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Small Reservoirs Simplified Risk Assessment Methodology

Research modelling report

Project: FD2658

Flood and Coastal Erosion Risk Management Research and Development Programme

Small reservoirs simplified risk assessment methodology

Research modelling report

MCR5053-RT002-R03-00

January 2014

Document information

Document permissions	Confidential - client
Project number	MCR5053
Project name	Small reservoirs simplified risk assessment methodology
Report title	Research modelling report
Report number	RT002
Release number	R03-00
Report date	January 2014
Client	Defra
Client representative	John Goudie
Project manager	Alex Topple
Project director	Craig Goff

Document history

Date	Release	Prepared	Approved	Authorised	Notes
14 Jan 2014	03-00	ATO	CAG	CAG	Final revision
06 Aug 2013	02-00	MVD	CAG	CAG	Minor edits following JG comments
10 Jun 2013	01-00	ATO	CAG	CAG	

Document authorisation

Prepared	Approved	Authorised

Executive Summary

This Report is a companion to the Guidance report “Small reservoirs simplified risk assessment methodology”, providing a description of the science underpinning the methodology in that report. It includes a brief technical summary of the science including:

- The breach process, a review of the potential breach prediction methods;
- Predicting local inundation, the spreading equation;
- Predicting the rate of flood attenuation and extent of valley inundation using the CIRIA methodology and comparing it to the Ponce method, InfoWorks RS and the RIM methodology.

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1. Introduction

This modelling report accompanies the 'Small reservoirs simplified risk assessment – guidance report' and provides details of the science behind the methodology.

1.1. Purpose of report

This project is concerned with facilitating development of new small reservoirs retained by embankment dams by:

- Providing a rapid screening method to assess risk to people;
- Facilitating, locating, and constructing small reservoirs where they do not pose a risk to people.

A methodology is required to model:

- Flood waves arising from dam failures;
- The hazard to people close to the dam;
- How the flood dissipates as it travels away from the dam for example down a valley.

The parameters required to assess the hazard to people comprise of depth (d) and flow velocity (v), simplified in some situation to total discharge (Q) divided by total width of flooding (W). The tests for when there is a significant risk to people are given in the guidance report.

This modelling report describes the science underling the guidance report, and should be read in conjunction with that volume.

1.2. Size of reservoir covered by this project

One of the early tasks necessary was to define the size of reservoir which was covered by this project, as this was important in defining the range of parameters considered in modelling dam breach failure and attenuation of the flood wave. Data was obtained from the register of reservoirs regulated by the Environment Agency, which contains information on dam heights and reservoir volumes (>25,000m^3) of dams in England and Wales, as shown in Figure 1.1 and Figure 1.2. It was decided that the size of reservoir covered by this project would be reservoirs not exceeding 100,000m^3, and dam height not exceeding 10m, on the basis that:

- This is a typical range of size of new small reservoirs;
- Larger dams are likely to have greater engineering input into their siting and design, such that this rapid screening would be of less value.

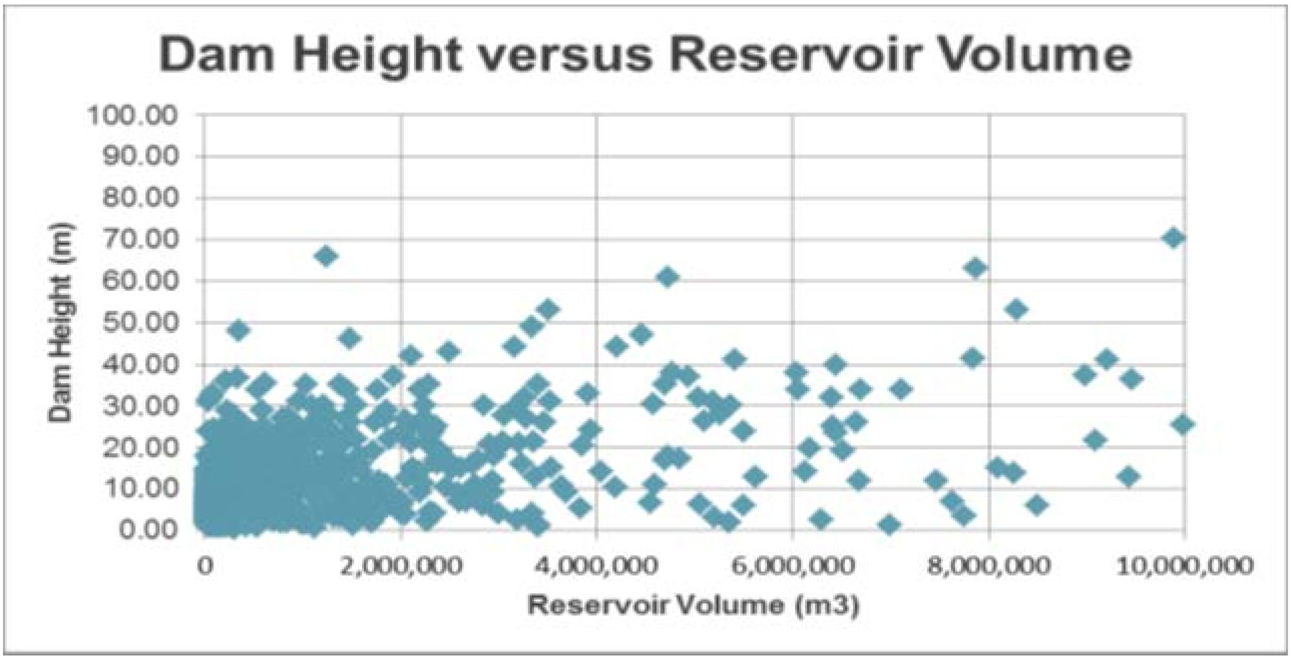

Figure 1.1: Dam height versus reservoir volume

Source: Environment Agency register of dams

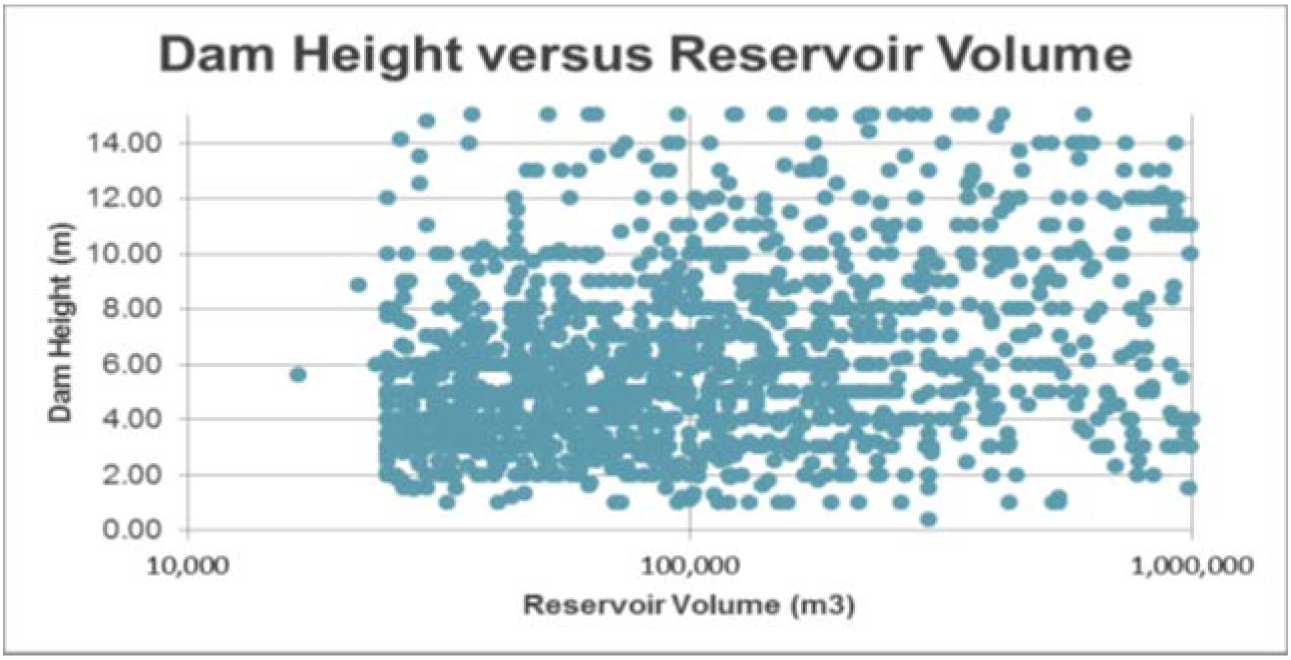

Figure 1.2: Dam height versus reservoir volume - <1Mm^3 and <15m

Source: Environment Agency register of dams

2. Predicting the dam breach hydrograph

The breach hydrograph defines the rate at which the reservoir water is released when the dam fails. The nature of the hydrograph depends upon the rate of breach growth combined with the stage-volume relationship for the reservoir. The same volume of water may be released quickly, with a high peak discharge, or more slowly, with a longer steady peak discharge. The nature of release can significantly affect flood impacts downstream, hence identifying these characteristics is important for a reservoir risk assessment.

2.1. Introduction

The flood hydrograph created as a result of dam failure (breach) depends upon factors including the:

a. Failure mode i.e. the processes initiating and progressing failure;
b. Erodibility of the soil;
c. Volume and shape of the reservoir;
d. Dam height;
e. Magnitude and shape of the incoming flood hydrograph (only relevant to impounding reservoirs);
f. Tail water level at the breach location.

The challenge for this project was to identify a method for predicting the breach flood hydrograph that combined accuracy of prediction with simplicity in calculation. Invariably these two factors are normally opposed, with more accurate methods typically requiring more complex analyses.

The approach taken to develop the method for small reservoir analyses was as follows:

1. Review of recent predictive equations and models to identify methods with the most potential;
2. Assess performance of selected methods;
3. Select (and refine if appropriate) one method;
4. Create a tabular or graphical summary of data for a range of dam heights, reservoir volumes etc. to support the small reservoir risk analysis method.

2.2. Breach processes

The recent PhD thesis by Morris (Morris, 2011) provides a useful summary of the state of art of breach modelling to ~2010 along with identification of key processes affecting the breach hydrograph – in particular the balance between soil erodibility and the reservoir stage volume relationship.

2.2.1. Effect of reservoir shape and soil erodibility on breach hydrographs

The difference in breach outflow arising from the interplay between reservoir stage volume and soil erodibility can be very large. Figure 2.1 shows an example of the two types of hydrograph that might occur. Hydrographs in-between these two shapes may arise when considering embankments built from layers of different soil erodibility. Both hydrographs in Figure 2.1 show a release of the same volume of water.

