RISK ASSESSMENT FOR HUME DAM - LESSONS FROM ESTIMATING THE CHANCE OF FAILURE

Leonard A McDonald[1] and Chi Fai Wan[2]

Abstract

A risk assessment has been undertaken as part of a comprehensive review of the safety of Hume Dam. Use of risk assessment techniques, to assist in evaluating the safety of existing dams, is a relatively recent trend. Hume Dam was a particularly challenging subject for the application of risk assessment techniques at their present stage of development. The challenge lay in the number and diversity of dam elements to be analysed, in the number and complexity of the potential failure modes and in the fact that there were significant safety issues under normal operating conditions.

This paper outlines some of the key lessons learned from that phase of the risk assessment that was concerned with estimating the chance of dam failure. Some of the issues discussed have not previously been addressed in the literature and some demonstrate a clear need for improved analysis procedures.

INTRODUCTION

Hume Dam is a major irrigation storage of 3,038,000ML on the Murray River some 15km upstream of Albury-Wodonga on the New South Wales-Victoria border. The dam is of national importance, mainly for food production.

Monitoring had shown movement in the Hume Dam Bank No. 1 over many years. At the section known as the "dog-leg", the rate of movement during periods of high storage level in the early 1990's, resulted in the owner (Murray-Darling Basin Commission, MDBC) initiating a comprehensive evaluation of the dam's safety. Traditional standards-based analyses were put in hand and remedial action has been taken progressively as significant deficiencies have been identified.

A risk assessment, commenced in 1996, will provide a comprehensive overview of the risks contributed by the various elements of the dam, a basis for priority ordering of further actions, and a basis for evaluation of the less urgent remedial options. The use of risk assessment to assist in evaluating the safety of existing dams is a recent trend. Methodologies to support the assignment of probabilities of failure are not yet well-developed. In some areas the necessary analysis tools are not yet available. The Hume Dam study was particularly challenging in the number and diversity of dam elements to be assessed, in the number and complexity of the failure modes, and in the fact that there were significant risks under normal operating conditions. There were five dams, three training walls (the fourth had negligible chance of failure) and a system of twenty nine spillway gates to be analysed. There were eighty one failure modes, with up to thirty two branches in an event tree. There were eight flood loading states, forty eight earthquake loading scenarios and six normal operating states. Some fifteen thousand conditional probabilities were assigned.

This paper deals with that phase of the risk assessment that addressed the questions: "What can happen?" and "How likely is it that it will happen?" Subsequent phases involve consequence assessment, residual risks of remedial options and presentation of the overall risk information. The purpose of the paper is to bring to notice selected key lessons from the Hume Dam risk assessment study, in the hope that these will stimulate discussion that will result in improved methodologies.

[1] Assistant Principal Engineer, Dams and Civil, NSW Department of Public Works and Services (now retired)
[2] Design Engineer, Dams and Civil, NSW Department of Public Works and Services

DESCRIPTION OF THE DAM

The dam was constructed in the late 1920's using the technology of the day. By modern standards the compaction of the earthfill embankments is poor. The concrete quality is reasonable. In the late 1950's, the storage capacity was increased by several metres by the installation of 29 vertical lift gates. Embankments were raised and a new saddle dam (Bank No. 3) was constructed. In the early 1960's, the concrete gravity dam was post-tensioned by non-restressable cables. In the mid-1980's, a new set of monitorable, restressable post-tensioned cables was installed because of concerns about possible corrosion of the original cables. In early 1996, a stabilising berm and improved drainage were constructed at the "dog-leg" area of Bank No. 1. Subsequently, further remedial works have been constructed. The dam has never experienced a reservoir level above Full Supply Level (FSL) due to high discharge capacity of the gates (7,000m^3/s at FSL and 14,000m^3/s at bank overtopping level). The layout of the dam is shown at Figure 1.

