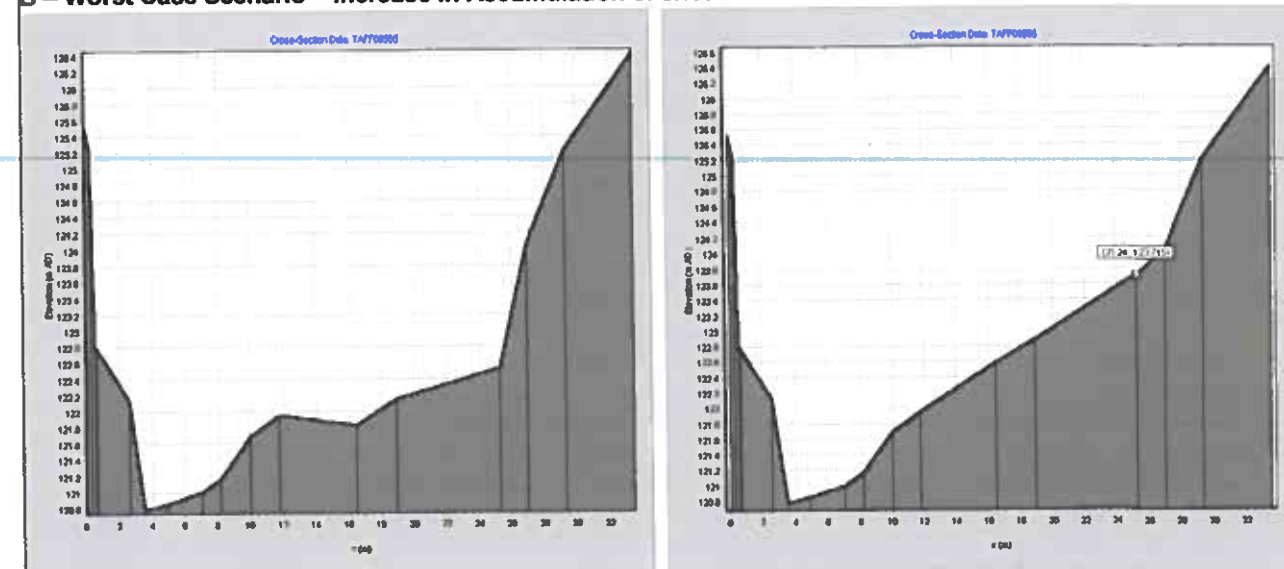
A De-shoaling Scenario



Step 1 - ISIS FF Model section with bed level modified at toe of the wall as per survey

Step 2 - Modification of bed level to represent de-shoaled conditions

B - Worst Case Scenario - Increase in Accumulation of shoal



Unmodified ISIS FF Model section

Modification of bed level to represent additional shoaling
On right bank further covering the toe of the wall

Source: MM

6.2 Results of Model Runs

6.2.1 Mapping Outputs

The mapping outputs to be delivered (specified in the Brief Revision 4) are summarised in Table 6.3. It should be noted that flood extents derived from 1D model results have been excluded from the deliverables by the EA Wales.

Maps have also been produced for Event 5 and the sensitivity model runs described in Section 6.1.4.

Although it was envisaged in the Brief to use the 1D model results in the reach between Merthyr Tydfil and Troedyrhiw to produce design flood extents, the EA Wales eventually decided not to make use of these flood extents due to the shortcomings in the model identified during the study. Further survey work is currently in progress which will help to address these issues. Consequently the results of the 1D model in terms of water levels and flood extent have not been discussed further.

Table 6.3: Mapping Deliverables

Mapping	Return Period (years)				
	5, 10, 15, 20, 50, 75	100	100+CC*	200	1000
Undefended scenario flood extents		✓			✓
Defended scenario flood extents	✓	✓	✓	✓	✓
Areas benefiting from defences		✓			
Hazard mapping (defended scenario)**	✓	✓	✓	✓	✓
Hazard mapping (undefended scenario)**		✓			✓
Animation**	✓	✓	✓	✓	✓

Flood maps have been produced from the outputs of the 2D modelling and are included in Appendix D. The maps show maximum flood hazard, depth and velocity for all of the scenarios that have been modelled. Table 6.4 details the model runs that have been completed, together with the provided maps.

Table 6.4: Summary of Model runs and Maps

Scenario	Drawing Number
Defended scenarios	
1 in 5	Hazard Map5yr_004.pdf
1 in 10	Hazard Map10yr_004.pdf
1 in 15	Hazard Map15yr_004.pdf
1 in 20	Hazard Map20yr_004.pdf
1 in 50	Hazard Map50yr_004.pdf
1 in 75	Hazard Map75yr_004.pdf
1 in 100	Hazard Map100yr_004.pdf
1 in 100 plus climate change	Hazard Map100yrCC_004.pdf
1 in 200	Hazard Map200yr_004.pdf
1 in 1000	Hazard Map1000yr_004.pdf
Undefended Scenarios	
1 in 100	Hazard Map100yr_undefended_004.pdf

Scenario	Drawing Number
1 in 1000	Hazard Map1000yr_undefended_004.pdf
Sensitivity scenarios	
1 and 2: 50% blockage at structures on the Ditch	Blockage279_Map_004.pdf Blockage358_Map_004.pdf
3 De-shoaled channel at Abefan	Deshoaling_Map_004.pdf
4 Additional shoaling at Abefan	Shoaling_Map_004.pdf
5 Floodplain roughness 0.1	Mannings_0.1_Map_004.pdf
6 Height of buildings in the 2D domain increased to 0.15 m	Building_015cm_Map_004.pdf
Event runs	
Event 5	Hazard Map Event5_004.pdf
Areas benefiting from defences	
100 year defended and undefended	ABD_004.pdf

Source: MM

6.2.2 Interpretation of Results

6.2.2.1 Defended Scenarios

The flood risk maps representing the current situation (defended scenario) show very little flooding up to a 1 in 20 year flood.

Flooding at lower return periods occurs close to the river course north of Aberfan where out-of-bank flow is likely to be related to water levels increased by a number of consecutive weirs. Flood water is confined between the river bank and the railway embankment and does not spread further.

