DEVELOPING A RISK ASSESSMENT FRAMEWORK FOR SAFETY EVALUATION OF EARTH DAMS IN SRI LANKA

Sothilingam Premkumar

(11/8012)

Degree of Master of Science

Department of Civil Engineering

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DECLARATION

I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree in any other university of institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published by another person except where the acknowledgement is made in the text.

Signature: .... Date: ....

The above candidate has carried out research for the Masters thesis under my supervision.

Signature of Supervisor: .... Date: ....

ABSTRACT

Sri Lanka has a rich history of earth dam construction with over 300 large and medium scale dams and over 12000 small scale earth dams currently in service. According to ICOLD (International Commission of Large Dams) classification, there are 76 large dams in Sri Lanka. A vast majority of those earth dams were built several centuries ago and limited scientific investigations have been conducted on the performance of such ancient earth dams from a geotechnical point of view.

After serving the nation for centuries, a large numbers of ancient earth dams are suffering partial failures due to excessive seepage, piping, slope instability, and excessive lateral deformations and cracking due to vibrations caused by heavy vehicles and tremors. No regular monitoring schemes were implemented to investigate the mechanisms of above failures.

The quantitative risk assessment seeks to enumerate the risk in terms of likelihood (probability) and consequences. The probability of failure for each mode involves engineering assessment of the particular failure mechanisms, and looking for solutions that can reduce the probability of those failure modes or minimize the consequences of a failure. There is no standard framework adopted in Sri Lanka for the risk assessment process of earth dams.

The main objectives of this report are to propose a quantitative risk assessment framework for safety evaluation of earth dams in Sri Lanka and to apply the developed risk assessment framework to an ancient earth dam of Sri Lanka to investigate its performance under different conditions. Here, as a case study, initial level risk assessment has been done for Nachchaduwa dam, using the developed framework. The critical loading conditions which are relevant to Sri Lanka were included in the study.

Nachchaduwa is an ancient tank, which was built 17 centuries ago to supply water for irrigation purposes. It was restored in 1906 and improved in 1917 by the Irrigation Department of Sri Lanka. According to an investigation carried out by Dam Safety and Water Resource Planning Project (DSWRPP), Nachchaduwa dam is selected as one of the dams with a higher risk of failure with some signs of excessive seepage and slope instability along the dam embankment. Risk assessment can provide valuable

information on the risk reduction measures and benefits of structural and non-structural risk reduction options. In addition, risk assessment outcomes can strengthen the case for funding capital improvements, additional investigations, and on-going dam safety activities, such as monitoring and surveillance and emergency management.

This report produces a quantitative risk assessment framework to be used for any type of earth dams in Sri Lanka and summarizes the risk assessment process, results, findings and recommendations for Nachchaduwa dam.

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It is with a sense of gratitude that we recall and appreciate the assistance received from different personnel to make this research a reality

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INTRODUCTION

CHAPTER 1

1.1 Overview

Sri Lanka has a rich history of earth dam construction. According to a survey conducted by Irrigation Department of Sri Lanka, there are 307 large and medium scale earth dams and over 12000 small scale earth dams currently in service. According to ICOLD (International Commission of Large Dams) classification, there are 76 large dams in Sri Lanka. Almost all of those earth dams were built by the great kings who lived centuries ago.

Currently, there is no central governing body for dam management. The authority is divided among Irrigation Department, Mahaweli Authority, Agrarian Development Department, Provincial councils, Ceylon Electricity Board, National Water Supply and Drainage Board and farmer organizations. However, there is no common structure in the above institutions regarding earth dam management. Irrigation Department manages a regional set up to cover the whole island. The department has practices and procedures developed over decades for earth dam management. Mahaweli authority is mainly governing the dams' constructed using modern technology. They have in house and independent inspection teams. Other institutions mentioned above have no proper mechanism for dam safety inspection. Maintenance and rehabilitation of earth dams managed by them is purely based on experience. It should be emphasized that, in all the above institutions, priority is given for water storage.

Therefore, limited scientific investigations have been conducted on the performance of ancient earth dams in Sri Lanka from a geotechnical point of view. Large numbers of ancient earth dams are suffering partial failures due to excessive seepage, piping, slope instability, and excessive lateral deformations and cracking due to vibrations caused by heavy vehicles and tremors. Currently, it seems that, constructing berms and repairing cracks based on previous experience is the only solution adopted by the governing organizations to address the underlying geotechnical issues.

No regular monitoring schemes were implemented to investigate the mechanisms of above failures. There are no national standards for design, construction, maintenance and rehabilitation of earth dams in the country. In addition, there is no national mechanism and standard for dam safety assessment and management as well. Lack of technical competency, financial limitations, ambiguity over ownership and authority, where priority is always for water storage has hindered the research and development in above areas.

Hence, it is a timely research with national importance to thoroughly investigate the performance of ancient earth dams from a geotechnical point of view and apply the findings to develop a framework for safety evaluation of these dams. Qualitative and Quantitative types of risk assessment are widely in practice around the word for dam safety assessments. Here, the guidelines on quantitative risk assessment for individual dams were developed. The overriding need for quantitative risk assessment is that the risk estimation procedures are logically correct and based so far as possible on accepted scientific knowledge. A main reason for doing quantitative risk assessment is that it allows comparisons of risk over a portfolio of dams.

Fully quantitative risk assessment seeks to enumerate the risks in terms of probability of failure and consequences. With the move to a risk based approach to dam safety there has been a concomitant focus on estimating the probability of failure of dams. The quantitative risk assessment allows assessment and ranking of the likelihood of failure and/or the risks of various components within the system. The quantitative risk assessment comprises the steps of risk identification, estimation, and evaluation.

The failure of a particular component of a dam under different loading conditions (e.g. Normal operating load, flood load and earthquake) involves various failure modes. Historic performance and event tree are the two broad categories of methods in use for estimating probability of failure. Potential consequences resulting from an uncontrolled release of a reservoir have several different dimensions, so the overall dam failure scenarios should be considered. Any of the dam failure could be a risk factor to Population in the inundation area. So priority should be given to the life safety consequences. Apart from the life safety consequences, financial and economical losses also should be considered.

In quantitative risk assessment there will be uncertainties in the estimation of risk. The need for reporting uncertainty may be less critical for studies that simply aim to rank the relative risk.

1.2 Objective

The objectives of this research are mainly the following:

- Detailed literature review to identify different risk assessment approaches used in other countries.
- Developing a risk assessment framework to be used in the safety evaluation of ancient earth dams in Sri Lanka.
- Applying the developed risk assessment framework to a selected ancient earth dam in Sri Lanka to investigate its performance under different conditions.

1.3 Outline of the Thesis

Here, in this report, the quantitative risk assessment framework was developed by considering the condition of earth dams in Sri Lanka. The suitable methodologies for estimating probabilities and consequences for given failure modes of a dam under different loading conditions are briefly discussed. In terms of risk evaluation, the tolerable risk criteria have been adopted based on widely accepted values currently in use around the world. Priority is given to the life safety consequences. The developed guidelines can be used for initial to very detailed analysis.

Dam safety and water resources planning project (DSWRPP) has done a risk assessment for Sri Lankan earth dams. According to their report there are 32 critical dams in Sri Lanka and Nachchaduwa is one of them. So, as a demonstration of the developed guidelines, a case study was done for Nachchaduwa dam. Due to the limited availability of data and time constraint, the initial level risk assessment process which is based on proper assumptions was selected. Apart from these, a case study has been done for Teton dam, which was failed in 1976.

This report consists of eleven chapters. The first two chapters are the introduction and literature survey respectively. The third chapter describes the framework for quantitative risk assessment, while the next five chapters discuss briefly on risk identification, estimation of likelihood of failure, estimation of consequences, reporting the risk and risk evaluation, respectively. The ninth and tenth chapters contain the case study for Nachchaduwa dam and Teton dam. Finally the conclusion is given under chapter eleven.

BACKGROUND

CHAPTER 2

2.1 Introduction

The origins and evolution of dam safety risk assessment can be traced back to a variety of engineering, societal considerations; and public policy and business issues. The 1972 failure of Buffalo Creek Dam led to the National Dam Inspection Act and the authorization by the Congress of the United States Army Corp of Engineers (USACE) to inventory dams located in the U.S. This resulted in the identification of some 2,900 unsafe dams of which 2,350 were found out to have inadequate spillways. Thus the early interest in applying risk-based approaches dates back to the study of ASCE Task Committee on the "Re-evaluation of the Adequacy of Spillways of Existing Dams" in 1973. Then the Teton Dam failure and later Taccoa Falls Dam failure led to an Executive Order that instructed federal agencies to explore risk-based approaches in their process of site selection, design, construction, and operation (John et al 2004).

In the latter part of the 1980s, the US Bureau of Reclamation (USBR) introduced guidelines for incorporating the results of risk analyses into the decision-making process (USBR 1989). During the 1990s, the use of risk-based procedures gained momentum. Later, the Water Resources Development Act of 1986 authorized USACE to maintain and periodically publish an updated National Inventory of Dams (NID). The Water Resources Development Act of 1996 established a National Dam Safety Program and named FEMA as its coordinator. It also required the reorganization of the Inter-agency Committee on Dam Safety (ICODS).

For federal agencies, the regulatory basis for the use of risk based prioritization decision methodologies was initiated with Executive Order 12866, "Regulatory Planning and Review" issued by the Office of the President on September 30, 1993, and its companion document, "Economic Analysis of Federal Regulations Under Executive Order 12866" issued by the Office of Management and Budget (OMB) on January 11, 1996. The Executive Order and the OMB implemented document, mandated promulgation of formal regulatory requirements by Government agencies and the encouragement of developing guidelines, and using risk based prioritization approaches in their investment decisions. With the encouragement of OMB, federal agencies developed guidelines using risk as a prioritization decision tool. Risk based guidelines developed by the Department of Transportation's Federal Aviation Agency, and the Department of Energy for their acquisitions investment analysis procedures are useful documents and are relevant to the purposes of this project (John et al 2004).

Meanwhile, the use of risk analysis to evaluate proposals for any major rehabilitation of water resources was initiated within the USACE in 1991. Thus, the Corps adopted a more methodical risk analysis approach to the engineering and economic evaluation of all flood damage reduction projects it plans and builds. Later, with the encouragement of OMB, the USACE (1996) recognized that major rehabilitation is an investment to avoid future increased operating and emergency repair costs and losses, and thus developed an economic-based decision framework that borrows heavily from the methods of risk analysis combined with probabilistic benefit-cost analysis.

By the middle of the 1990s, the Australian Committee on Large Dams (ANCOLD 1994) published guidelines on dam safety that explicitly addressed tolerable life loss risk criteria based on nuclear power and industrial facility risk practices, mirroring similar work that had been published by BC Hydro (but was subsequently abandoned in 1997). Starting in 1995, USBR developed risk assessment procedures and is currently one of the largest users of risk based methodologies.

In Canada, research was undertaken by Hatch Energy (then Acres International) to develop a computerised, risk-based procedure to assist in decisions with respect to the optimum timing and alternatives for competing rehabilitation options (Donnelly and MacTavish 1997; Westermann 1998; de Meel et al. 1998). During this same period, BC Hydro adopted a qualitative, risk-based approach to assist in the assessment of complex dam safety issues (Nielson 1993; Salmon and von Hehn 1993; Nielson et al. 1994; Salmon and Hartford 1995).

In 2003, the Australian Committee on Large Dams (ANCOLD 2003) has upgraded the guideline published in 1994. The 1994 ANCOLD Guidelines on Risk Assessment set out the conceptual foundations of risk assessment, as understood at the time and the 2003 Guidelines were directed to the practical application of risk assessment, as an aid to better dam safety management.

2.2 Risk

Risk is a measure of the probability and severity of an adverse effect to life, health, property or the environment. Generally risk is estimated by the combined impact of all triplets of scenario, probability of occurrence and the associated consequence.

$$\textit{Risk} = \textit{Probability of Failure} \times \textit{Consequences}$$

2.3 Risk Assessment

Risk assessment is the process where the understanding of the risk (Risk Analysis) is compared to societal tolerated risks of a similar nature (Risk Evaluation), allowing a decision regarding the requirements for control of the risk. The decision may involve consideration of legislated requirements, codes and standards, authoritative good practice, engineering judgement, risk based analysis, societal values and expectations, and the owners' own values.

Depending on the decision, risk reduction measures may be needed. Risk assessment is an ongoing process, with periodic review of risks to ensure they remain tolerable.

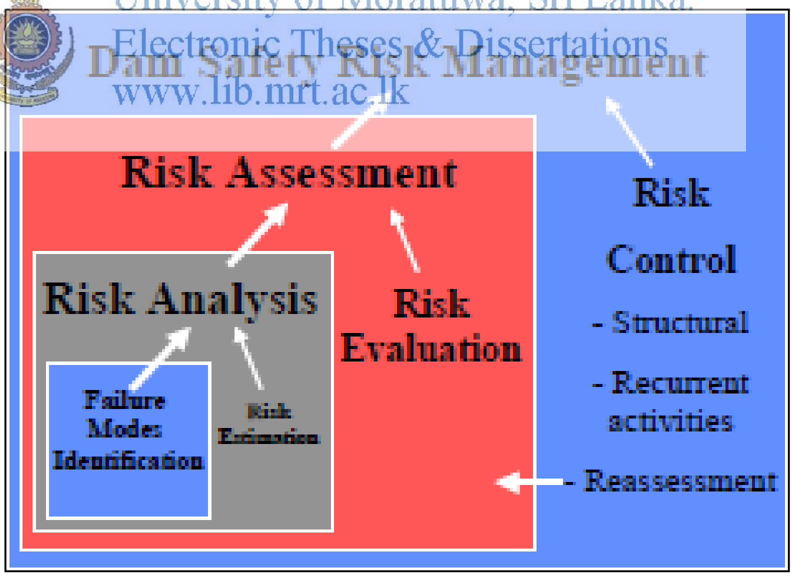


Figure 2.1: Interrelationship between components of risk assessment and risk management (Bowles et al 1999)

2.4 Risk Assessment Methods

There are several methods available to analyze risk depending on the desired output. Some of the more common methods are summarized in Table 2. **1**. In general, risk assessment methods may be considered to be qualitative or quantitative, although some methods such as Fault Tree Analysis (FTA) provide enough flexibility to be used for either approach.

