International Conference "Water, Environment, Energy and Society" (WEES–2009)
New Delhi, 12–16 January 2009

Assessing Spillway Design Flood and the Collapse of Gasper Dam, June 29, 1917

Colin Clark

Charldon Hill Research Station,
Shute Lane, Bruton, Somerset, BA10 0BJ, UK
E-mail: Colin4chrs@hotmail.com

ABSTRACT: The spillway design flood of some UK dams has recently been under review. Although there have been no fatalities caused by dam failure in recent years, it is now realised that some dams will have to be strengthened in order to avoid a future disaster. Gasper dam in Wiltshire, UK was built in the late Victorian era and was destroyed during the great storm of 28–29 June 1917. It was rebuilt in 1920. The dambreak flood discharge has been estimated at three locations downstream of the dam. They are much higher than estimates using current methods such as Hughes *et al.,* (2000) showing that the hazard posed by dam-break floods is greater than previously supposed.

Since 1978 the dam has been inspected five times. The most recent inspection (2004) noted that it is presently classified as a category B dam in the UK dams are graded A-D – which means that some overtopping during the maximum flood is permissible. However, since 1920 the total number of houses at risk from a dam breach exceeds 10. This means that the dam should be upgraded to category A, such that the dam should safely pass the probable maximum flood (PMF). The current inspector's report estimated the PMF as 59 $m^3 s^{-1}$. However, five new estimates of the PMF have been produced. These range from 71–150 $m^3 s^{-1}$. These new results were obtained by using: 1. The ICE (1996) rapid method. 2. The FEH UH (1999) approach. 3. Transposition of the Martinstown storm of 1955 which took place 45 km to the south. 4. Transposition and maximisation of the same storm. 5. Analysis of two historic floods and an estimate of bankfull discharge.

When runoff rates from other floods in the UK are applied to the Gasper dam catchment a PMF of at least 100 $m^3 s^{-1}$ is suggested. Therefore unless Gasper dam is modified to safely pass the higher estimate of the PMF then it remains a serious threat to both people and property downstream. These results have serious implications for the safety of other dams in the UK and perhaps elsewhere.

INTRODUCTION

Following the discovery (MacDonald and Scott 2000) that the UK Flood Estimation Handbook (IOH, 1999) extreme rainfall estimates exceeded the estimate of probable maximum precipitation (PMP) the UK government asked for the problem to be investigated further, with the reports of Babtie (2000), HBR (2002) and Cox, (2002) coinciding with the publication of new estimates of PMP which were homogenized with estimates of less extreme rainfall (Clark, 2002a). Since then Defra (2005) have published new guidelines for engineers to design and reassess dams, but the question of using higher estimates of PMP has yet to be resolved. In the UK there are several thousand dams which are located upstream of communities: these are designated category A which have to be able to safely pass the probable maximum flood without being destroyed. The less critical dams are styled B, C and D. There are standard methods which have by law to be used to assess the spillway design flood of a category A dam (ICE, 1996). More recently quantitative risk analysis (Brown and Gosden, 2004) has been introduced as well as guidance for the production of emergency plans in the event of a major dam incident (Gosden *et al.,* 2006). The matter is not without precedent because on June 19, 2005 there was serious damage to a dam at Boltby in the North York Moors when a flash flood with an estimated return period in excess of 10,000 years took place (Clayden *et al.,* 2007). If the rarity assessment is correct then even more severe floods will take place since according to the Reservoir Safety Guide (ICE, 1996) the PMF is about double the magnitude of the 10,000 year flood. It is important to learn from previous dam break incidents (Charles, 2005), even when they have taken place a long time ago. The flood of 1917 that resulted in the collapse of Gasper dam has recently been reviewed (Clark and Pike, in press) and this paper describes methods of assessing the spillway design flood of Gasper dam and its relevance to dams in general.

The structure of this report is as follows: first, the catchment area is described second a brief account of the storm, flood, and breaching of Gasper dam in 1917 will be given. Third, the methods of assessment of the spillway design flood and reservoir routing are described. Fourth, the results of these methods are presented. Fifth, an analysis of a more recent overtopping incident at Gasper will be described and put into the context of dam safety. Sixth, a flood frequency curve for the Gasper Dam site has been prepared and its significance discussed in relation to dam safety.

THE GASPER CATCHMENT AREA

Gasper dam is situated on the upper Stour in SW Wiltshire (Figure 1). It receives runoff from a catchment area of 6.75 km^2. The local geology consists of Gault Clay overlain by Upper Greensand, which is mapped as three separate formations: the Cann Sand, the Shaftesbury Sandstone, and the Boyne Hollow Chert. There are 11 separate landslips which the Geological Survey have identified on the map (NERC, 1996). The dam impoundment level is about 129 m OD. The dam is 5.95 m in height as measured from the toe to the dam crest. Originally built in 1875 Gasper dam was destroyed in 1917 by a flood and then rebuilt in 1920. The original spillway still exists and is located at the western end of the dam. A stone plaque on the road bridge at the site commemorates this event. The dam impounds a lake with a surface area of 4.6 Ha., but in 1917 the surface area was 5.98 Ha. At the outlet a circular concrete shaft 4.8m diameter, 2.8 m deep leads to a culvert with an effective area of 3.246 m^2. About 10 m from the distal end of the culvert is a fish trap with a compound rectangular weir constructed of concrete blocks. This trap presents a 0.4 m backwater to the culvert that reduces the capacity of the latter. The freeboard between the spillway and the dam crest is 1.388 m. This was resolved by the apportionment of the freeboard at the lower level, comprising 0.3 of the circumference, and the higher level that makes up the remainder.