Minor flooding at lower return periods is also shown at Merthyr Vale where Crescent Street is the location first affected by the flood waters.

The 1 in 15 year flood maps still show very little flooding in addition to the areas discussed above. Photographs taken in September 2008 during Event 5 which has an estimated return period of approximately 15 years generally confirm the model results for the lower return periods.

The photographs also show that in some areas, such as at the weir upstream of Nixonville Road Bridge, the water level came close to bankfull. The flood maps for the 1 in 20 year flood show the onset of flooding in this area. Water spills onto the floodplain in the area upstream of the weir and spreads further downstream.

The flood maps for the 1 in 50 year event show a significant increase in the extent of flooding and also an increase in the depth of flooding in areas where flooding starts at lower return periods.

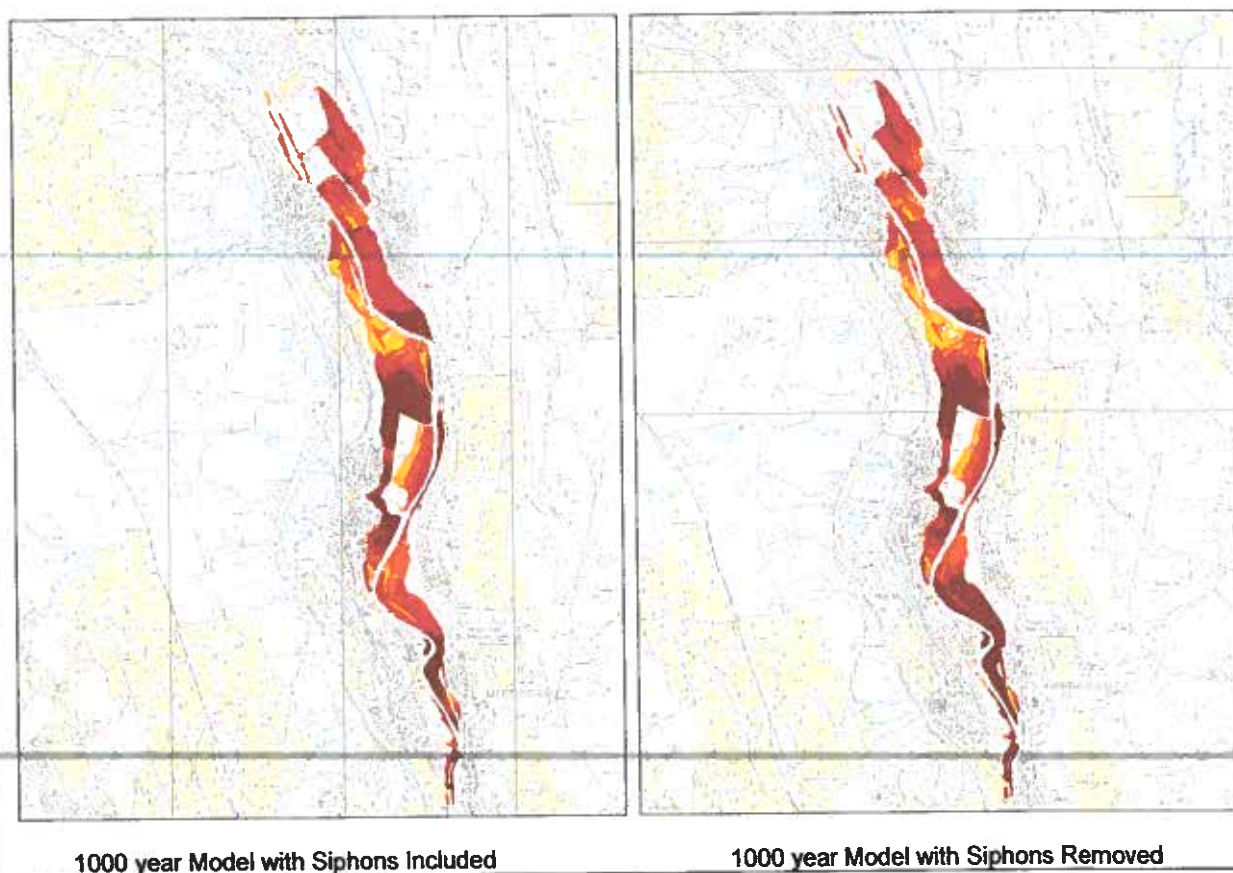
The area most affected by the spreading of floodwaters is Troedyrhiw, where the water overtops the embankment on both sides of the river. On the right bank water backs up behind raised ground at the footbridge leading to the school grounds, and appears to outflank the defence following an overland flow path causing flooding further downstream. The left bank side shows flooding of the school and playing fields.

An area further north in Troedyrhiw is affected by flooding from the Ditch. Water overtopping the embankment of the ditch upstream of the railway bridge flows parallel to the railway embankment.

The 1 in 75, 1 in 100 year and 1 in 200 year flood maps show an overall increase in extent and depth of flooding in all areas discussed above. Depth exceeds 2 m in some areas such as at Crescent Street and north of Aberfan on the right bank floodplain.

The 1 in 1000 year flood maps show severe flooding of most of the valley with a high flood hazard attached. Figure 6.3 shows the comparison of the 1000 year design event flood depths. The 1000 year model with the siphons included shows a significant increase in the depth of flooding in the upstream reach of the model, between south of Troedyrhiw and Aberfan on the right bank floodplain. Therefore, less water flows through the channel near the downstream end of the model. Table 6.5 shows the modelled peak flows at Fiddlers Elbow gauging station for both the 1000 year models with and without the siphons.

Figure 6.3: Flood Depth for the 1 in 1000 year Design Event With and Without Siphons



Source: MM

Table 6.5: Modelled Peak Flows at Fiddlers Elbow Gauging Station

	1000 year model with siphons included	1000 year model with siphons removed
Modelled Peak Flows at Fiddlers Elbow	558m ³ /s	651m ³ /s

Source: MM

6.2.2.2 Undefended Scenarios

The undefended 1 in 100 year flood map shows that without defences the school at Troedyrhiw is worse affected by flooding. The flood maps of the undefended 1 in 100 year event show a significant increase in the flood depth at Troedyrhiw on the left bank floodplain. Increased flooding in this area consequently leads to a significant reduction in flooding on the right bank side; hence it remains dry.

