Loyne Dam - Stability Review based on a QRA, Event Tree Approach

DR PETER J MASON, MWH Ltd, High Wycombe, UK

SYNOPSIS. Quantitative Risk Assessment (QRA) methods are being increasingly used to assess the safety of UK dams, but the emphasis to date has been on the erosion of fill dams. The author has used the approach followed in the recent USBR Unified method for assessing such risks and has adapted them for assessing the stability of concrete gravity sections at Loyne dam. The results are compared to Persons at Risk (PAR) from potential flood releases and are demonstrated to fall below the ALARP region, indicating acceptability.

INTRODUCTION

The Loyne and Cluanie Dams were inspected under the UK Reservoirs Act by the author in July 2005. The inspection included a basic stability assessment assuming linear behaviour and commonly used parameters and material properties. Without further information to better define material parameters this approach indicated the stability of Loyne Dam to be marginal. This also mirrored the findings of an earlier Halcrow Ltd report which reviewed the stability of a number of Scottish & Southern Energy (SSE) concrete gravity dams.

One of the key factors in determining acceptable stability proved to be establishing the condition of the internal concrete lift joints of the dam and whether the joints could be considered as having sufficient tensile strength to resist failure by uplift and toppling. Cored drilling was considered but it was decided that such sampling would be too limited and location specific to be taken as representative of the dam as a whole. A risk assessment approach was therefore adopted.

In recent years Quantitative Risk Assessment (QRA) procedures have been developed as a means of assessing the failure of embankment dams due to a number of factors, such as internal erosion and piping. A number of different approaches have been developed for such QRA procedures. A very recent one is the "*Unified Method for Estimating Probabilities of*

Managing Dams: Challenges in a Time of Change. Thomas Telford, London, 2010

Failure of Embankment Dams by Internal Erosion and Piping", *version Delta Issue 2, August 2008, USBR, US Army Corps of Engineers, University of New South Wales and URS.*

While the above procedures were not specifically developed for concrete dams, they are based around a logical event tree approach for reviewing the events necessary for failure to occur and the likely probabilities of those events occurring in any specific failure mode. The author has used these methods in workshops facilitated by one of the USBR method's authors and so considered adapting the approach for the particular failure mode(s) envisaged at Loyne Dam. The approach used at Loyne was also reviewed by the USBR method's author before proceeding with the assessment.

Once the basic methodology had been confirmed a meeting was held in which the author and another All-Reservoir Panel Engineer with extensive experience of Scottish concrete dams reviewed:-

- the failure modes to be considered for Loyne Dam
- the associated events which would have to take place for failure to occur
- likelihood factors for each of those events.

It should be noted that the values in the last item above were assessed jointly by both Engineers based on discussion, reviews of associated documents including photographs of the original construction and on their collective experience of concrete gravity dams, including many operated by S&SE. The reasons for decisions and values were recorded as required by the USBR method and this paper includes the results of some of those discussions by way of example. Aspects such as stability calculations and reservoir level probability assessments are covered only briefly in the following sections as they are not the prime purpose of the paper.

FAILURE MODE ANALYSIS - DISCUSSION

Earlier calculations had indicated that for certain load cases Loyne dam does not meet normally required safety factors in terms of sliding stability on some lift joints. Such analyses tend to use an arbitrary friction (Ø) angle such as 45° whereas in the absence of excessive confining stress, the friction angle on a rough concrete crack or surface may be nearer 54°. Similarly where confining stresses are significant, apparent cohesion will develop due to the need for asperities to shear before sliding movement can occur. Therefore, although sliding safety factors do not always meet conventional levels this is not necessarily seen as the most probable failure mode. It can also be noted that while internal failures of masonry dam bodies have occurred, such failures are generally absent in the case of concrete gravity dams other than through the foundations.

Calculations showed that the failure mode giving most concern was the combined case of high assumed reservoir levels with lift joints being fully pressurized due to the internal drainage system being ineffective.

The joint review concluded that, should overturning occur, it would be progressive. Any initiation of overturning and toppling would increase uplift forces further reducing sliding friction capacity and causing the affected section of dam to dislocate downstream slightly. The toppling section might re-seat itself; however, the process would begin again with a now reduced overturning safety factor as well as a reduced contact area to resist sliding. Thus a progressive mixture of toppling and sliding would occur until it was eventually displaced completely. It was also noted that any given 2D slice would be retained in shear and torsion by the sections immediately adjacent to it. Therefore the failure would have to occur over a sufficient length of dam for the effects of such lateral support to be minimal.