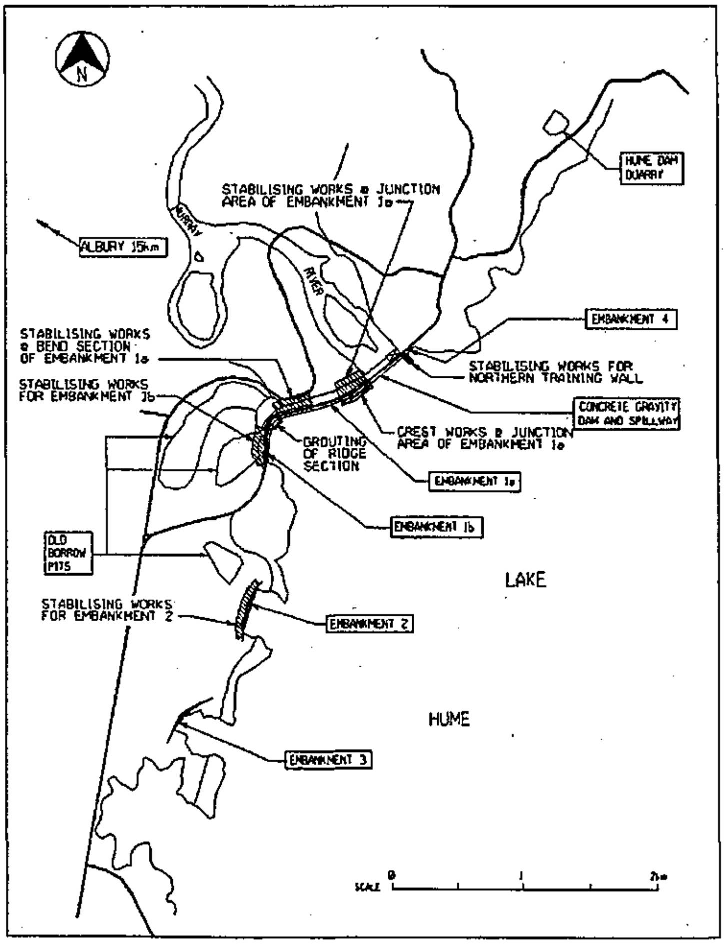

Figure 1 Layout of Hume Dam

The concrete gravity dam is 336 m long and 45 m high to the top of the parapet wall. There are 9 spillway blocks and 6 non-overflow blocks. Figure 2 is a typical cross-section of a spillway block. Bank No. 1 is 1,184m long and has a maximum height of just over 40m (adjacent the concrete dam) to the top of the parapet wall (195.30m AHD). The central concrete core wall is slotted into the foundation rock whilst the upstream and downstream earthfill shells are founded on recent or Tertiary alluvium (according to location); or on residual soil (at the higher foundation levels). The embankment fill was borrowed from a deposit of strongly dispersive Tertiary alluvium just downstream of the dam.

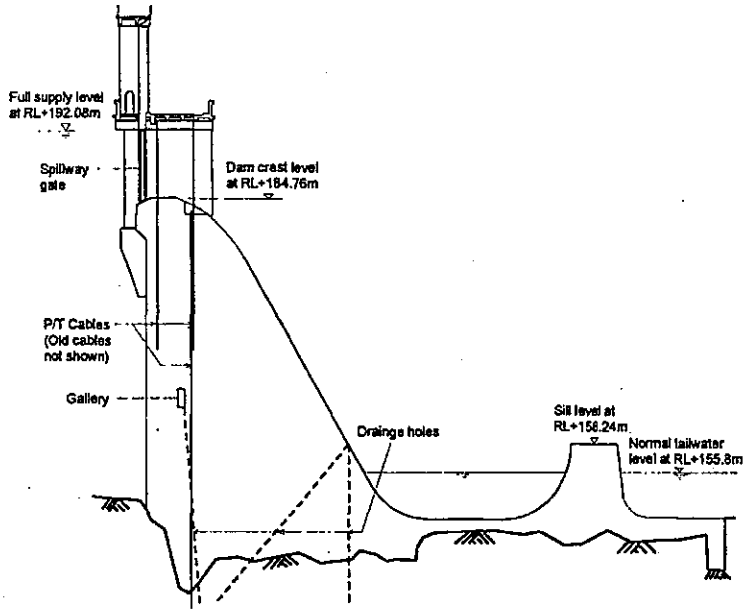

Figure 2 Typical Cross-section of the Concrete Dam Spillway Section

The core wall of Bank No. 1 is slotted into a concrete block (known as the Southern Tower Block) which is monolithic with the concrete dam. Originally, the interface space was sealed with bitumen. This area is known as the Southern Junction. Bank No. 2 is a homogeneous embankment, 488m long and up to 18.3m high, founded on Tertiary alluvium and constructed of the same strongly dispersive fill as Bank No. 1. Bank No. 3 is 447m long and of maximum height 7.4m, founded on strongly dispersive Tertiary alluvium but with embankment fill that is only mildly dispersive. Bank No. 4, of non-dispersive fill and with a central core wall, adjoins the northern side of the concrete dam. It is founded on non-dispersive residual soil and is some 100m long with a maximum height of 18 m.

RISK ANALYSIS PROCESS

Probabilities were assigned by engineering judgments made in the light of as much detailed analysis as practicable. There were three groups involved in the analysis: the Analysis Team, the Review Panel and the International Review Consultants. The Review Panel comprised independent specialists, representatives of the client (NSW Department of Land and Water Conservation, DLWC, acting on behalf of MDBC), personnel involved in monitoring and surveillance, and sometimes site personnel responsible for operation of the dam. The Analysis Team played a key role in review sessions. Reviews were held at milestone points, such as at the completion of event tree construction. It was important to agree on the logic of failure processes, before proceeding to assign conditional probabilities. Those involved in review sessions had documentation of work to date provided to them before the sessions. Sessions involved a process of challenge and debate that was focused on methodology along with global comparisons and assessment of probabilities. The outcome of sessions was documented and circulated to participants.

The benefits of detailed analyses and a structured peer review process, were clearly demonstrated by the Hume risk analysis experience. A common problem of large risk assessment studies is the confusion that arises in dealing with very large numbers of probabilities. For the Hume study, a new concept, that of probability assignment protocols, was developed to deal with this problem. The protocols are concise instructions for the assignment of a conditional probability to each branch of each event tree, according to the load or condition state. Each protocol was followed by the reasons supporting it. These protocols enabled the analysis team to maintain logical consistency over a large number of failure modes and to check against double counting, they were the main vehicle for review (it is not feasible to individually review 15,000 probabilities), they acted as an instruction sheet to the person entering the probabilities onto the event trees and, most importantly, they provide permanent documentation of reasons for probability values. An example from the study report follows:

"Protocol:

- *Probability of "Slide Occurs"*

a) For each water level, being the mid-level of bands in Table 8.1,

- *select the Lower Quartile (LQ) value of F that is representative of each embankment zone (zone boundary mid-way between each F value)*
- *convert LQ values of F to Mean Value F using Figure 7.14*
- *select multiplier from Table 8.12*
- *adjust each sunny day Factor of Safety, F, by the multiplier*
- *compute the probability of a slide from Equation 7.18 for each F*
- *assign the zone length for each F value*
- *adjust the probability of sliding for each zone length using Equation 7.25, with "n" equal to the length in metres divided by 100*
- *compute the overall probability of an embankment slide, combining the probabilities for each zone according to Equation* 7.24"

Such an example cannot be fully understood out of the report context, but does convey the idea of the probability assignment protocols.

The logic of failure processes was represented by event trees. However to estimate the chance that spillway gates would not operate when needed, it was necessary to use fault trees.