There is an increase in flooding north of Aberfan and in Aberfan in the area upstream of the Nixonville Road Bridge on the right bank side. The area at Crescent Street also shows a small reduction in flood depth.

The undefended 1 in 1000 year flood maps show severe flooding of most of the valley with a high flood hazard attached. Although the extent does not increase, the depth increases and consequently so does the flood hazard. The area most affected by the spreading of floodwaters is Aberfan near the area downstream of the Nixonville Road Bridge on the left bank side.

6.2.2.3 Areas benefiting from Defences

As discussed above, the areas benefiting from defences are not necessarily limited to areas behind defences as the flow dynamics are complex, and flooding in one particular area reduces the amount of flow which might lead to less flooding in other areas not spatially linked to the protected area.

Based on the 1 in 100 year defended and undefended flood maps, areas benefiting from defences have been assessed. These are shown on a map in Appendix E.

6.2.2.4 Sensitivity Scenarios

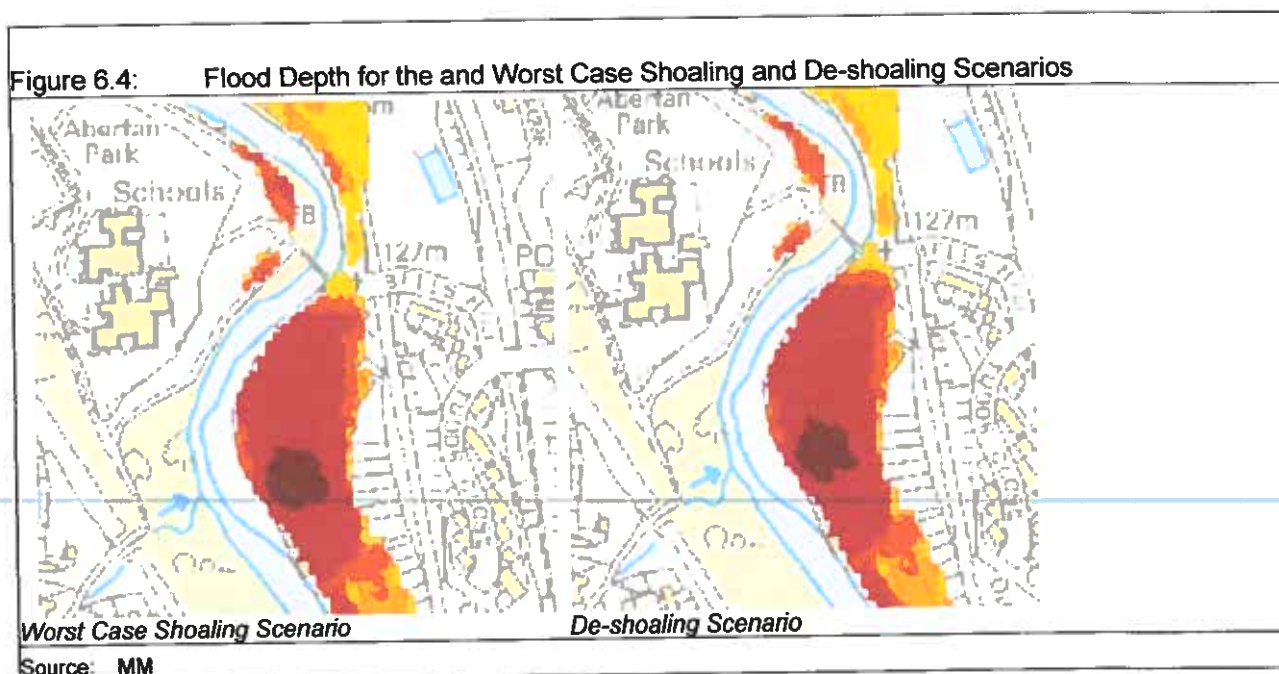
The blockage of the two bridges on the Ditch does not lead to a significant change in flood extent, flood depth or velocity.

The change of the floodplain roughness to 0.1 leads to some changes in flood depth in Troedyrhiw where the depth increases on the left bank side and is reduced on the right bank. At Crescent Street the depth of flooding is also reduced.

The increase in the height of the buildings represented in the 2D domain has a relatively small overall impact. Areas with large buildings such as the school at Troedyrhiw show some changes (increases and reductions) in the flood depth.

Two scenarios alternatively explore the impact of the accumulation of shoaling material and its removal in the Aberfan area upstream and downstream of the footbridge. The results of the worst case scenario assuming more shoaling, in particular in the area upstream of the river bend near the footbridge, and the de-shoaling scenario are shown in Figure 6.4. Figure 6.4 shows the flood depth for the two scenarios. The figure on the left shows the worst case scenario. Increased accumulation of material consequently leads to an increase in flood extent on the left bank side in the area upstream of the footbridge compared to a de-shoaled channel. Further downstream the flood depth increases in the Crescent Street area.

Although the scenarios are based on the 1 in 100 year flood, they demonstrate that regular de-shoaling can help avoid or reduce flooding in Crescent Street, an area where residents were affected by flooding in the past and which will remain a hot spot in the future.



7. Flood Threshold Levels

7.1 Introduction

The Brief envisaged the identification of “trigger levels at Merthyr Tydfil and Merthyr Vale level recorders which mark the onset of flooding within the town” of Aberfan and Merthyr Vale.

During the model development and model calibration a number of problems have been identified which consequently lead to a revision of the approach:

- Water levels at Merthyr Vale (Aberfan) level gauge are affected by the accumulation of shoaling material and are therefore unsuitable for trigger levels.
- The representation of the gauge at Merthyr Tydfil as Q-H boundary is likely to lead to errors in the modelled high flows.
- The calibration results at Merthyr Town Weir are unsatisfactorily. The source of the discrepancy between modelled and observed water levels has not been identified as there is little data available.

Water levels at Troedyrhiw gauge have been considered reliable and it was therefore decided to use the levels recorded at Troedyrhiw as trigger levels indicating the onset of flooding.