A qualitative method is an approach which relies mostly on tables and descriptors including expert knowledge to assess the risks of a system. Qualitative methods provide a general sense of the major risks which, once ranked in likelihood of occurrence or severity of consequences, can then be more closely analyzed using quantitative methods and compared with acceptable risk criteria. Oftentimes, however, risks identified using qualitative methods can only be relatively compared to one another. As a result, qualitative methods do not provide an absolute value for the risks considered and lack the capacity to compare risk levels between different sources (Mayrai et al 2007)

A quantitative approach, on the other hand, relies on point estimates to assess system risk and performance (Mayrai et al 2007). For event tree or fault tree analysis, for example, probabilities of occurrence are estimated based on the available information and assigned to each branch to reflect the best estimate of the likelihood of an occurrence to a particular outcome.

Table 2. 1: Summary of risk assessment methods (Mayrai et al 2007)

Method	Abbreviation	Qualitative or Quantitative
Preliminary Hazards Analysis	PHA	Qualitative
Failure Mode and Effects Analysis	FMEA	Qualitative
Hazard and Operability Studies	HAZOP	Qualitative
Failure Mode Identification	FMI	Qualitative
Management Oversight Risk Trees	MORT	Qualitative
Safety Management Organization Review Technique	SMORT	Qualitative
Failure Mode and Effect and Criticality Analysis	FMECA	Quantitative
Probable Failure Mode Analysis	PFMA	Quantitative
Cause Consequence Analysis	CCA	Quantitative
Fault Tree Analysis	FTA	Quantitative

2.5 Levels of Risk Assessment

ANCOLD guidelines on risk assessment, describes four different levels of risk assessment as:

- Screening;
- Preliminary;
- Detailed;
- Very detailed.

The following tables give supplementary guidance to enable better understanding of the intent of the levels given above.

Table2.2: Levels of risk assessment (ANCOLD, 2003)

Level	Type	Engineering Inputs	Estimation of Probability of Failure	Estimation of Consequences	Risk Evaluation Method
Screening	Qualitative or Quantitative	Basic	Screening to preliminary	Basic to Moderate	Basic
Preliminary	Quantitative	Moderate to basic	Preliminary	Moderate	Moderate to basic
Detailed	Quantitative	Advanced to moderate	Detailed	Advanced to moderate	Detailed to moderate
Very detailed	Quantitative	Advanced to very advanced	Very detailed	Advanced to very advanced	Detailed or very detailed

Screening and preliminary studies should only be used to rank risk, or to get early identification of issues not found by standard based approach (ANCOLD, 2003)

Table2.3: Levels of engineering inputs to risk assessment (ANCOLD, 2003)

Levels	General Description of typical issues
Basic	• Assemble readily available design, construction, monitoring and surveillance data and report. • Use and adapt existing flood and earthquake studies. • Use existing analysis of embankment stability or used judgement. • Assess filters and piping based on existing data. • Assess liquefaction by presence of liquefiable materials and judgements using existing data. • Assess concrete dam stability using judgement, existing calculations, or basic calculations. • Assess gates, valves by judgement.
Moderate	• Detailed search for and assembly of design, construction, monitoring and surveillance data and report • Flood studies to modern standards. • Reassess embankment stability using existing data. • Assess filter and piping in detail using existing data minor additional data. • Assess liquefaction by H. B. Seed type methods to give AEP of liquefaction using existing data or limited additional data. • Assess concrete dam stability under flood loading using conventional analysis with estimated properties for dam and foundation. For earthquake use spectral analysis pseudo static. • Assess gates and valves reliability by historic performance data and judgement.
Advanced	• Detailed search for and assembly of design, construction, monitoring and surveillance data and report • Flood studies to modern standards • Assess embankment stability and potential post failure deformations in detail, with new investigations of conditions if needed. • Assess filters and piping in detail using existing data supplemented by sampling and testing of as-constructed materials as necessary. Carry out erosion/filter testing if needed. • Assess liquefaction by H. B. Seed type methods to give AEP of liquefaction and post liquefaction stability, using existing and additional data as necessary. • Assess concrete dam stability under flood loading using conventional analysis with measured or estimated, uplift pressures and foundation properties investigated in details. For earthquake, use rigid block/Newmark analysis. • Assess gates and valves by basic reliability analysis as needed
Very Advanced	• Detailed search for and assembly of design, construction, monitoring and surveillance data and report • Flood studies to modern standards • Assess embankment stability and potential post failure deformations in detail, with new investigations of conditions if needed. • Assess filters and piping in detail using existing data supplemented by sampling and testing of as-constructed materials as necessary. Carry out erosion/filter testing if needed. • Assess liquefaction by H. B. Seed type methods to give AEP of liquefaction and post liquefaction stability, using existing and additional data as necessary. Post failure deformations may be estimated numerically, or dynamic numerical analysis carried out. • Assess concrete dam stability under flood loading using conventional analysis with measured or estimated, uplift pressures and foundation properties investigated in details. For earthquake, use rigid block/Newmark analysis, modelling the uncertainty in the parameters. If critical, use linear or non-linear dynamic numerical analyses. • Assess gates and valves by basic reliability analysis as needed.

Table2.4 : Levels of consequence assessment for risk assessment (ANCOLD, 2003)

General Description				
Level	Dam Break Scenarios	Downstream Inundation Assessment	Life Safety Consequence Assessment	Economic and Financial Consequences Assessment
Basic	Single sunny day and flood related scenarios.	Empirical breach hydrograph, flood peak routed or simple one dimensional flood inundation model.	LOL (Loss of Life) estimated by engineering judgement based on PAR (Population at Risk).	Often not quantified.
Moderate	One or more scenarios for important dam components and reservoir flood levels.	Empirical breach hydrograph, simple one dimensional flood inundation model.	Estimate LOL from PAR, warning times using empirical formulae and judgement.	Often not quantified formally. If quantified usually only direct consequences are estimated.
Advanced	One or more scenarios for important dam components and reservoir flood stages.	Empirical breach hydrograph, one or two dimensional flood inundation model, with antecedent and tributary flows.	Estimate LOL from PAR, warning times using empirical formulae, judgement, and assessment of PAR, evacuation routes, and emergency management procedures.	Direct financial and economic consequences estimated in some detail, indirect consequences estimated approximately or not at all.
Very Advance	More than one scenario for important dam components and reservoir flood stages.	Empirical breach hydrograph or mathematical model, two dimensional flood inundation model, with several breach models, and tributary and antecedent flows.	Estimate LOL from PAR, warning times using empirical formulae, judgement, and detailed assessment of PAR, evacuation routes, and emergency management procedures.	Direct and indirect financial and economic losses estimated in detail.

Table 2.5: Risk evaluation methods (ANCOLD, 2003)

Level	General Description
Basic	Qualitative or life safety – societal and individual – risk, using ANCOLD and USBR criteria to rank risks and identify high risk failure modes.
Moderate	Life safety – societal and individual – risk, using ANCOLD and USBR criteria to rank risks and identify high risk failure modes. Assess direct economic and financial risks to identify high business risk failure modes.
Advanced	Life safety – societal and individual – risk, using ANCOLD and USBR criteria to rank risks and identify high risk failure modes. Assess direct economic and financial risks to identify high business risk failure modes. Remedial works assessed in terms of cost to save a statistical life.
Very advanced	Life safety – societal and individual – risk, using ANCOLD and USBR criteria to rank risks and identify high risk failure modes. Assess direct economic and financial risks to identify high business risk failure modes. Remedial works assessed in cost per statistical life saved terms. Analysis is likely to include modelling uncertainly of the inputs.

2.6 Key Participants in the Study

The **owner** – who is legally responsible for dam safety, and must take responsibility for the answer to the question: Are the risk tolerable;

A **decision-maker** – may sometimes act on behalf of the owner, in taking the results of the risk analysis and the decision recommendation of the analysis team, and deciding what actions, if any, should flow from that information and from other relevant considerations. The owner and decision-maker may sometimes be the one party;

The **analysis team** – which is responsible for undertaking the risk analysis and for the soundness of the results. The owner needs to employ an analysis team with knowledge and skills appropriate to the purpose of the study;

The **regulator** of dam safety – if there is one – is not necessarily a direct participant, bit will often have set minimum requirements relation to those risk that s\affect the interests of the community. The regulator’s acceptance of risk reduction options may also be required.

2.7 Available Risk Assessment Framework

2.7.1 Risk assessment framework of ANCOLD

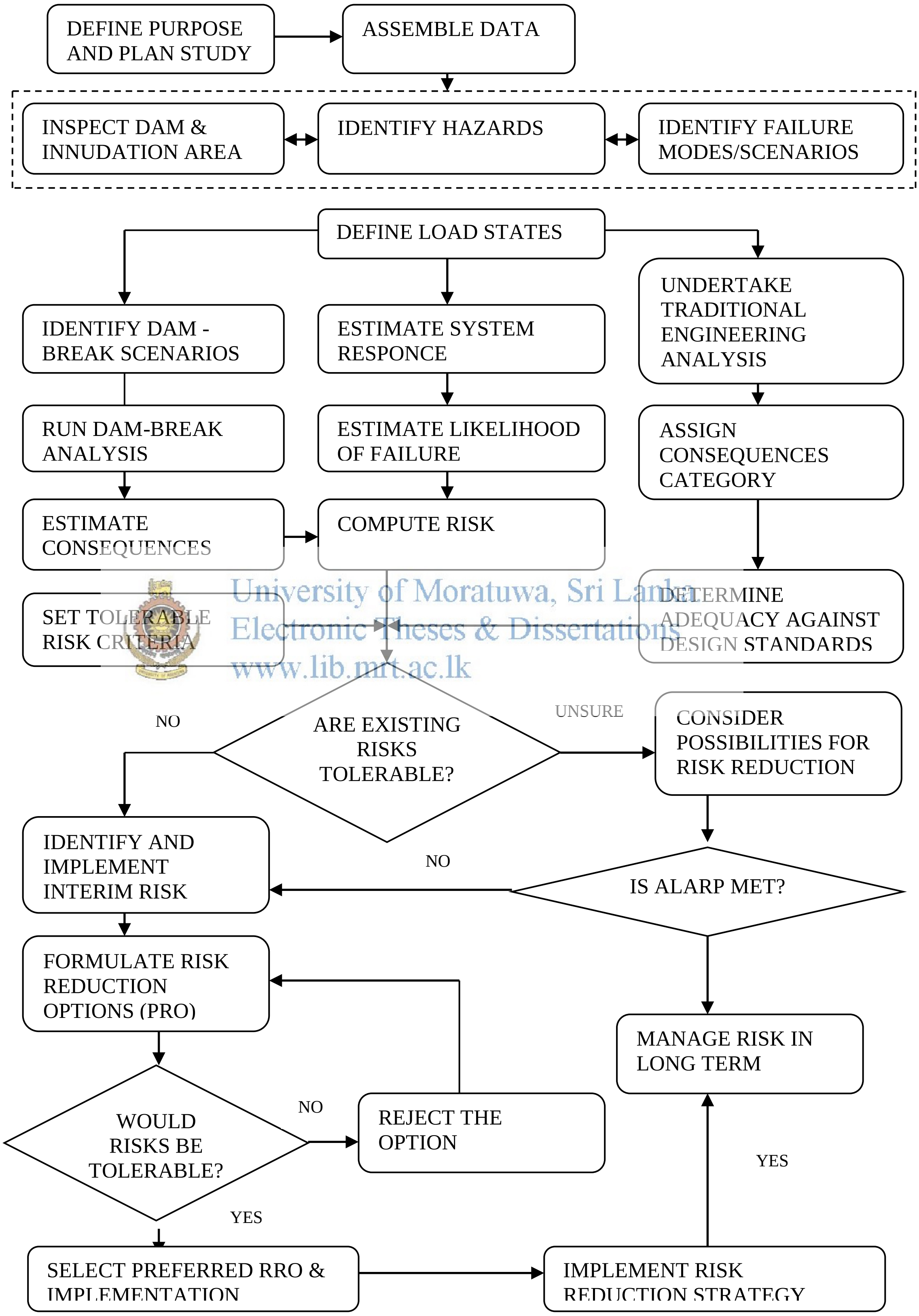


Figure 2.2: Typical risk assessment process for a dam (ANCOLD, 2003)

2.7.2 FEMA framework

Table 2.6: Quantitative risk assessment framework of FEMA

	Initiating Event	System Response	Outcome (Breach/ No Breach)	Exposure	Consequence
1) Risk Identification	External: Earthquake Upstream Dam Failure Internal: Piping	Overtopping Deformation Slope Instability	Breach No Breach	Time of Day Season Warning Time	Economic Damage Loss of Life Environmental Social
2) Risk Estimation	Loading Problem →	Response Problem →	Outcome Problem →	Exposure Problem →	Losses
3) Risk Evaluation	Using Event Tree Method or Historical Method				
4) Risk Treatment	Upstream Watershed Changes Upstream Dam Improvements	Structural Modification Safety Inspections Instrumentation Operating Restrictions	Structural Modifications	Warning Systems Flood Proofing Emergency Preparedness	Relocation Land Use Zoning

2.8 Documents Needed for Risk Assessment Study

The following are typical of the documents, which should be obtained (ANCOLD, 2003):

- Scheme option report;
- Concept design reports;
- Geological reports;
- Site investigation reports;
- Materials investigation reports;
- Design reports;
- Design calculation folders;
- Records of discussions with designers;
- Environmental impact statements;
- Construction reports;
- Construction photograph;
- Geological record reports;
- Geological mapping;
- "as-built" drawings;
- Full details of any modifications to the dam;
- Safety inspection reports (routine, annual and comprehensive);
- Instrumentation monitoring records and plots;
- Operation logs;
- Safety review reports;
- Incident reports.