EVENT TREE

For failure to occur in the manner described above it was considered that the following events would need to occur:-

(a) reservoir levels would need to be sufficiently high

(b) a flawed lift joint(s) would need to exist

(c) the flawed lift joint would need to be capable of pressurization

(d) the pressurized lift joint would need to be sufficiently open to permit flow or seepage

(e) the internal drainage relief would need to be incapable of providing sufficient relief, due either to partial blockage or a generally insufficient capacity

(f) a sufficient length of dam would need to be affected for rotation to take place and not be prevented by lateral (side) restraint

(g) there would need to be the lack of ability to intervene to prevent the above occurring

(h) the above would need to combine to cause the toppling/sliding failure mode

This closely mirrors the similar number and sequence used in the USBR approach for erosion at fill dams, but of course with different mechanisms. In addition to these events, discussions also took place to establish most likely bond stresses, or tensile stress capacity, available at the lift joints at Loyne dam and also the likely percentage of area, over any given lift, where this capacity could be assumed to apply.

The likelihoods of the events above were discussed and judgment was used, as described earlier, to assign general descriptors to each event. The descriptors used and the associated probabilities were taken from a similar QRA study elsewhere as well as being reviewed by the UBS method author mentioned earlier and are given in Table 1.

Table 1. Event Likelihood Descriptors and Associated Estimated Factors

Likelihood	**Likelihood Factor**
Virtually Certain	0.999
Highly Probable	0.99
Very Probable	0.9
Probable	0.7
Neutral	0.5
Possible	0.3
Unlikely	0.1
Very Unlikely	0.05
Highly Unlikely	0.01
Virtually Impossible	0.001

EVENT LIKELIHOODS – DAM LIFT JOINTS AND FOUNDATIONS

It was considered that there could be three broad types of internal concrete lift joint failure location at Loyne:-

- Non-overspill crest blocks adjacent to the spillway, where one end of the block is unrestrained.
- Crest blocks elsewhere where failure is likely to disrupt adjacent blocks.
- Abutment and mid-level blocks where a number of blocks to the side and above the block in question would also have to yield for failure to occur.

For brevity only the results of assessing probabilities for (b), (c) and (d) are given as examples of the type of review and record needed. However, the results for all events are summarized in Tables 2, 3 and 4. It should be noted that at this stage the probability of (a), relating to reservoir rise, was put at unity with actual probable values assessed later. Similar probabilities were assessed for likely failure of the dam at the concrete/rock foundation level. The results for this are summarized in Table 5.

The probability of a flawed joint existing (b)

It was noted that original construction took place with full awareness of the importance of forming good quality lift joints. The record drawings and photographs indicated lift joints with steps and aligned so as not to coincide with joints in the upstream pre-cast facings. It was also considered that adequate liaison took place at that time between designers and those on site, such that the importance of good lift preparation would have been known. Nevertheless, it was also acknowledged that there would have been times of bad weather including both wet conditions and/or frozen conditions and therefore that some flaws are likely to be present. It was concluded that the probability of horizontal lift joint flaws existing somewhere in the structure as a whole was **Very Probable**. It was also concluded that the probability of horizontal lift joint flaws being present at any given location was **Possible**.

The Probability of the lift joints being able to pressurise (uplift relief being ineffective) (c)

It was noted that the internal drainage relief system had been carefully designed with vertical 150mm diameter riser pipes at approximately 5m centres (three per block) and connecting on each lift with lateral half-round drains. It was considered that sufficient awareness of the system's importance, coupled with probably good liaison between the designers and site, would have ensured reasonable construction care. Indeed the description of the works issued on completion included a description of the internal drainage system. It was noted that the system had good interconnections with built-in redundancy should blockage occur of any one drain. Furthermore the use of Trief (blast furnace slag) cement would have minimized calcite deposition and hence the likelihood of blockage. It was noted that S&SE water-jet the lower relief drains every few years to try and keep them clear and that the drainage system is continuous from the foundations through to the dam crest at all points. It was concluded that the probability of the lift joints being able to pressurize through the uplift relief system being ineffective was **Unlikely**.