Until further work has been carried out to improve the representation of the gauge at Merthyr Tydfil, the level records are used to identify trigger levels indicating the onset of flooding.

7.2 Identification of Thresholds

Figure 7.2 shows the location of the gauge at Merthyr Tydfil and Troedyrhiw together with the hot spot areas at Aberfan and Merthyr Vale.

The animation and the results of the 100 year design model run have been used to identify the trigger levels which mark the onset of flooding in the areas identified by the EA Wales:

- School at Troedyrhiw
- Overtopping into plantation & onset of flooding of properties
- Nixonville
- Crescent Street

The results are summarised in Table 7.1 Figure 7.2 shows the 100 year design water level hydrograph together with the trigger levels at Merthyr Tydfil indicating the onset of flooding in a number of hot spots.

The travel time between Merthyr Tydfil and Troedyrhiw is short. During the historic events the travel time was around 15 to 30 minutes. Therefore the additional lead time gained from the use of the gauge at Merthyr Tydfil is small. However, during critical situations this might allow some extra time for emergency response.

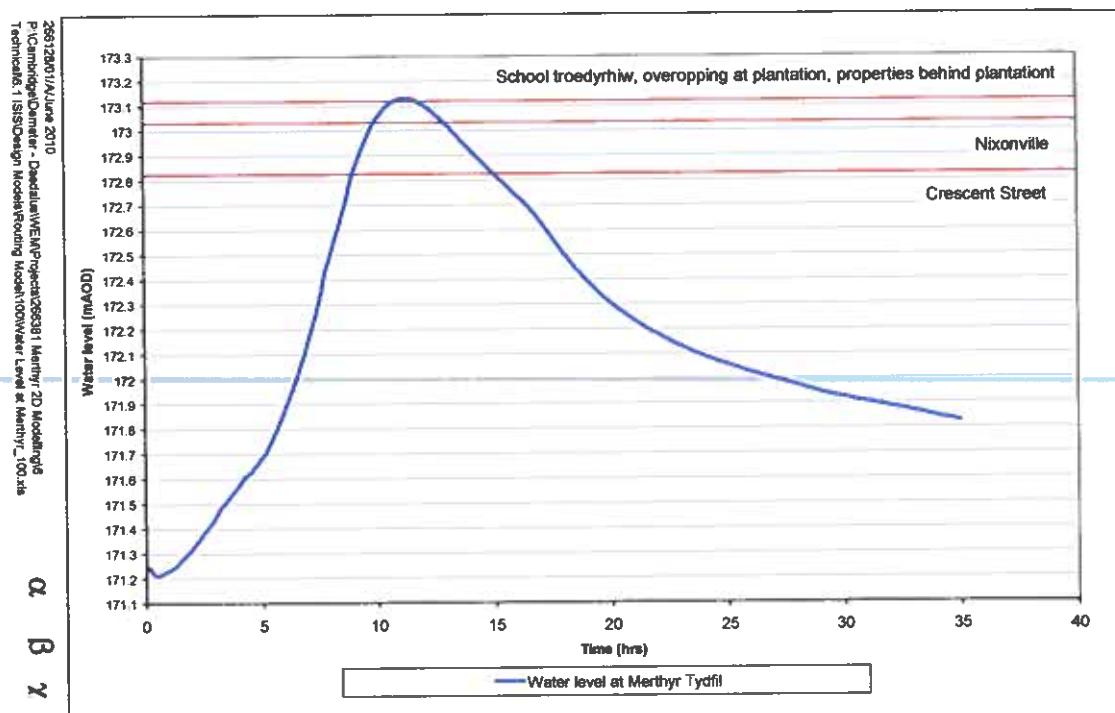
Table 7.1: Trigger Levels at Troedyrhiw and Merthyr Tydfil marking the Onset of Flooding in Hot Spot Areas

Area	Time of onset of flooding (hrs)	Trigger Level (mAOD)	
		Merthyr Tydfil	Troedyrhiw
School at Troedyrhiw	10:30	173.10	136.45

Area	Time of onset of flooding (hrs)	Trigger Level (mAOD)	
		Merthyr Tydfil	Troedyrhiw
Overtopping into plantation	11:45	173.11	136.53
onset of flooding of properties behind plantation	12:15	173.09	136.48
Nixonville	10:00	173.04	136.29
Crescent Street	9:00	172.83	135.71