2.9 Hazards for Earth Dams

ANCOLD guidelines on risk assessment summarize the following hazards for earth dams (ANCOLD, 2003):

Obvious hazards are:

- The storage water is itself a hazard, given that the dam is an imperfect container (hence the need to consider failure modes under normal operating conditions);
- Floods;
- Earthquakes.

Other hazards are less obvious:

- Operator error;
- Vandalism is conceivable, and is usually sufficiently likely to require analysis;
- Inadequacy of maintenance;
- Security breaches;
- Terrorism is conceivable;
- Act of war is conceivable;
- Fire is conceivable, and is credible for electrical equipments;
- Reservoir rim landslide is conceivable and would warrant analysis for some dams;
- Lightning strike, particularly as regards its effects on vital operating, control or monitoring equipment;
- Wind, particularly set-up and seiche effects;
- Barometric pressure in regards to seiche effects for large reservoirs;
- Upstream dams, in regard to both their operations (for example, emergency releases) and potential for failure;
- Upstream natural landslide dams, as regards potential for failure;
- Impact of a large air craft is conceivable, but is extremely unlikely except in areas of high air traffic (for example, approach to a major airport);
- Tsunami is conceivable for some low dams in coastal areas, but likelihood needs consideration.

2.10 Failure Modes Analysis

The Failure Mode Effect and Analysis (FMEA) process is descriptive and qualitative and provides the engineers a comprehensive understanding of the dam. The process is described in more detail by Hartford (1999) and summarized as follows (Raymond. A. S):

"FMEA is a very versatile design-based tool with significant scope for application in dam risk management. The process is aimed at systematically developing a picture of the dam system, its components and their interactions, and presenting details of how component failure could lead to system failure, the magnitudes of the failure effects and the criticality of the various components in preventing the risks from materializing.

Failure Mode, Effect and Criticality Analysis (FMECA) extend FMEA to provide a means of ranking the failure modes in terms of an index of risk that incorporate representations of probability and consequences. This provides a sound basis for prioritizing corrective or remedial actions. While the general nature of the FMEA worksheet can be established and made transportable from one situation to the next, it may be necessary to tailor the generic worksheet for individual situations, to incorporate the necessary detailed information.

A comprehensive FMEA/FMECA can be expected to generate a very large number of potential failure modes. In a well designed and maintained system, the analysis can be expected to demonstrate that the potential failure mode has been 'designed out' of the system or controlled in some other manner. Since the analysis is required to first identify all significant potential failure modes and then identify all compensating provisions, FMEA/FMECA often requires a great deal of time and a very significant resource commitment.

From a technical perspective, the analysis can become extremely complex if the effects of multiple failures are taken into account. Much time and effort is often but unavoidably expended on the analysis of failure modes that have a negligible effect on the performance of the system." As Einstein said "A theory should be as simple as possible but no simpler" (Morgan & Henrion, 1990, p.289).

Table 2.7: Example of a failure mode effects an analysis (FMEA) worksheet (ANCOLD, 2003)

Component	ID Number	Primary Function	Auxiliary Functions	Failure Modes	FM No	Causes	Failure Effect		Failure Detection	Mitigating Action	Severity
							Local Effect	End Effect			

2.11 Categories of Methods for Estimating Probability of Failure

There are two broad categories of methods available for estimating probability of failure (Fell et al, 2000):

a) Historic performance methods

These methods use the historic performance of dams similar to the dam being analyzed to assess a historic failure frequency, and assume that the future performance of such dams will be similar. In some cases, the performance of dams during first filling, or in the first 5 years, is separated from later performance. These methods do not directly account for the reservoir loading, including normal operating loads or floods, nor do they allow for the detailed characteristics of the dam or for the ability of those responsible for the operation of the dam to detect a problem developing and to intervene. Generally speaking, these methods are only applicable for initial or portfolio risk assessments, and for checking more detailed event tree methods, and should not be used alone for detailed assessments (Fell et al, 2000).

b) Event tree methods

Event tree methods have the advantage that the mechanics of the failure, from initiation to breach can be modelled; as can the reservoir level, the details of the dam and its foundation and the ability to intervene to prevent breaching. However, as discussed below, sometimes there is little objective basis for estimation of the conditional probabilities within the event tree and much subjective judgement is needed. It may therefore be necessary to relate back to historic performance data as a "credibility check" on the answers (Fell et al, 2000).

c) Fault tree methods can be useful for representing logical combinations of system states and possible causes that contribute to a specified event (top event) in a dam system. They are particularly well suited to the representation of mechanical and electrical systems such as spillway gates (Fell et al, 2000).

d) Deterministic analyses; Care must be taken in selecting inputs for deterministic analyses and interpreting their results when their results are to be used to support estimates of probabilities in a risk analysis. They can be used, for example, for estimating a threshold of failure of a concrete gravity dam. It is important to use best estimates of loadings and properties, not the conservative ones usually used for design (Fell et al, 2000).

e) Stochastic analyses, including Monte Carlo approaches, are not widely used in practical dam safety risk analyses at this time. However, they can be used to estimate, at least in a partial sense, the uncertainties associated with estimated probabilities (Fell et al, 2000).

f) Judgement informed by information obtained from the preceding categories. Judgement is unavoidably woven into the fabric of all dam safety investigations and analyses whether they are performed under a traditional deterministic framework or a risk-based framework. It is also the basis for combining information from the different categories (Fell et al, 2000).

2.12 Flood Routing Studies for Serial Dam Failure

When dams are located in series, an upstream dam can be both a threat and a means of protection to a downstream dam. Typically, at lower flows associated with a regional or local run off event, an upstream dam will safely pass floods with some attenuation that will thereby reduce the magnitude of the flood imposed on downstream dams. However, at higher flows, the possibility of failure of the upstream dam may exist, and that would usually lead to outflows that are higher than either natural or no-failure flows under the same runoff conditions. Thus, inflows to downstream dams will be increased and the likelihood of their failure may also increase.

2.13 Evaluating the Risks

There are two main approaches to societal risk criteria;

- F-N lines;
- Expected annual life loss values (fxN).

where,

"f" – Estimated probability of occurrence of each overall failure scenario

"N" – Corresponding estimated number of lives that would be lost

"F" – Cumulative distribution function, the estimated annual probability of a failure expected to result in the loss of "N" or more lives.

ANCOLD follows the F-N lines approach while the USBR (1997) follows the expected value approach.

Table 2.8: Summary of risk evaluation criteria (Bowles et al, 1999)

Risk Evaluation Type			Rating Code	Explanation
Life Safety-Societal Risk	ANCOLD (1998) Interim Amended Societal Risk Criteria *(for all failure modes combined)*	Limit	N	Does not meet limit criterion - F-N plots above limit criterion
			Y	Meets limit criterion - F-N plots below limit criterion
		Objective	N	Does not meet objective criterion - F-N plots above objective criterion
			Y-ALARP?	Meets objective criterion, but ALARP (As Low As Reasonably Practicable) must be evaluated - F-N plots below objective criterion
	USBR (1997) Interim Tier 1 Public Protection Guidelines *(for flood, earthquake and static failure modes separately)*		N-Strong L&S	Strong justification for long- and short-term risk reduction measures – Expected incremental loss of life exceeds 0.01 lives/year
			N-Strong L	Strong justification for long-term risk reduction measures - Expected incremental loss of life between 0.01 and 0.001 lives/year
			Y-ALARP?	Diminished justification for long-term risk reduction measures (i.e. ALARP must be evaluated) – Expected incremental loss of life less than 0.001 lives/year
	USBR (1997) Interim Tier 2 Public Protection Guidelines *(for total of failure modes)*		N	Increasing justification to reduce probability of failure - Probability of failure exceeds 1 x 10 $^{-4}$ /year
			Y-ALARP?	Decreasing justification to reduce probability of failure (i.e. ALARP must be evaluated) – Probability of failure less than 1 x 10 $^{-4}$ /year
	BC Hydro (1993) Interim Societal Risk Criteria *(for total of failure modes)*		N	Does not meet criterion - Expected incremental loss of life exceeds 0.001 lives/year
			Y-ALARP?	Meets criterion, but ALARP must be evaluated - Expected incremental loss of life less than 0.001 lives/year
Life Safety– Individual Risk	ANCOLD (1994) Average over PAR *(for total of all failure modes)*	Limit	N	Does not meet limit criterion – Incremental probability exceeds 1 x 10 $^{-5}$
			Y	Meets limit criterion - Incremental probability less than 1 x 10 $^{-5}$
		Objective	N	Does not meet objective criterion - Incremental probability exceeds 1 x 10 $^{-6}$
			Y-ALARP?	Meets objective criterion - Incremental probability less than 1 x 10 $^{-6}$, but ALARP must be evaluated
	ANCOLD (1994) Person at most risk *(for total of failure modes)*	Limit	N	Does not meet limit criterion - Incremental probability exceeds 1 x 10 $^{-4}$
			Y	Meets limit criterion - Incremental probability less than 1 x 10 $^{-4}$
		Objective	N	Does not meet objective criterion - Incremental probability exceeds 1 x 10 $^{-5}$
			Y-ALARP?	Meets objective criterion - Incremental probability less than 1 x 10 $^{-5}$, but ALARP must be evaluated
	BC Hydro (1993) Interim Person at most risk *(for total of failure modes)*		N	Does not meet criterion - Incremental probability exceeds 1 x 10 $^{-4}$
			Y-ALARP?	Meets criterion - Incremental probability less than 1 x 10 $^{-4}$, but ALARP must be evaluated
Economic/ Financial	NSW Total Asset Management Risk Example Guidelines *(for flood, earthquake and static failure modes separately)*		N-Major	Major risk - Imperative that risk reduction be implemented
			N-Medium	Medium risk - Risk reduction required in a reasonable time
			Y-ALARP?	Low risk - Risk reduction to be ALARP

FRAMEWORK FOR QUANTITATIVE RISK ASSESSMENT

CHAPTER 3

3.1 Define Type and Level of Risk Assessment

The Australian standard on risk management (SA/NZS, 1999) describes three types of risk analysis (ANCOLD, 2003):

- *Qualitative analysis* – uses word form or descriptive scales to describe the potential consequences and the likelihood they will occur,
- *Semi-quantitative analysis* – the qualitative scales are given numeric values, but these do not have to bear an accurate relationship to the magnitude of consequences or likelihood,
- *Quantitative analysis* – uses numerical values of consequences and likelihood that are intended to accurately reflect their magnitude.

Both qualitative and quantitative types of analysis are typically combined in to one study. For example hazard identification and failure mode analysis are forms of qualitative analysis, but they are essential elements of quantitative analysis. It was selected the quantitative type risk assessment as appropriate for Sri Lankan earth dams.

3.1.1 Quantitative type risk analysis and assessment

Quantitative risk assessment seeks to enumerate the risk in terms of likelihood (probability in terms) and consequences. The quantitative risk assessment allows assessment and ranking of the likelihood of failure and/ or the risks of various components within the system. The quantitative risk assessment comprises the steps of risk identification, estimation, and evaluation. The overriding need for quantitative risk assessment is that the risk estimation procedures are logically correct and based so far as possible on accepted scientific knowledge.

In case where initial level (screening or preliminary) risk analysis demonstrates that risks are obviously and seriously intolerable, timely action to reduce risks may be required.

3.1.2 Levels of quantitative type risk assessment

The level of risk assessment depends on the purpose of the study and the information needed by the decision maker. Levels of risk assessment range over a continuum and there are no unique clear-cut definitions of levels. Different owners vary in the level of detail that they require, but none rely on risk assessment alone for making such decisions. The following guidelines cover the methodologies which satisfy different levels of risk assessment.

3.2 Quantitative Risk Assessment Framework

The framework of quantitative risk assessment comprises the steps as risk identification, estimation and evaluation. The risk identification includes the activities such as inspection of dam and inundation area, hazard identification and failure mode identification. The next step is to estimate the probability of failure and the corresponding consequences, where improved approaches to estimation of probabilities and consequences are needed. The last step is to evaluate whether the risk are tolerable. If risk is not tolerable the proper risk reduction method should be identified and implemented.

The risk assessment process can be sub divided in to two sections as risk analysis and risk evaluation, where risk analysis is the combination of risk identification and risk estimation.

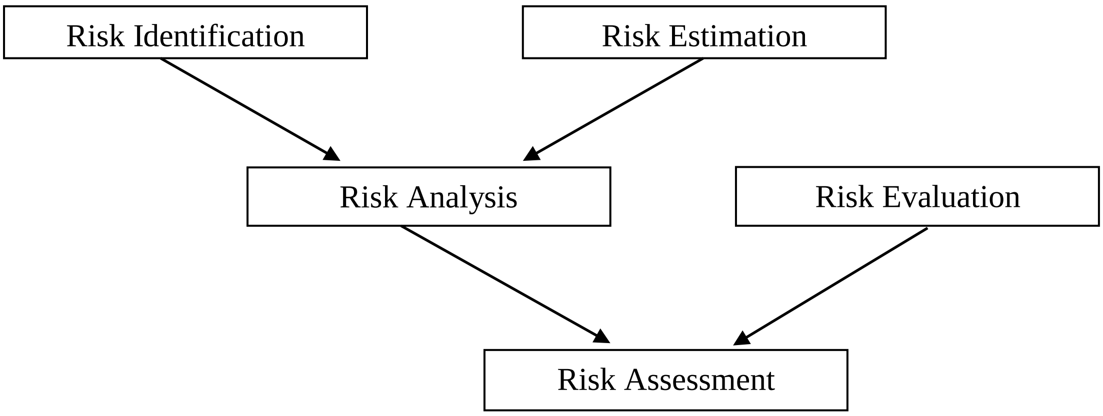


Figure 3.1: Interrelationship between the components of risk assessment

With the above process, the "selection of risk reduction measures" is also included in the frame work. The following sections discuss about all the individual steps in the following quantitative risk assessment framework.