The probability of lift joints being able to open and permit flow when the surrounding concrete zones are in tension (d)

It was noted that the internal horizontal lift joints have been specifically located so as not to coincide with the external horizontal joints between the upstream facing panels. In addition it was noted that these facing panels incorporate rear lifting and locating blocks which are embedded in the dam mass concrete and which includes some reinforcement which will pass across the horizontal dam lift joints. The in-situ concrete in these upstream areas is also richer than the internal hearting concrete; nevertheless there may have been some increased difficulty in ensuring full compaction around the rear blocks. It was also noted that the physical distance between the reservoir and the drains in these locations is relatively small at little more than 1m. It was concluded that when imposed loads produced tensile conditions, the probability of horizontal lift joints being able to open and permit flow was **Unlikely to Possible**.

Supplementary issues - Concrete Parameters and Lift Joint Characteristics

A review of core results from Loyne dam some 50 years after construction, indicated mean concrete densities at Loyne to be 2.350 tonnes/m^3, however, test strengths indicated considerable variability. Results from extensive coring and testing of such joints at a number of US mass concrete dams is given in the report, "*Uplift Pressures, Shear Strengths and Tensile Strengths for Stability of Concrete Gravity Dams" Vol.1, Stone & Webster, Aug 1992*. These indicated a mean tensile capacity of 1.2 MPa, with 90% of results exceeding 0.49 MPa. Results came from a range of dams built between 1918 and 1991, but averaging 1967.

The percentage of any lift joint that can be assumed to have remained intact and not de-bonded varies and conservative values of 50% to 67% are often used. However the full depth coring of another, older, dam in Scotland found the rate of intact lift joints to be nearer 80% and so in the case of Loyne a figure of 67% was adopted. Combining probable tensile strengths with percentage of effective contact area the following effective values over 100% of the lift joint area became:-

Effective mean tensile capacity = 1.20 x 0.67 = 0.80 MPa = 78 t/m^2

Effective 10% tensile capacity = 0.49 x 0.67 = 0.33 MPa = 32 t/m^2

Foundation Level

Similar reviews to those described above were held for the concrete/rock contact at foundation level. Photographs of the original construction were reviewed and the foundations are described as Granulite with veins of Granite and Pegmatite and also included the presence of Schists. The foundation rock was seen to be fairly massive in nature. The concluded probabilities will not be discussed in detail here but the conclusions are summarized in Table 5.

For the likely tensile strength capacity at any concrete/rock foundation contact, a similar review was undertaken to that for concrete/concrete contact as described earlier and with results as follows:-

Effective mean tensile capacity = 0.73 x 0.80 = 0.58 MPa = 57 t/m^2

Effective 10% tensile capacity = 0.345 x 0.80 = 0.28 MPa = 27 t/m^2

Tables 2 to 5 summarize the results of the above discussions and evaluations.

Table 2. Net Failure Probabilities - Crest Lift Joints Adjacent to the Spillway

Concrete Lift Joints Non-Overspill Crest Blocks Adjacent to Spillway			
Event	**Ref**	**Judgement**	**Factor**
Reservoir levels sufficiently high	(a)	Assumed as unity	1.0
Flawed (lift) joint(s)	(b)	Possible	0.3
Joints able to be pressurised	(c)	Unlikely	0.1
Joint sufficiently open	(d)	Unlikely to Possible	0.2
Insufficient drainage capacity	(e)	Possible	0.3
A sufficient length affected	(f)	Highly Probable	0.99
Inability to intervene	(g)	Virtually Certain	0.999
The above causing failure	(h)	Virtually Certain	0.999
Net Probability			0.001778438
Net Probability (Scientific)			1.78E-03

Notes: *Reservoir levels assumed sufficiently high once a year*
Some values possibly interpolated between standard values

Table 3. Net Failure Probabilities - Embedded Crest Concrete Lift Joints

Concrete Lift Joints Crest - Random Embedded Blocks			
Event	**Ref**	**Judgement**	**Factor**
Reservoir levels sufficiently high	(a)	Assumed as unity	1.0
Flawed (lift) joint(s)	(b)	Very Probable	0.9
Joints able to be pressurised	(c)	Unlikely	0.1
Joint sufficiently open	(d)	Unlikely to Possible	0.2
Insufficient drainage capacity	(e)	Possible	0.3
A sufficient length affected	(f)	Probable	0.7
Inability to intervene	(g)	Virtually Certain	0.999
The above causing failure	(h)	Virtually Certain	0.999
Net Probability			0.003772444
Net Probability (Scientific)			3.77E-03