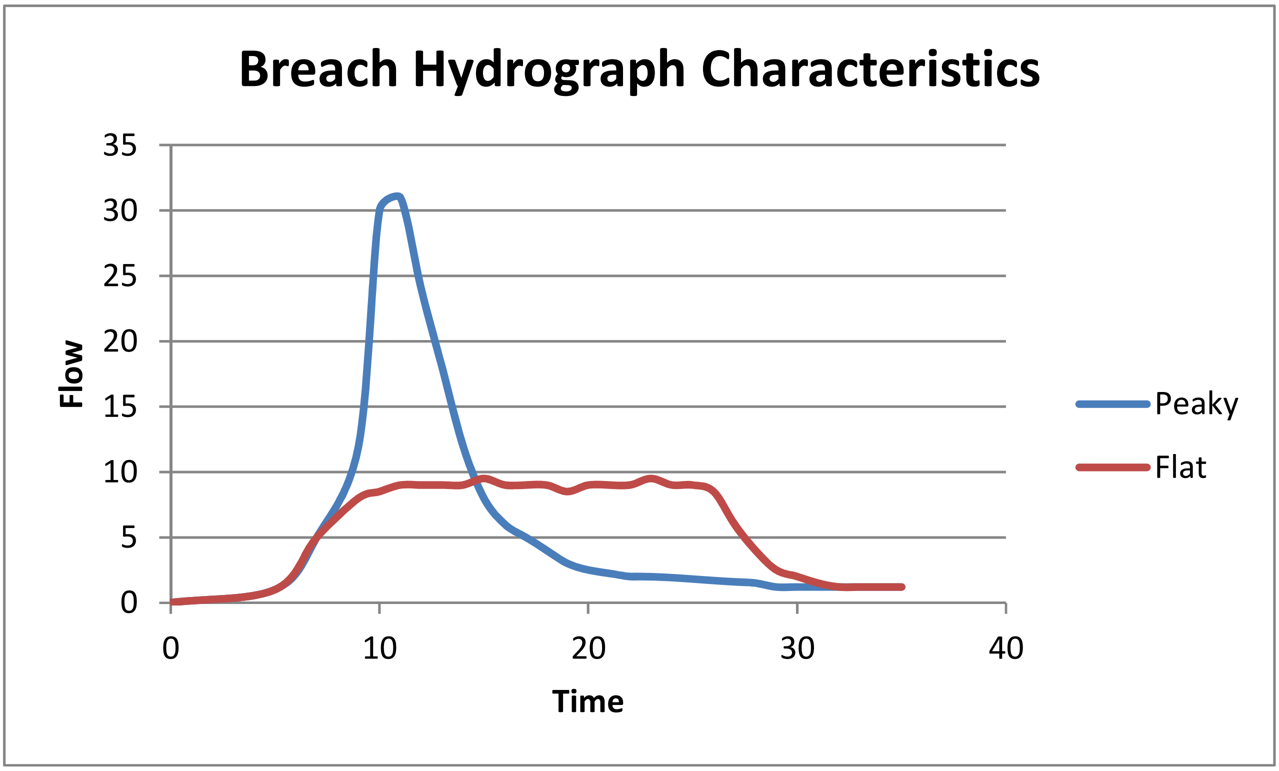

Figure 2.1: Different types of breach outflow hydrograph

Source: HR Wallingford

Potential variation in the reservoir stage volume relationship is shown in Figure 2.2 and Figure 2.3. The plot on the left shows different stage – area relationships, whilst on the right, the associated stage - volume relationships. These relate to uniform stage area (i.e. box shaped reservoir volume), uniformly changing in 1D (i.e. wedge shaped volume) and uniformly changing in 2D (i.e. pyramid shaped volume). These correlate to factors of 1, 0.5 and 0.33 against the product of area and depth of the reservoir. These plots show uniform or average change; site specific change rates will vary about these values. [An option would be for the user to plot their stage volume relationship as a means of seeing how their dam compares to given data, and hence what predicted values they should extract].

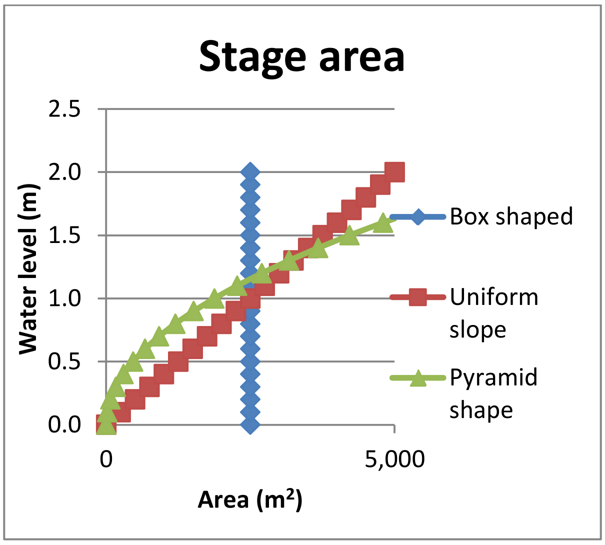

Figure 2.2: Different stage – area relationships

Source: HR Wallingford

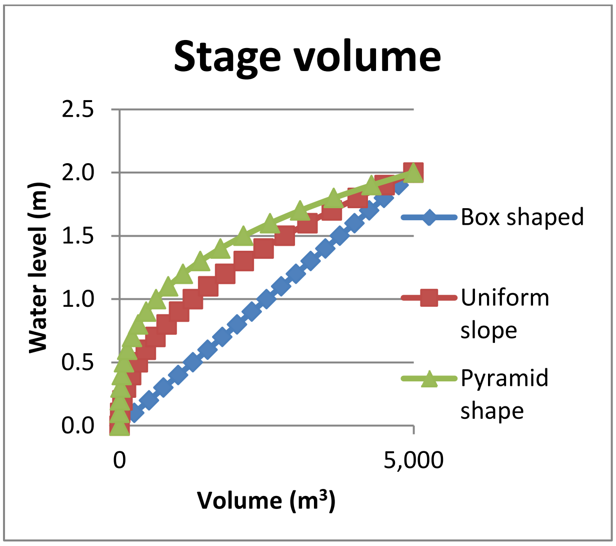

Figure 2.3: Associated stage – volume relationships

Source: HR Wallingford

If we provisionally assume that the appropriate level of detail for predicting breach includes identification of the different hydrograph shapes, then the objective for the project is to provide data which will allow the user to select an appropriate Q_p and hydrograph shape for their dam. From this, the user could then (i) use Q_p for local inundation prediction and (ii) rebuild the hydrograph for use in long field inundation prediction (which needs the hydrograph shape for prediction of Q_p attenuation).

In order to define the different hydrograph shapes the following parameters would be needed:

- Q_p Peak discharge
- T_i Time of breach hydrograph initiation
- T_p Time of peak discharge (or initial time of peak discharge)
- T_{ap} Time at peak discharge (for flat shaped hydrograph)
- T_f Time of failure (from initiation to drain down.

The type of hydrograph (‘peaky’ or ‘flat’) will be implicit from the data given – when $T_{ap} = 0$ the hydrograph is considered peaky.

2.2.2. Erodibility

Erodibility of the soil will affect the way in which erosion initiates and progresses. In some situations, dams may not fail – or will only partially fail.

The erosion of soil can be predicted using two key values and the equation below:

$$E = K_d b(\tau - \tau_c)^a$$

Where:

E is the erosion rate in $m^3/s/m^2$ (bulk volume hence rate of bed elevation change or retreat)

K_d is the erodibility or detachment coefficient in $cm^3. N^{-1}.s^{-1}$

τ is the effective shear stress

τ_c is the critical stress

a and b are empirical coefficients dependent upon soil properties (but often set to 1).

Equation 2.1: Soil erodibility

Source: Nearing et al (1988)

For the purposes of these analyses, τ_c has been assumed to be zero. This is a conservative assumption in terms of initiation of erosion (i.e. we always assume that erosion can occur regardless of soil type and state).

For the purposes of screening the erodibility of UK dams, it will be assumed that the erodibility can be linked to the underlying geology, i.e. that local soils have been used to construct the embankment (at least the shoulders, even if imported clay or an artificial liners such as concrete or HDPE has been used to form the watertight element). A simple overview of UK soils was included as Appendix D to the draft Engineering Guide to Early detection of internal erosion (KBR, 2007, available on BDS members area). A simple assessment could be as shown in Table 2.1 and Table 2.2. If this were part of the adopted methodology the challenge would be in providing simple guidance so that a farmer can assess what the erodibility coefficient is likely to be for his site.

Table 2.1: Range of erodibility of soils used to construct UK dams

Erodibility	Soil type	Examples	
		Geology	Dams where soil used in shoulders
More erodible	Sandy silty soils SM	Greensand	Mill Leese FSR, Nr Hythe
		Alluvial sands	Less likely as in 20th century sand likely to have been exploited in quarries than used as bulk fill in dam construction
		River terrace deposits	Thames water NI embankments
Medium	Low plasticity clays	Glacial (boulder) clays	Selset
Least erodible	High plasticity clays CH	London clay, Gault clay, Lias clay	Empingham

Table 2.2: Range of Erodibility of UK soils

	Method	Morris PhD	Key features	Range of Kd shown in PhD	Range of Kd considered reasonable for UK dams: Most erodible SM	Range of Kd considered reasonable for UK dams: Least erodible - CH
	Equation					
1	Temple and Hanson, 1994	Eqn 6.5	Function of dry density and clay content		10 (dry density 1.2, 0 % clay)	0.04 (Dry density 1.8, 80% clay)
	Regazzoni, 2009	Eqn 6.11-6.13	Liquid limit, clay content, degrees of saturation		No data to allow application to UK soils	
2	Hanson, 2007	Eqn 6.6	Compactive effort and water content		Lower values than above	
3	Qualitative description	Table 6.4		<0.001 to >20	20	0.01
4	Link to degree of compaction and clay content (Hanson)	Table 6.5, Figure 6.14		0.001 to 1000	100	0.01
5	Direct measurement JET HET		The two tests seem to give different results			

2.3. Review and analysis of potential breach prediction methods

Building from the findings of Morris (Morris, 2011), the following models were identified as potential solutions for predicting breach:

1. Froehlich equation (Froehlich, 1995) (as used for the RIM analyses). This is a simple equation based upon a regression analysis of ~20 historic dam failures in the US. The equation uses just dam height and volume to predict peak outflow, timing and breach width.
2. Xu equation (Xu and Zhang, 2009) based on analysis of 75 dam failures but including consideration of dam type, soil erodibility, reservoir type etc.
3. AREBA model – a simplified predictive model, designed to predict breach through homogenous embankments.
4. HR Breach model – a time stepping predictive breach model based upon observed physical processes.