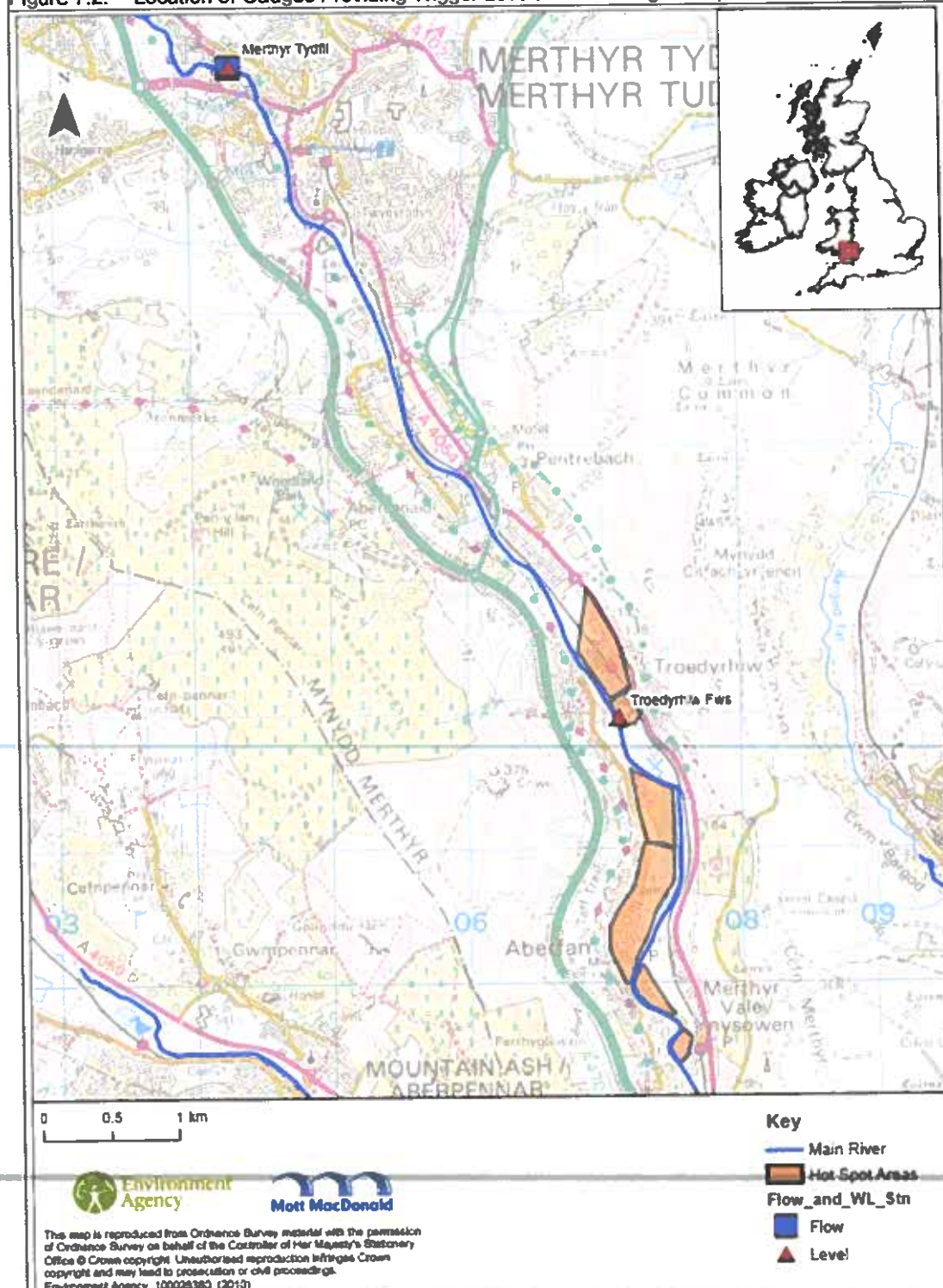
Source: MM

Figure 7.1: 100 Year Design Hydrograph and Trigger Levels Merthyr Tydfil indication the Onset of Flooding in Flooding Hot Spots



Source: MM

Figure 7.2: Location of Gauges Providing Trigger Levels for Flooding Hot Spots



Source: MDT

8. Recommendations

8.1 Replacement of Q-H Controls

The gauges at Merthyr Tydfil and Fiddler's Elbow are currently represented as Q-H boundaries, the set-up used in the FF model. Although this ensures consistency between the conversion used to calculate flows from the observed water levels and the modelled flow, this approach is unsuitable as high flows are likely to be poorly represented.

- The Q-H control does not accommodate for the impact of backwater from rising water levels downstream of the weir.
- The existing rating of which the Q-H control is replication is based on extrapolation of flow over the gauging structure and spot gaugings which might not correctly represent the level – flow relationship once the flood waters have been spilled onto the floodplain. Therefore modelling the gauging structure and the floodplain at the gauge is likely to result in a different rating and consequently a discrepancy between observed and modelled water levels.

Work is currently being undertaken by Edenvale Young Associates and it is recommended to update the model according to the findings of the undertaken study.

The EA Wales also commissioned the survey of the two gauges to ensure accurate information of the gauging structures is available for any further work.

It is important that the revised rating curve and the model are kept consistent. However, there are limitations, in particular when backwater affects the rating curve which consequently shows a hysteresis.

At the time of the final submission of this document a rating review has been carried out based on the latest survey and spot gaugings. The intention is to replace the Q-H control with the partial model developed for the revision of the rating curve. Although this work has been finished in September 2010 updates to the model could not be made at this stage.

8.2 Closing of Gaps in the Survey

The 1D model between Merthyr Tydfil and Troedyrhiw is characterised by river reaches which are represented by routing cross sections, a method suitable for flood forecasting purposes but unsuitable for flood risk mapping as water levels are not calculated and the accuracy of the model is insufficient for flood risk mapping.

Survey work is currently being undertaken to close the gaps identified in this study. The survey also covers the bank levels to support future 1D-2D modelling work in this area.

It is recommended to incorporate the survey into future modelling work.

8.3 Updating FF Model

We recommend considering the following changes be made to the FF model based on improvements which have been made for the purpose of flood risk mapping:

1. Remove in-built glass walling and extend cross sections onto the floodplain. Note that for this study this has only been carried out in the reach between Merthyr Tydfil and the start of the 1D-2D linked model (TAFF09400). Where the 2D model represents the floodplain the cross sections have not been extended.
2. Where existing cross sections have been updated using more recent survey, the FF model; should also be updated.
3. Where cross sections based on more recent survey have been added (e.g. in the Aberfan area) it should be checked whether the accuracy of flood forecasts would significantly gain from more densely spaced cross sections.
4. The spill at Aberfan which has been removed from the model as the weir structure had been modelled twice should be removed from the FF model.
5. The changes to the model at Merthyr Town Weir taking the recently constructed fishpass and the subsequent modifications to the weir crest into account should also be considered in the FF model.

Any further development of the modelling of the gauging structures at Merthyr Tydfil and Fiddler's Elbow as recommended in Section 8.1 above should be considered consistently in all models independent from their respective purposes.

Details of the changes made to the FF model have been recorded in a log book which has been provided together with the other deliverables.

9. References

- [1] WS ATKINS (2007): River Taff Flood Risk Mapping Update - Addendum. November 2007
- [2] WS ATKINS (2007): River Taff Flood Forecasting Modelling Project – Final report. April 2006
- [3] WELSH WATER AUTHORITY (1979): Cardiff Flood Protection Scheme.
- [4] WS ATKINS (200): Aberfan and Merthyr Vale Flood Alleviation Scheme Engineering Feasibility Report – Final report. October 2000.

Note: a number of references where reports were not available or specifically used but information has been used are listed in Chapter 3.