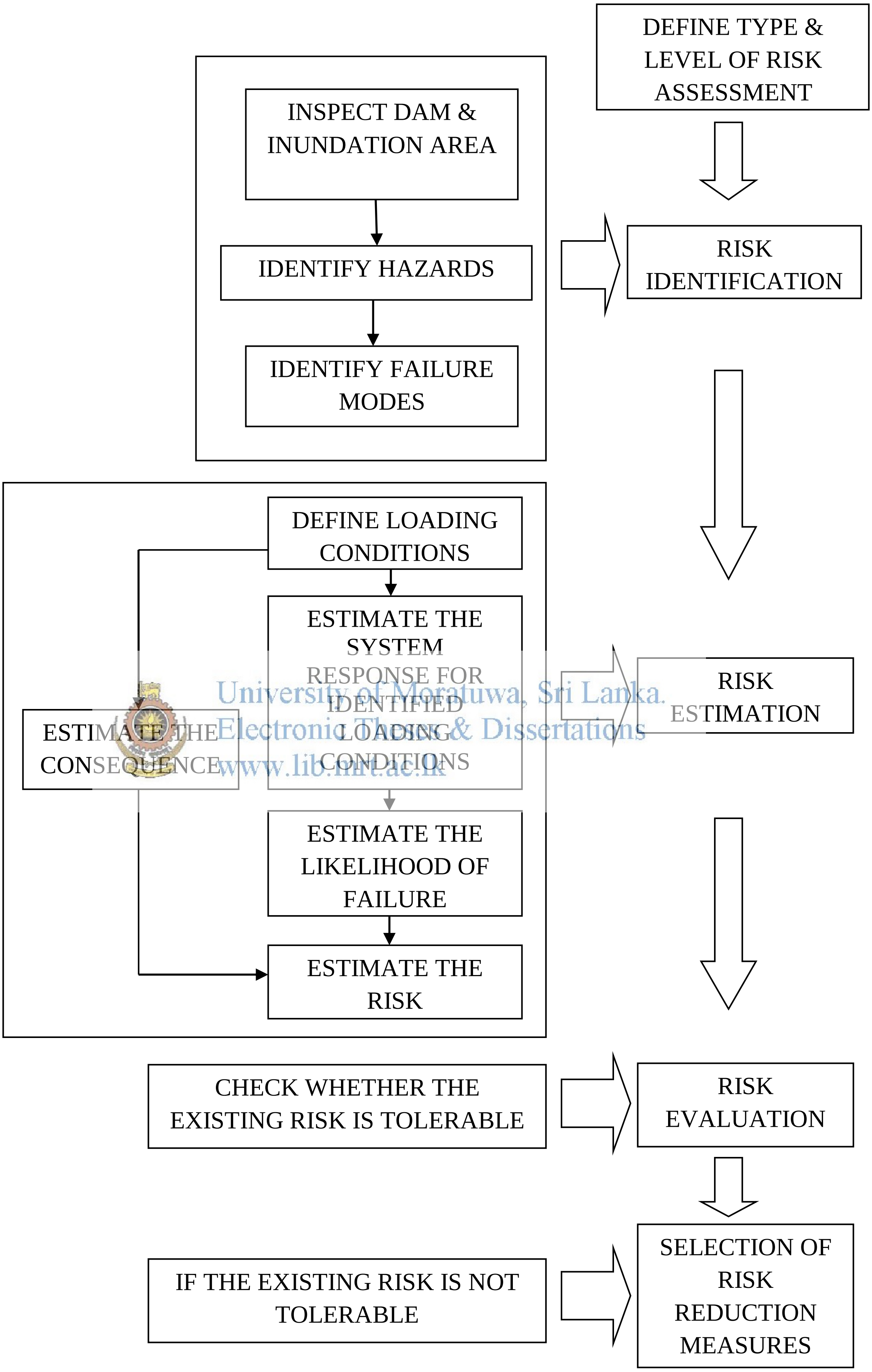


Figure 3.2: Quantitative risk assessment framework for an ancient earth dam

RISK IDENTIFICATION

CHAPTER 4

4.1 Inspection of Dam and Inundation Area

Before starting this step, relevant data should be assembled and their sources should be recorded.

In preparation for the inspection:

- Assemble data on previous dam failures;
- Make an initial list of hazards and failure modes;
- Prepare the inspection check list.

During inspection:

- Systematically work through the inspection checklist and make an on-site record against each item;
- Measure key dimensions.
- Gather necessary information.

Immediately following the inspection, prepare a report, which records all of the matters that were noted and which includes the photographic record. Include the report in the documentation of the study.

The dam-break inundation area should be inspected with the task of estimating consequences in mind. The consequences sub-team should visit local authorities and interest groups to find out details of planning restrictions, expected future developments, areas of heritage or special environmental value and to establish contact for later inquiries.

4.2 Identify the Hazards

This activity is primarily related to quantitative analysis. Based on Standard guidelines on risk assessment around the world the selected obvious hazards for Sri Lankan earth dams are;

- The storage water is itself a hazard, given that the dam is an imperfect container (hence the need to consider failure modes under normal operating conditions)
- Extreme Floods

Data on earthquakes felt in Sri Lanka suggest that earthquakes of magnitude 4 have not occurred in Sri Lanka during historical times for which records are available. However, the possibility of earthquakes of magnitude greater than 4 occurring at these dam sites cannot be ruled out (Welikala). In this guideline, based on the studies and present status of earth dams in Sri Lanka, earthquake loading is considered as less obvious. So normal operating load and flood loads were selected as obvious hazards for earth dams in Sri Lanka.

Most of the Sri Lankan dams are interconnected and failure of an upstream dam may cause other dams failure. But, the failure of upstream dams should not be considered as loading conditions in a risk analysis (USBR, 1999). The risk of multiple dam failures/incident are addressed by assigning the cause of failure to the most upstream dam failure and including the resulting dam failures as consequences for that dam (USBR, 1999).

4.3 Identify the Failure Modes

Identifying the failure modes to be analyzed is the one of the important parts in risk assessment of earth dams. A failure mode is a sequence of system response events, triggered by an initiating event, which could culminate in dam failure. Procedures for failure mode identification vary, but in a typical approach, a small team of dam engineers, who have knowledge of historical dam failure mechanisms, would develop a list of failure modes.

Failure modes analysis can be undertake using systematic and comprehensive process such as FMEA (Failure Modes and Effects Analysis) or FMECA (Failure Modes, Effects and Criticality Analysis) (ANCOLD, 2003). In quantitative risk assessment, the usual process is FMEA; because the later parts of the risk assessment will define criticality. FEMA is a quantitative technique by which the effects of individual component failures are systematically identified.

ANCOLD guideline on risk assessment divides the FMEA in to nine steps as (ANCOLD, 2003);

- Establish the basic principle and corresponding documentation in performing the analysis;
- Define the system which may be defined at various levels;
- Define the components of each sub-system;
- Identify the causes of the failure modes and operating conditions under which the failure can occur;
- Identify the failure modes;
- Identify the effects of the component failure on system considering local and global effects;
- Identify the failure detection method;
- Identify compensating or mitigating provisions including isolation and redundancy;
- Assign the severity classification.

Most important failure modes to be considered for Sri Lankan earth dams are;

- Internal erosion and piping;
- Embankment overtopping;
- Slope instability;
- Spillway and spillway energy dissipation scour, and overtopping of spillway chute wall.

Some failure modes are repeated under number of loading conditions. For example, an embankment dam may have a likelihood of piping under normal operating conditions, with an additional increment of piping risk of piping under flood loading. Some of the failure modes related to earthquake loading has been considered as less critical for Sri Lankan earth dams.

Failure modes should be listed in sufficient detail to capture all of the significant failure scenarios. For example, based on the failure path, internal erosion and piping can be sub divide as;

- Internal erosion and piping through the embankment;
- Internal erosion and piping through the foundation;
- Internal erosion and piping from embankment to foundation.

Furthermore piping through the embankment can be sub-divided into;

- Internal erosion and piping through the dam;
- Internal erosion and piping along or into conduit.

These failure scenarios can be further sub-divided into potential piping process such as; initiation, continuation, progression and breach mechanism in order to identify the causes of the failure modes (Fell et al, 2005).

RISK ESTIMATION – LIKELIHOOD OF FAILURE

CHAPTER 5

5.1 Evaluation of Load States

Loading on the dam needs to be partitioned over the full range of possible loads. The amounts of partitioning of the load states should take account of the type of analysis and the system response to the loads. Preliminary or initial studies will use less partitioning, or may not formally partition the loads.

5.1.1 Normal operating load

A reservoir level-duration relationship is used to estimate the likelihood that normal operating loads will occur in a specified range (Fell et al, 2000). This relationship should be based on a continuous record of water levels, and not peak water levels. It is important that this relationship be representative of operating conditions for the period of time for which the risk analysis is to be carried out.

If operating rules, inflow characteristics, or reservoir release patterns have changed over the life of the reservoir, the historical record should be adjusted, using reservoir simulation, to represent future conditions before the reservoir level-duration relationship is developed (Fell et al, 2000).

Normal operating reservoir levels and flood levels are now commonly combined into the one distribution. For initial level risk assessment, it can be assumed that the reservoir is always at the full supply level (FSL).

5.1.2 Extreme flood load

ANCOLD guidelines on risk assessment divide the flood load evaluation in to three tasks as (ANCOLD, 2003);

i. Production of event magnitude versus frequency/probability curves to define a loading domain.
ii. Partitioning of the loading domains into load states that will be used in the risk analysis.
iii. Identify the load scenarios. One or more load states define a load scenario.

The term loading domain is used to refer to the total range in magnitude of loads, together with their associated probability of occurrence, expressed as a continuous relationship – peak flood discharge versus annual exceedance probability (AEP). Wind effects may cause only a very small increase in likelihood of failure due to flood, if it can be reasoned that there is little or no correlation between peak water level in the reservoir and wind velocity (ANCOLD, 2003).

There are two approaches have been taken for partitioning of the loading domain (ANCOLD, 2003);

- Manual partitioning of the loading domain into a relatively few states – typically 3 to 10;
- Automated partitioning by use of available software to produce a large number of load states.

In manual approach, the load state covers a range of load values is represented by a single value representative load, usually the mean of the portion end point loads, which is the basis for assigning estimated conditional probability of failure (ANCOLD, 2003). An example of manual portioning of an inflow flood domain for quantitative analysis is given in Table 5.1.

Table 5.1 : Manual portioning of inflow flood domain (ANCOLD, 2003)

Partition Point Peak Inflow Discharge (m^3/s)	Partition Point Annual Exceedance Probability	Representative Inflow Discharge (m^3/s)	Annual Probability of Flood with Peak Inflow in Partition
250	1 in 1		
		1725	9.980E-01
3200	1 in 500		
		4475	1.714E-03
5750	1 in 3500		
		7375	2.571E-04
9000	1 in 35000		
		10500	2.571E-05
12000	1 in 3500000		
		12750	1.857E-06
13500	1 in 1000000		
		13500	1.000E-06
Total			0.9999993

In the above table, the two right hand columns define the load states for use in risk analysis. With flood frequency relationship based on annual series, the loading domain should commence at AEP 1 in 1 event, because, load partitions being mutually exclusive, the sum of the annual probabilities of all of the partitions would then be 1.0.

5.2 Estimation of Probabilities

There are two tasks under this guideline:

- To estimate the system response for the flood loading and normal operating conditions – that is, the estimated conditional probability of failure (e.g.: probability of failure for slope instability), given load magnitude;
- To estimate the annual likelihood of some load condition or event that could initiate a failure mechanism.

At this time there is no widely accepted method for estimating probability of failure for dams. The probabilities should be estimated with considering the range of accuracy in mind.

The following, two broad categories of methods are most suitable for the estimation of probabilities of failure:

a) Historic performance methods

These methods use the historic performance of dams similar to the dam being analysed to assess a historic failure frequency, and assumes that the future performance of such dams will be similar. These methods do not directly account for the reservoir loading, nor do they allow for the detailed characteristics of the dam or for particular intervention. Generally speaking, these methods are only applicable for screening and preliminary level portfolio risk assessments, and for checking more detailed event tree methods, and should not be used alone for detailed assessments.

b) Event tree method

An event tree is a graphical representation of a series of events, which form failure or accident scenarios for a dam. Event tree methods have the advantage that the mechanics of the failure, from initiation to breach can be modelled; the details of the dam and its foundation and the ability to intervene to prevent breaching. However, sometimes there is little objective basis for estimation of the conditional probabilities within event tree and much subjective judgement is needed.

5.2.1 Estimating the probability of internal erosion and piping

The probability of failure of internal erosion and piping can be estimated using historic performance method or event tree method. The method of estimating the probability of failure of embankment dams by piping, have been summarized by Foster et al (2000). Here, the event tree method is used to estimate the probability of failure by internal erosion and piping.