LOADING SCENARIOS

Flood Loading

In the early days of risk assessment for dams, there was a view that loading domains should be split into only a few partitions in order to limit the complexity of the analysis (BC Hydro, 1993 at pages C-9 and C-10). It soon became apparent in the Hume study, that a small number of partitions would make the assignment of conditional probabilities difficult because of the large span of each partition and the difficulty of predicting, at the time of partitioning, where the safety thresholds would be. Subsequently it was found that the program developed to route floods through the reservoir (Dyer, 1996) required 10 peak discharge states to operate in any case. Eventually a total of 21,384 flood loading scenarios were considered, made up from 10 natural flood peak discharge states, 1 dam failure flood state (Dartmouth Dam upstream), 6 prior storage states, 4 gate operating states, 6 wind velocity states (one of which was "calm") and 16 fetch/direction states (up to 4 for each bank).

$$N = (10+1)\cdot(6)\cdot(4)\cdot(6-1)\cdot(16)+264 = 21{,}384$$

The question was how to combine such a large number of loading scenarios with the many event trees and their many branches. It was realised that what influences conditional probability of dam failure is peak water level in the reservoir. Thus it was decided to produce a curve of Peak Water Level (PWL) versus Annual Exceedance Probability (AEP). The PWL's, each with its annual chance of occurrence, from the many flood loading scenarios were sorted into rank order to obtain the cumulative distribution. Figure 3 is the curve for calm conditions. The PWL versus AEP curve was then divided into 7 partitions which captured all of the significant risk thresholds; for example, the zone at and immediately above the level (195.30 m AHD) at which the embankments overtop.

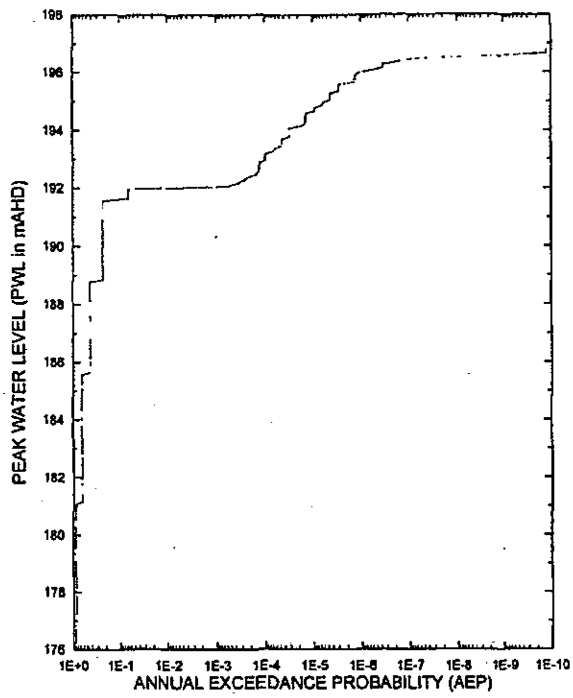

Figure 3 Peak Water Level versus Annual Exceedance Probability - Calm

The Hume experience demonstrates how rapidly the total number of loading scenarios escalates as an apparently modest increase is made in the number of partitions in each loading domain. On the other hand, finer partitioning removes a major source of potential inaccuracy. Having 6 to 10 partitions, as compared with say 3 to 5, does not involve a major additional effort if computer processing is used, particularly if conditional probabilities can be expressed as a function of loading in the form of a response curve. One exception is the task of flood routing. For Hume, 264 basic flood routings were required (the first three terms of the equation above) but more than double that number were needed to find critical durations. The optimum number of partitions will vary from case to case, but striking the best balance is clearly a key consideration.

The study showed that wind effects made a negligible addition to chance of failure, the reason being that wind velocity and direction were treated as not correlated with peak stage in the reservoir. For Hume Dam, this seems reasonable because there is a large time lag between the rainfall event and peak reservoir stage. The situation may be otherwise elsewhere, particularly on small catchments.

Earthquake Loading

Earthquake loading scenarios cannot be rolled into a single parameter analogous with PWL for flood loading. There were 48 loading scenarios (8 peak ground acceleration states by 6 prior storage states) for the study. Many probabilities were expressed as an 8 by 6 matrix in the protocols.

The innovation in Australian practice for the Hume study was the consideration given to magnitude contributions. This is an issue for any failure process that depends on duration and number of cycles of earthquake shaking, these being a function of magnitude. A given Peak Ground Acceleration (PGA) at a dam site may be due to a smaller event close by or to a larger event far away. The latter will typically have a greater duration and number of cycles; and will thus be more likely to cause liquefaction, may result in greater displacement of concrete dam blocks and may cause greater deformation in embankment dams. Figure 4 is based on the plot of magnitude contributions produced for the study by the Seismology Research Centre, Royal Melbourne Institute of Technology. For the Hume study, load state annual probabilities were based on the following PGA versus AEP curves

:

- events of M5.0 and above for occurrence of liquefaction
- events of any magnitude for failure modes at the Southern Junction
- events of M4.5 and above for all other failure modes