In theory, the methods listed above are listed in order of accuracy, but also in terms of increasing complexity and time required for application. A comparison of performance was undertaken to see where an acceptable balance between accuracy and complexity of analysis fell for this project solution.

A range of parameters were identified for analysis. These included, for example, different dam heights, reservoir volumes, and soil erodibility. Initial checks were made assuming that breach initiated through overtopping. A comparison of results showed significant differences in prediction.

2.3.1. Comparing Froehlich, Xu, AREBA and HR Breach

Figure 2.4 provides an example of the difference in results seen between Froehlich and Xu. For comparative purposes, this plot also includes prediction using a simple broad crested weir equation based upon breach width equal to dam height or 3 x dam height.

The trend shown here is that the Froehlich prediction is conservative, being similar to the Xu prediction for a highly erodible dam material. Using a simple weir equation over predicts conditions greatly.

The next stage in assessment was to compare results from HR BREACH against the regression analyses. This showed that the predictive model generally predicted even lower peak discharges than the Xu regression equation. The difference was sufficient to warrant closer investigation, with model runs repeated using AREBA and cross checked. This trend in results was also confirmed with the AREBA model; the AREBA model gave similar results to HR BREACH (for this application, which considers only homogeneous earth embankments).

The interim conclusion at this stage was that the AREBA model should be used to create tabular data for the risk assessment methodology. AREBA was used over HR BREACH due to the shorter model run times, it's reduced complexity, and because it is more realistic than Froehlich and Xu.

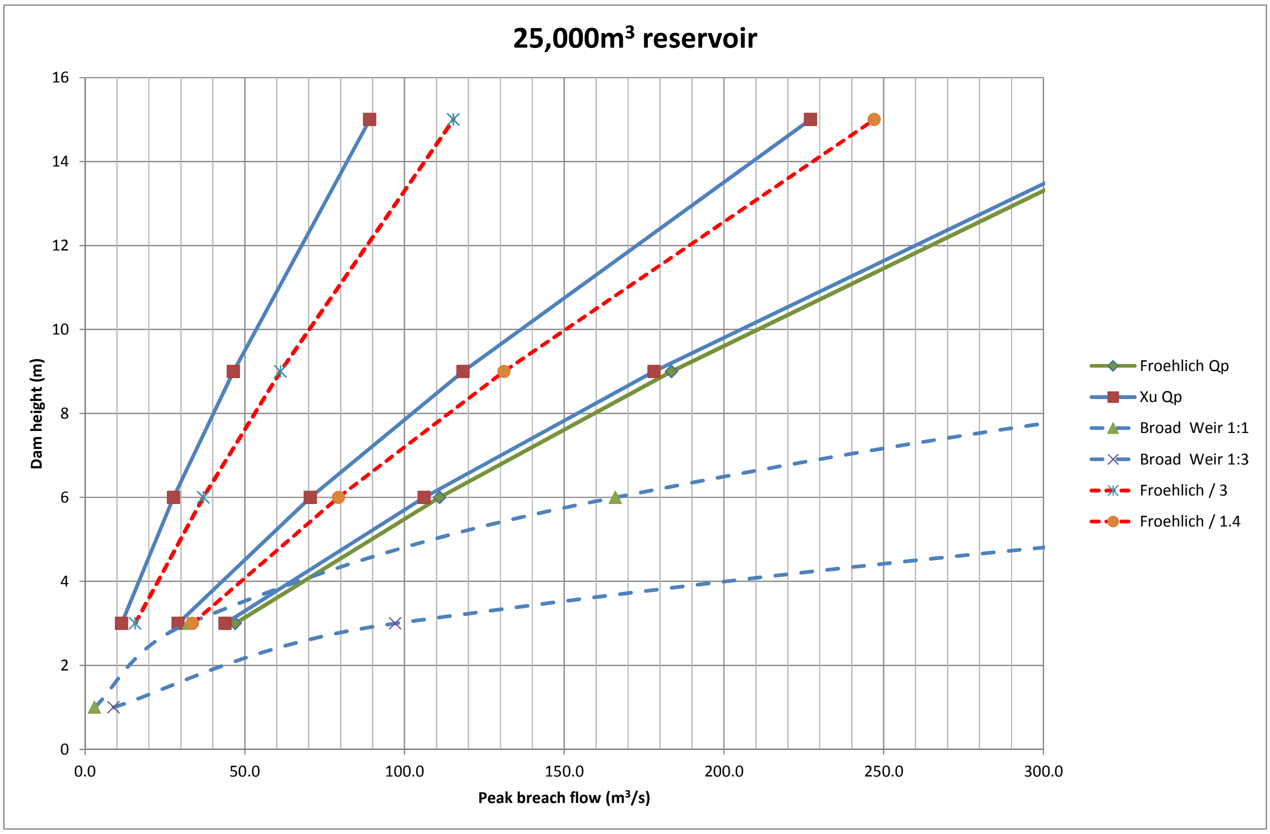

Figure 2.4: Comparison of regression equations and simple broad crested weir equation

Source: HR Wallingford

2.3.2. More detailed assessment of AREBA for breach prediction

Following on from the initial sensitivity study, a series of AREBA runs were carried out to determine an appropriate complexity of analysis and results for use. Variables that were considered at this stage included:

- 3 magnitudes of erodibility (typically high, medium, low soils)
- Overtopping failure
- Range of dam heights
- Range of reservoir volumes
- 3 different reservoir shapes (i.e. stage volume relationships)
- Seeking outputs which identified both Q_p and the hydrograph shape (i.e. Peaky or flat).

These analyses were undertaken and a set of results produced. Two issues were identified which then affected the final choice of approach for the risk assessment methodology:

1. Volume of data required and complexity of method (for a non-technical user)
2. Non failure breach scenarios.

2.3.3. Volume of data; complexity of method

When the approach was tested, it was considered that the volume of information produced, and hence the options needed to be considered as part of the analysis method, was too complex – given that the goal was a simple, relatively quick analysis method that could be performed by non-technical users.

2.3.4. Non failure scenarios

Runs undertaken using a range of erodibility considered applicable to the UK shows that certain reservoir combinations, and in particular small non-impounding reservoirs built of high plasticity clay, are unlikely to erode at a rate that would lead to a catastrophic failure.

However, as this project relates to government regulation of dam safety it has been agreed with Defra that the Guide would provide peak breach flows resulting from catastrophic failure for all dam sizes. This is in effect acknowledgment that at any individual dam there may be site specific failure modes that could lead to rapid failure (for example, physical damage by digger, aircraft impact etc.), and that to demonstrate beyond reasonable doubt that none could lead to catastrophic failure at a specific dam it would be necessary to carry out a full failure modes analysis for all credible failure modes. For the purposes of this project, catastrophic is defined as a time base for the breach hydrograph similar to that for a breach hydrograph defined using Froehlich (1995).

2.3.5. Concluding approach and summary of parameters used to derive guidance in main report

Based upon the issues detailed in Section 2.3.2 above, a simplified approach to breach analysis was adopted. This approach allowed for a conservative estimate of breach conditions and presented less options for analysis by the user, so making the analysis simpler to undertake. The proposed approach comprised failure analyses assuming:

- Highly erodible soil only;
- Initiation through a pipe (or hole) at the base of the dam;

- No inflow considered within the calculation, but conditions are assumed to be such that the reservoir level is full to the embankment crest, with any spillway or outflow blocked or closed.

These assumptions meant that:

- The dam always failed in a rapid manner. This then removes consideration of peaky or flat type hydrographs (all failures are peaky in nature) so simplifying the data used for risk assessment;
- There are no non failure scenarios;
- There are no varying assumptions regarding different inflows.

Failure by piping often results in a more extreme (higher peak) discharge than through overtopping. In this case, an initiation hole of 0.5m diameter was assumed reflecting a severe problem with the dam that could have arisen for a number of reasons. It was noted that the modelling parameters used predicted rapid failure of the pipe due to roof collapse (arising from the use of low strength – highly erodible material) and subsequent erosion through overtopping. This combination of parameters was intended to reflect a structure that would clearly fail quickly; closer analysis of the interactive effects of selecting different failure processes and different material strengths was not feasible within the scope of work, but the combination used should provide a conservative estimate of conditions.

A summary of the final parameters and assumptions used to create the breach data for the risk assessment is shown in Table 2.3.

Table 2.3: Variables used in dam break analysis

Parameter	Values used for sensitivity/ Guide	Comment
Dam height	2, 4, 8, and 16m	Defined as the height from dam crest to the flood plain i.e. includes freeboard.
Reservoir stored volumes	5,000, 15,000, 25,000 and 100,000m^3	
Reservoir shape	3 different stage volume relationships	Reservoirs constructed on flat, sloping ground, and across valleys.
Reservoir escapable volumes	Full to dam crest Freeboard 0.3m	Volume used to calculate hydrographs is greater than the stored volume figure since its allows for the reservoir to fill to crest level.
Soil erodibility (Kd):	A single high erodibility value of $K_d = 100\ cm^3.\ N^{-1}.s^{-1}$ A low value of critical shear stress was used (0.5 N.m^{-2})	
Failure mode	Internal erosion, initiated by a 0.5m diameter hole at the base of the dam.	Representative of problems with the dam sufficient to initiate catastrophic failure
Embankment geometry	The embankment geometry has been taken as having a side slope of 1V:2.5:H on both faces, with a crest width of 0.5H (hence crest widths of 1m, 2m, 4m and 8m respectively). Any surface protection measures are ignored.	The crest width is important in determining time to failure, but once catastrophic failure has occurred does not significantly change the peak breach discharge.

3. Predicting the local inundation area

The method for analysis of flood spreading is dependent upon whether your reservoir is constructed on flat ground, on sloping ground, or across a valley. This chapter presents the equations used for predicting flow conditions arising from flood water spreading on flat land and sloping ground.

3.1. Introduction

Flow conditions in the immediate vicinity of the dam are important because it is:

- Where risk to an individual (Test 1) is normally highest;
- A transition zone between high velocity flow from the breach, and flow which has spread and been steered by the local topography;
- Where flow may be affected by downstream obstructions and topography such that the raised tail water may affect flow through the breach.