The event tree method involves the decomposition of the failure process into a sequence of events, starting from initiating events through to breaching. Conditional probabilities are assigned to each branch of the event tree, often by a panel of "experts". These are generally judgmental probabilities and are based on the expert's experience, review of information on the design, construction, and performance of the dam, and the reading of selected dam incident and performance case histories from the literature (Foster. et. al, 1999).

Internal erosion and piping can be sub divided as;

- Internal erosion and piping through the embankment
- Internal erosion and piping through the foundation
- Internal erosion and piping from embankment to foundation.

Furthermore piping through the embankment can be sub-divided into;

- Internal erosion and piping through the dam
- Internal erosion and piping along or into conduit

The conditional probability of failure is influenced by reservoir water level. Reservoir water level is recognized as an important factor on the likelihood of a concentrated leak forming, piping hole enlargement and of the formation of a breach mechanism.

Fell et al (2005), suggest using mapping scheme by Bameich et al (1996), for the assessment of probabilities to relate subjectively judged descriptions of likelihood of an event to quantitative probabilities. This mapping scheme was developed for use in dams risk assessment, by Bameich et al (1996) from Military Standard (1993), using Baysian theory to assess historical data. This was done by a group of dams and geotechnical experts, and reviewed by Professor A. Cornell (Fell. et. al, 2005).

To the mapping scheme developed by Bameich et al (1996), five different levels of likelihood ranges were included by considering the method used to estimate the probability of failure for internal erosion and piping. Table 5.2 compares the levels with the mapping scheme developed by Bameich et al (1996) from Military Standard (1993), using Baysian theory to assess historical data.

Table 5.2 : Mapping scheme linking description of likelihood to quantitative probability (Barneich et al 1996) with included likelihood ranges

Description of condition or event	Order of Magnitude of Probability Assigned	Likelihood Range
Occurrence is virtually certain.	1	Very High
Occurrences of the condition or event are observed in the available database.	10^{-1}	High
The occurrence of the condition or event is not observed, or is observed in one isolated instance, in the available database; several potential failure scenarios can be identified.	10^{-2}	Average
The occurrence of the condition or event is not observed in the available database. It is difficult to think about any plausible failure scenario; however, a single scenario could be identified after considerable effort.	10^{-3}	Low
The condition or event has not been observed, and no plausible scenario could be identified, even after considerable effort.	10^{-4}	Very Low

In the following sections, the factors influence on the likelihood of each potential process of internal erosion and piping is discussed. The probability can be calculated by engineering judgement using Table 5.2.

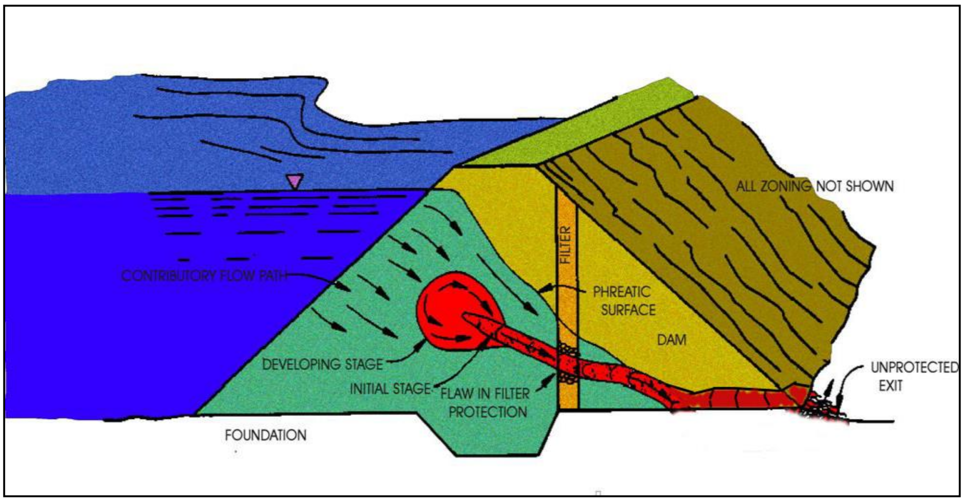


Figure 5.1: Internal erosion and piping through the embankment

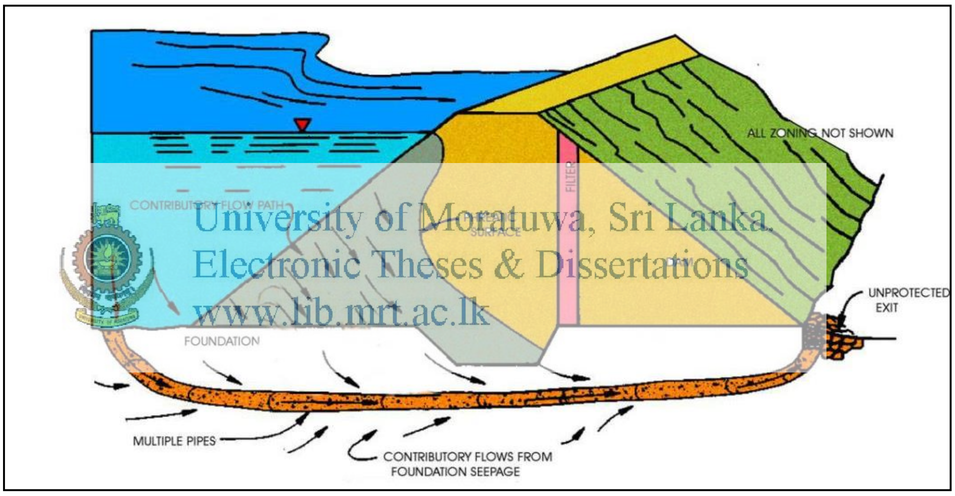


Figure 5.2: Internal erosion and piping through foundation

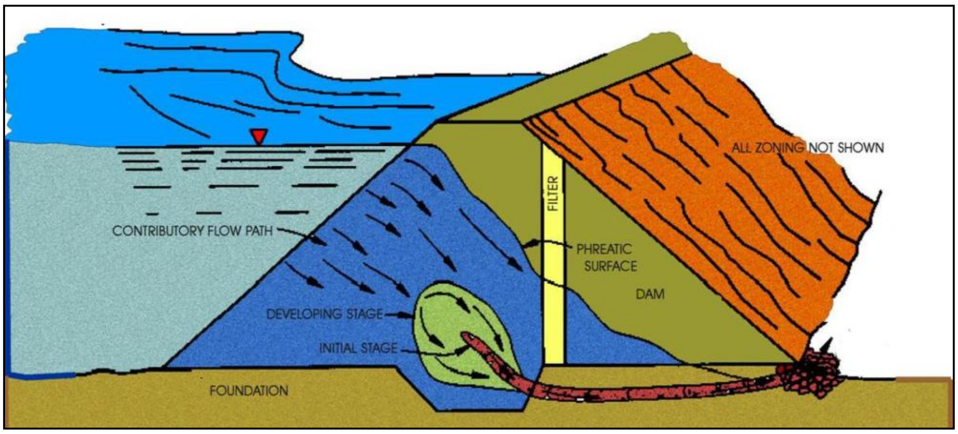


Figure 5.3: Internal erosion and piping through embankment into the foundation

5.2.1.1 Internal erosion and piping through the embankment

In order to develop a framework for an event tree for piping through the embankment, it is necessary to consider the potential piping processes. The potential piping processes are backward erosion piping, concentrated leak piping and suffusion.

Backward erosion - piping refers to the process in which erosion initiates at the exit point of seepage and progressive backward erosion results in the formation of a continuous passage or pipe.

Concentrated leak - piping involves the formation of a crack or concentrated leak directly from the source of water to an exit point and erosion initiates along the walls of the concentrated leak.

Suffusion - refers to the internal migration of fines by seepage flow through internally unstable soils.

The sequence of events leading to failure by backward erosion piping and concentrated leak piping are essentially the same, however, the mechanisms involved in the initiation and progression stages are different. But the factors influencing these mechanisms are similar and therefore it is possible to develop a single event tree framework which encompasses both piping processes (Foster. et. al, 1999).

The piping process initiates by suffusion leads to backward erosion and the event tree is almost similar from that point. So here, it was considered all potential initiation process of piping in to one and developed a single event tree for piping through the embankment. Reservoir water level is recognized as an important factor on the likelihood of a concentrated leak forming, piping hole enlargement and of the formation of a breach mechanism.

5.2.1.1.1 Assessment of likelihood of initiation of internal erosion and piping through the embankment

Initiation of Erosion - Concentrated Leak

Potential sources of concentrated seepage paths are (Foster. et. al, 1999):

- Horizontal or vertical transverse crack through the core
- A continuous high permeability zone in the core due to defects in construction such as poor compaction, layer of coarse grained materials, ice lenses in fill and desiccation cracking

- High permeability zone or a crack adjacent to a conduit through the core or adjacent to a wall

Transverse crack

Transverse cracks through the core are considered as those formed by hydraulic fracture, differential settlements or collapse compression.

Table 5.3 and

Table 5.4 summaries the influence the factors have on the likelihood of the formation of a crack or wetting induced collapse in the core.

Table 5.3 : Influence of factors on likelihood of cracking or wetting induced collapse susceptibility of core materials (Foster. et. al, 1999)

Factor	Influence on likelihood		
	More likely	Neutral	Less likely
Compaction density ratio (1)	Poorly compacted, <95% standard density ratio (2)	95-98% standard density ratio	Well compacted, ≥98% standard density ratio
Compaction water content	Dry of standard optimum water content (approx. OWC -3%)	Approx OWC - 1% to OWC -2%	Optimum or wet of standard optimum water content
Soil types (3)	Low plasticity clay fines	Medium plasticity clay fines	High plasticity clay fines Cohesionless silty fines

Notes: (1) For cracking, compaction density ratio is not a major factor. It is more important for wetting induced failure
(2) <93% standard compaction, dry of owe, much more likely.
(3) Soil type not as important as compaction density and water content.

Table 5.4 : Influence of factors on the likelihood of cracking or hydraulic fracturing - features giving low stress conditions (Foster. et. al, 1999)

Factor	Influence on likelihood		
	More likely	Neutral	Less likely
Overall abutment profile	Deep and narrow valley. Abrupt changes in abutment profile, continuous across core.	Reasonably uniform slopes and moderate steepness, e.g. O.25H: 1V to O.5H: IV	Uniform abutment profile, or large scale slope modification. Flat abutment slopes (>O.5H:IV)

	Near vertical abutment slopes		
Small scale irregularities in abutment profile	Steps, benches, depressions in rock foundation, particularly if continuous across width of core. (examples: haul road, grouting platforms during construction, river channel)	Irregularities present, but not continuous across width of the core	Careful slope modification or smooth profile
Differential foundation settlement	Deep soil foundation adjacent to rock abutments. Variable depth of foundation soils. Variation in compressibility of foundation soil.	Soil foundation, gradual variation in depth	Low compressibility soil foundation. No soil in foundation
Core characteristics	Narrow core, H/W > 2, Particularly core with vertical sides.	Average core width, 2 < H/W < 1	Wide core H/W < 1
	Core material less stiff than shell material	Core and shell materials equivalent stiffness	Core material stiffer than shell material
	Central core		Upstream sloping core
Closure section (during construction)	River diversion through closure section in darn, or new fill placed a long time after original construction		No closure section (river diversion through outlet conduit or tunnel)
Reservoir operation	During first filling or reservoir never reached full supply level. Rapid annual filling. Long periods of low reservoir level followed by rapid filling	At full supply level, steady or slow annual filling of reservoir after first filling.	Steady low reservoir level.

High Permeability Zone

Table 5.5: Influence of factors on the likelihood of a concentrated leak – high permeability zone (Foster. et. al, 1999)

Factor	Influence on likelihood		
	More likely	Neutral	Less likely
Compaction density ratio	Poorly compacted, <95% standard density ratio (1)	95-98% standard compaction density ratio	Well compacted, ≥98% standard density ratio
Compaction water content	Dry of standard optimum water content (approx. OWC - 3%)	Approx OWC - 1 % to OWC -2%	Optimum or wet of standard optimum water content
General quality of construction	Poor clean up after wet, dry or frozen periods during construction, No engineering supervision of construction		Removal of dried, wet or frozen layers before resuming construction Good engineering supervision
Instrumentation details	Poor compaction around Instrumentation, particularly if pass through. the core		No instrumentation in the core
Characteristics of core materials	Large variability of materials in borrow area, moisture content, conditioning and grain size Core materials susceptible to shrinkage cracks due to drying Widely graded core materials susceptible to segregation		Low variability of materials in borrow areas Low shrinkage potential Narrow grading.

Note : (1) < 93% Standard compaction, dry of OWC much more likely

Initiation of Erosion – Suffusion

Suffusion involves the washing out of fines from internally unstable soils. Soils susceptible to suffusion are gap-graded soils and soils with flat "tails" in the finer part of the grain size distribution (Foster. et. al, 1999).