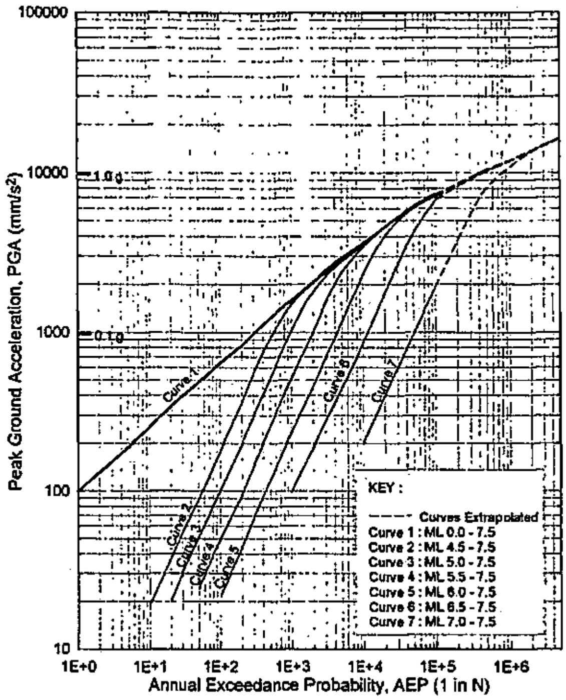

Figure 4 Magnitude Contributions to Earthquake Shaking for Hume Dam

The Southern Junction was seen to be in a particularly vulnerable position, with the possibility of a sudden brittle failure, due to progressive movement of the core wall downstream over the years. This movement (in the order of 200 mm) had cracked the wall and the Southern Tower Block, and had induced high stresses in these elements. The view was that a single sharp jolt, resulting from an earthquake of any magnitude, could trigger a failure at this location.

Normal Operating Conditions

Load condition was defined by the 6 prior storage states. The Hume storage fluctuates over a large range on an annual cycle. The approach was to assign conditional probabilities of failure for storage at FSL and to then apply a multiplier to adjust for storage condition. This multiplier was dam specific, since the chance of a prompt failure had to be zero when storage was at the base of the dam.

LENGTH AND NUMBER EFFECTS

Factors, such as the length of an embankment dam, or the number of independent blocks in a concrete gravity dam, affect the probability of failure. Simply put, the more throws of a die, the greater the chance of getting at least one unfavourable outcome. The effect depends on the degree of correlation between blocks or between segments of an embankment. If elements are not correlated, so that knowing the properties of one element says nothing of the properties of the others, the overall chance of failure is given by the union of events; which, by de Morgan's Rule, is:

$$P[T] = P[E_1 \cup E_2 \cup \ldots \cup E_i \cup \ldots \cup E_n]$$
$$= 1 - (1 - P[E_1]) \cdot (1 - P[E_2]) \cdot \ldots \cdot (1 - P[E_i]) \cdot \ldots \cdot (1 - P[E_n]) \quad \cdots\cdots (1)$$

where $P[T]$ = the overall probability of failure
E_i = Element i fails, $i = 1 \ldots n$
$P[E_i]$ = the probability that Element i fails
$\cup$ signifies the union of events

If there is perfect correlation, the overall probability of failure is the maximum value out of the individual element probabilities. The two extremes can be seen as upper and lower bounds (CUR, 1990). It can be shown that if an embankment is notionally divided into segments, the probability of failure of the segment depends on its length, since the overall upper bound probability of a failure must be independent of any notional partitioning. These considerations raise two difficult questions:

- if the chance of failure is estimated by reliability analysis, for example based on FOSM (First Order Second Moment) analysis, to what segment length does that chance relate?
- if the chance of failure has been estimated from the historic database, as was the case with piping, to what segment length does that chance relate?

These questions were considered in relation to the 1,184 m long Bank No. 1 at Hume Dam. Vanmarcke (1977) shows how Factor of Safety, F, reduces as bank length increases due to declining influence of side restraint. Beyond 100 m length, F is effectively the two dimensional value. Because the Hume stability analyses were generally two dimensional, the estimated probability of failure was taken to apply to a segment length of 100 m. Probabilities estimated from the historic database, were taken to apply to a length of 500 m. This was about the average length for all dams in the US Register of Large Dams and was seen to be a rough measure of the average length of dams of the historic database. From the union of events, the chance of failure for any zone of a dam was based on the following formula:

$$P[Z] = 1 - (1 - P[S])^n \quad \cdots\cdots (2)$$

where n = the ratio of the zone length (part or whole of dam) to the basic segment length (100m for stability risks from reliability analysis, 500m for risks from the historic database)
$P[Z]$ = the probability of failure for the zone
$P[S]$ = the probability of failure for the basic segment length

This formula was used whether "n" was greater than, or less than, 1.0. When "n" is less than 1.0, the resulting zone value is in fact a lower bound. As an example of zoning, each cross-section analysed for stability was taken to be representative of a zone, the ends of which were defined by the mid-point between analysis sections. These zones were typically 200m to 300m long.

These questions of length and number are difficult and do not seem to have had much attention in the literature. They deserve wider discussion.

RETROSPECTIVE VERSUS PROSPECTIVE PROBABILITIES

Reliability analysis is used to estimate chance of failure, given the uncertainties in load and resistance (Whitman, 1984 and Ang and Tang, 1975). Provided mean values of parameters are used in the analysis, reliability analysis estimates the chance that the true value of some performance parameter (such as Factor of Safety, F) will be less than 1.0 (the failure point), given the computed value of the performance parameter. Thus reliability analysis is useful for prospective situations; that is, those that have not yet been put to the test. Where the die has not yet been thrown. Examples are where a dam has not yet been built, or where an existing dam may be subjected to a load greater than the load of record

What of situations that have been put to the test; where the die has been thrown and the outcome is a success? The dam has not failed. In the case of slope stability, the true value of F must be greater than 1.0, although there is no way of knowing by how much it exceeds 1.0. If load and resistance properties were invariable with time, the probability of failure would be zero. But is it? This question arises with existing dams under normal operating conditions.

So far as is known, no concrete gravity dam has suffered a delayed (it is necessary to exclude infant mortality cases) structural failure under a previously experienced load. Failure within the first five years is seen as "infant mortality" (McCann et al, 1985). But earthfill dams have suffered piping or slope failures many years after construction, without any apparent change in loading condition. USCOLD (1975) report three failures and several incidents involving embankment slides many years after construction. Some internal perturbation involving load, resistance or both has occurred; most probably a change in seepage regime. Perhaps the true value of F was not much above 1.0 in the first place. But there can be occurrences that cause a dramatic reduction in the value of F. Wetting up of the fill from seepage around the end of the core wall at the Hume Dam Southern Junction appears to have reduced F substantially from its original value. These latter situations can be treated as prospective, if the initiating cause is not at work at the time of consideration.