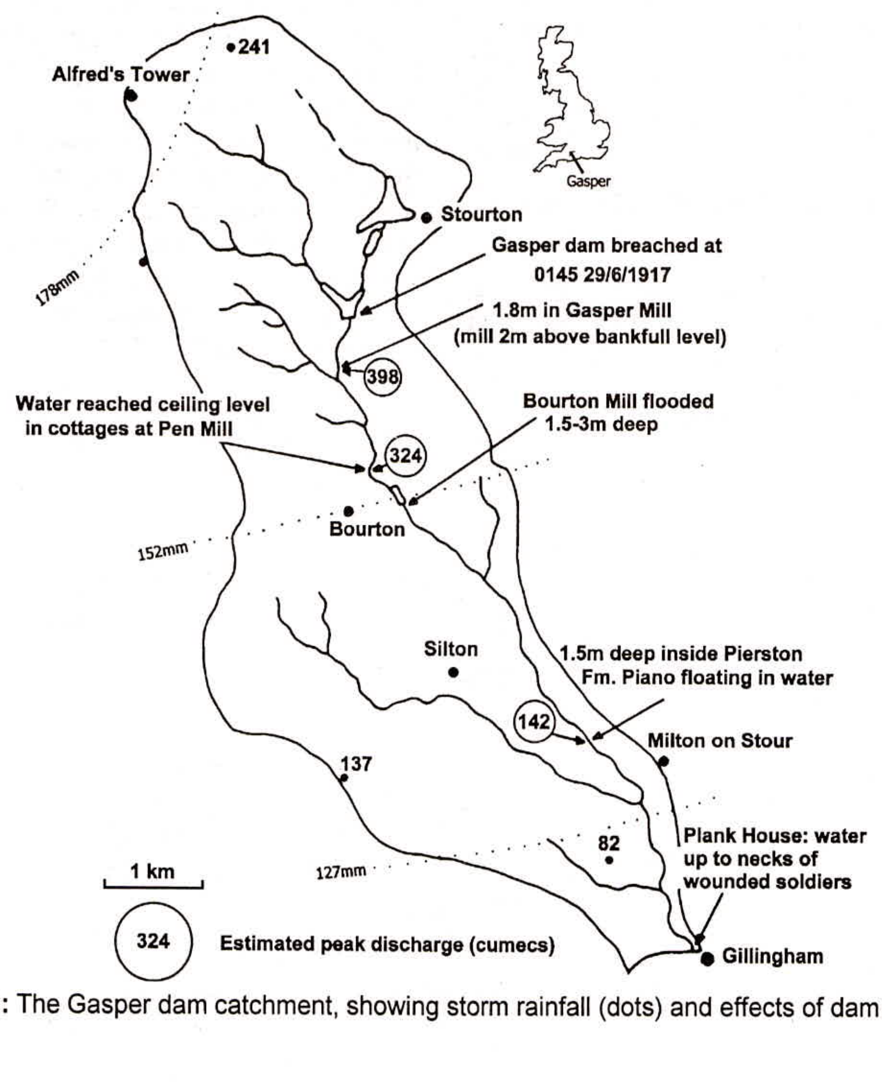

Fig. 1: The Gasper dam catchment, showing storm rainfall (dots) and effects of dam collapse

About 2 km downstream of Gasper Lake are the villages of Penselwood and Bourton. In recent years the number of houses in these two places has increased. This means that more than 10 persons could be affected by a major breach of Gasper dam, thereby making it a Category A dam (ICE, 1996). Dams that belong to this category of dams have to be designed to safely pass the Probable Maximum Flood (PMF), which is widely considered to result from the PMP. In recent years doubts have been expressed about the magnitude of extreme rainfall and floods, especially in SW England (Bootman & Willis, 1981; Clark, 1991). These and other studies have led to new estimates of Probable Maximum Precipitation (PMP) for the whole of Britain being produced (Clark, 2002a). At the same time evidence from historic floods has shown that current methods of flood assessment such as the Flood Estimation Handbook (IOH, 1999) cannot simulate very rare events since the PMF has been exceeded in the past. This situation has led to considerable concern amongst hydrologists and can be addressed by making several estimates of the PMF and then making an informed judgement using as much local knowledge as possible. Whilst there is always some uncertainty in the assessment of the maximum flood, when suitable empirical, local, and historic information is presented, this uncertainty is considerably reduced.

THE BREACHING OF GASPER DAM, 29 JUNE 1917

On 28 June 1917, a thundery low was moving ENE towards the Kent coast with a secondary triple point depression in the English channel which had moved over Belgium by 0100 GMT on 29 June. An area of southern England became trapped between the cold front of the first depression and the occlusion of the second. A wide band of rainfall was thus below the pincer like motion of the two fronts. Similar cases have been reported in May 1993 by Pike (1994), and the Easter storm of 1998 (Galvin and Pike, 2001). The heaviest rainfall took place over West Wiltshire and East Somerset with Bruton receiving 243 mm and the Gasper dam catchment, about 12 km to the east catching 165 mm. There are no gauging stations on the Upper Stour which rises in the Gasper catchment, but the contemporary newspaper report tells us:

"Shortly before two o'clock on Friday morning, by which time the lake had received an addition of many thousand tons of water by direct rainfall, and also as a result of the drainage from surrounding hills, a thunderous roar awakened inhabitants in the vicinity. Subsequent investigation revealed the entire disappearance of the lake through a channel created by the destruction of the road of about 20 yards and to a depth of about 20 feet…The raging torrent leapt forward at great speed and poured towards Gasper Mill which was about 500 yards distant…and the mill house was flooded to a depth of from six to seven feet…Pen Mill was the next important object of destruction…Cottages too were flooded in some cases the water rising to the ceilings of the lower rooms…Pierston Farm..was flooded to a depth of about five feet…smaller articles of furniture were swept out of the rooms by the force of the water, in which a piano was subsequently discovered floating." Salisbury Journal 7/7/1917.