Factors which are important in determining the velocity and direction of flow in this area include:

a. The location, and type of breach;
b. Slope of the ground;
c. Obstructions such as walls, property, vegetation etc.

3.2. Spreading equation

The Guide presents an equation for predicting flow conditions arising from flood water spreading across flat ground from a point source. This equation is obtained in a 2 step calculation:

- Simplification of the 1D shallow water equations into a non-linear diffusion equation by neglecting the acceleration terms and combining the continuity and momentum equations.
- Integration of the non-linear diffusion equation in a polar coordinate system assuming a radial steady state flow on a flat ground.

Although attractive in its simplicity there are a number of important simplifications, including that:

- It is a steady state solution, and so takes no account of the finite volume of the reservoir (and assumes a sink boundary condition at the downstream limit);
- It neglects momentum of flow through the breach;
- It is a point source and neglects the finite width of the breach;
- It assumes imaginary frictionless ‘glass’ walls at the edge of the flow (in reality there will be a boundary zone where flow depth and velocity taper off to zero);
- The conditions immediately downstream of the dam will affect how the flow spreads. Figure 3.1 shows that for a 45 degree angle of spread any occupied space more than 30m from a 25,000m^3 reservoir retained by a 4m high dam would be categorised as low risk in terms of threat to an individual (Test 1). This suggests that perhaps 100m is a reasonable limit at which local flow conditions are important, and risk becomes governed by the general width of flooding and thus total number of people at risk. Note, however, that this is a general observation, and there may always be site specific features which might

lead to high depth velocity values further down the valley – for example, a narrowing of the valley with property located adjacent to the river or stream.

A full solution would need to take all of these factors into account and so would be complex, negating the whole concept of a simplified screening tool. Figure 3.1 shows how depth velocity (dv) varies with distance for an angle of spread of 45 or 90 degrees. The guide suggests using an angle of spread of 45 degrees (widens by 1m for every 2.5m distance) on the basis that:

- A 45 degree spread results in total structural destruction (>7m^{2}s) at a distance of 15m from the dam (this is credible i.e. a 2 storey building 15m away from a 4m high dam surely is severely damaged) plus partial structural damage (3m^2/s) at up to 30m distance;
- If using a 90 degree spread there is no total structural destruction and the limit of partial damage is at 20m;
- 45 degrees is a precautionary approach which is more applicable at screening stage.

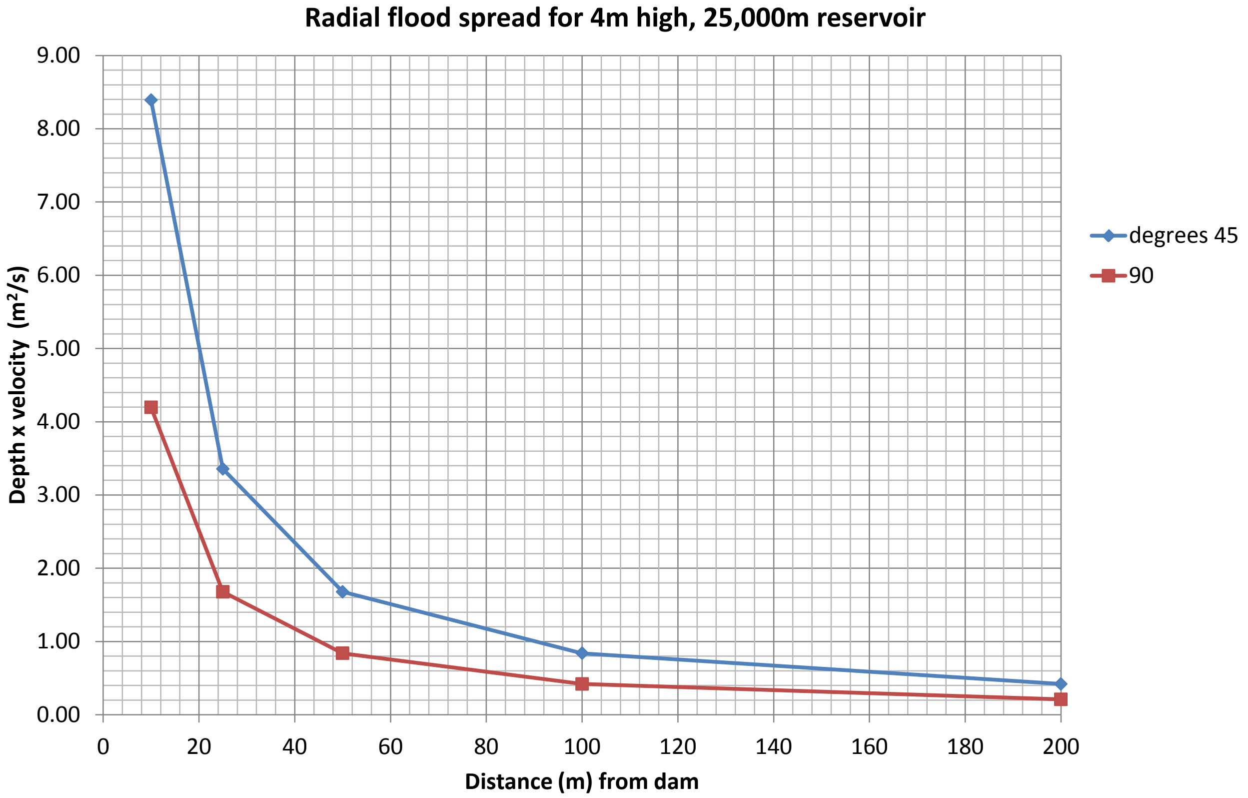

Figure 3.1: Depth x velocity versus distance for different types of radial spread

Source: HR Wallingford

To calculate the depth x the velocity using the radial method for flat land, we have developed and used the following equations:

$$d = \left(\frac{13}{3}\frac{n^2 {Q_P}^2}{\Omega^2}\right)^{3/13} r^{-3/13}$$

Equation 3.1: Depth (d)

Source: HR Wallingford

$$v = \sqrt{\frac{3}{13}}\, d^{7/6} n^{-1} r^{-1/2}$$

Equation 3.2: Velocity (v)

Source: HR Wallingford

As for radial spread, the angle of spread is the key assumption, noting that the approach adopted should be conservative (precautionary) and provide data relevant to the central section of the jet where risk to people is highest. The Guide thus suggests adoption of a spread of 1m for every 2.5m, with width at the dam of twice dam height (conservative in relation to typical breach widths which are normally greater than this).

Other rules of thumb considered but rejected as more complex include Design of small dams (USBR) guidance that supercritical flow should not contract or expand in width at an angle (α) greater than tan α=1/ 3F, where F is the Froude number

3.3. Estimating local flood impact conditions in other situations

In the situation where the dam has been constructed on sloping ground, or the breach flow path is constrained by topography or man-made obstructions such as hedges, walls and houses, it was decided that for the near field affects the flow width can be calculated assuming either a triangle shaped cross section on the flow path or a rectangular cross section on the flow path.

- For the triangular shaped cross section, with top width of water W and mid height depth of water d, the flow depth times velocity (dv) (which is a measure of the damage that the flow might do) can be calculated as:
  $dv = 2Q_p / W$
- Where the cross section of the flow path is more of a rectangular cross section, the flood damage potential can be calculated as:
  $dv = Q_p / W$
  This assumes a broadly rectangular shaped flood flow section, with flood water width W and depth of water d.

For those reservoirs within a valley is was decided that the user should go straight to using more detailed methodology such as the CIRIA method to calculate the impact conditions. Section 4 goes on to discuss the CIRIA methodology used for the far field effects.

4. Predicting the rate of flood attenuation and extent of valley inundation

4.1. Introduction

The challenge here is to identify an approach that allows the simple calculation of flood flow conditions along a valley, including for the attenuation of the flood flow as it progresses away from the dam. When you consider the number of variables needed to define the input, the flow system and the output it can be seen that at least ten variables are needed (as shown in Table 4.1). This number of variables is too great to be able to provide a quick and simple graphical method (which would probably only be viable for a maximum of 4 variables).

The CIRIA Report C542 rapid assessment method was taken as a starting point for a potential analysis method and a review of different simplified calculation methods for predicting such flow conditions was undertaken. Whilst a number of different solutions were found, the methods invariably required a more complex (as compared to the CIRIA method) analysis. As such it was finally concluded that the method presented in the CIRIA Report still offered an appropriate balance between accuracy of solution and complexity of calculation needed by the user. Since there has been a decade of experience in using the CIRIA method since publication, and the original method had not been validated, some testing and refinement of the method was implemented.

Table 4.1: Variables relevant to attenuation of a flood wave down a valley

Variable (input or output)	Sym.	Number	Range of values
Output - Flood hazard Q, velocity, flooded width and distance down the valley	Q, v, W , L	4	Ideally velocity would be presented as a range across the flooded width, so that both peak and point value at individual receptors (houses) were available
Inflow – peak flow and time base (or volume)	Q_o, T_h	2	Reservoir volume up to say 100,000m^3, but increased by volume of incoming flood
Valley cross section - base width and side slopes	W, S_s	2	Base width could vary from 5m up to 100m. Similarly side slopes could locally vary from steep (1H:1V) up to 20H;1V or flatter
Valley bed slope and roughness	n, S_o	2	Bed slope could vary from say 2% down to 0.01%
Total		10	

Experience in use of the CIRIA equations suggests that it under predicts the rate of attenuation, with Supplement No 1 to the Interim Guide (Brown & Gosden, 2006) suggesting that k be reduced until "L_a remains within the range of say generally 5 to 100km". The CIRIA methodology was therefore reviewed as outlined in the following sections.

4.2. CIRIA Routing method

4.2.1. General

The CIRIA routing equation is presented in Equation 4.1. Derivation of the equation can be found in Appendix 8 of CIRIA (2000), being derived from a convection-diffusion equation for flood routing.