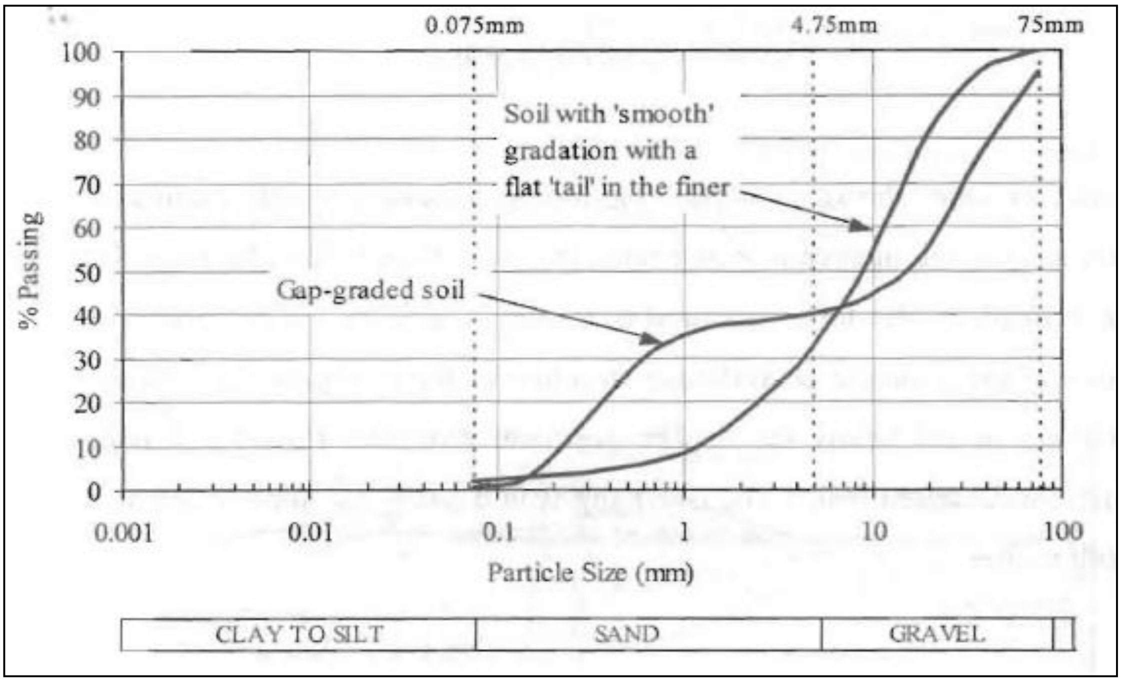


Figure 5.4: Examples of soils susceptible to suffusion.

Table 5.6 summarizes the factors influencing the likelihood of suffusion occurring in the core. The susceptibility of soils to suffusion depends on the particle size distribution.

Table 5.6 : Influence of factors on the likelihood of suffusion (Foster. et. al, 1999)

Factor	Influence of likelihood		
	More likely	Neutral	Less likely
Particle size distribution: 1) General 2) Gap-graded soils (Sherard, 1979) 3) Smooth gradations with a tail of fines based on Kenney and Lau (1985) or Burenkova (1993)	Gap – graded. Flat tail in finer sizes d15c /dI5f > 5 Potentially unstable		Uniform gradation, well graded d15c /d15f < 5 Stable
Compaction Density	Poorly compacted, <95% standard compaction density ratio (1)	95-98% standard compaction density ratio	Well compacted, ≥98% standard compaction density ratio
Permeability	High	Moderate	low

Notes: (1) <93% standard compaction, dry of OWC much more likely.
(2) d15c = particle size on the coarse side of the distribution for which 15% is finer. d15f= particle size on the fine side of the distribution for which 15 % is finer.

Suffusion can occur in non-cohesive soils, as evidenced by laboratory tests including those by Burenkova (1993) and Kenney and Lau (1985), and also in cohesive soils as evidenced by field performance (Sherard, 1979: CFGB, 1997) (Foster. et. al, 1999).

5.2.1.1.2 Assessment of likelihood of continuation of internal erosion and piping through the embankment

Continuation of internal erosion is mainly depending on the filter criteria. If filter not present, the probability can be taken as 1.0 (Fell. et. al, 2005).

In case where filter is present, the judgmental approach for estimating the probability involves;

- Plotting particle size distributions for the base soil and the filters.
- Assess the particle size distributions against the no erosion, excessive erosion and continuing erosion criteria.

From this, use judgement to assign a probability that given the soil and filter gradations and other factors such as potential segregation, the filters will be:

- No erosion (filters finer than no-erosion criteria).
- Some erosion (filters between no erosion and excessive erosion)
- Excessive erosion (filters between excessive erosion and continuing erosion)
- Continuing erosion (filters coarser than continuing erosion criteria)

Table 5.7 summarizes the results of testing to define the no erosion boundary and compares these with the Sherard and Dunnigan (1989) criteria. Proposed criteria for the no erosion boundary are also shown. The boundary is different for dispersive soils (Fell. et. al, 2005).

Table 5.7 : Summary results of statistical analysis and proposed criteria of the no erosion boundary of filter tests for the assessment of filters of existing dams (Foster 1999, Foster and Fell 2001)

Base Soil Group	Fines Content (1)	Design Criteria of Sherard and Dunnigan (1989)	Range of DF15 for No Erosion Boundary From Tests	Proposed criteria for no erosion boundary
1	≥ 85%	DF15 ≤ 9 DB85	6.4 - 13.5 DB85	DF15 ≤ 9DB85 (2)
2A	35 – 85%	DF15 ≤ 0.7mm	0.7 - 1.7mm	DF15 ≤ 0.7mm (2)
3	< 15%	DF15 ≤ 4 DB85	6.8 - 10 DB85	DF15 ≤ 7 DB85
4A	15 – 35%	DF15 ≤ (40-pp% 0.75 mm) x (4DB85 – 0.7)/25 + 0.7	1.6 - 2.5 DF15 of sherard and Dunnigan design criteria	DF15 ≤1.6DF15d, (2) where DF15d = (35 – pp%0.075 mm) (4DB85 – 0.7)/20 + 0.7

Notes: (1) The subdivision for soil group 2 and 4 was modified from 40% passing 75μm, as recommended by Sherard and Dunnigan (1989), to 35% based on the analysis of the filter test data. The modified soil groups are termed group 2A and 4A.

The fine content is the % finer than 0.075mm after the base soil is adjusted to a maximum particle size of 4.75mm.

(2) For highly dispersive soils (pinhole classification D1 or D2 or Emerson class 1 or 2) it is recommended to use a lower DF15 for the no erosion boundary.

- For soil group 1 soil, suggest use the lower limit of the experimental boundary; i.e. DF15 ≤6.4 DB85.
- For soil group 2A soils, suggest use DF15 ≤0.5mm.
- The equation for soil group 4A would be modified accordingly.

(3) DF – diameter of filter particle at which 15% of the particle present are finer, DB – diameter of base soil (or core material) particle at which 85% of the particle present are finer.

It may be assumed that a filter which is finer than the no erosion boundary will have a very low probability of continuation. Based on Table 5.2, it is usual to assign a probability of continuation of 10^{-4} this case (Fell. et. al, 2005).

Table 5.8 summarizes the criteria for excessive and continuing erosion boundaries (Fell. et. al, 2005). The probability of continuing erosion would be very low probability where the compatibility of adjacent soils falls into the no or some erosion category.

Table 5.8 : Excessive and continuing erosion criteria (Foster (1999), Foster and Fell (1999, 2001))

Base Soil	Proposed Criteria for Excessive Erosion Boundary	Proposed Criteria for Continuing Erosion Boundary
Soils with DB95 < 0.3 mm	DF15 > 9 DB95	For all soils: DF15 > 9DB95
Soils with 0.3 < DB95 < 2 mm	DF15 > 9 DB90	
Soils with DB95 > 2 mm and fine content > 35%	average DF15 > DF15 which gives an erosion loss of 0.25g/cm^2 in the CEF test or coarse limit DF15 > DF15 which gives an erosion loss of 1.0g/cm^2 in the CEF test	
Soils with DB95 > 2 mm and fine content < 35%	DF15 > 9 DB85	
Soils with DB95 > 2 mm and fine content 15 - 35%	DF15 > 2.5 DF15design, where DF15design is given by: DFI5design=(35-pp%0.075mm)(4DB85-0.7)/20 +0.7	

Note: Criteria are directly applicable to soils with DB95 up to 4.75mm. For soils with coarser particles determine DB85 and DB95 using grading curves adjusted to give a maximum size of 4.75mm.

Probability of continuation can be assumed with engineering judgement, according to the filter criteria given in Table 5.7 and Table 5.8 based on the mapping scheme given in Table 5.2.

5.2.1.1.3 Assessment of likelihood of progression of internal erosion and piping through the embankment

In the context of an event tree framework, the progression of piping refers to the formation and enlargement of the pipe. There are two issues affecting the progression of piping through the embankment:

(i) The ability to support a roof of the pipe, i.e. will the pipe remain open or collapse?

(ii) Enlargement of the hole

In the event tree the progression process is divided in to three separate sections as;

(i) Ability to support a roof

(ii) Limitation of flow

(iii) Soil erodibility

Ability to support a roof

Information on the ability of embankment materials to sustain open in piping tunnels is obtained by reviewing the case studies of failures and accidents involving piping through the embankment. The ability to support an open roof is indicated by materials which were observed to contain piping tunnels without the development of a sinkhole or where the sinkhole developed a long time after the piping incident. These latter cases indicate slow upward migration of the void (Foster. et. al, 1999).

Materials with piping tunnels or sinkholes that developed slowly generally have the following characteristics:

- Fines content (< 0.075 mm) greater than about 15%
- Core materials that are well compacted.

The most important factor influencing the ability of a material to support a roof is the fines content. Embankment materials with fines contents greater than about 15% have the potential to support open piping tunnels for sufficient time for piping to develop (Foster. et. al, 1999). The factors influencing the ability to support a roof of the pipe are summarised in Table 5.9.

Table 5.9 : Influence of factors on the likelihood of progression of erosion - ability to support a roof (Foster. et. al, 1999)

Factor	Influence on likelihood of fill materials supporting a roof of a pipe		
	More likely	Neutral	Less likely
Fines content (% finer than 0.075 mm)	Fines content > 15 %	Fines content < 15% and > 5 %	No fines or fines content < 5 %
Degree of saturation	Partially saturated (first filling)		Saturated

Enlargement of a Pipe

The most critical issue distinguishing the case histories of piping failures and accidents appears to be related to the limitation of flows through the damaged core. Another issue influencing enlargement of the pipe is related to the rate of erosion. This is influenced by the erodibility of the embankment materials, the hydraulic gradient across the core and the volume of water in the reservoir to continue the erosion process. The factors influencing the enlargement of the pipe are summarized in Table 5.10 and Table 5.11.

Limitation of flow

Mechanisms of flow limitation which were identified from the case studies are:

(a) Filtering action,

(b) Crack filling action, and

(c) Flow restriction from an upstream zone.

These mechanisms are influenced by the zoning and the presence of filters.

Table 5.10 : Influence of factors on the enlargement of the pipe - limitation of flows (Foster. et. al, 1999)

Factor	Influence on likelihood of pipe enlargement		
	More likely	Neutral	Less likely
Action of filter downstream of core	Considered in assessment of filter performance		
Fillings of cracks by washing in of material from upstream	Homogeneous zoning. Upstream zone of cohesive material		Zone upstream of core capable of crack filling (cohesionless soil)
Restriction of flow by upstream zones or concrete element in dam	Homogeneous zoning. Very high permeability zone Upstream of core.	Medium to high permeability zone upstream of core	In zoned dam, medium to low permeability granular zone upstream of core. Central concrete core wall and concrete face rock fill dam

Erodibility

The presence of erodible embankment materials is an important factor influencing piping failure. It is necessary to first consider the difference between the erosion resistances of soils for the two different types of piping process: concentrated leak piping and backward erosion piping. In concentrated leak piping, the soil particles are eroded by flow along the walls of the crack, whereas in backward erosion piping, soil particles are eroded by flow into the head and walls of the pipe. Both involve the removal of soil particles by the shear stresses exerted by the flow of water, and so whilst the mechanisms are different, the relative erosion resistance of different soils is most likely similar for the two mechanisms.

Three factors affecting the erosion resistance of soils are postulated (Foster. et. al, 1999):

(i) Soil type - cohesionless or cohesive
(ii) Soil/water chemistry for cohesive soils, and
(iii) Compaction characteristics.

Table 5.11 : Influence of factors on the progression of erosion - likelihood of pipe enlargement – erodibility (Foster. et. al, 1999)

Factor	Influence on likelihood of pipe enlargement		
	More likely	Neutral	Less likely
Soil type	Very uniform, fine cohesionless sand. (PI<6) Well graded cohesionless soil. (PI<6)	Well graded material with clay binder (6<PI<1 5) (3)	Plastic clay (PI> 15)
Pinhole Dispersion Test (4)	Dispersive soils, Pinhole DI, D2.	Potentially dispersive soils, Pinhole PD1, PD2.	Non-dispersive soils, Pinhole ND1,ND2.
Critical shear stress (Arulanandan and Perry, 1983)	Soils with $\tau_c < 0.0004$ kN/m^2	Soils with $0.0004 < \tau_c < 0.0009$ kN/m^2	Soils with $\tau_c > 0.0009$ kN/m^2
Compaction density ratio (1)	Poorly compacted, <95% standard compaction density ratio (2)	95-98% standard compaction density ratio	Well compacted, ≥98% standard compaction density ratio

Compaction water content	Dry of standard optimum water content (approx. OWC - 3%)	Approx OWC -1 % to OWC -2%	Standard optimum or wet of optimum water content
Hydraulic gradient across core (2)	High	Average	low

Note: (1) <93% Standard compaction, dry of OWC much more likely.
(2) Even dams with very low gradients, e.g. 0.05 can experience piping failure.
(3) PI = Plasticity index
(4) Using Sherard Pinhole Test.

Hydraulic gradient

The applied shear stress by the water flowing through the pipe, and therefore the rate of erosion, are proportional to the hydraulic gradient along the pipe (and hence across the core). The higher the hydraulic gradient, is larger the rate of erosion. It is concluded that the hydraulic gradient across the core has some influence on the enlargement of the pipe, but it is less influential than the erodibility of the embankment materials (Foster. et. al, 1999).

5.2.1.1.4 *Assessment of likelihood of breach mechanism of internal erosion and piping through the embankment*

Potential mechanisms of breach formation are classified into four categories (Foster. et. al, 1999):

- Gross enlargement of pipe,
- Crest settlement or sinkhole leading to overtopping,
- Unravelling of the downstream slope, and
- Instability of the downstream slope.