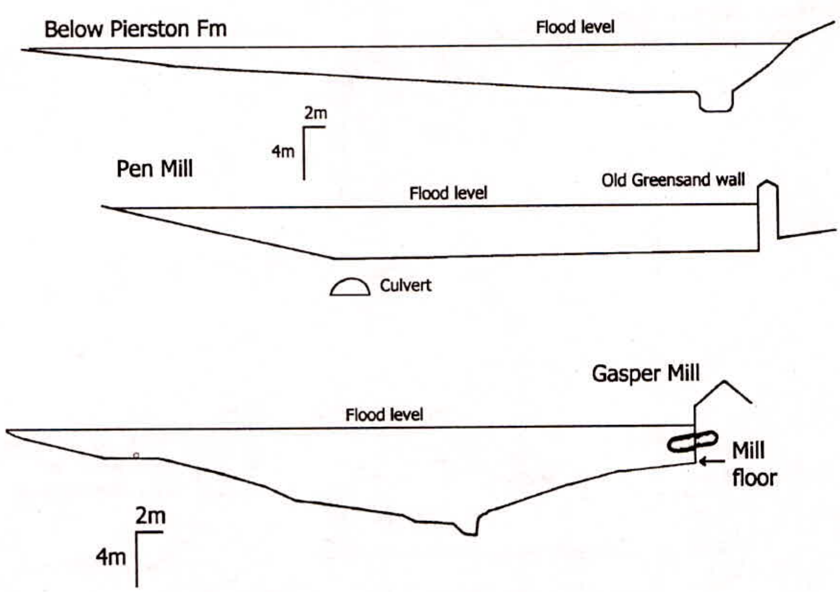

Fig. 2: Valley cross sections below Gasper dam

It is important to assess the magnitude of the dam-break flood. In addition to the newspaper description of the incident several photographs were taken and these confirm the likely level of the flood at Gasper Mill which is about 0.5 km downstream of Gasper dam. Figure 2 shows the three surveyed cross sections at Gasper Mill, Pen Mill and Pierston Farm. The estimated peak discharges are shown in Figure 1. The estimates at Pen Mill and Pierston Farmhouse are a minimum since they are based on reported levels inside dwellings when the level outside can be higher. The likely peak outflow of about 400 cumecs can be compared with two empirical formulae which have been used to estimate dam-break floods. The first is that of Hughes *et al.*, (2000),

$$Q_p = 330(BFF)^{0.42} \qquad \dots (1)$$

Where Q_p = peak outflow (cumecs); BFF = breach formation factor ($m^4\ 10^6$) = VH, where V = volume of water in the reservoir at the time of failure ($m^3 \times 10^6$); H = peak water level (m) above base of the dam. Application of this method with H = 5.95 and V = 0.170 Mm^3 gave a peak outflow of 339 cumecs.

An alternative method was proposed by Froehlich (1995),

$$Q_p = 0.607 V^{0.295} H^{1.24} \qquad \dots (2)$$

Where V = volume (m^3), H = height of peak water level (m) above the base of the dam. Clearly, the breach of Gasper dam exceeded these two estimates although was closer to that of Hughes *et al.*, (2000). This may have been because the dam failed very rapidly. Reports mentioned a continuous roar of water but further details have not been found. However, a water balance check of the likely volume of water which would have left the dam show that a peak outflow of about 380 cumecs could have flowed from Gasper Lake if it had failed in about 0.25 hour.

METHODS OF SPILLWAY DESIGN FLOOD ESTIMATION

1. The ICE (1996) rapid method.
2. The FEH rainfall runoff method.
3. Transposition of a historic storm and method 2.
4. Transposition and maximisation of a historic storm and method 2.
5. Non-linear flow model applied to the data in methods 3 and 4.
6. New estimates of PMP and the non-linear flow model.
7. Flood frequency analysis.
8. Reservoir routing of the inflow hydrograph.

The ICE (1996) Rapid Method

This method only gives a rough estimate of the PMF. Normally an estimate of the flood surcharge is given which also includes an estimate of wave surcharge. Since the spillway at Gasper is complicated by the culvert beneath the highway, no assessment of the flood surcharge is given at this stage,

$$\text{Peak inflow (cumecs)} = \frac{0.454\ A^{0.937}\ S1085^{0.328}}{SAAR^{0.319}} \qquad \dots (3)$$

Where: A = catchment area (km^2)
$S1085$ = mainstream slope in m/km
$SAAR$ = standard average annual rainfall

Values of the first two variables were obtained from the 1:25000 OS map and the value of SAAR obtained from the FEH CDROM (IOH, 1999). The catchment area is also given on the CDROM as 6.71 km^2 although a value of 6.75 km^2 was obtained from the OS map. The higher value was adopted in this report.

An allowance for wave surcharge should be made in the assessment of dam safety. The standard method as described in ICE (1996) was applied to Gasper dam. The method involves the calculation of the mean annual maximum wind speed from published data for the 50 year maximum speed, an adjustment for height of the impounded water, duration, and direction,

$$U = ft \,.\, fa \,.\, fw \,.\, fd \,.\, fn \,.\, U50 \qquad \dots (4)$$

Where U = mean annual maximum wind speed m/s; ft = ratio of 50 year: mean annual speed; fa = 1.0 + (0.001 × altitude, m); fw = adjustment for fetch; fd = 1.05; fn = adjustment for wind direction; $U50$ = wind speed with a return period of 50 years.

The recommended wave height is the significant height, which is the height of the highest third of all waves. About 14% of all waves will be higher than this height. The significant wave height,

$$H_s = UF^{0.5}/1760 \qquad \dots (5)$$

Where: H_s = significant wave height (m); U = mean annual maximum wind speed m/s; F = fetch (m). The design wave height has to be increased according to the dam construction and where no overtopping of the dam crest can be tolerated the design wave height $H_D = 1.67H_s$.

The FEH Rainfall-runoff Method

This method is described in the FEH (Houghton-Carr, 1999). It uses design rainfall depths, adjusted for the areal reduction factor, and percentage runoff in a unit hydrograph convolution process. A key parameter in the calculation is the time to peak for the unit hydrograph. In the estimation of the PMF the time to peak is reduced,

$$\text{PMF}_{\text{time to peak}} = \text{Time to peak } 0.67 \quad \ldots (6)$$

The percentage runoff consists of three components: 1. The standard % runoff, obtained from the CDROM. 2. A dynamic component based on rainfall depth. 3. A catchment wetness component.

The rate of runoff per mm of net rainfall is calculated according to the peak runoff rate multiplied by a set of values related to the time to peak and the time base (2.52 × time to peak). These values change in a linear manner during the duration of the storm.

In the description of this method (Houghton-Carr, 1999), there is no level of uncertainty given to the estimate of PMF. However, in the FEH the example of PMF estimation which is given, namely the West Lyn in North Devon, was equalled during the 1952 flood at the same site and exceeded during the flood of 1770 (Clark, 2001).