Notes: *Reservoir levels assumed sufficiently high once a year*
Some values possibly interpolated between standard values

Table 4. Net Failure Probabilities - Abutment and mid-level Concrete Lift Joints

Concrete Lift Joints Abutment Crest Blocks and Random Mid-Level Blocks			
Event	**Ref**	**Judgement**	**Factor**
Reservoir levels sufficiently high	(a)	Assumed as unity	1.0
Flawed (lift) joint(s)	(b)	Possible	0.3
Joints able to be pressurised	(c)	Unlikely	0.1
Joint sufficiently open	(d)	Unlikely to Possible	0.2
Insufficient drainage capacity	(e)	Possible	0.3
A sufficient length affected	(f)	Possible	0.3
Inability to intervene	(g)	Virtually Certain	0.999
The above causing failure	(h)	Virtually Certain	0.999
Net Probability			0.000538921
Net Probability (Scientific)			5.39E-04

Notes: *Reservoir levels assumed sufficiently high once a year*
Some values possibly interpolated between standard values

Table 5. Net Failure Probabilities - Foundation Level

Foundations Anywhere at or Below Foundation Level			
Event	**Ref**	**Judgement**	**Factor**
Reservoir levels sufficiently high	(a)	Assumed as unity	1.0
Flawed (lift) joint(s)	(b)	Very to Highly Probable	0.95
Joints able to be pressurised	(c)	Virtually Certain	0.999
Joint sufficiently open	(d)	Probable	0.7
Insufficient drainage capacity	(e)	Very Unlikely	0.05
A sufficient length affected	(f)	Unlikely	0.1
Inability to intervene	(g)	Virtually Certain	0.999
The above causing failure	(h)	Possible Certain	0.3
Net Probability			0.000995506
Net Probability (Scientific)			9.96E-04

Notes: *Reservoir levels assumed sufficiently high once a year*
Some values possibly interpolated between standard values

ANNUALISED PROBABILITIES OF RESERVOIR ELEVATIONS

A review of the flood hydrology of Loyne reservoir and historic water level records produced the annualized probabilities shown in Table 6.

Table 6. Assumed Water Levels versus Probability for Loyne Reservoir

Flood Event	**Probability**	**Modified Probability**	**Assumed Outflow (m^3/s)**	**Reservoir Elevation (mOD)**	**Tail W Elevation (mOD)**
PMF	2.50E-06	2.50E-05	349	228.85	214.5
1 : 10,000	1.00E-04	1.00E-04	175	228.23	212.5
1 : 1,000	1.00E-03	1.00E-03	105	227.92	211.5
1/150	6.67E-03	6.67E-03	70	227.73	210.9

Note: *PMF probability modified by a factor of 10 as reservoir is more than 300mm above precedent levels*

STABILITY ASSESSMENTS

The initial phase of the studies above indicated internal concrete failure mode probabilities based on the assumption that water loadings had reached levels necessary to initiate failure. The second phase assessed annualized water level probabilities in more detail. The third phase considered the inherent structural stability of the works for loadings associated with the reservoir elevations described above and with due consideration to the parameters derived in the second phase.

Previous studies had concluded that stability problems were potentially associated with blocked drainage rather than earthquake and so this was the load case focused upon.

The analyses derived the safety factors and/or stresses for a series of lift joint elevations and also an assumed foundation contact at Elevation +208 mOD. The non-overspill sections of Loyne dam were found to be inherently stable, however, the upper zones of the overspill sections indicated marginal stability when cohesion was ignored. Furthermore the stability of the very upper elevations was shown to be sensitive to quite small changes in reservoir level.

The analyses showed the stability of the central spillway about lift joints at +222 mOD to be satisfactory but with concern at or above +224 mOD where both overturning safety factors and peak tensile stresses for 1 in 10,000 year and PMF events were noticeably worse than for the 1 in 1,000 year reservoir level. The net failure probabilities for such crest concrete lift joints are given in Table 3 as corresponding to 3.77E-03, or 0.00377. If we now combine that with the more probable scenario of the PMF or 1 in

10,000 year flood events, we achieve a combined annualized failure probability of 0.00377 / 10,000 = 3.77E-07. Such values, in fact any values below 1.00E-06, are generally considered to indicate adequate safety. Nevertheless this risk was also assessed in terms of potential downstream consequence.