Retrospective probability of failure was the most difficult problem encountered in the Hume Dam study. The approach of McCann et al (1985) was considered, but the evaluation scales are not suitable for detailed analyses. For the concrete dam, the retrospective probability of failure was taken to be negligible. For training walls and the earth dams, the historic delayed failure rates were increased or decreased according to curves that took account of the computed value of the performance parameter, on the basis that the lower the computed value, the greater the chance that the true value is only marginally above 1.0. However the procedure was regarded as no more than an aid to judgment. Clearly, better procedures are needed for estimating retrospective probabilities.

For prospective situations, reliability analysis was used. For situations where the original prospective chance of failure under the load of record was significant (although not realised), a modification of classic reliability analysis was developed, as illustrated in Figure 5.

Since the dam did not fail under the load of record, all combinations of resistance parameters that would have resulted in a value of F less than 1.0 are ruled out of contention; that is, the true value of F does not lie within the space marked "Prospective Major Slide Domain, Original Load". The sample space for true F is now known to be the complement of that domain. A load, greater than the load of record, shifts the probability density function (pdf) to the left by the distance:

$$\text{Shift} = E[F_o] - E[F_N] \qquad \cdots\cdots \ (3)$$

Where E[ ] = expected value

F_o = computed F based on mean parameters for load of record

F_N = computed F based on mean parameters for new load

Combinations of parameters that would place true F in the original major slide domain remain out of contention. But there is now a slide domain, marked "Prospective Major Slide Domain, New Load", which could contain the true value of F. The conditional probability of a slide is:

$$P[S] = \frac{\Phi(\beta_o) - \Phi(\beta_N)}{\Phi(\beta_o)} \qquad \cdots\cdots \ (4)$$

where P[S] = conditional probability of a slide, given the new load

$\Phi(\ )$ = standardised Normal cumulative distribution function

β_o = reliability index under the original load

β_N = reliability index under the new load

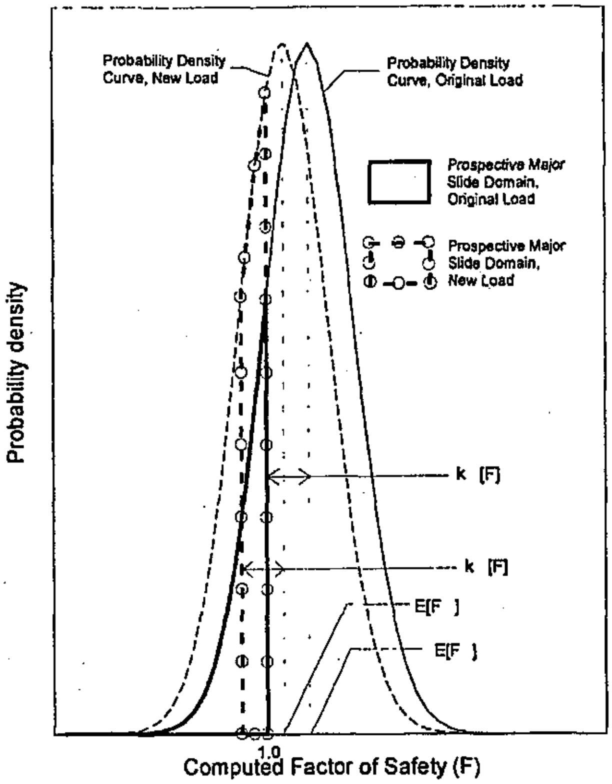

Figure 5 Modified Reliability Analysis for Loads Higher than the Load of Record

Equation (4) derives from the following relationships of classic reliability analysis (Whitman, 1984):

$$\beta = \frac{E[F]-1.0}{\sigma[F]} \qquad \cdots\cdots \ (5)$$

$$P[f] = 1.0 - \Phi(\beta) \qquad \cdots\cdots \ (6)$$

where P[f] = probability of failure for a performance parameter where a true value of 1.0 defines the point of failure

$\sigma[F]$ = the standard deviation in F

β = reliability index

β as given by Whitman (1984)

Equation (4) can be applied to other performance parameters, such as Shear Friction Factor for concrete gravity dams. However, it does require that the standard deviation of F (or other performance parameter) remain constant, or nearly so, over the span of the shift. Crum (1996), discussing the work of Christian et al (1994), suggested that the coefficient of variation was closer to being constant than is the standard deviation. Monte Carlo simulation for the Hume concrete dam gave a nearly constant coefficient of variation. This is an issue requiring further clarification. At this stage it would be prudent to restrict use of the relationship to modest shifts in the performance parameter.

SLOPE STABILITY

At the Southern Junction, the stability of Bank No. 1 was of concern because of the downstream movement of the core wall, the cracking of the wall and tower block, and the wetting of the fill by seepage through the interface space. Several boreholes were drilled in the embankment, with continuous cone penetrometer logs taken, providing unusually good data availability. An FOSM approach to reliability analysis was taken, following the procedures of Christian et al (1994). This yielded a mean computed value of F of 0.96 (based on mean values of input parameters) and a standard deviation of 0.21. Christian et al estimated standard deviations in the range of 0.12 to 0.29 for the James Bay banks. Analyses by University of New South Wales, using numerical modelling of deformation, confirmed the precarious stability in this area. Taking the standard deviation value as constant, and representative of all the banks, the system response curve of Figure 6 was obtained.