Transposition of a Historic Storm and the FEH Rainfall Run-off Method

In the estimation of PMP, storms can be physically moved from their actual location to other places providing certain conditions are met (WMO, 1986). These include moving the storm only to a meteorologically homogeneous area, keeping the storm to within specified limits of elevation, and within areas with similar levels of precipitable water. Whilst this method gives useful estimates of possible storms in areas where they have not been recorded, it also allows an estimate of the resulting flood. SW England is noteworthy for extreme rainfall (Clark, 1991). In particular the largest one-day fall of 279 mm was recorded at Martinstown in Dorset, 45 km to the south of Gasper. An unofficial fall of 355 mm (Rodda, Downing, & Law, 1976) has also been noted for this event, while a new analysis has recently been made of manuscript details in the care of Binnie and Partners, now Black and Veatch, (Clark, 2005a). A new isohyet map of the 1955 storm was produced for the Gasper catchment area, this gave an areal rainfall depth of 165 mm in three hours. The estimated storm profile used in the unit hydrograph convolution was the same as given in Clark (2005a). It is important to note that this profile is less peaky than the standard 50% summer storm profile as given in the FEH and also storm profiles that are used to estimate the PMF.

Transposition and Maximisation of a Historic Storm and the FEH Rainfall-run-off Method

The estimation of the PMF in the UK by means of transposition and maximisation of storms has not been widely undertaken. However, in their assessment of the PMF at Bruton dam, 7 km to the WNW, Black and Veatch (2006), considered the Martinstown storm and a maximisation factor of 1.46 (Clark, 2005a) in order to provide estimates of the PMF at that dam site. The result considerably exceeded the spillway capacity. For the Martinstown storm over the Gasper catchment area the three-hour storm depth was 241mm and with the same rainfall profile.

Application of a Non-linear Unit Hydrograph Model to the Martinstown Storm

Studies of the Upper Brue at Bruton (Clark, 2004a) and the Valency at Boscastle (Clark, 2005b, 2006) have shown that the FEH method of calculating unit hydrograph ordinates tends to underestimate the peak discharge of floods but also overestimates the volume of runoff. This is probably because of the non-linear way in which small catchments respond. Furthermore, the FEH percentage runoff has been found to be too low for more extreme floods. With these considerations in mind and the need to improve flood warning at Bruton, the author developed a non-linear flow model (Clark, 2004a), which is in use by the Environment Agency for Bruton and under consideration for Boscastle. The structure of the model is now briefly described.

Time to peak: This is based on observed flood events which are used to produce an empirical formlation. For the upper Stour at Gasper dam this becomes,

$$Y = \{(2.4073R + 10.1005)/R\} - 1.55. \quad \ldots (7)$$

Where Y = time to peak in hours. Time base = 2.52 time to peak.

Rainfall is partitioned into slope runoff = sinθ rainfall mm, where θ = mean catchment slope, and Ksat runoff where Ksat = saturated hydraulic conductivity based on a large sample of soil measurements made in the field and laboratory. A weighted estimate of the percentage runoff is made using the frequency distribution of Ksat values. Quickflow is the sum of slope runoff and Ksat runoff. Delayed flow is rainfall – quickflow.

Ordinates of the unit hydrograph, rising stage,

$$Y = [\text{INVLOG } 2(t - 0.9T_p) / 1 + \text{INVLOG } 2 \\ (t - 0.9T_p)\, Q_p \quad \ldots (8)$$

Where T_p = time to peak; Q_p = peak runoff rate per mm net rainfall = $330/T_p$ (catchment area/1000).

Falling stage,

$$Y = \{[\text{INVLOG } (t_1 - 0.88(TB - T_p)] / 1 + [\text{INVLOG} \\ (t_1 - 0.88(TB - T_p)]\}\, Q_p \quad \ldots (9)$$

Where TB = time base; $t_1 = TB - t$, where t = time

Delayed flow: For Gasper this has a one hour delay after the storm commences and one hour time to peak. This is based on empirical observations made on the upper Brue and elsewhere.

Delayed flow: rising limb, 0.30T, where T = time since the start of delayed flow. Falling limb: $0.031T + 0.30$ where T = hours since peak flow.

The non-linear flow model requires a storm hyetograph and an estimate of the SMD prior to the storm. Observations made over a twenty-year period at CHRS 8 km to the WNW and daily rainfall measured at North Brewham allow an estimate of SMD to be made. Comparison of these and other results with MORECS (Meteorological Office Evaporation Calculation System) shows that the latter are often in serious error. Comparison of water balance of the upper Brue at Lovington with lysimeter and MORECS estimates of water losses shows that direct measurements made at CHRS are more realistic (Clark, 2002b).

Assessment of PMF Using Flood Frequency Analysis

It is widely acknowledged that the PMF has a return period of 10^6 years. Where the peak discharge and return period of historic floods can be estimated, together with an estimate of bankfull discharge which for clay catchments has a return period of about 2 years, it is possible to produce an estimate of the PMF. A new method of estimating the return period of a historic flood has recently been described (Clark, 2007). Briefly the return period of a flood can be given,

$$Rp_{\text{flood}} = Rp\ ER \, . \, Rp\ SMD \quad \ldots (10)$$

Where Rp flood = return period of the flood; $Rp\ ER$ = return period of effective rainfall; $Rp\ SMD$ = return period of antecedent SMD. The effective rainfall is storm rainfall – SMD. The method was developed from 10 years of lysimeter data gathered at CHRS that were analysed using an extreme value distribution. From this analysis the return period of the antecedent SMD can be calculated. Adjustments to the method can be made so that it can be used in other areas of England and Wales. Full details are given in Clark (2007).

Reservoir Routing

At the outlet of New Lake there are two control structures, the compound service weir and the culvert. As the water level rises behind the dam during a flood these two structures, will, in turn, exert their influence on the rate of discharge from the lake.

The weir: discharge over the weir is assessed using,

$$Q = CBH^{1.5}, \text{ where } C = \text{discharge coefficient of } 0.7 \quad \ldots (11)$$

At heads lower than 0.4 m the lower retention level of 129.42 m is effective. Above this level the complete diameter of the weir comes into operation.