Appendices

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Appendix A. Hydrometric Data

Table A.1: Summary of Water Level and Flow Records Provided for this Study

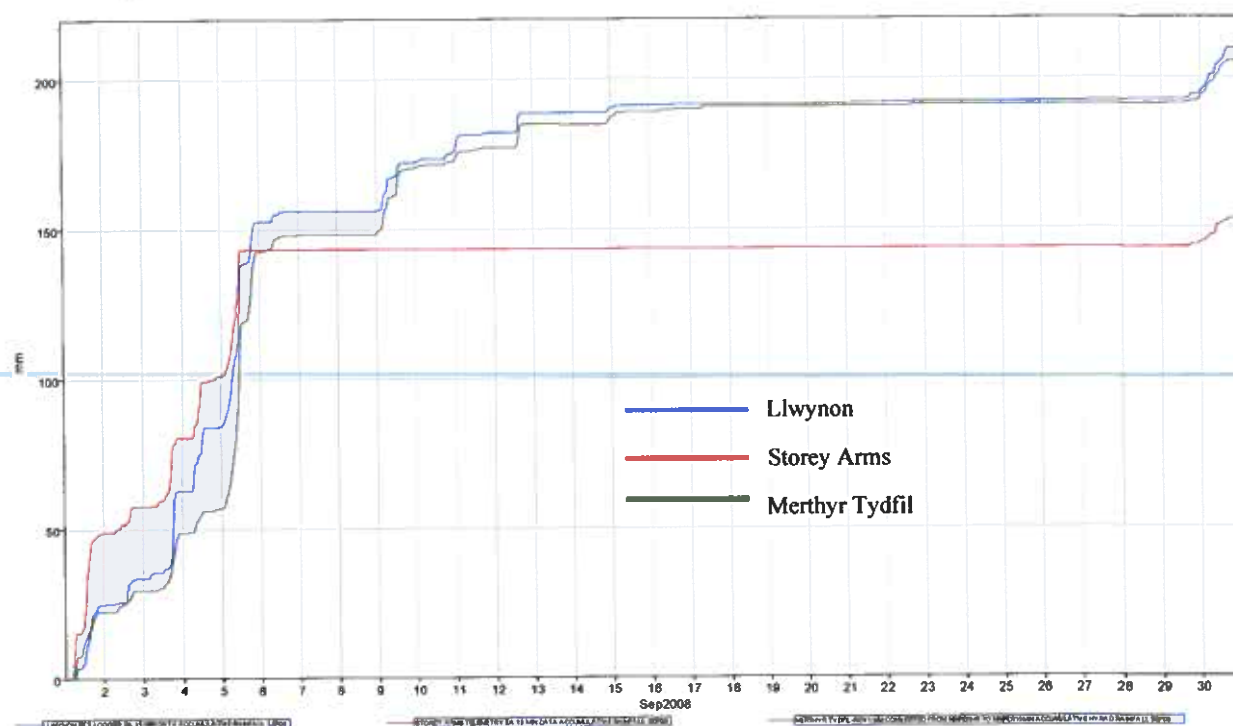
Name of record	Measurement	Period provided	Period processed	Interval	Comment
ABERFAN DS	STAGE	01 Jun 2005 – 01 Nov 2005	01 Jun 2005 – 01 Nov 2005	15MIN	
ABERFAN US	STAGE	01 Jun 2005 – 01 Nov 2005	01 Jun 2005 – 01 Nov 2005	15MIN	
ABERFAN FWS	STAGE	01 Aug 1999 – 01 Jul 2009	01 Aug 1999 – 01 Jul 2009	15MIN	
FIDDLERS ELBOW	FLOW	01 Jan 1982-01 Oct 2009	01 Jun 1999 – 01 Oct 2009	15MIN	
FIDDLERS ELBOW	STAGE	01 Jan 1982- May 2003	01 Dec 1995 – 01 May 2003	15MIN	Processed by MM1)
LLWYNON FLOOD CHANNEL	FLOW	16 Oct 1987 – 01 Oct 2009	01 Apr 1999 – 01 Oct 2009	15MIN	Processed by MM1)
LLWYNON FLOOD CHANNEL	STAGE	16 Oct 1987 – 06 Oct 2009	01 Apr 1999 – 01 Nov 2008	15MIN	Processed by MM1)
LLWYNON FLUME	FLOW	16 Oct 1987 – 01 Oct 2009	01 Mar 2005 – 01 Oct 2009	15MIN	Processed by MM1)
LLWYNON SPILLWAY	STAGE	01 Jun 2000 – 01 Mar 2009	01 Jun 2000 – 01 Mar 2009	15MIN	Processed by MM1)
MERTHYR TOWN WEIR	STAGE	01 Jun 2000 – 01 Mar 2009	01 Jun 2000 – 01 Mar 2009	15MIN	
MERTHYR TYDFIL -	STAGE	01 Jan 1982- 01 Oct 2009	01 Dec 1996 – 01 Oct 2009	15MIN	Processed by MM1)
MERTHYR TYDFIL	FLWS	01 Jan 1982 01 Oct 2009	01 Jan 1982 – 01 Oct 2009	15MIN	Processed by MM1)
PONTSTICILL RES	STAGE	01 Jul 2000 – 01 Oct 2009	01 Jul 2000 – 01 Oct 2009	15MIN	Mostly zeros or missing data
TROEDYRHIW	STAGE	01 Aug 1999 – 01 Oct 2009	01 Aug 1999 – 01 Oct 2009	15MIN	
QUAKERS YARD	STAGE	01 May 1996 – 01 Jun 2002	01 May 1996 – 01 Jun 2002	15MIN	

Table A.2: Summary of Rainfall Data Provided for this Study

Name of record	Period provided	Period processed	Interval	Comment
Storey Arms, Brecon Beacons	01 OCT 2003 -01 OCT 2009	01 OCT 2003 -01 OCT 2009	15 MIN	
Llwynon Reservoir	01 JAN 1990 -01 Jan 2009	01 JAN 1990 -01 Jan 2009	15 MIN	
Nant Wen Farm, Bedlinog	01 NOV 2000 -01 Oct 2009	01 NOV 2000 -01 Oct 2009	15 MIN	
Catchment to Merthyr Tydfil	01 AUG 2008 – 01 JUN 2009	01 AUG 2008 – 01 JUN 2009	15 MIN	
Rhymney Reservoir		01 JAN 2001 – 01 OCT 2008	15MIN	
Aberdare Filters Logger	16 Nov 1990 – 01 Mar 2005	01NOV2003 - 01MAR2005	15MIN	