The CIRIA method is compared with the RIM flood routing approach in Table 4.2. The CIRIA method is obviously significantly simplified, compared to a detailed 2D flood routing approach.

$$L_a = k\, W_T^{-0.2}\, S_o^{1.9}\, n^{-1.8}\, Q_p(0)^{0.2}\, T_h^{2}$$

$$Q_p(x) = Q_p(0)\exp(-x / L_a)$$

where:

k	Constant. Suggested value 2.5.
L_a	Length over which Q falls to 37% of its value at the start of the reach
n	Manning's roughness coefficient – as Table 5.2 in the main report
$Q_P(0)$	Discharge at upstream end of calculation
S_o	Valley bed slope (measured along the flood plain, not watercourse)
T_h	Time period at half discharge
W_T	Surface width of flooding
X	Distance from the considered zone upstream point

Equation 4.1: CIRIA Routing equation

Source: CIRIA (2000)

Table 4.2: Comparison of CIRIA valley routing method with RIM

Feature	Assumption made in RIM Phase 2	Assumption made in CIRIA, 2000
Topographic data	Lidar – spot levels with accuracy of ±0.25m, so significantly more accurate than CIRIA	Contours at 5m centres from 1:25,000 scale OS map
Roughness	Can vary over short distances, reflecting model nodes	Average in each reach (several km)
Mathematical model	Two dimensional	One dimension, simplified to trapezoid cross section

4.2.2. Comments on the use of the CIRIA method

It should be noted that the use of the CIRIA formula is not meant to be a single calculation giving a single value of L_a. See p213 Appendix 8 of the CIRIA report. "The change of L_a along the valley is most likely to occur because of the change in the hydrograph peak. It is recommended that the value of L_a is recalculated, if the peak flow rate changes, Q_p by more than about 10 per cent."

However as the peak discharge attenuates away from the dam, T_h increases and L_a becomes larger, hence progressively reducing the attenuation. Also important is the fact that x should be the length of the zone across which the calculation is made, not the chainage downstream of the dam (cf. CIRIA report, p53).

Q_p, T_h and L_a should be recalculated at regular intervals, rather than calculated once for the location of interest. This leads to a more realistic attenuation of the peak discharge.

Conversely, an attenuation length of 10m is only valid over a short distance, it needs to be revaluated for calculating the attenuation at 2km downstream of the dam for example. If L_a is not recalculated the peak discharge is reduced to almost zero after 100m with such a small value of L_a.

Table 4.3 shows the percentage of reduction in the peak discharge after a distance of 2km for three values of L_a.

Table 4.3: Effect of La on the reduction of Q_p

Attenuation length L_a (km)	Reduction of Q_p after 2 km
100	2%
10	18%
1	86%
0.1	100%
0.01	100%

4.2.3. Sensitivity

The CIRIA equation has been applied to a variety of reservoirs and valley slope to investigate the sensitivity to the various parameters, as shown in Figure 4.1 and Table 4.4.

The following comments can be made:

- Effect of slope:
  - L_a varies by two orders of magnitude for slope varying by one order of magnitude (Figure 4.1).
  - This is due to the exponent of 1.9 on the valley slope in the CIRIA formula. Although this is a significant variation, there is no evidence to modify the value of the exponent.
- Maximum value of L_a:
  - Values greater than 100km for large volumes reservoirs with small height.
  - Such large values of L_a can appear excessive, but as explained in section 4.2.2, L_a should not be mistaken for a characteristic inundation length. Large values of L_a indicate that the attenuation of the peak discharge is very small.
- Minimum value of L_a:
  - Values lower than 10m for small reservoirs on slope of 1 in 1000.
  - These values are only applicable over a very small distance (a few meters), as the very high attenuation of the peak discharge means that L_a should be recalculated. L_a will naturally increase away from the dam as the hydrograph flattens and T_h increases.
- Localised storage:
  - The CIRIA method makes no account for losses into temporary storage behind road embankments , hedges etc. as the flood wave travels down the valley.

- Volume losses that result in a reduction of the volume of the hydrograph cannot be represented. A detailed study, ideally with 2D modelling, would be necessary to represent these localised processes.
- Energy losses generated by dissipation (eddies) in these localised storage can be represented by increasing the friction coefficient n (thereby reducing L_a).

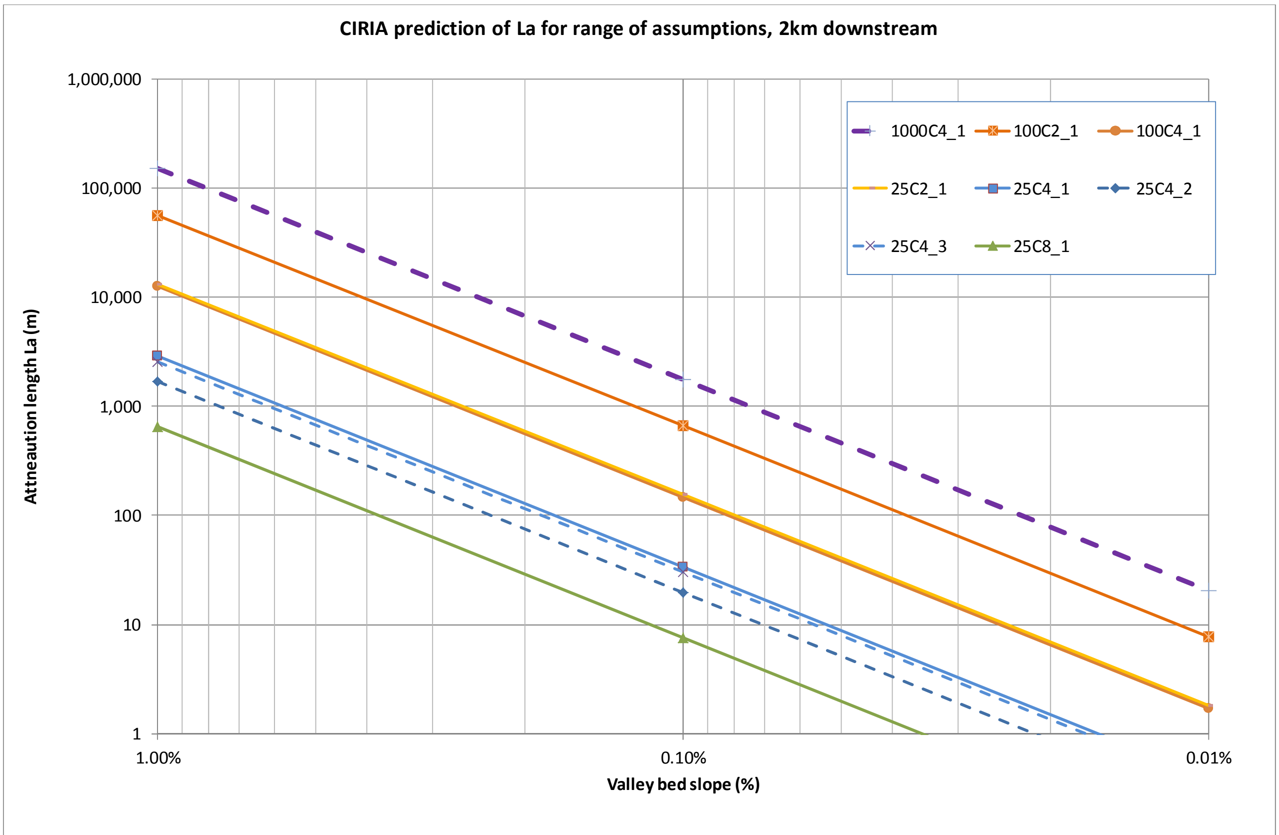

Figure 4.1: CIRIA prediction of L_a for range of assumptions, 2km downstream

Table 4.4: Detail of sensitivity cases shown on Figure 4.1

Run name				Details of content in subset			
Escapable volume (1000 m^3)	Valley shape	Dam Height (m)	Subset	Base width / side slope	Manning n	Q_p (m^3/s)	T_h (s)
25	C	2	1	10/10	0.075	60	525
25	C	4	1	10/10	0.075	106	235
25	C	4	2	10/10	1.000	106	235
25	C	4	3	30/20	0.075	106	235
25	C	8	1	10/10	1.000	206	107
100	C	2	1	10/10	0.075	118	1,031
100	C	4	1	10/10	0.075	210	467
1,000	C	4	1	10/10	0.075	1,396	1,433

4.3. Comparison with the Ponce method

Ponce et al. (2003) provide an analytical method for estimating peak discharge attenuation along a channel of rectangular section. In this method, the perturbation form of the gradually varied unsteady flow equations are derived first, and this is solved with the assumption of a solution having a sinusoidal form. Eventually the discharge is expressed in a similar way to the CIRIA method as:

$Q_p(x) = Q_p(0) \exp(-\alpha X / L_0)$

where:

- X distance along the river reach from the dam site
- L_0 reference channel length (length in which the steady equilibrium flow drops a head equal to its depth)
- α discharge attenuation factor (the expression for calculating α relies on many intermediate variables, it is too long and complex to be shown here).

Therefore we have $L_a^{Ponce} = L_0 / \alpha$.

This method is more complex to use than the CIRIA method, but it has provided data for comparison with the CIRIA method (Table 4.5). The main observations are:

- For average to steep slope (0.001 to 0.01) there is roughly a multiplying coefficient 3 between the attenuation length calculated by the two approaches (the CIRIA attenuation length being greater). The coefficient k could be reduced so the CIRIA method gives similar output to the Ponce method.
- However for the mild slope (0.0001) the interpretation is not clear since the Ponce method and the CIRIA method behave very differently (non monotonic variation of La when Q_p decreases with the Ponce method).

Although there is not enough evidence here to draw firm conclusions, this comparison exercise seems to indicate that in the majority of cases the CIRIA and Ponce methods give consistent values of L_a. This also seem to show that the k coefficient in the CIRIA method could be reduced to make the discrepancy between the two methods smaller.