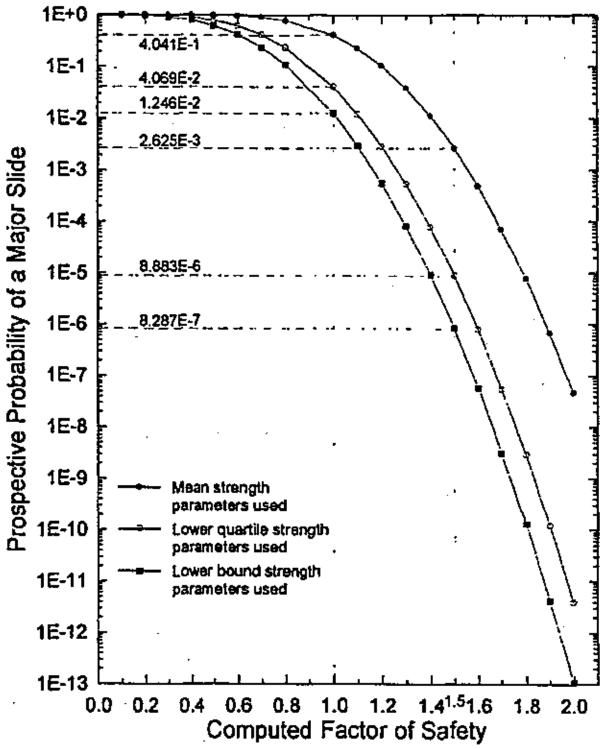

Figure 6 System Response Curve for Prospective Probability of Sliding

Reliability analyses gave some interesting insights into the relationship between probability of a slide and computed factor of safety, F. If mean values of input parameters are used, a computed F value of 1.0 corresponds to a 50 percent prospective chance of sliding (leaving model error out of consideration - Figure 6 includes an estimate of model error). That is so, regardless of the standard deviation in F. This is obvious from Equation (5) in the preceding section. Uncertainty in the standard deviation in F is most critical at high values of F (that is, for low probabilities of sliding) and does not matter greatly for low values of F (that is, at high probabilities of sliding). If computed F is greater than 1.0, a higher standard deviation gives a higher probability of sliding. But if F is less than 1.0, a higher standard deviation gives a lower probability of sliding. The conservative shifts in soil strength properties (that is, understating the strengths) that are common in traditional standards-based analyses, make a significant difference to probability of sliding at low values of F, but have a far greater impact at high values of F. This is obvious from Figure 6. The terms "lower quartile" and "lower bound" are indicative only as regards soil strength; they actually represent shifts of 1.5 and 2.0 standard deviations in F.

The Hume experience revealed an inherent problem in applying reliability analysis to slope stability of earth dams. Estimation of the standard deviation in F requires estimation of the correlation distance for soil strength. This is a measure of how erratically soil strength varies, and can be visualised as the "wave length" of soil strength variation in a continuous plot of cone penetrometer results. This parameter governs the extent to which some of the variation in strength cancels out due to spatial averaging over the failure surface. Figure 7 shows the critical failure surface at the Southern Junction.

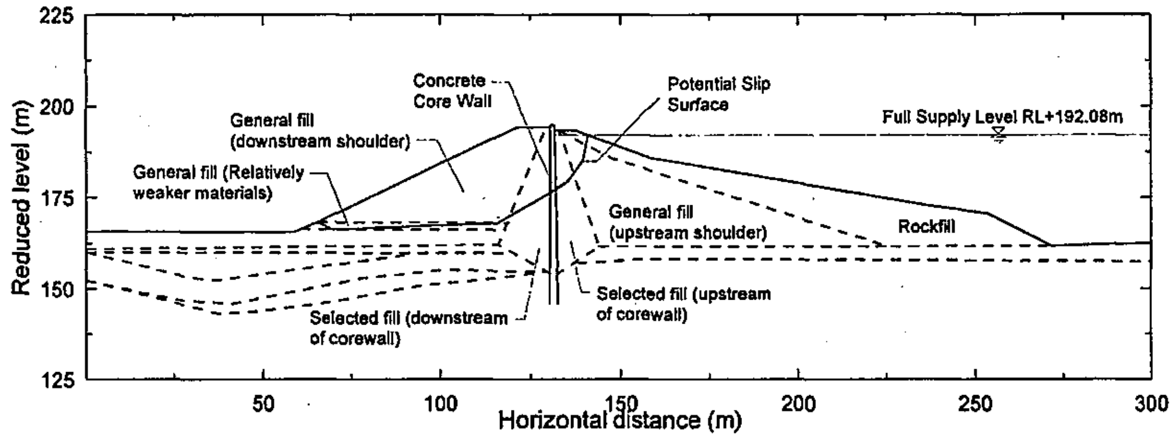

Figure 7 Critical Slide Surface at the Southern Junction

The FOSM analysis showed that 93 percent of the variance in F was accounted for by the thin weak horizontal layer at the base of the slide surface. But there are no continuous soil strength data in the horizontal direction (either normal to or parallel to the dam axis) from which to estimate the horizontal correlation distance. The good data are in the vertical direction. Since earth dams are constructed in horizontal layers (so that it will be common that such layers dominate stability) and boreholes are normally vertical (so that the best data are in the least important direction for reliability analysis), it seems that estimating the standard deviation in F will typically be plagued by uncertainty. Construction in layers suggests that uniformity will be typically much greater in the horizontal direction than in the vertical, and that therefor the horizontal correlation distance (10m was used) will be much greater than the vertical correlation distance (a value of about 0.5m was computed from the penetrometer logs).