Name of record	Period provided	Period processed	Interval	Comment
Pontsticil Telemetry	26 Jun 2006 - 19 Jan 2004	-	15MIN	
Pontsticil Reservoir	01 Jan 2001 - 07 Oct 2009	-	15MIN	
Old Pontsticil	01 Oct 1989 - 01 Oct 2009	01NOV2000 - 01OCT2009	15MIN	
Nueadd Res	02 Feb 1961 - 01 Oct 2009	01OCT2003 - 01OCT2009	15MIN	
Storage Gauges	Varies	Varies	Varies	

Figure A.1: Comparison of TBR and HYRAD rainfall in the Upper Catchment



Source: Environment Agency Wales, Processed by MM

Appendix B. FEH Catchment Descriptors

B.1. FEH Catchment Descriptors Upstream of Merthyr Tydfil Gauge

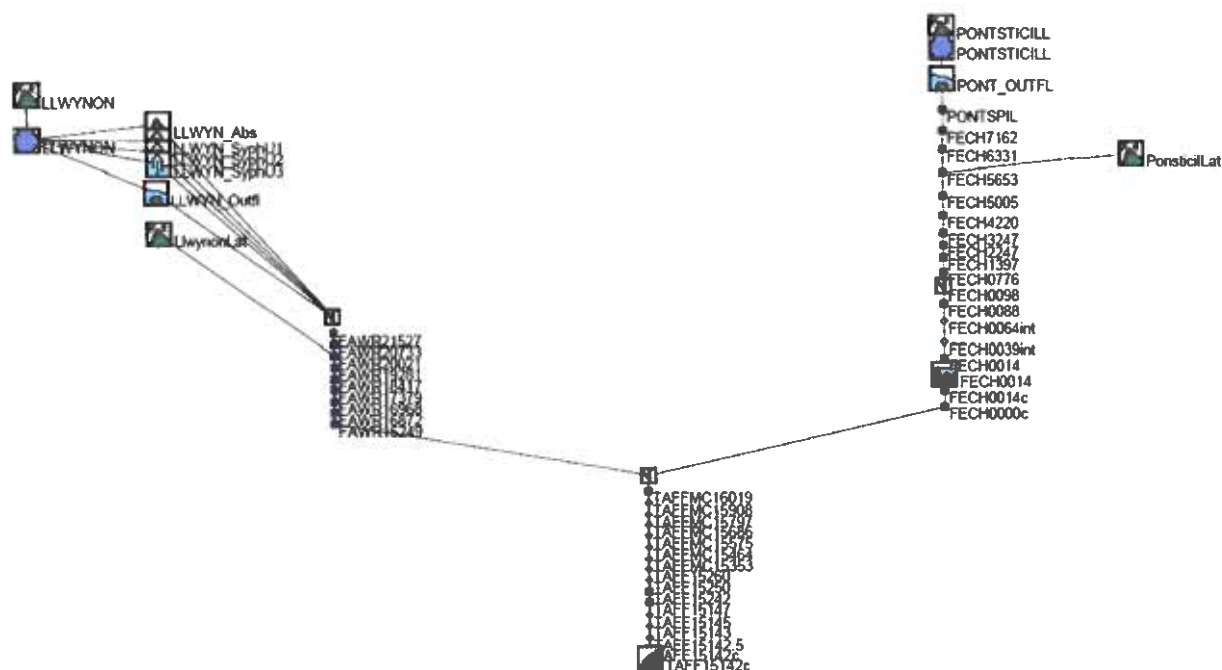
Descriptor	Llwynon	LlwynonLat	Pontsticill	PontsticillLat
AREA	44.01	13.30	33.49	15.22
ALTBAR	461.00	420.44	484.00	349.58
ASPBAR	187.00	160.48	188.00	178.40
ASPVAR	0.25	0.08	0.25	0.28
BFIHOST	0.31	0.50	0.31	0.52
DPLBAR	6.42	4.13	5.75	4.45
DPSBAR	166.40	163.94	169.00	135.72
FARL	0.83	0.85	0.94	0.90
LDP	13.50	5.64	12.77	7.56
PROPWET	0.56	0.70	0.54	0.54
RMED-1H	12.30	14.90	12.10	11.78
RMED-1D	63.00	69.52	61.70	56.26
RMED-2D	85.30	94.16	84.60	77.24
SAAR	1981.00	2057.83	1938.00	1589.16
SAAR4170	2057.00	2043.31	1911.00	1638.97
SPRHOST	54.28	56.63	52.68	35.88
URBCONC1990	-999999.00	-999999.00	-999999.00	-999999.00
URBEXT1990	0.00	0.02	0.00	0.03
URBLOC1990	-999999.00	-999999.00	-999999.00	-999999.00
C	-0.03	-0.03	-0.03	-0.03
D1	0.51	0.60	0.51	0.49
D2	0.47	0.58	0.47	0.47
D3	0.36	0.41	0.35	0.33
E	0.29	0.37	0.29	0.29
F	2.58	3.16	2.57	2.53
C(1 km)	-0.03	-0.03	-0.03	-0.03
D1(1 km)	0.50	0.60	0.48	0.52
D2(1 km)	0.48	0.61	0.46	0.52
D3(1 km)	0.36	0.44	0.35	0.37
E(1 km)	0.29	0.37	0.29	0.31
F(1 km)	2.54	3.01	2.53	2.45