Table 4.5: Comparison of L_a (m) for different values of slope and hydrograph shape

Slope S_0 (m/m)	Hydrograph duration T (h)	Peak breach discharge $Q_p(x=0)$	Reservoir volume V_w (m^3)	L_a^{Ponce} (m)	L_a^{CIRIA} (m)	ratio L_a
0.01	0.75	20	27,000	65,325	182,250	2.79
0.01	1.5	10	27,000	224,675	634,631	2.82
0.01	3	5	27,000	773,998	2,209,915	2.86
0.01	6	2.5	27,000	2,670,306	7,695,370	2.88
0.001	0.75	20	27,000	708	2,294	3.24
0.001	1.5	10	27,000	2,608	7,990	3.06
0.001	3	5	27,000	9,151	27,821	3.04
0.001	6	2.5	27,000	31,875	96,879	3.04
0.0001	0.75	20	27,000	256	29	0.11

Slope S_0 (m/m)	Hydrograph duration T (h)	Peak breach discharge $Q_p(x=0)$	Reservoir volume V_w (m^3)	L_a^{Ponce} (m)	L_a^{CIRIA} (m)	ratio L_a
0.0001	1.5	10	27,000	147	101	0.68
0.0001	3	5	27,000	86	350	4.05
0.0001	6	2.5	27,000	374	1,220	3.26
0.01	0.75	40	54,000	76,136	209,350	2.75
0.01	1.5	20	54,000	261,334	729,000	2.79
0.01	3	10	54,000	898,723	2,538,525	2.82
0.01	6	5	54,000	3,096,005	8,839,659	2.86
0.001	0.75	40	54,000	784	2,636	3.36
0.001	1.5	20	54,000	2,985	9,178	3.07
0.001	3	10	54,000	10,516	31,958	3.04
0.001	6	5	54,000	36,650	111,285	3.04
0.0001	0.75	40	54,000	447	33	0.07
0.0001	1.5	20	54,000	256	116	0.45
0.0001	3	10	54,000	149	402	2.71
0.0001	6	5	54,000	413	1,401	3.39
0.01	0.75	80	108,000	88,953	240,480	2.70
0.01	1.5	40	108,000	304,601	837,401	2.75
0.01	3	20	108,000	1,045,373	2,916,000	2.79
0.01	6	10	108,000	3,594,916	10,154,102	2.82
0.001	0.75	80	108,000	845	3,027	3.58
0.001	1.5	40	108,000	3,406	10,542	3.10
0.001	3	20	108,000	12,081	36,710	3.04
0.001	6	10	108,000	42,144	127,833	3.03
0.0001	0.75	80	108,000	780	38	0.05
0.0001	1.5	40	108,000	447	133	0.30
0.0001	3	20	108,000	258	462	1.79
0.0001	6	10	108,000	443	1,609	3.63

Source: HR Wallingford

4.4. Comparison with InfoWorks RS

A limited number of InfoWorks RS (1D) model runs were performed to assess the rate of attenuation with two very different synthetic hydrographs. A rectangular channel of constant slope, 10m wide and 12km long, has been used. The two hydrographs, a "peaky" one and a "flat" one, described in Figure 4.2, are the upstream boundary condition in the channel. The downstream boundary condition is defined as normal depth condition.

The peak discharge along the channel computed by InfoWorks has been plotted on the first 5km of the channel (Figure 4.3). The peak discharge calculated with the CIRIA method has been plotted on the same figure. It should be noted that the attenuation length L_a has been recalculated at regular intervals along the channel. There are no clear conclusions coming out of this comparison. In some configurations (peaky 1:1,000, flat 1:100, flat 1:1,000), the peak discharge curves are close, in the other configurations (peaky 1:100, peaky 1:10,000, flat 1:10,000) they are very different. Except for the peaky hydrograph on the steep slope, the peak discharge is attenuating more with the CIRIA method than with the InfoWorks computation.

Limitations of the analysis are:

- a rectangular channel 10m wide is used, rather than a wider valley section
- the relatively low breach flows used are associated with medium and low erodibility soils in this study.

The discrepancies between the attenuation rate in CIRIA and InfoWorks are significant in some cases, this analysis should be completed to be able to draw clear conclusions.

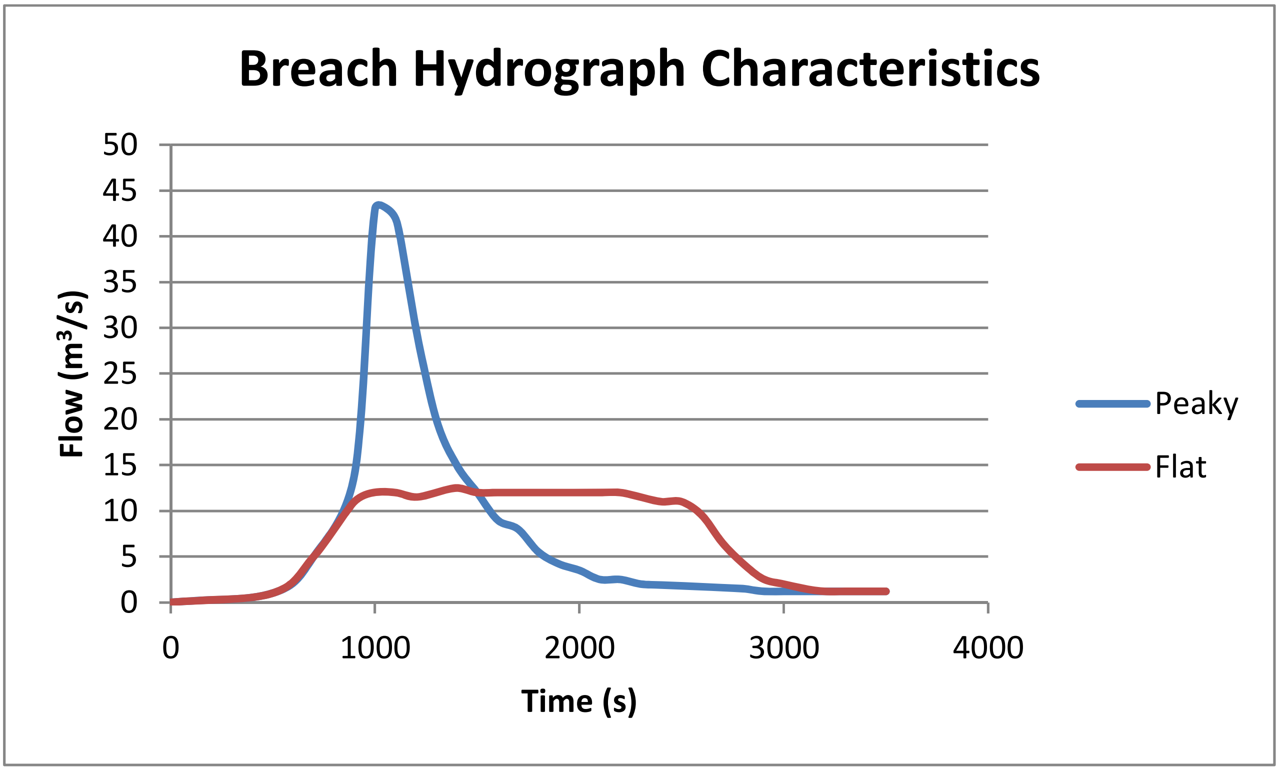

Figure 4.2: Synthetic hydrograph used in the InfoWorks runs

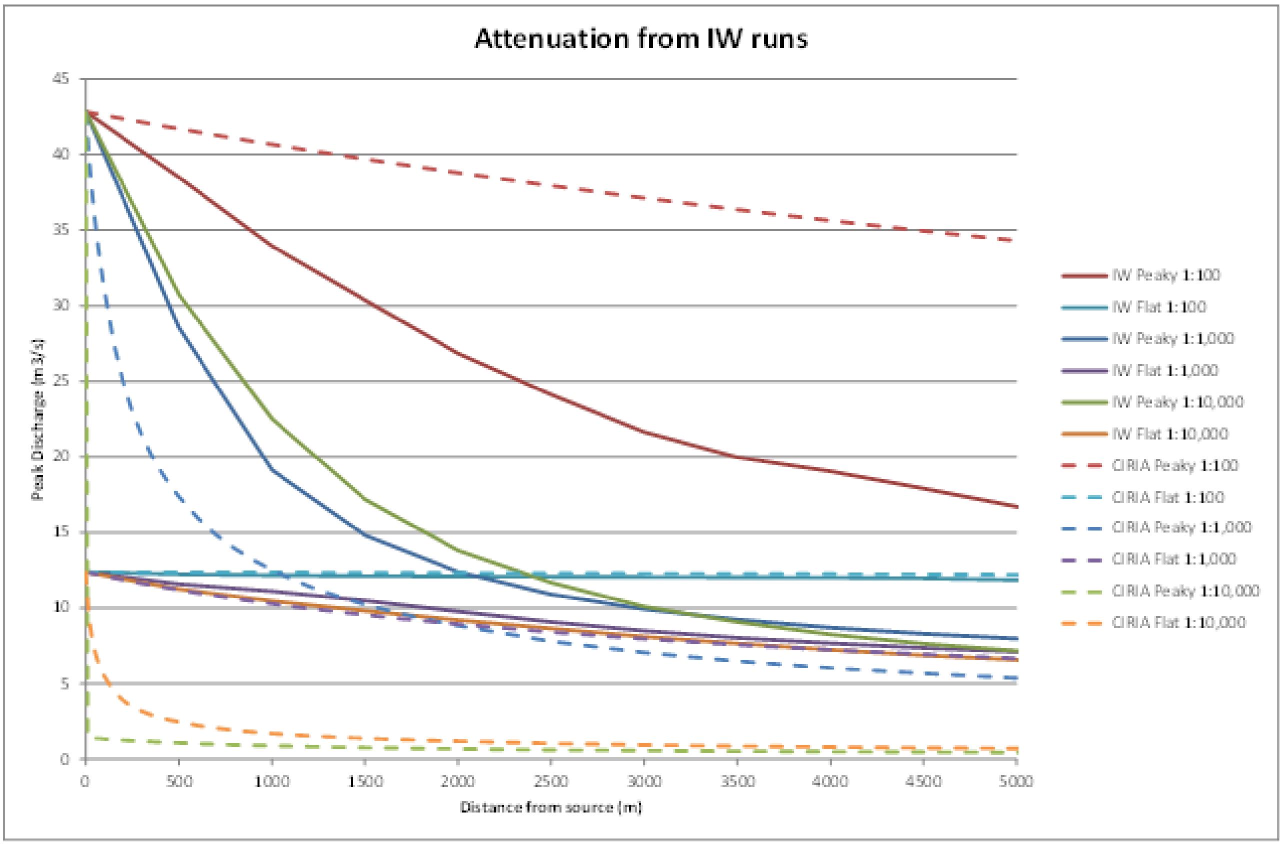

Figure 4.3: Comparison of peak discharge attenuation from CIRIA and InfoWorks

4.5. Comparison with the RIM method

An additional comparison is to summarise the output from detailed dam break carried out as part of the RIM project to provide the range of extent of flooding, and distance to where hazard to people drops below defined value(s). The following summaries of detailed flood routing are available.

- A sample of the recent RIM mapping, provided by Mott MacDonald as an example of the range of outputs.
- BDS paper with a review of 35 dambreak studies using DAMBRK – Tarrant et al, 1994 Inundation mapping for dam failure – lessons learnt from UK experience.