Most breach mechanism involved gross enlargement of the pipe and few piping failures have resulted from crest settlement/sinkholes (Foster. et. al, 1999). So we have considered these two mechanisms as critical.

The breach mechanisms involved unravelling of the downstream slope and instability of downstream slope are considered as less critical, based on the historical data available in Foster et al (1999). The probability is assumed as lump of enlargement of the pipe and crest settlement/sinkholes.

Gross enlargement of pipe

Continuing erosion and enlargement of a pipe passing through the dam can result in either collapse of the crest and formation of a breach, or emptying of the reservoir through the pipe. This breach mechanism requires the pipe to pass through the downstream zone of the dam and therefore the zoning of the dam and the characteristics of the downstream zone are important factors. The storage volume influences the time water passes through the pipe.

Table 5.12 summarizes the factors influencing on the likelihood of breaching by gross enlargement.

Table 5.12 : Influence of factors on the likelihood of breaching by gross enlargement (Foster. et. al, 1999)

Factor	Influence on likelihood of breach		
	More likely	Neutral	Less likely
Zoning	Homogeneous type zoning. Zoned type dam with a downstream zone able to support a roof	Zone type dam, downstream zone of sand or gravel with fines.	Zone type dam, downstream zone of gravel or rockfill
Storage volume	Large storage volume		Small storage volume

Sinkhole or crest settlement

Localized subsidence of the crest resulting from piping through the dam can lead to loss of freeboard sufficient for localized overtopping and formation of a breach. It is assumed breaching is more likely the higher the reservoir level over the base of the sinkhole. It is also assumed breaching will not occur by this mechanism if the reservoir level is lower than the base of the sinkhole.

Table 5.13 summarizes the influence of other factors on the likelihood of breaching crest settlement or sinkhole. The nature of the downstream zone has some effect if overtopping of the crest occurs.

Table 5.13: Influence of factors on the likelihood of breaching by sinkhole or crest settlement (Foster. et. al, 1999)

Factor	Influence on likelihood of breach		
	More likely	Neutral	Less likely
Freeboard at time of incident (1)	< 2m Freeboard	≈3m Freeboard	> 4m Freeboard
Crest width	Narrow crest	Average crest width	Wide crest
Downstream zone (2)	Fine grained, erodible	Fine grained, non-erodible. Gravel.	Rockfill

Note: (1) Much more likely if < 1m, very unlikely if > 5m.
(2) Minor influence

5.2.1.1.5 Assessment of likelihood of successful early intervention of internal erosion and piping

Some factors helps to assess the likelihood of early intervention are summarized as below.

Likely factors;

- Leakage generally accessible on reservoir-side slope;
- Early phases of erosion process can be controlled;
- Conducting proper monitoring;
- Embankment is instrumented;
- Proper access to the location;
- Less effect of shock and stress on site staff.

Unlikely factors;

- Long embankment will be more difficult to monitor;
- Cracks on the crest;
- Long time required for decisions;
- Quick breach development;
- High work load on site staffs;

The probability of early intervention can be estimated by proper engineering judgement. The probability of early intervention can be assumed as equal for all the failure mode, since all the factor are same except the time needed for the formation of breach mechanism. But if the time needed for the formation of breach mechanism varies by a large value then it should be considered in the estimation of the probability of early intervention.

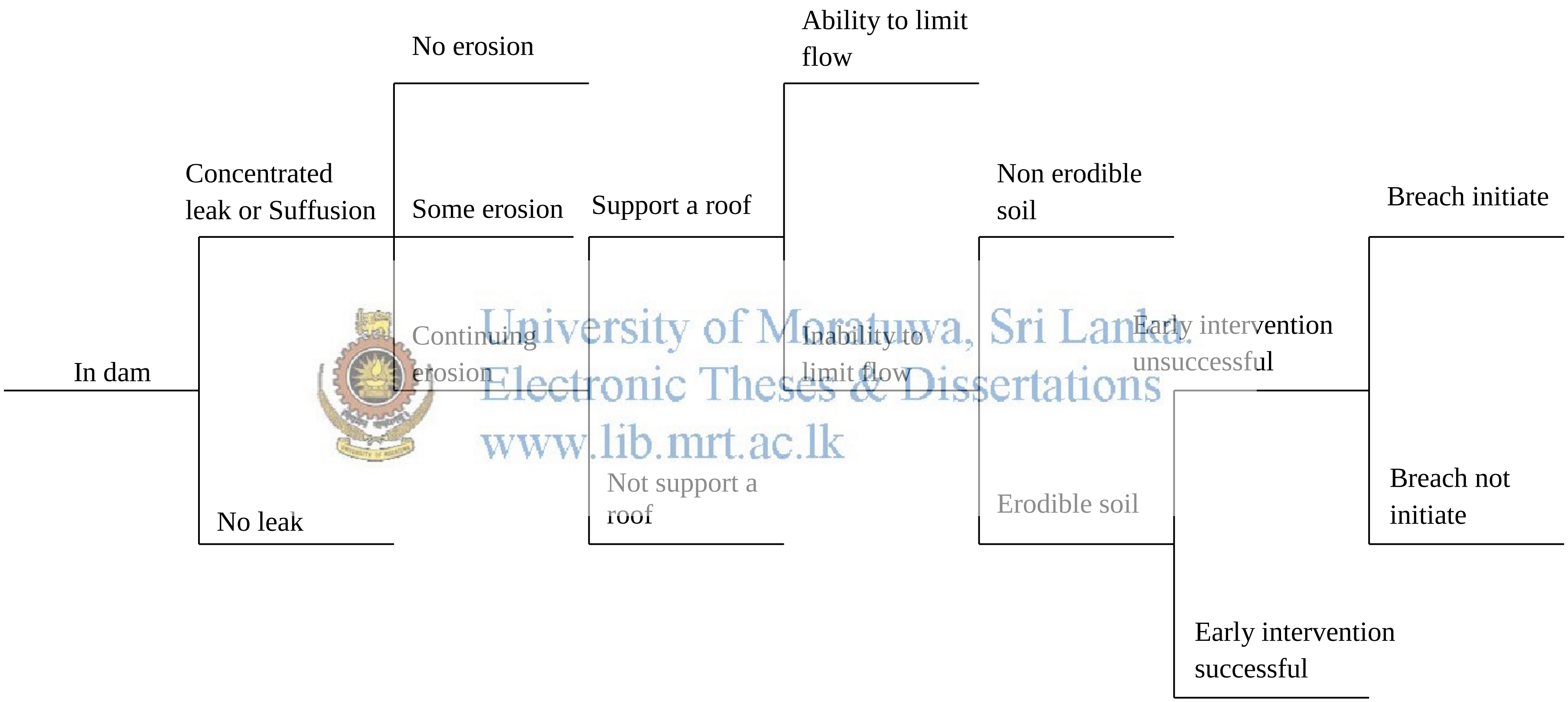


Figure 5.5: Event tree for internal erosion and piping through embankment – in the dam

5.2.1.2 Internal erosion and piping through the embankment - along and into the conduit

The statistics of dam incidents indicate conduits through the embankment are a common source of initiation of piping through the embankment.

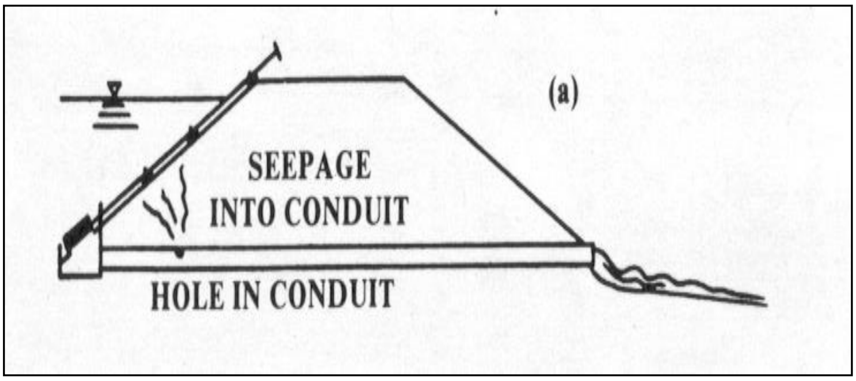


Figure 5.6: Seepage into conduit

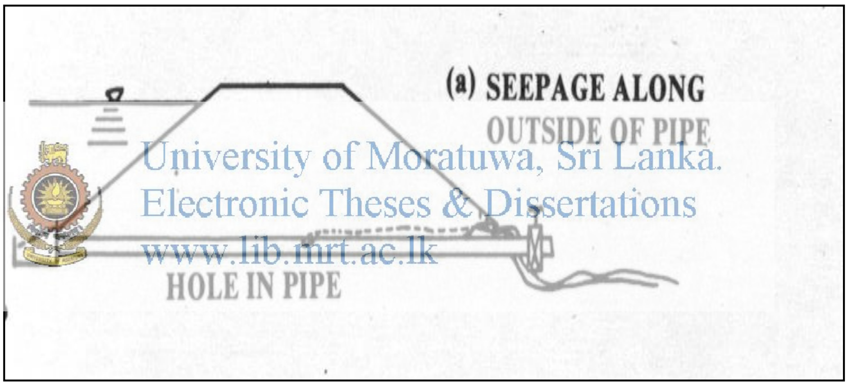


Figure 5.7: Seepage along the conduit

Separate event trees are used to assess the probability of failure by:

- Piping through the dam,
- Piping along, into the conduit

This is necessary because of the difference in factors influencing the initiation and progression of piping at these locations within the embankment.

Most of the conduits in Sri Lanka are circular. In addition there is no way of monitoring the condition of the conduit as well.

The following characteristics are logical when one considers the continuation, progression and breach components of piping failures (Foster. et. al, 1999).

- Homogeneous dams have little to prevent continuation, progression and breaching from occurring (no filters; no high permeability rockfill zones).

- They may also be more likely to be poorly constructed.
- Zoned earthfill dams are likely to have a downstream zone of sand/gravel which can act as a filter, and gives a higher (than the core) permeability.
- Earth and rockfill and concrete face rockfill dams have high discharge capacity rockfill zones, which prevent a breach mechanism from forming. However accidents can develop if filters are not properly designed.
- Puddle core earthfill dams have experienced accidents due to the use of masonry outlets, which are unable to withstand cracking due to differential settlement across the puddle core or when the conduit abuts an outlet gate tower. (this is the case for most of Sri Lankan earth dams)
- Corewall dams are susceptible to differential movements leading to the initiation of cracking of a conduit, but the corewall is likely to prevent piping failure.

5.2.1.2.1 Assessment of likelihood of initiation of internal erosion and piping - Conduits

Factors influencing on initiation of piping along and into conduit (Foster. et. al, 1999):

- Dam height - most failures and accidents occur in dams less than 30m height
- Dam zoning
- Compaction of earthfill - virtually all accidents and failures can be related to modest compaction control or no compaction. The experience in some case histories is that failures have occurred even though compaction control in the dam as a whole is good, but the difficulty of compacting around the conduit, or against poor conduit detailing, such as corrugated surfaces, has led to poor compaction along the conduit.
- Conduit details (e.g.: type of pipe, diameter of pipe)
- Piping incident mode (e.g.: erosion into conduit)
- Cause of initiation of piping into conduit (e.g.: opening of joints due to settlement)
- Time of piping incident (e.g.: on first filling, long term incident)

The influence of conduits on the initiation of piping is considered for each of the modes of piping incident; piping into the conduit, or out of the conduit, or along and above the conduit.

i. Erosion into the conduit

The conduit allows erosion into the conduit if it is cracked, corroded, or joints have opened. This is most likely to occur if (Foster. et. al, 1999):

- Settlement and associated foundation spreading of the conduit has occurred due to compressible (soil) foundations and the joints are not designed or constructed to accommodate this. Rutledge and Gould (1973) have taken measurements of joint opening of concrete outlet pipes for a number of earth dams, with heights of 5 to 15m, constructed on soil foundations. The maximum vertical settlements were 90-950mm and the average stretching of the outlet conduit varied between 0.3 and 0.9% of the original length.
- The conduit is joined to a "stiff" structure, ego outlet shaft or concrete core wall
- Poor detailing of joints in design or construction.
- Water flows in the conduit under pressure and fluctuating flows giving a surging effect.
- If the conduit is of steel or iron construction and is old and corroded.

ii. Erosion along the conduit

The conduit facilitates the initiation of piping by:

- Causing stress distributions due to the stiff conduit and its surround which lead to low principal stresses and hydraulic fracture.
- Making compaction of soil difficult.

Compaction of the embankment materials around the conduit would be difficult if (Foster. et. al, 1999):

- Collars are provided at close intervals.
- The concrete is formed with corrugated steel sheet or other non-smooth formwork, preventing compaction of the soil adjacent to the conduit.
- The conduit is a pipe not surrounded in concrete. Compaction under the pipe is not practicable.

Table 5.14 : Influence of factors on the likelihood of a concentrated leak associated with a conduit (Foster. et. al, 1999)

Factor	Influence on likelihood			
	Much more likely	More likely	Neutral	Less likely
Conduit type	Masonry, brick corrugated steel	Steel, cast iron, not encased	Cast iron, concrete encased concrete precast	Concrete encased steel Concrete cast in-situ
Conduit joints	Open joints, or cracks signs of erosion	Open joints	High quality joints, "open" up to 5 mm but with waterstops	High quality joints, no openings, waterstops
Pipe corrosion	Old, corroded cast iron or steel	Old cast iron, steel.		New steel with corrosion protection
Conduit details	Significant settlement or deep compressible foundation soils. Junction with shaft in embankment	Some settlement, shallow compressible foundation soils		Little or no settlement or rock foundation
Conduit trench details	Narrow, deep, near vertical sides. Vertical sides, trench in soil (backfilled with concrete)	Medium depth, width, slopes. Excavated through dam.	Wide, side slopes flatter than 1H : 1V	Trench totally in rock, back filled with concrete.