PROBABLE FAILURE MODES AND CONSEQUENCES

A review of the construction drawings indicated that the lowest likely rupture surface would be the lift joint at or around El +223 mOD. It is clear from the construction drawings that such a failure plane would be fairly ragged with some shearing or breakage of upstream and downstream precast units. Nevertheless a broad crested weir discharge coefficient of 1.705 was deemed appropriate to describe the outflow control. Table 7 summarizes the resulting outflows from these scenarios for a range of floods and associated upstream water levels.

Table 7. Outflow Floods for a Range of Breach Scenarios

		No of Failed Blocks				
		1	2	3	4	5
Flood Event (yrs)	Upstream Reservoir Level (mOD)	**Effective Crest Length (m)**				
		13.716	27.432	41.148	54.864	68.58
		Outflow (m^3/s)				
150	227.73	241	481	722	962	1,203
1,000	227.92	255	510	766	1,021	1,276
10,000	228.23	280	559	839	1,119	1,399
PMF	228.85	331	662	993	1,324	1,654

Notes: *Assumed Weir Cd* = *1.705*
Effective Weir Crest Elevation = *223 mOD*

For reasons discussed earlier, shear key restraint at the block joints means that failure of a single block would be almost impossible and that a more likely scenario would be the failure of three of more crest blocks. Similarly it has been shown that failure is highly unlikely for reservoir levels up to +228 mOD, as the crest has already experienced such levels without incident. These zones have therefore been shaded on the table, leaving the most probably failure scenarios corresponding to the 1 in 10,000 yr and PMF flood events and involving three or more crest blocks.

It can be seen from the table that the corresponding outflow floods range from approximately 840m^3/s to 1,650m^3/s. However the highest value seems overly conservative. It combines the worst possible flood, with the lowest credible rupture elevation and the greatest possible number of block failures. It seems more likely that some intermediate failure would have occurred, perhaps at a higher elevation and at a more frequent flood event, before this case is reached. More realistic conservative cases would therefore seem to be either the four block failure case at PMF or the five block failure case at the 1 in 10,000 year event. These correspond to outflow floods with peaks at 1,399m³/s and 1,324m³/s respectively. For all practical purposes it is considered that these can be represented by a flood in the order of 1,350m³/s.

Dam safety risk assessment guidelines, such as the, *Interim Guide to Quantitative Risk Assessment (QRA) for UK Reservoirs*, Thomas Telford, 2004, generally consider risk as a function of persons at risk or the cost benefit of mitigating measures. Specific reservoir risk is depicted graphically in terms of whether the reservoir is above or below a so-called "As Low as Reasonably Practical" (ALARP) region in terms of risk and consequence. A similar graph is given by the United States Bureau of Reclamation's, *Guidelines for Achieving Public Protection in Dam Safety Decision Making*, 2003. It should be noted that the lower bound line for the US "ALARP" range is one magnitude higher than that proposed in the UK interim guide.

The most probable outflow from a crest failure at Loyne dam, in the order of 1,350m³/s, is close to one dam-break scenario already studied by SSE. Considering those results proportionally it could be seen that a 1,350m³/s outflow from Loyne would attenuate to an inflow to the downstream Dungreggan reservoir of approximately 990m³/s.

A previous inspection report on Dundreggan reservoir by the author had concluded that such an inflow would not overtop Dundreggan dam and would therefore be passed downstream to Invermoriston via operation of the main gates. At Invermoriston flows would dissipate into Loch Ness.

SSE has reviewed the results of previous dam-break studies for both Loyne and Dundreggan in terms of the above and information provided to the author by SSE concluded that a conservative value for persons at risk (PAR) in the combined reach from Loyne to Ivermoriston under the above scenario would be approximately 50.

The estimated failure risk for the crest at Loyne of 3.77E-07 is combined with the estimated PAR value of 50 on the ALARP graphs from both the UK, QRA Interim Guide and the USBR Guide as shown in Figure 1. The risk at Loyne can be seen to be in the "Tolerable" or "Broadly Acceptable" regions using both UK and US criteria.