The culvert: water flowing over the weir immediately enters the culvert. Some head losses are expected on account of the turbulence but after the weir is drowned out this effect would become less significant. Discharge through the culvert has been estimated using the Colebrook-White equation which in metric units,

$$V = [-2\,(21.532g\ Di)^{0.5}]\ 0.3048\text{Log}\ [\ 0.00328\ Ks/(12.1389\ D) + 2.15\ v/3.2808\ D\,(13.123\ Di)^{0.5}\ ] \quad \ldots (12)$$

Where V = velocity m/s; D = equivalent diameter of culvert m^2; Ks = roughness (mm), i = water surface slope; v = kinematic viscosity of fluid. Using an effective culvert diameter of 3.246 m^2 gives a velocity at full bore of 3.2 m/s. Thus the culvert capacity is 10.4 cumecs. Discharges in excess of this value can be accommodated with a head loss that is expressed in higher water levels on the upstream side. The head loss is calculated using the velocity head equation,

$$V^2/2\,g \quad \ldots (13)$$

The culvert has right-angled edges at entry and exit hence the coefficients are 1.3 and 0.5 respectively. The friction losses are calculated from,

$$HL_{\text{friction}} = L\,(V\,n/\,R^{0.66})^{2} \quad \ldots (14)$$

Where L = length of culvert (m); V = velocity m/s; n = Mannings n; R = hydraulic radius.

Stage Inflow-outflow Relationships

The routing of a flood through a reservoir involves the solution to the equation of continuity,

$$I\,\Delta t = O\,\Delta t + \Delta S \quad \ldots (15)$$

Where I and O are average rates of inflow and outflow for a given time period; ΔS = change in storage; Δt = time interval.

This equation can be rewritten,

$$[S_2/\Delta t + O_2/2] = [S_1/\Delta t - O_1/2] + [(I_1 + I_2)/2] \quad \ldots (16)$$

Where the subscripts are for the start (1) and end (2) of each time interval. The storage of the reservoir is related to the height of water and the outflow determined by a rating equation. Table 1 shows the values of storage volume, rate of outflow, storage per unit time and the storage outflow functions. The area of New Lake is 4.6 Ha. Once the water level increases significantly above the retention level the area will increase according to the average slope of the land surrounding the lakeside. The water level is initially controlled by the weir but above discharges of about 15 cumecs the rate of outflow is controlled by the culvert. Table 1 shows the stage discharge inflow-outflow parameters. These were then applied to the inflow hydrograph during flood events.

Table 1: Reservoir Routing Parameters

Water Level Above Weir (m)	*Storage (m^3)*	*Outflow $m^3 s^{-1}$*	*S/Δt*	*(S/Δt – O/2)*	*(S/Δt + O/2)*
0.0	0.0	0.0	0.0	0.0	0.0
0.1	4737	0.24	2.63	2.39	2.87
0.25	11802	0.94	6.55	6.08	7.02
0.5	24237	2.66	13.46	12.13	14.79
0.75	37277	7.19	20.70	17.10	24.29
1.00	50950	11.97	28.30	22.31	34.28
1.25	65227	16.25	36.23	28.10	44.35
1.388	73390	16.8	40.77	32.37	49.17
1.638	*88637*	*23.2*	*49.24*	*37.64*	*60.84*
1.888	*105754*	*41.2*	*58.75*	*38.15*	*79.35*
2.138	*121002*	*77.4*	*67.22*	*28.52*	*105.94*
2.388	*138090*	*105.8*	*76.71*	*23.81*	*129.61*
2.588	*152259*	*151.8*	*84.58*	*8.68*	*160.48*

Note: water level reaches crest level at a level above mean weir level of 1.388 m.

In making the reservoir routing calculations the effect of the original and small spillway has not been taken into account. It is understood that this spillway may become blocked during a flood event.

RESULTS AND DISCUSSION

The results of the various methods that have been described make up the contents of this section. This will enable a direct comparison between the results and to allow a judgement to be made as to the most likely spillway design flood.

ICE (1996) Rapid Method

For Gasper New Lake Area = 6.75; S1085 = 25.7; SAAR = 965. Therefore Q = 71 cumecs.

Wave Surcharge:

U50 = 21 m/s

U = 0.79 × 21 = 16.59 m/s

Altitude adjustment = 1.0(0.001 × 129) = 1.129

Direction adjustment (bearing 315°) = 0.86

Fetch (350 m) adjustment = 1.05

Duration adjustment = 1.05

(U) = 16.59 × 1.129 × 1.05 × 1.05 × 0.86 = 17.7 m/s

Significant wave height = $UF^{0.5}/1760$ = 0.19 m

Wave run-up was not considered significant Figure 6, (ICE 1996).

Design wave height H_D = 1.67 × 0.19 = 0.32 m

NB: the factor 1.67 refers to an earth dam where no overtopping can be accepted. In ICE (1996) the minimum standard for wave surcharge is 0.6m. However, given the small size of the lake and shelter provided by the trees, a value not exceeding 0.4m is recommended.

FEH (1999) and Other Unit Hydrograph Estimates

The FEH uses the estimates of PMP in the FSR and an updated version of the rainfall-runoff method.

The 2.5 hour storm profile: 14 22 99 22 14 mm 30 min^{-1}.

ReFH time to peak 1.91 hours. Reduced by one third = 1.28 hours.

Storm rainfall 171 mm

Antecedent rainfall = 42.8 mm

CWI = 164.8

DPR_{CWI} = 9.95

DPR_{RAIN} = 13.6

SPR_{HOST} = 31.5

% runoff = 31.5 + 9.95 + 13.6

The estimated peak discharge from the 2.5 hour PMF = 86 cumecs. This does not include a value for baseflow which is less than 1 cumec. Figure 3 shows the inflow and outflow hydrographs, where it can be seen that the lake does not reduce the flood at all. The possible effect of Garden lake has not been investigated in this study but from an inspection of the lake in the field the reduction in flood magnitude is unlikely to be significant. The same applies to the much smaller Turner's Paddock Lake.

Table 2 shows the results of other unit hydrograph simulations.

Discussion of the results in Table 2 mainly centres around three basic concerns. Uncertainties in the input data, validity of transposition and maximisation, and the accuracy of the FEH rainfall runoff method versus the non-linear flow model.

Table 2: Estimates of Extreme Floods at Gasper Lake

Method	*Inflow ($m^3 s^{-1}$)*	*Outflow ($m^3 s^{-1}$)*	*Peak Level (m OD)*	*Head Over Crest (m)*	*Head with Wave Surcharge (m)*
FEH/PMF	86	86	131.96	0.86	1.18
Transposition of Martinstown storm	49	48	131.67	0.56	0.88
Trans. & max.of Martinstown storm	70	70	131.86	0.76	1.08
Trans. Using Non-linear FM	68	68	131.84	0.74	1.06
Trans. & max Non-linear FM	94	94	132.03	0.93	1.25
PMP/PMF	146	146	132.28	1.18	1.50
Flood freq. analysis	150	150	132.30	1.20	1.52

Uncertainties with the Input Data

The unit hydrograph method is very sensitive to the value of time to peak. In all cases the FEH PMF time to peak of 1.28 hours was used in preference to the T-year value of 1.91 hours. When the equation for time to peak for the upper Brue was applied to Gasper, the result was 2.65 hours. This was reduced according to the ratio of mainstream channel length 3/9.2 to give a value of 0.86 hours. This was then increased to 1.0 to allow for a small reservoir lag.