POST-LIQUEFACTION STABILITY

Deformation under earthquake shaking was not an issue for the Hume banks because of the large freeboard at FSL (3.22m). The failure mode requiring analysis was a bank slide due to loss of strength in foundation soils due to liquefaction. The reduction of strength is so large, that the situation is treated as prospective (away from the Southern Junction the banks generally had a high computed F value under normal operating conditions). The much reduced post-liquefaction value of F was entered into a curve, similar to Figure 6 above, to obtain the conditional probability of a slide, given liquefaction. The curve was based on a different standard deviation in F because the uncertainties in the undrained residual strength of the liquefied zones now governed. Undrained residual strength was represented as an equivalent friction angle, typically between 4 and 9 degrees. The probability of liquefaction was found by applying the procedure of Liao et al (1988) to every potentially liquefiable data point from the exploration boreholes. The probability of earthquake shaking was taken from the curve of PGA versus AEP for events of M5.0 and greater.

Three lessons emerged from the work. Firstly, liquefiable materials are continuous over large areas of the foundation but there are relatively few boreholes. It is possible that the boreholes have not found the most susceptible zones. Secondly, there is the matter of extent of liquefaction. At some sections, there were three boreholes. The highest probability of liquefaction from each hole was taken to represent that hole (provided it could form a plausible failure surface with the highest values from the other holes). There could be localised liquefaction in the vicinity of one hole (associated with a small scale slide if the liquefied zone was near the toe of the dam) or liquefaction could extend over the area represented by all three holes (associated with a large scale slide). A search process was followed to find the critical case. The question was: "What is the probability of liquefaction extending over the area represented by all three holes, given the probability of liquefaction representative of each hole?" No direct guidance was found in the literature. After considering the theory of uni-modal bounds (Ang and Tang, 1975 - Sub-section 7.2), it was concluded that the lowest of the three representative probabilities was the upper bound probability for liquefaction of the whole area, and the product of the three probabilities (the intersection of events) was the lower bound. The lower bound was always negligibly small, and raises conceptual difficulties because it is a function of the number of boreholes; and, as with embankment length effects, the only conclusion is that the probability associated with each borehole must be a function of the area represented by the borehole. Otherwise, the lower bound probability of liquefaction could be influenced by drilling more holes! Such issues are yet to be fully clarified. Thirdly, strength may reduce to a value intermediate between the static value and undrained residual strength, either because of drainage relieving pore pressure during ground shaking or due to too few cycles of shaking that result in only a partial increase in pore pressure. There was no opportunity for drainage at Hume Dam because of low permeability strata capping the liquefiable zones; and using the AEP for events greater than M5.0 was seen to account for number of cycles. But, in the general case, there is a need for a methodology to deal with intermediate strength states.

STABILITY OF THE CONCRETE DAM

The retrospective probability of failure under normal operating conditions was taken to be negligibly small.

The FOSM approach was tried for the prospective flood loading case but produced erratic results. The standard deviation in SFF (Shear Friction Factor - the ratio of horizontal shear capacity to the net horizontal force) was found to be unstable, varying from 1.6 to 7.9 with small differences in the assumed tensile capacity of lift surfaces. Apparently this is because, using 2D cantilever analysis, the safety status of the dam changes dramatically as soon as cracking initiates. Monte Carlo simulation was therefor used to find the pdf for SFF. For the concrete dam, 2,000 runs were used, but this was increased to 20,000 for the training walls. Stable results were obtained, with the mean value of SFF increasing from 6.4 (with standard deviation of 1.29) at the highest natural flood level of 196.76m AHD, to over 8.0 (with standard deviation of 1.45) at FSL, giving a negligible chance of failure.

Under earthquake loading, the approach was to estimate the yield acceleration taking account of the dam's dynamic response and to then estimate displacement using a sliding block model (Newmark, 1965). Monte Carlo simulation was used to find the pdf of displacement, given the pdf of yield acceleration. For this purpose, the number of runs had to be limited to 1,000 to avoid excessive computer running time. The accelerogram used for the sliding block analysis, was that recorded at Pacoima Dam for the Northridge earthquake of 17th January, 1994. The accelerogram was scaled in proportion to PGA, a procedure that is open to challenge. But the whole question of an appropriate accelerogram is inherently uncertain. It was assessed that displacements in excess of 60mm would shear the post-tensioning cables. However, under the highest earthquake load state (0.92g), the mean displacement was estimated as 2mm and the maximum displacement out of the 1,000 runs was only 36mm. The yield acceleration had been estimated as asymmetrically distributed, with a long tail on the high yield acceleration side. It was reasoned that the adopted Normal distribution for yield acceleration, with mean of 0.63g and standard deviation of 0.10g, would overstate the displacements slightly.

Post-earthquake analysis, which took account of the changed uplift pressures in the cracked section, showed the dam was safe unless rocking had failed the post-tensioned cables. There was a chance of cable failure at the extreme PGA states, which contributed a small risk of dam failure. Subsequent flood, during the period required for repair of the dam, was assessed not to be a problem on the basis that the storage would be drawn down once post-earthquake inspection revealed the dam had cracked and the cables had failed. Subsequent earthquake was not analysed. By the traditional standards-based approach (ANCOLD 1991), the safety limit for the dam would be a PGA appreciably less than 0.63g (since that value is based on estimated pdf's for parameters, such as tensile capacity of lift surfaces, whereas these are assigned lower bound values in the traditional analyses). But the risk assessment shows negligible chance of failure until earthquake load State 7 (0.76g) which gives a small risk (2.4×10^{-6} per annum upper bound). Work on another gravity dam has shown a safety limit of 0.3g by the traditional approach but negligible chance of failure at any credible level of shaking by risk assessment; except for risk related to subsequent flood. Subsequent flood risk is dam specific, since it depends on catchment area, storage volume and the capacity of the discharge facilities to maintain a low reservoir level until repairs are made.

These considerations seem to point to a marked disparity in assessed safety under a risk assessment approach (which is trying to define the failure point), as compared with the traditional standards-based approach (ANCOLD, 1991). There is no doubt that the traditional approach has produced safe dams, but is it asking for an unnecessarily large margin of safety on existing dams? No gravity dam has yet failed due to earthquake. The key question is whether it is reasonable to accept a possibility of cracking with displacement (say up to a few hundred millimetres for gravity dams without post-tensioned cables) under extreme earthquake loads. It could be argued that such displacement will not result in flooding that would endanger downstream residents and property; although it may slowly empty the reservoir. These questions would seem to be worthy of wide discussion and debate.