The RIM sample comprised 60 dams, with a spread of height and volume as shown in Figure 4.4 (maximum height was 10m). The data was extracted from the flood maps of risk to people. For the purposes of this project reservoirs with a capacity greater than 200,000m^3 were discarded, so the following plots of extent of damage and flooding are on a sub-sample of 52, which have the range of parameters summarised below and shown graphically in Figure 4.5.

Table 4.6: Distance at which dambreak returns to watercourse channel

	Dam Height (m)	Dam Volume (m^3)	Distance (km) at which hazard (D(v+0.5)+DF falls to 2.0	Distance (km) at which dambreak returns to watercourse channel
Max	9.2	200,000	28.3 (6.3 if neglect highest 2)	28.3 (19.4 if neglect highest)
Median	4.4	68,095	1.6	4.8
Min	1.0	25,000	0.0	0.4

The maps produced under RIM showed hazard to people in terms of the classes of hazard to people as defined in FD2321 (i.e. depth (velocity + 0.5) + Debris factor", although the maximum adopted was 2.0, being the value defined in the earlier FC2320, rather than damage to property (dv). Thus the maps do not show the extent of structural damage where dv > $3m^2/s$.

It would be possible to extract the data on distance to where dv fell below 3 m^2/s this, but this would require extract from the GIS files, would be much more time-consuming and is outside the current scope of this project.

Additional plots have been constructed using the RIM sample of 60 dams

- Figure 4.6: Reservoir volume vs. distance to end of dambreak
- Figure 4.7: Reservoir volume vs. hazard distance
- Figure 4.8: Dam height vs. distance to end of dambreak
- Figure 4.9: Dam height vs. hazard distance.

The Hazard has been calculated as defined in FD2321 (depth x (velocity +0.5) + debris factor) = 2.0. The Limit of partial structural damage is derived from Tarrant et al, 1994 (median and upper bound).

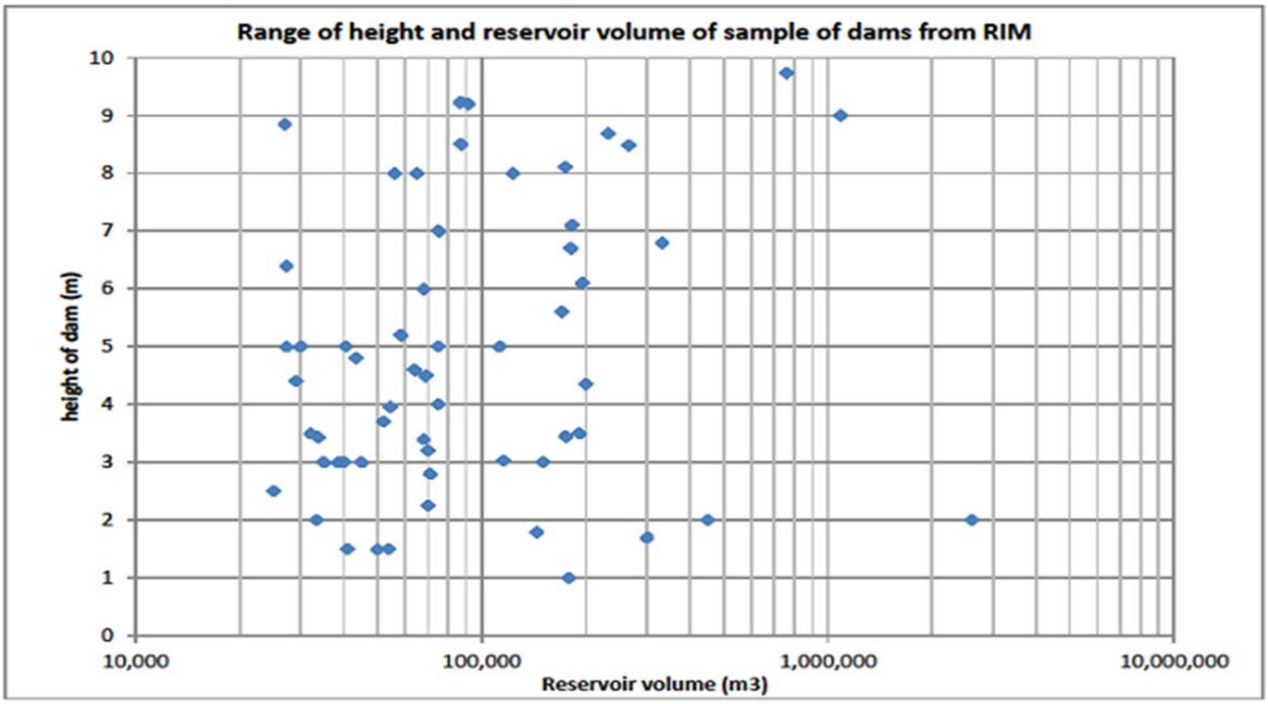

Figure 4.4: Range of dam height and reservoir volume in RIM sample

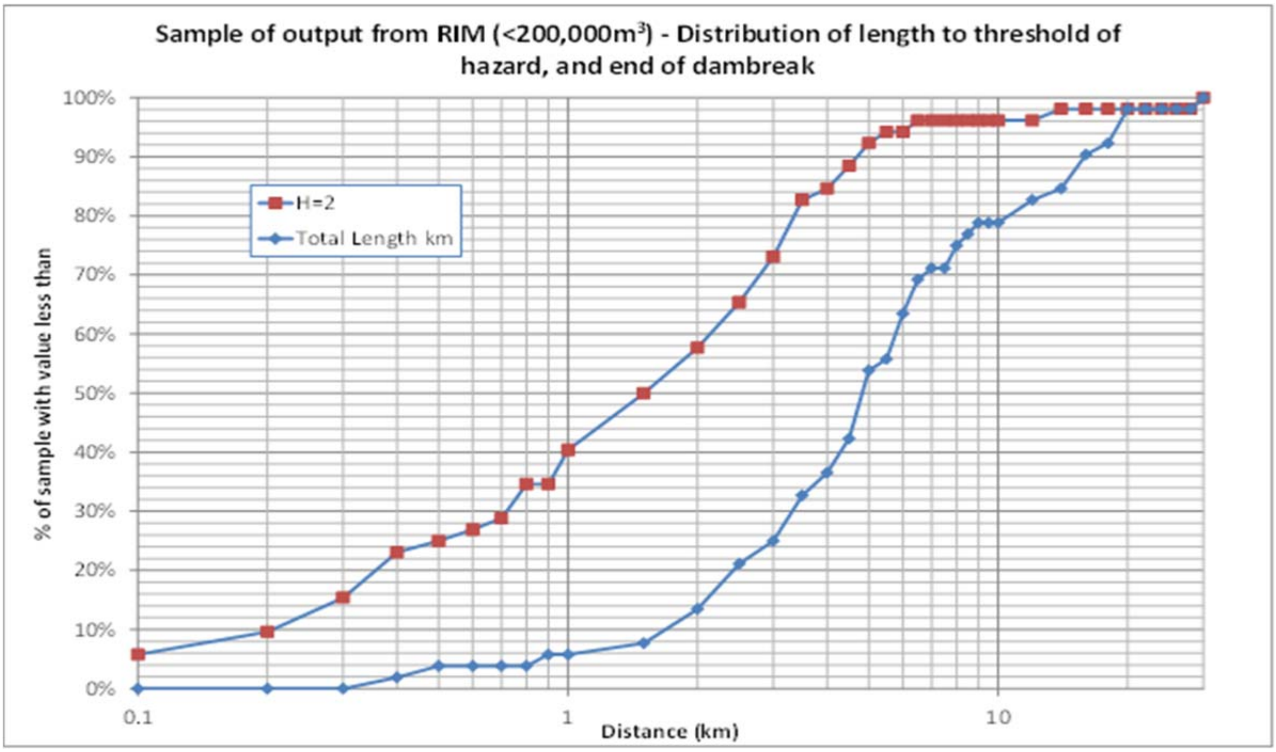

Figure 4.5: Sample of output from RIIM (<200,000m^3) – Distribution of length to threshold of hazard, and end of dambreak