There is some uncertainty as regards the percentage runoff. Measurements of the saturated hydraulic conductivity of soils in the catchment has shown that FEH values are too low. This is also the conclusion of a recent study of the upper Brue (Black & Veatch, 2005). When the Ksat based percentage runoff data are applied to the FEH transposition and transposition and maximisation of the Martinstown storm the peak discharges are 75 and 123 cumecs respectively.

Validity of Transposition and Maximisation of Storms

The WMO manual for estimating PMP (WMO, 1986) explains the hydrometeorological reasoning behind the process of moving and maximising storms. In the present case the Martinstown storm took place about 45 km to the south of Gasper and the value of SAAR is similar in both areas. The detailed storm profile at this stage can only be an educated guess since apart from a spot reading taken at about 1900 hours during the storm by Norris Symonds, the hourly rainfall intensities are a matter of speculation. However, the peak hourly value given in Clark (2005a) of 55 mm hr^{-1} is well below the 1 hour PMP given in the FEH. The process of maximisation has a long history, and while there is the possibility of excessive maximisation being achieved, the value in this case was only 1.46, and certainly well below values of 1.75 or above which have been quoted elsewhere.

The Accuracy of the FEH Rainfall-run-off Method Versus the Non-linear Flow Model

The relative accuracy of the two methods of unit hydrograph simulation can be tested by applying the storm of 12 July 1982 to the Gasper catchment.. This resulted in New Lake being filled up with some slight overtopping. The expected inflow discharge was 24 cumecs and the resulting outflow discharge was about 17 cumecs which, when combined with runoff between Gasper and Bourton Mill, where the catchment area is 11.2 km^2, produced a peak discharge of about 25 cumecs (Black & Veatch, 2005, Clark, 2004b) at Bourton Mill. On the other hand the FEH method predicted a peak discharge of 12.2 cumecs at Gasper dam. This would not have caused a significant backwater effect and would also have prevented the observed flood at Bourton Mill taking place. The flood of July 1982 is sufficiently important from the point of view of frequency of overtopping at Gasper to be discussed below.

Given the uncertainties in estimating extreme floods, from the results presented in Table 2, a PMF in excess of 100 cumecs is suggested. The highest two estimates are within 12% of each other and the lower of these estimates has a runoff rate of 19 cumecs per square km. This is slightly less than the run-off rate of the PMF at Bruton dam whose catchment area is four times as big. While the FEH PMF based estimate is close to 90 cumecs, it is an underestimate because the method tends to underestimate the % runoff during extreme events: there is no allowance for rainfall intensity, only rainfall depth in the dynamic term.

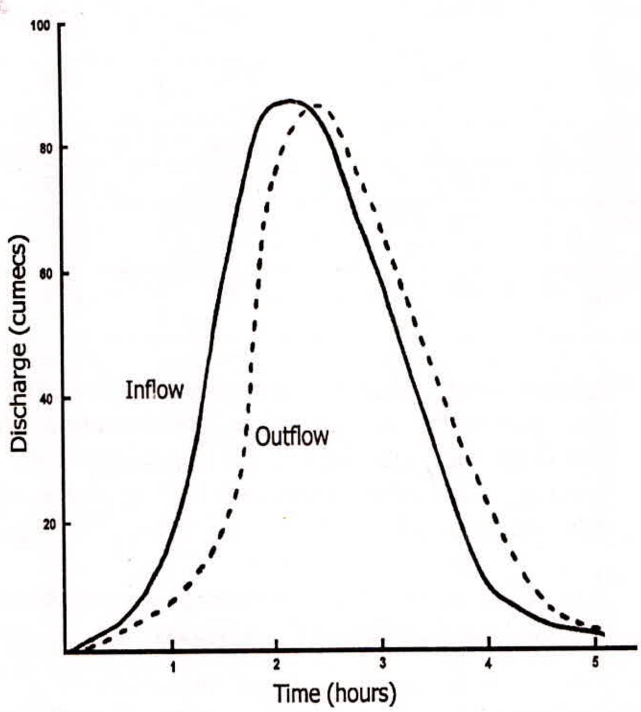

Fig. 3: FEH inflow-outflow hydrographs at Gasper dam

FLOOD FREQUENCY ANALYSIS

The method of Clark (2007) was applied to the storms of 1917 and 1982 whose peak discharge were 31 and 24 cumecs respectively. The estimated return periods of the two floods are presented below:

1982 storm

Effective rainfall: 52 mm in 2.0 hours

SMD: 30 mm.

Seasonal return period of rainfall: 118 years

Return period of SMD: 2.1 years.

Return period of flood: $2.1 \times 118 = 248$ years

1917 storm

Effective rainfall: 70 mm in 4 hours

SMD: 30 mm.

Seasonal return period of rainfall: 230 years

Return period of SMD: 2.2 years

Return period of flood: $2.2 \times 230 = 506$ years

Bankfull discharge was estimated from current meter readings in the channel 80 m below Gasper dam at a stable reach. The result was 3 cumecs. This was slightly lower than a wider reach just downstream of the measuring site but felt to be representative of the river as a whole.

Simple regression was then applied to the discharges of the 1917 and 1982 floods (31 and 24 cumecs respectively) and the modified reduced variate (Rakhecha & Clark, 1999) and updated (Clark, 2007) to give a flood frequency relation for the upper Stour at Gasper Dam and shown graphically in Figure 4,

$$\text{Log } Q = 0.1849y + 0.4413 \quad \ldots (17)$$

Where Q = peak discharge,

$$y = [(-\ln \ln (1 - 1/T) - 3.3842) * 1.09348 * T^{-0.046518}] + 3.3842 \quad \ldots (18)$$

where T = return period (years).