RESULTS OF THE STUDY

The greatest contribution to chance of failure was estimated to come from normal operating conditions, due to the movements and fill softening in the Southern Junction area of Bank No.1. The contribution from earthquake loading was only slightly smaller, whilst that from flood loading was estimated to be several orders lower. Bank No. 1 was estimated to have the highest chance of failure (some 400 times that of the next most vulnerable bank) and the concrete dam the lowest. The most significant modes in terms of chance of failure were static instability, post-liquefaction instability, piping and instability of the northern training wall. Wind effects increased the chance of failure from flood loading by only 2.7 percent. The chance of failure due to subsequent flood, where the dam is initially damaged but there is no prompt failure, made a negligible contribution to overall risk. This outcome was due to the capacity of the spillway gates, and power and irrigation outlets, to maintain a reduced reservoir level until repairs are effected. The greatest contribution to chance of failure from flood and earthquake came from the lowest loading partition, reflecting the high chance of occurrence of the loads as well as the vulnerability of the Southern Junction area. Whilst it was estimated as virtually certain that power to the spillway gates would fail under high flood conditions, there is only a very small chance of loss of gate capacity, given available backup systems, provided there are sufficient trained personnel on site. The dam owner has already taken action to reduce the serious risks.

CONCLUSION

The Hume experience has highlighted the fact that the traditional standards-based approach has a different purpose to risk assessment. The traditional approach evolved in response to the need to design new dams that are safe. There is no doubt that it has been successful. The approach has been to overstate load and to understate resistance so as to ensure safety. The procedures are codified and the result is, by and large, not analyst dependent. The analyst does not know what the true margin of safety is. It is sufficient to know that the procedure will yield a safe dam. The question being answered is: "How can I ensure that this dam will be safe?" Risk assessment is asking the more difficult question: "What is the probability that this dam will fail, given load or condition?" The object of the question is usually an existing dam. Answering it requires that the analyst understands the failure process in a detailed way, so as to predict the point of failure! There are as yet no codified procedures and results are almost certainly analyst dependent to a large degree (hence the importance of group review). The analytical tools needed to assist judgment are poorly developed or lacking.

Despite the problems, the Hume experience has shown that the struggle to answer the risk assessment question yields real benefits. Firstly the process is, by its nature, comprehensive. It systematically examines all elements of a dam and all failure modes, thereby avoiding the situation of dealing with some obvious deficiency (inadequate spillway capacity?) while some less obvious, but more dangerous, problem (a rusting conduit?) is left unattended. Secondly, it inherently exposes uncertainty and brings it into consciousness, thus constantly reminding the analyst that the results are only a best estimate of the real situation. Thirdly, risk assessment provides a quantitative basis for ordering of priorities. Finally, and most importantly, the struggle to identify and analyse the failure processes greatly enhances understanding of the safety threats to a dam. This understanding is the most valuable benefit of the exercise.

This paper has briefly outlined some of the insights gained in the process of estimating the chance of failure for Hume Dam. It is hoped that the matters raised will provoke discussion and will ultimately result in improved analysis tools.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the assistance of their colleagues in the NSW Department of Public Works and Services, and the cooperation and support of officers of the NSW Department of Land and Water Conservation and the Murray-Darling Basin Commission.

REFERENCES

ANCOLD (Australian National Committee on Large Dams), *Guidelines on Design Criteria for Concrete Gravity Dams,* November 1991

Ang, A.S. and Tang, W., *Probability Concepts in Engineering Planning and Design,* John Wiley and Sons, New York, 1975.

Christian, J.T., Ladd, C.C. and Baecher, G.B., *Reliability Applied to Slope Stability Analysis,* Journ. of Geotechnical Engineering, ASCE, Vol. 120, No. 12, December 1994.

Crum, D., Discussion on paper *Reliability Applied to Slope Stability Analysis,* Journ. of Geotechnical Engineering, ASCE, Vol. 122, No. 5, May 1996.

CUR (Centre for Civil Engineering Research and Codes, Technical Advisory Committee on Water Defences, The Netherlands), *Probabilistic Design of Flood Defences,* June 1990.

Dyer, B.G., *Hume Dam Gate Operation Model,* Murray-Darling Basin Commission Technical Report 96/9, September 1996.

BC Hydro, Hydroelectric Engineering Division, *Guidelines for Consequence-Based Dam Safety Evaluations and Improvements,* Report No. H2528, August 1993.

Liao, S.S.C., Veneziano, D. and Whitman, R.V., *Regression Models for Evaluating Liquefaction Probability,* Journ. of the Geotechnical Engineering Division, ASCE, Vol. 114, No. 4, April 1988.

McCann, M.W., Franzini, J.B., Kavazanjian, E. and Shah, H.C., *Preliminary Safety Evaluation of Existing Dams,* Report prepared for Federal Emergency Management Agency by Department of Civil Engineering, Stanford University, November 1985.

Newmark, N.M., *Effects of Earthquakes on Dams and Embankments,* 5th Rankine Lecture, Geotechnique, Vol. 15, No. 2, 1965.

USCOLD (United States Committee on Large Dams), *Lessons from Dam Incidents, USA,* Joint ASCE/USCOLD Publication, 1975.

Vanmarcke, E.H., *Reliability of Earth Slopes,* Journ. of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT11, November 1977.

Whitman, R.V., *Evaluating Calculated Risk in Geotechnical Engineering,* Journ. of the Geotechnical Engineering Division, ASCE, Vol. 110, No. 2, February 1984.