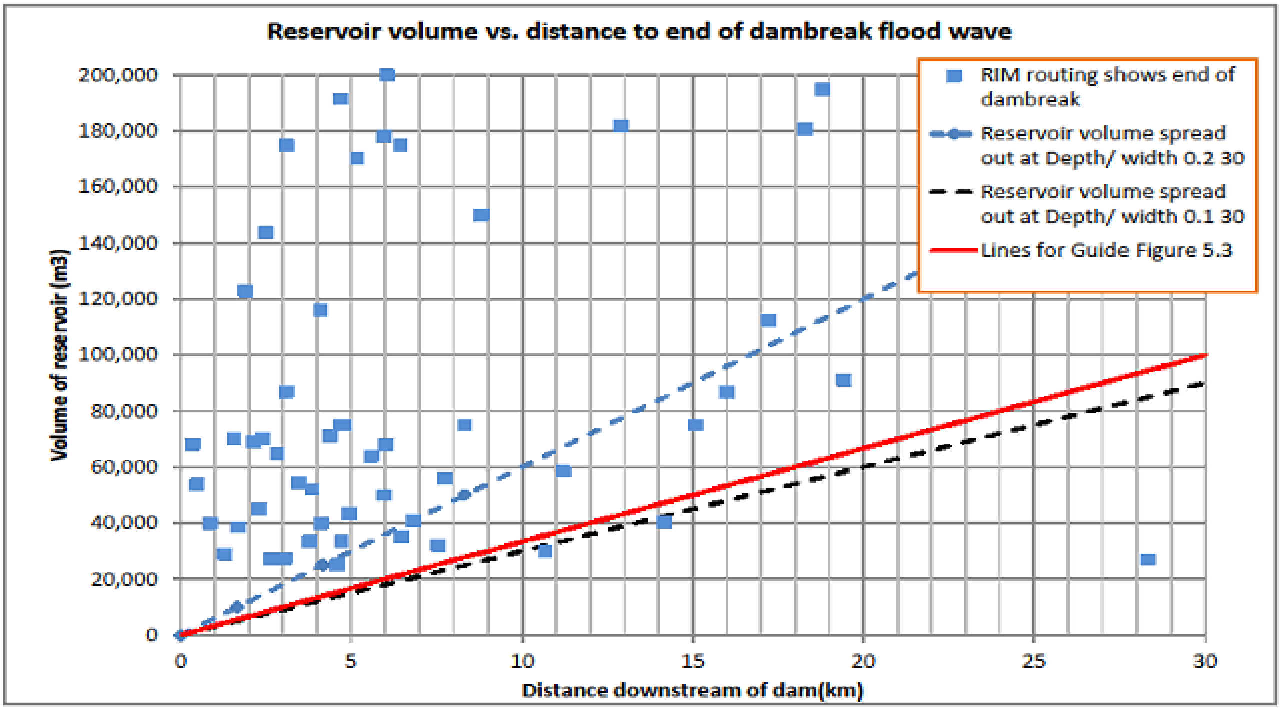

Figure 4.6: Reservoir volume vs. distance to end of dambreak

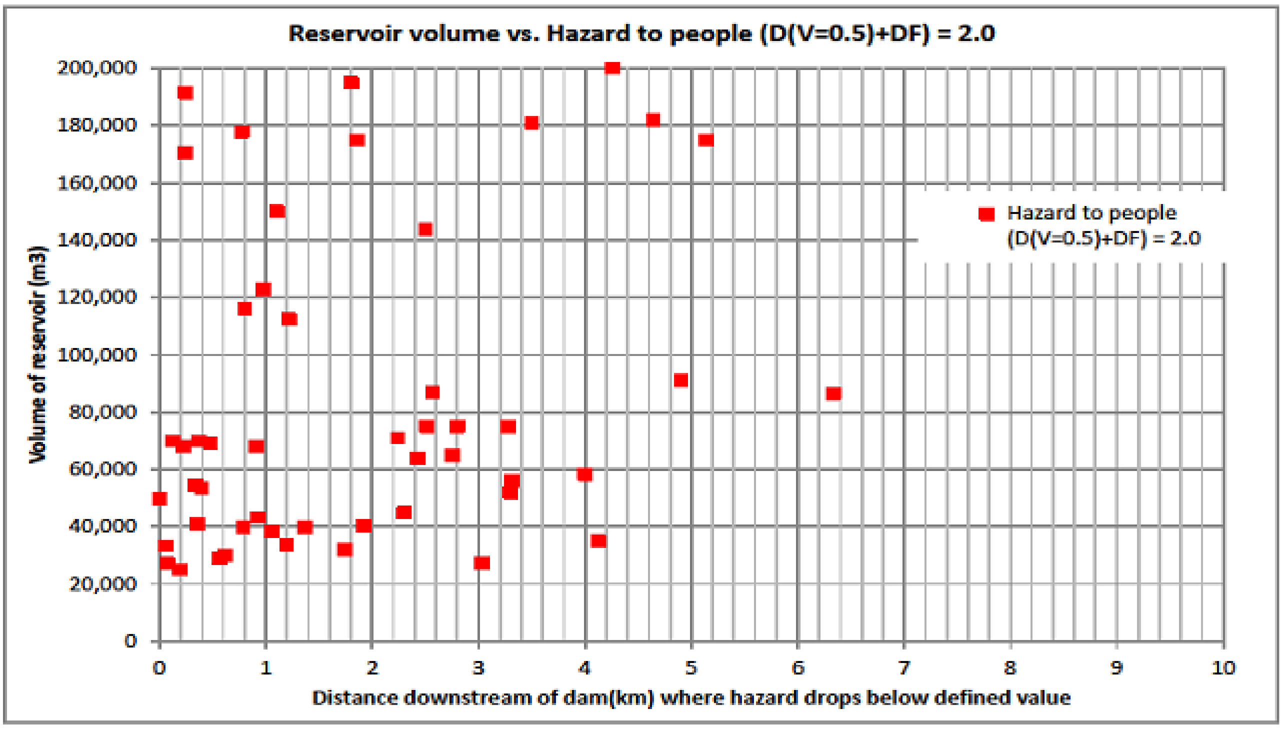

Figure 4.7: Reservoir volume vs. hazard distance

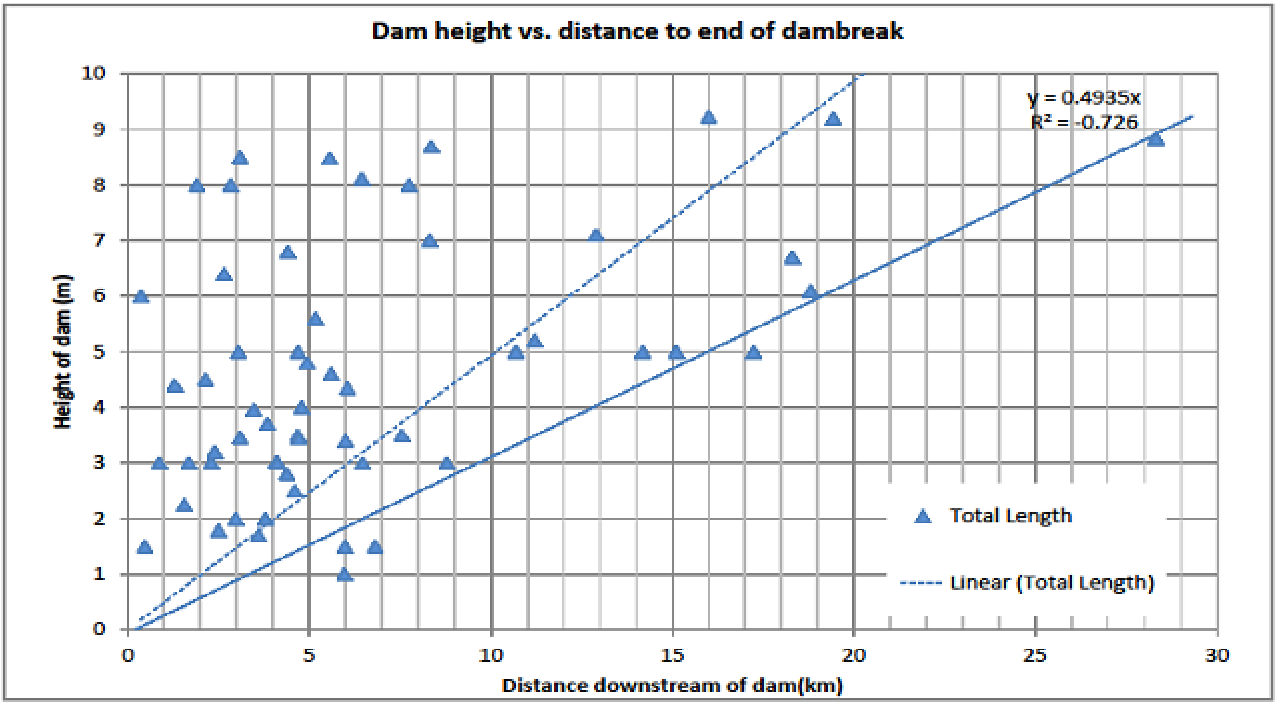

Figure 4.8: Dam height vs. distance to end of dambreak

Figure 4.9: Dam height vs. hazard distance

4.6. Conclusions

4.6.1. The existing routing method

The CIRIA routing equation suggests that the key variables are bed slope, Manning's n and the time period of the flood wave (i.e. it is these that have an exponent of around 2, compared to other variables which have much smaller exponents). The sensitivity study shows that as valley slope has potentially a larger range of credible values than Manning's n it is valley slope that has the biggest influence on L_a.

An issue with the CIRIA equation is that it does not account for losses into temporary flood storage, hence underestimating the attenuation of the peak discharge. However, this can only be accounted properly by a detailed, ideally 2D, modelling study. It should be noted that accounting for losses would increase the attenuation, i.e. reduce the attenuation length. It is appropriate that the CIRIA method, as a simplified approach, is more conservative.

Comparison with Ponce et al (2003) suggests that the CIRIA method over predicts the attenuation length by a factor of 3 in most situations, i.e. the CIRIA method is more conservative as it gives less attenuation of the peak discharge. Comparison with results from a few simulations with InfoWorks RS were inconclusive.

The following observations are made on the RIM sample of 60 dams:

- There is a trend of distance to a given level of damage (and end of the dambreak) increasing for larger reservoir volumes/ higher dams, although there is considerable scatter as would be expected (Figure 4.1 suggests two orders of magnitude difference depending on valley slope, and potentially greater if changes in valley cross-section and roughness are considered).
- As a screening tool it is necessary to be conservative, thus an envelope is drawn to reservoir volume vs. distance to end of the dambreak on Figure 4.6 for purposes of the main report.
- In terms of application to the Guide, it is considered simpler to go direct to a suggested check value of La (which can be assessed for physical credibility), than adjusting k.

4.6.2. Proposed refinements

If or when further refinement of the CIRIA equation is promoted as a project, the following should be noted in relation to its scope:

- With ten variables affecting the routing (see Table 4.1), it would be a major exercise to carry out sufficient detailed analysis to try and compare the CIRIA equation with output from a 2D mathematical model (presumably simplified to an idealised uniform trapezoidal channel).
- It would be further limited by the fact that real life dam break would be affected by temporary flood storage behind small obstructions across the flood plain, thus effectively both increasing roughness but also and more importantly reducing the volume of the main flood wave as it travelled down the valley.
- An alternative, and probably more fruitful approach would be an assessment of the Phase 2 RIM analysis (say 800+dams ?) to establish whether curves of best fit and upper bound could be established for sub-groups, such as upland reservoirs (likely to have steeper valley slope), reservoir size etc. To make this work you would need to sub divide the RIM dams into different categories which is simply breaking down the variables (volume, height, valley slope, valley shape etc.).

In the light of the various reviews described above, the following refinements to the CIRIA (2000) methodology are proposed (Table 4.7).

Table 4.7: Summary of proposed refinements to CIRIA methodology, as included in the Small reservoirs simplified risk assessment methodology – Guidance Report

	Refinement	Comment
1	Valley profile approximated by trapezoid rather than V shape	Many valleys (especially where glaciated) have a trapezoidal shape. Where the valley is V shaped, W can be set to zero. This refinement was included in the Interim Guide (2004).
2	Roughness	No change
3	Check L_a, if greater than Figure 5.3 of the Guide (Figure 4.7 of this report) reduce to that value	This is so that the dambreak cannot continue indefinitely
	Potential Refinements not pursued in this project	**Comment**
4	Reduce Factor K on rate of attenuation from 2.5 to 1.0	Supplement No 1 to the Interim Guide (2006) identified this was a problem. Ponce suggests a factor of 3 different from CIRIA
5	Correction for losses into temporary storage	Not included at this stage, as insufficient data to provide guidance. Rely on reducing k and increasing the friction coefficient
6	Adjustment to L_a for maximum extent of level of damage	Would increase complexity of method; not justified in screening process for farmers and other non-technical users

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