B.2. FEH Catchment Descriptors Downstream of Merthyr Tydfil Gauge

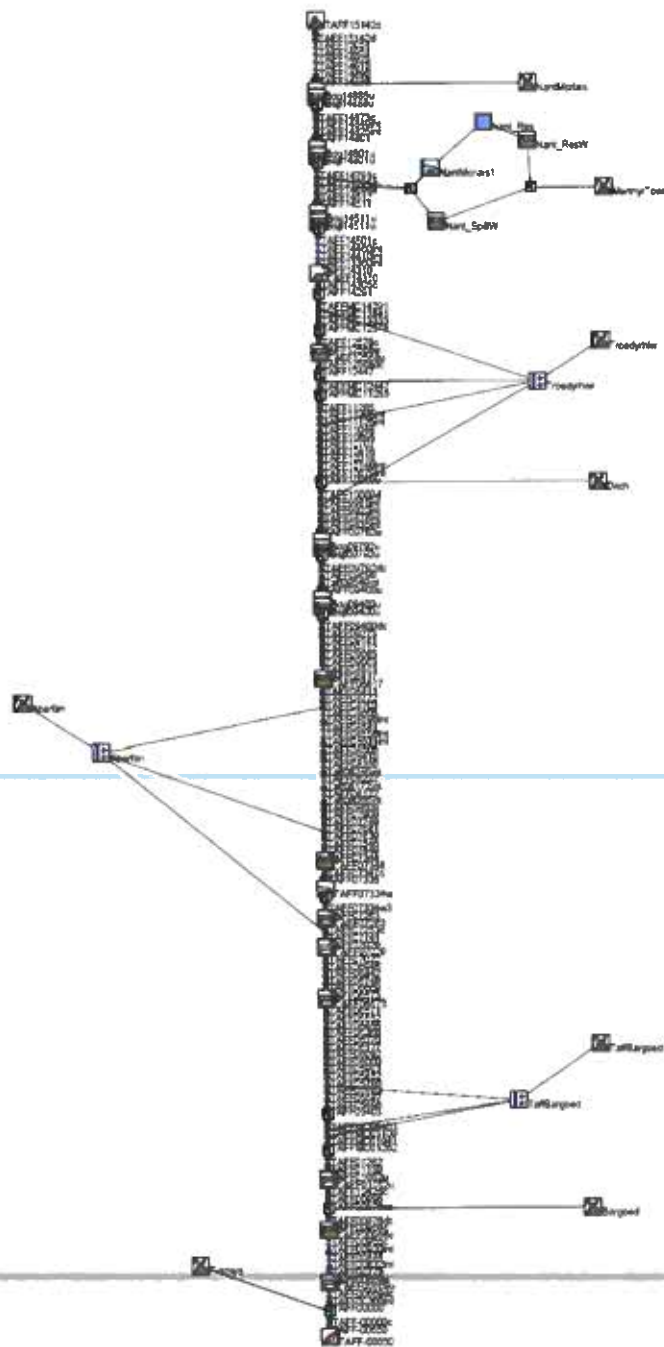
Descriptor	Aberfan	Bargoed	Ditch	Fiddlers	Merthyr Town	Nant Morlais	TaffBargoed	Troedyrhiw
AREA	5.83	28.75	1.49	5.97	11.37	1.52	6.97	22.68
ALTBAR	269.46	304.00	263.00	163.79	370.00	290.00	189.46	269.05
ASPBAR	127.78	186.00	250.00	257.76	220.00	202.00	132.32	167.19
ASPVAR	0.20	0.33	0.78	0.20	0.49	0.70	0.19	0.20
BFIHOST	0.57	0.48	0.59	0.52	0.36	0.53	0.69	0.38
DPLBAR	2.63	6.65	1.15	2.66	4.09	2.01	2.90	5.53
DPSBAR	227.82	148.30	240.00	98.89	94.30	68.60	232.43	137.60
FARL	0.89	0.98	1.00	0.91	0.95	1.00	0.89	0.98
LDP	2.84	13.97	2.31	2.15	8.60	3.87	5.51	8.35
PROPWET	0.55	0.49	0.54	0.13	0.54	0.54	0.32	0.55
RMED-1H	12.00	11.90	11.80	12.48	11.60	11.60	12.00	11.71
RMED-1D	59.10	54.20	54.30	57.30	53.60	54.70	61.38	55.46
RMED-2D	77.69	71.50	72.60	73.38	72.10	74.50	80.20	74.86
SAAR	1549.03	1505.00	1501.00	1496.39	1512.00	1466.00	1569.46	1580.34
SAAR4170	1514.81	1526.00	1543.00	1491.72	1549.00	1475.00	1471.27	1542.08
SPRHOST	27.33	36.05	24.87	32.06	45.62	38.71	20.06	47.07
URBCONC1990	0.53	0.47	0.49	1.11	0.73	0.80	0.47	-999999.00
URBEXT1990	0.04	0.01	0.05	0.05	0.15	0.23	0.03	0.08
URBLOC1990	2.56	0.34	0.58	3.85	0.65	0.75	3.18	-999999.00
C	-0.03	-0.03	-0.03	-0.03	-0.03	-0.03	-0.03	-0.03
D1	0.49	0.47	0.48	0.48	0.47	0.48	0.51	0.48
D2	0.44	0.43	0.44	0.44	0.45	0.47	0.44	0.45
D3	0.38	0.37	0.37	0.38	0.36	0.35	0.40	0.35
E	0.29	0.29	0.29	0.29	0.29	0.29	0.29	0.29
F	2.55	2.54	2.52	2.57	2.52	2.51	2.59	2.53
C(1 km)	0.00	-0.03	-0.03	-0.03	-0.03	-0.03	-0.05	-0.03
D1(1 km)	-0.32	0.51	0.49	0.34	0.49	0.48	0.97	0.65
D2(1 km)	0.21	0.44	0.45	0.38	0.48	0.47	0.55	0.27
D3(1 km)	0.52	0.35	0.36	2.36	0.37	0.36	-0.54	0.55
E(1 km)	0.19	0.29	0.29	0.26	0.29	0.29	0.34	0.29
F(1 km)	2.36	2.58	2.53	2.52	2.51	2.51	3.15	2.90

Appendix C. ISIS Model Routing Model Schematics

Figure C.1: ISIS Model Schematic – Routing Model Upper Catchments to Merthyr Tydfil



Source: MM



Source: MM

Appendix D. Flood Maps

Appendix E. Areas Benefiting from Defences