Application of this equation for a return period of 10^6 years when $y = 9.382$ gives a peak discharge of 150 cumecs. Also shown on Figure 4 is the estimated flood frequency using the ReFH (Kjeldsen *et al.,* 2005) up to a return period of 150 years and the FEH PMF. There is a very big difference between the two results, but the floods of 1917 and 1982 strongly suggest that the more severe estimate is more realistic. The estimated PMF of about 150 cumecs gives a rate of run-off well below the extreme catastrophic flood of Allard, Glasspole, and Wolf (1960).

PREVIOUS OVERTOPPING INCIDENTS

In the inspection report (Stourhead Western Estate, 2004) of Gasper Dam carried out under Section 10 of the Reservoirs Act (1975), the Water Bailiff Malcolm Bullen stated that the dam briefly overtopped two times during the past 30 years while he has lived at the Laundry House which is next to New Lake. If this statement is correct then it is great cause for concern, not only because of its occurrence but also because of the light that it may throw upon some of the estimates of PMF. Mr Bullen was interviewed during February 2007 and asked to confirm the two incidents of overtopping. They were confirmed, but no year or date could be recalled. However, he was clear that during these two events the culvert was not blocked by any debris. Furthermore, when questioned as to the time of day and season he was clear that they both took place during the day and in the summer, when, according to him, a heavy storm took place. There is no doubt that one of those incidents was during the storm of July 1982 when the area received about 100 mm, much of which fell in 5 hours.

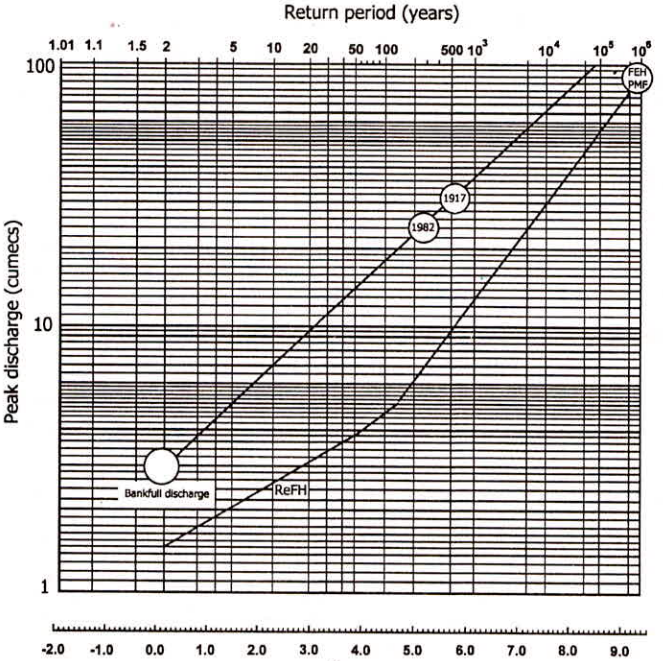

Fig. 4: Flood frequency estimates for Gasper dam site

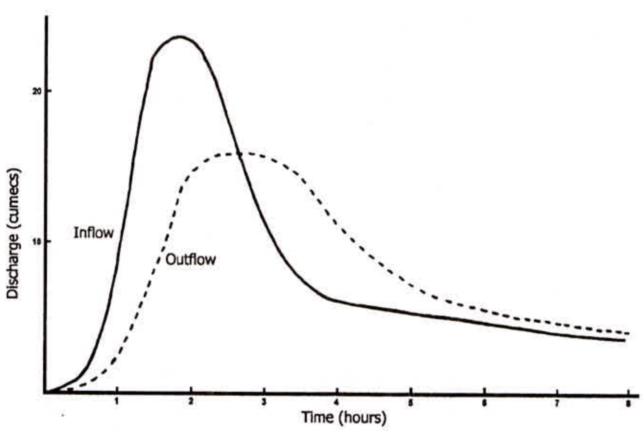

Fig. 5: Inflow-outflow hydrographs for 12 July 1982 at Gasper dam

The autographic rainfall trace at Gillingham, 8 km to the SE, is very similar to the estimated trace at North Brewham 7 km to the WNW (Clark, 1986). Since the former has more detail this has been used in a unit hydrograph simulation of the resulting flood. Figure 5 shows the resulting inflow and outflow hydrographs. The peak outflow would require a head loss of 2.16 m which gives a peak water level of 131.07 m, 0.028 m below crest level. Since it is likely that some wave action existed at the time then there is a very good chance that this flood did indeed overtop Gasper Dam. An outflow of about 15–17 cumecs would have been necessary in order to produce a flood of about 25 cumecs downstream as observed at Bourton Mill. Since the Water Bailiff was clear there was no blockage of the culvert then the simulation of the hydrograph has confirmed both the likely water level behind the dam and the size of the flood 2 km dowstream.

Given that the likely value for the PMF at Gasper New Lake is around 140 cumecs it is pertinent to question the likely probability of a damaging overtopping incident. Reference to the flood frequency equation (17) would suggest a peak discharge of at least 50 cumecs. This has a return period of less than 4000 years. In the Inspector's report (Stourhead Western Estate 2004), it is suggested that the dam could retain floods up to about the 1 in 10000 year event. Since there have been two minor overtopping incidents during the past 30 years this cannot be correct. A flood as low as 40 cumecs could, at present, lead to serious damage to the dam. This flood has an expected frequency of about 1 in 1600.

DISCUSSIONS

It should be clear from this report that some of the raw data in the inspection report are in error. With the level of uncertainty in any study of this nature it is important to give enough detail to enable the reader to understand how the results have been produced. The summer PMF was given as 59.8 cumecs. This is exceeded when the ICE (1996) rapid method is applied and considerably exceeded when storm transposition and maximisation procedures are carried out. A PMF of 60 cumecs is equivalent to a runoff rate of 8.88 cumecs per km^2 which is much less than the runoff rate during the 1768 flood at Bruton with a catchment area of 31 km^2. Higher rates of run-off are to be expected in small catchments where the areal reduction in point rainfall is less.

In view of the large area of woodland upstream of the lake some provision should be in place for catching large trees which could cause partial blockage of the culvert. Although the culvert should be enlarged to carry at least 100 cumecs and therefore be less liable to being obstructed by trees, in a major event this could still happen.