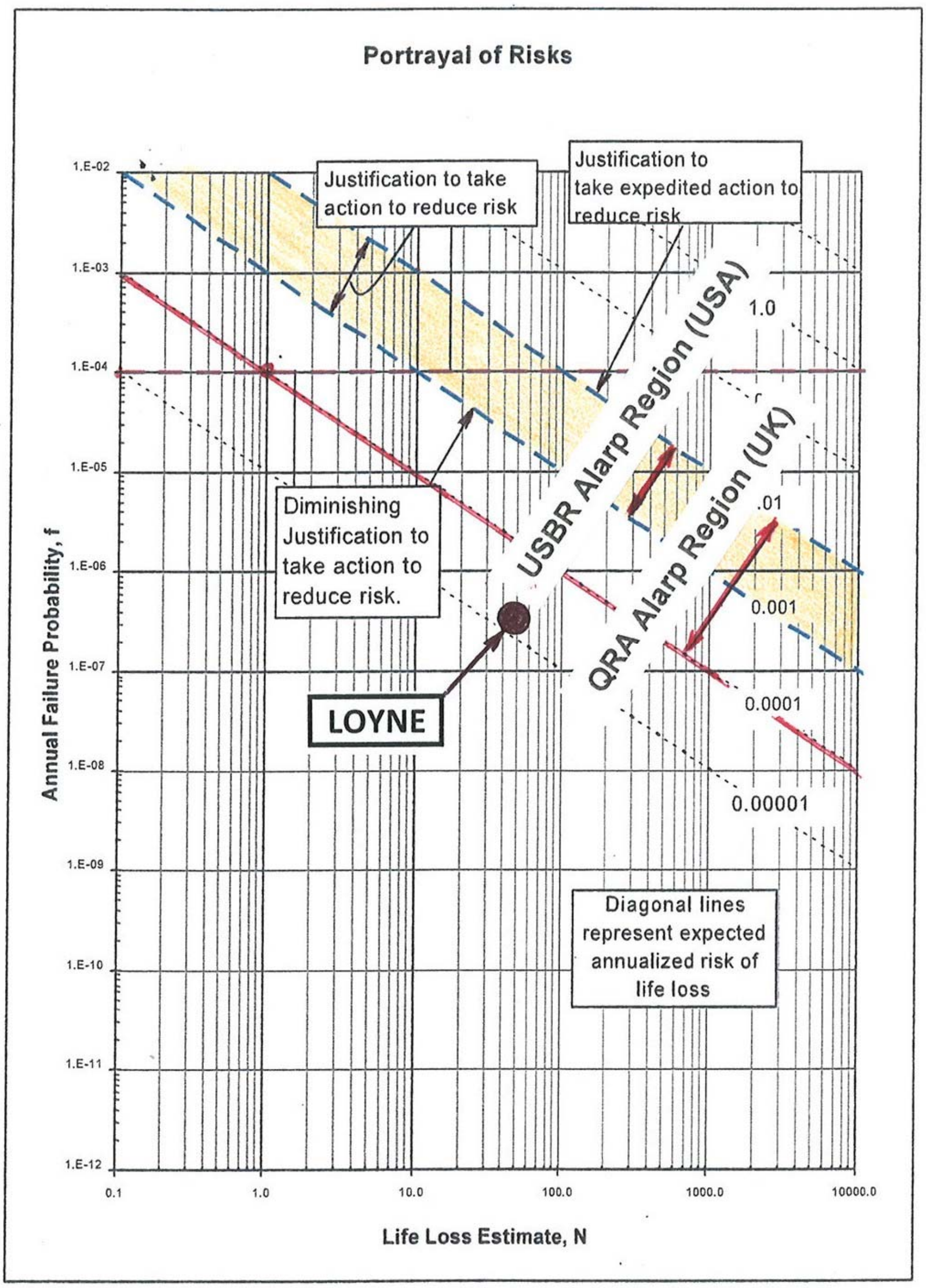

Figure 1. Loyne dam in relation to UK and USBR Risk Guidelines

ACKNOWLEDGEMENTS
The author would like to express his appreciation to SSE for permission to publish this article and in particular to Stuart King for his valuable support and inputs into this study, also to Tony Morison of Halcrow for working with the author on assessing individual event probabilities.