Flood risk analysis is an evolving science, but enough progress has been made in recent years to adopt, with confidence, the higher standards of safety that this report has suggested. The local geological map shows the presence of 11 landslips. Of these, two are located close enough to the lake to warrant inspection in the field in order to detect any evidence that they may have been active in the recent past. It is known that a slip near to the Mere reverse fault some three km to the SW is still active (Prudden pers com).

The recommendations in the inspector's report are all worthwhile. In addition, the capacity of the culvert must be enlarged to safely pass 130 cumecs. Furthermore, a suitably designed trash rack should be installed which would capture large debris passing downstream. This should be sited well away from the spillway in order to allow safe passage of floodwaters through the enlarged culvert below the highway.

REFERENCES

Allard, W., Glasspole, J. and Wolf, P.O. (1960). Floods in the British Isles. *Proc. Inst. Civ. Engrs.* 15, 119–144.

Babtie (2000) *Clarification on the use of FEH and FSR design rainfalls.* http://www.defra.gov.uk/environment/rs/index.htm

Black & Veatch (2006). *Bruton flood storage reservoir: report on hydraulic and hydrological studies.* Environment Agency.

Black & Veatch (2005). *Bourton Mill Pond. Dam Safety Report.* Clublight Developments.

Bootman, A.P. and Willis, A. (1981) Discussion on paper 5 (Folland *et al.*) in Flood Studies Report – Five years on. Thomas Telford, 62–63.

Brown, A.J. and Gosden, J.D. (2004). *Interim guide to quantitative risk assessment for UK reservoirs.* Thomas Telford, London.

Charles, J.A. (2005). Use of incident reporting and data collection in enhancing reservoir safety. *Dams and Res.* 13, (2), 29–35.

Clayden, J., Dyke, T. and Carruthers, G. (2007). *The Boltby incident.* ICE 21 May.

Clark, C. (1986). *Floods at Bruton: past present and future.* Charldon Publications. Bruton, 52pp.

Clark, C. (1991). A four parameter model for the estimation of rainfall frequency in South West England. *Meteorl. Mag.* 120, 21–31.

Clark, C. (2001). Great storms and floods over Southern England, August 1770. Weather, 56, 327–337.

Clark, C. (2002a). New estimates of PMP for Great Britain. *Internat. Water Power & Dam Constr.* 54, 4, 18–26.

Clark, C. (2002b). Measured and estimated evaporation and soil mositure deficit for growers and the water industry. *Meteorl. Appl.* 9, 85–93.

Clark, C. (2004a). A non-linear unit hydrograph model for real time flood forecasting below a dam. *Internat. Water Power & Dam Constr.* 56, 12, 20–26.

Clark, C. (2004b). *Flood frequency analysis for the Upper Stour at Bourton, Dorset.* Rept. Commissioned by Bourton Parish Council. 33pp.

Clark, C. (2005a). The Martinstown storm 50 years on. *Weather*, 60, 9, 251–257.

Clark, C. (2005b). Catastrophic floods: magnitude and frequency investigations. *Internat. Water Power & Dam Constr.* 57, 2, 14–21.

Clark, C. (2006). Planning for floods: will we ever learn? *Internat. Water Power & Dam Constr.* 58, 6, 20–27.

Clark, C. (2007). Flood risk assessment using hydrometeorology and historic flood events. *Internat. Water Power & Dam Constr.* 59 (4), 22–30.

Clark, C. and Pike, W.S. In press. The Bruton storm and flood after 90 years.

Cox, D.R. (2002). *Some comments on 10,000 year return period rainfall.* Report to Defra. 24pp. http://www.defra.gov.uk/environment/water/rs/pdf/coxreport.pdf

DEFRA (2005). Floods and reservoir safety – Revised guidance for Panel Engineers. *Dams and Res.* 15, (1), 16.

Froehlich, D.C. (1995). Peak outflow from breached embankment dam. *ASCE, J. Water Res.* Plan. *& Mangt.* 12, (1), 90–97.

Galvin, J.F.P. and Pike, W.S. (2001). A comparison of Eastertide storms in Southern Britain in 1998 and 1999. *Weather,* 56, (4), 125–134 & 137–138.

Gosden, J., Brown, A., Williams, N., Gardiner, K., Clarke, N. and Green J. (2006). Emergency planning for UK reservoirs. *Dams and Res.* 16, (2), 24–28.

Halliburton, Brown & Root Ltd. (2002). *Supplement to report by Sir David Cox on review of methodology for extrapolation of extreme rainfall in FEH.* HBR, Leatherhead, Surrey.

Houghton-Carr, H. (1999). *Restatement and application of the Flood Studies Report rainfall-runoff method.* Vol. 4 Flood Estimation Handbook. IOH. Wallingford.

Hughes, A., Hewlett, H., Samuels, P.G., Morris, M., Sayers, P., Moffat, I., Harding, A. and Tedd, P. (2000). *Risk management for UK reservoirs.* CIRIA report C 542

ICE (1996). *Floods and reservoir safety.* 3ed. Thomas Telford, London.

IOH (1999). *Flood Estimation Handbook.* 5 vols. Wallingford.

Kjeldsen, T.R., Stewart, E.J., Packman, J.C., Bayliss, A.C. and Folwell, S. (2005). *Revitalisation of the FSR/FEH rainfall-runoff method.* Final report to Defrs/EA. CEH Wallingford.

MacDonald, D.E. and Scott, C.W. (2000). Revised design storm rainfall estimates obtained from the Flood Estimation Handbook (FEH). *Dams 2000.* Proc. 11th British Dam Society conference, Bath, 12–13.

NERC (1996). British Geological Survey. *Wincanton, solid and drift geology.*

Pike, W.S (1994). The remarkable early-morning thunderstorm and flash flooding in Central Southern England on 26 May 1993. *J.Meteorl.,* 19, 43–64.

Rakhecha, P.R. and Clark, C. (1999). Revised estimates of one-day probable maximum precipitation (PMP) for India. *Meteorl. Appl.* 6, 343–350.

Rodda, J.C. Downing, R.A. and Law, F.M. (1976). *Systematic Hydrology.* Newnes-Butterworths. London.

Stourhead Western Estate (2004). *Report on an inspection carried out under Section 10 of the Reservoirs Act 1975*: Gasper New Lake.

WMO (1986). *Manual for estimation of probable maximum precipitation.* 2nd Ed. Operational Hydrology Report No 1. WMO No 332.