Note: (1) Conduits type, joints, corrosion and details mostly affect piping in to conduit
(2) Conduit trench details mostly affects piping along conduit

5.2.1.2.2 Assessment of likelihood of continuation of internal erosion and piping – Conduit

The assessment of probability of continuation of internal erosion along conduit is discussed in section 5.2.1.1.2 . Here filter around conduit should be considered.

The joints in sluice barrel wall should have sufficient opening to erode material into the sluice barrel. Erosion into a conduit, depending on the relationship between joint opening and diameter of the soil surrounding is shown in Table 5.15.

Table 5.15 : Joint opening for no, excessive and continuing erosion into conduits (Fell. et. al, 2005)

Erosion condition	Joint opening width, w	
	Clays, sandy clays, clayey sands	Silt, sand, gravel soils
No erosion	$w \leq D_{85}$ surrounding soil	$w \leq 0.5\ D_{85}$ surrounding soil
Some erosion	$D_{85} < w < D_{90}$ surrounding soil	$0.5\ D_{85} < w < D_{85}$ surrounding soil
Excessive erosion	D_{90} surrounding soil $< w < D_{95}$	D_{85} surrounding soil $< w < D_{95}$
Continuing erosion	$w \geq D_{95}$ surrounding soil	$w \geq D_{95}$ surrounding soil

5.2.1.2.3 *Assessment of likelihood of progression of internal erosion and piping – Conduit*

Conduits and spillway structures passing through the dam facilitates the continuation and progression of piping by providing "side" to the potential erosion hole which will not collapse. The likelihood of erosion developing beyond the initiation stage is greater than without a conduit or wall (Foster, et. al, 1999).

For the situation where piping involves erosion into a conduit, the conduit facilitates continuation and progression by:

- Maintaining an open joint.
- Carrying away the soil which erodes into the conduit and thus the seepage pressures do not have to transport the eroded soil a long distance through the dam.

However the progression of piping may be limited or slowed due to the limited width of the open joint or crack. Filtering of the embankment materials against the crack, particularly if the crack is narrow and the well graded embankment materials may prevent the continuation of piping.

The assessment of probability of progression of internal erosion along conduit is discussed in section 5.2.1.1.3.

5.2.1.2.4 *Assessment of likelihood of breach mechanism of internal erosion and piping – Conduit*

The assessment of probability of breach mechanism of internal erosion along conduit is discussed in section 5.2.1.1.4.

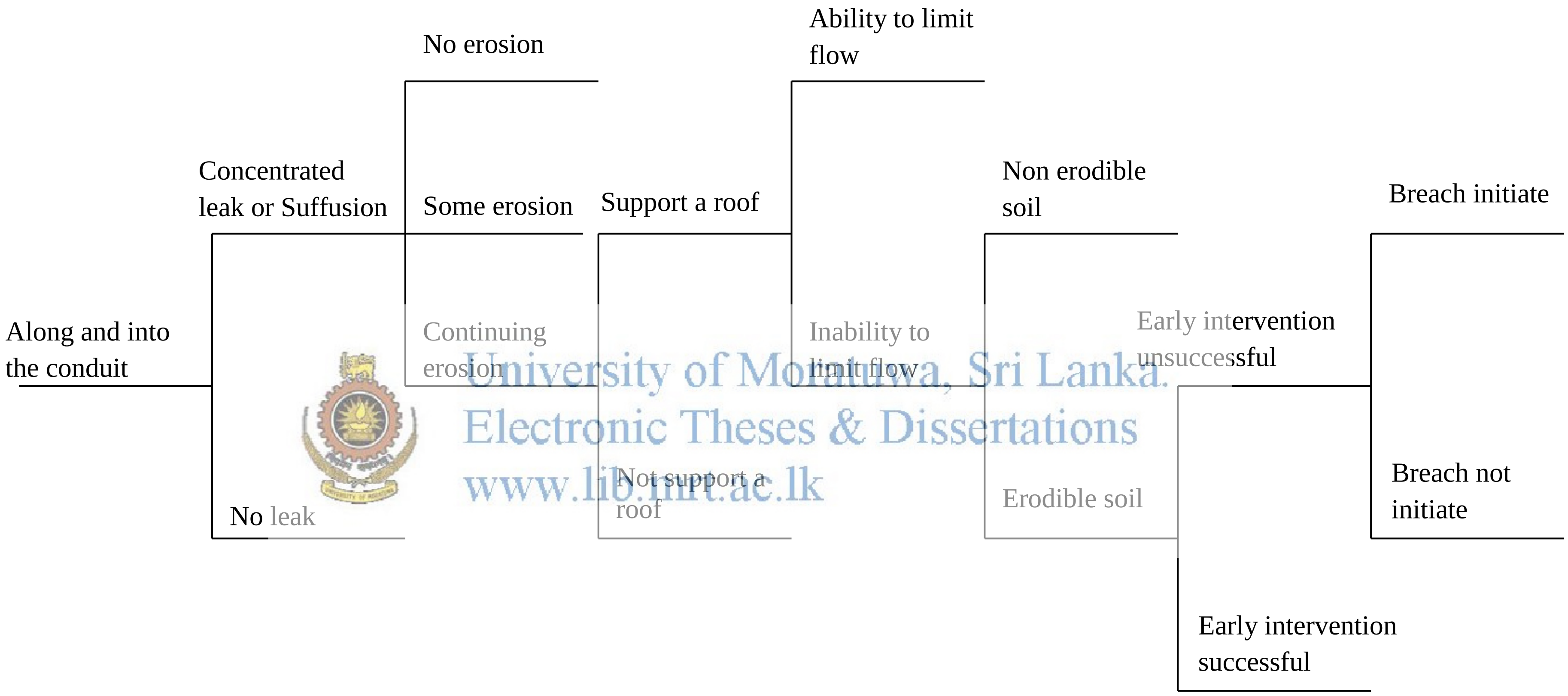


Figure 5.8: Event tree for internal erosion and piping embankment - along and into conduit

5.2.1.3 Internal erosion and piping through foundation

Piping processes that can occur in the foundation are:

(i) Concentrated leak piping

(ii) Backward erosion piping

(iii) Suffusion

(iv) Blowout / heaves

As for piping through the embankment, it is possible to develop a single event tree framework for the assessment of concentrated leak piping and backward erosion piping. Both processes result in the formation of an open pipe through the foundation (Foster. et. al, 1999).

The piping process initiates by suffusion and blow out/heave leads to backward erosion and the event tree is almost similar from that point. So here, it was considered all potential initiation process of piping in to one and developed a single event tree for piping through the foundation.

5.2.1.3.1 Assessment of likelihood of initiation of internal erosion and piping through the foundation

Initiation of Erosion - Concentrated leak

For piping through the foundation, potential concentrated seepage paths are high permeability geological features within the dam foundation. The statistics suggest dams founded on glacial, colluvial and volcanic ash deposits are more likely to experience piping incidents. Fell et al (1992) note that due to their mode of deposition and structure, these deposits commonly have high permeability features present. Volcanic ash soils are also characteristically highly erodible (Foster. et. al, 1999).

Dams founded on alluvial soils appear to be less likely than the average to experience piping incidents. Residual, Aeolian, and lacustrine soils are neutral.

Other geological features or environments which are commonly associated with concentrated seepage paths through the foundation are (Foster. et. al, 1999):

- High permeability sands and gravels. Particularly those with open work gravel layers, and buried river channels, and
- Lateritic profiles.

The statistics of rock geology types involved in piping incidents indicate limestone foundations are particularly susceptible. Other rock geology types that are considered to be more likely to experience piping incidents, based on the statistics are (Foster. et. al, 1999):

- Dolomite - similar to limestone
- Saline rocks (e.g., gypsum) - soluble rocks
- Basalt and rhyolite - open jointed, cooling joints
- Interbedded sandstone and shale - open jointed, stress relief joints.

Dams with no or partially penetrating cut-offs (i.e., cut-off not penetrating to bedrock) are 15 times more likely to fail by piping through the foundation than those with fully penetrating cutoffs (Foster. et. al, 1999).

Table 5.16 : Influence of factors on likelihood of a concentrated seepage path through the foundation (Foster. et. al, 1999)

Factor	Influence on likelihood		
	More likely	Neutral	Less likely
Geological environment • Soil foundations	Glacial Colluvial Volcanic (ash) Lateritic profile	Residual Aeoline Lacustrine	Alluvial
• Rock foundations	Limestone, dolomite Gypsum Basalt, rhyolite Interbedded sandstone and shale		Shale Sandstone (only) Conglomerate Igneous (other than basalt and rhyolite) Metamorphic
Geological features	Open jointed rock Openwork gravel Buried river channels Solution features Weathered faults and dykes		Demonstrated absence of such features
Continuity of high permeability features	Continuous from upstream to downstream, Perpendicular to axis	Discontinuous feature	Not continuous below dam, cut-off by trench

Initiation of Erosion - Suffusion

The assessment of the potential of soils to suffusion is discussed in piping through the embankment in Section 5.2.1.1.1. The factors are essentially the same as for piping through the embankment except compaction density of the core materials is replaced with relative density (for cohesionless soils) and consistency (for cohesive soils).

Table 5.17 : Influence of factors on the likelihood of suffusion (Foster. et. al, 1999)

Factors	Influence on likelihood		
	More likely	Neutral	Less likely
Particle size distribution: • General • Gap-graded soils (1) • Smooth gradations with a tail of fines (2)	Gap-graded. Flat tail in finer sizes d I5c / d15f > 5 Potentially unstable		Uniform gradation. well graded d15c / d15f < 5 Stable
Permeability	High	Moderate	Low
Density	Loose	Medium dense	Dense

Notes: (1) Based on method of splitting grain size curve (Sherard, 1979).
(2) Based on Kenney and Lau (1985) or Burenkova (1993).
(3) d15c = particle size on coarse side of the distribution for which 15% is finer. D15f= particle size on the fine side of the distribution for which 15% is finer.

Initiation of Erosion – Blow out

The mechanism of ‘blowout’, also termed 'heave', involves high pore pressures in the foundation at the downstream toe of a dam leading to low effective stresses. Blowout occurs when the effective stress becomes zero. High pore pressures in the foundation can occur downstream of the dam if there is a surface layer of lower permeability than an underlying permeable layer.

The factor of safety (F_u) against blowout of the confining layer occurring can be calculated by (Foster. et. al, 1999):

$F_{uv}. = \sigma_v / u$

where σ_v = total vertical stress at any point in the foundation u = pore pressure at the same point

For a point directly below the confining layer, Point X in Figure 5.9, the factor of safety is (Foster. et. al, 1999):

$F_{uv} = h_{layer}\gamma_{sat} / h_p\gamma_w$

where h_{layer} = thickness of the confining layer

h_p = head of water pressure at point X

γ_{sat} = unit weight of saturated foundation soil

γ_w = unit weight of water.

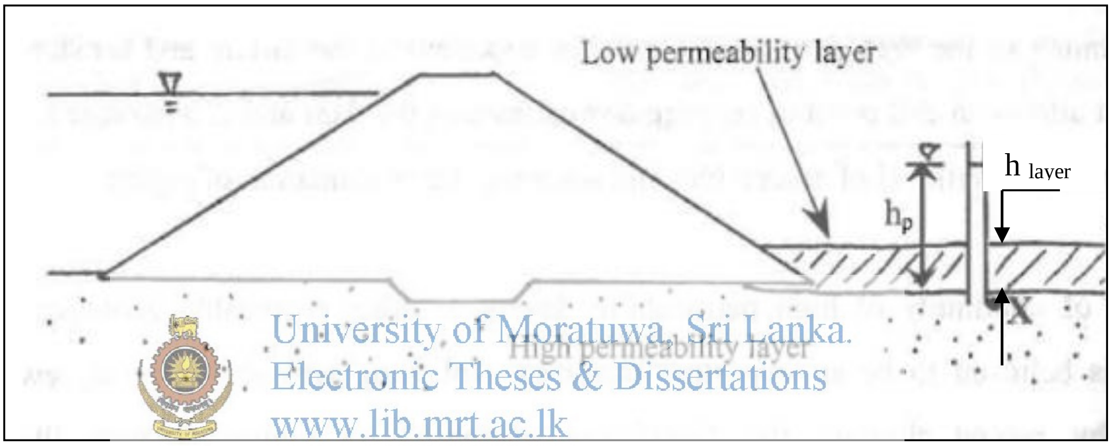


Figure 5.9: Influence of confining layer on pore pressures in the foundation (Foster. et. al, 1999)

Table 5.18 : Influence of factors on the likelihood of blowout (Foster. et. al, 1999)

Factor	Influence on likelihood		
	More likely	Neutral	Less likely
Foundation conditions at the downstream toe	Low permeability layer overlying high permeability layer		High or low permeability layer only
Observed behaviour	Sand boils at downstream toe "Quick sand" conditions		No sand boils
Factor of safety for effective stress condition $F_{uv} = \sigma_v / u$	$F_{uv} < 1.2$	$F_{uv} \approx 1.5$	$F_{uv} > 2.$

5.2.1.3.2 Assessment of likelihood of continuation of internal erosion and piping through the foundation

One of the necessary requirements for piping in the foundation to occur is the presence of an exit point of seepage which allows the continuing removal of eroded materials. Two types of exit points of seepage are possible (Foster. et. al, 1999):

i. 'Free" or unfiltered exit point;
ii. Filtered exit point.

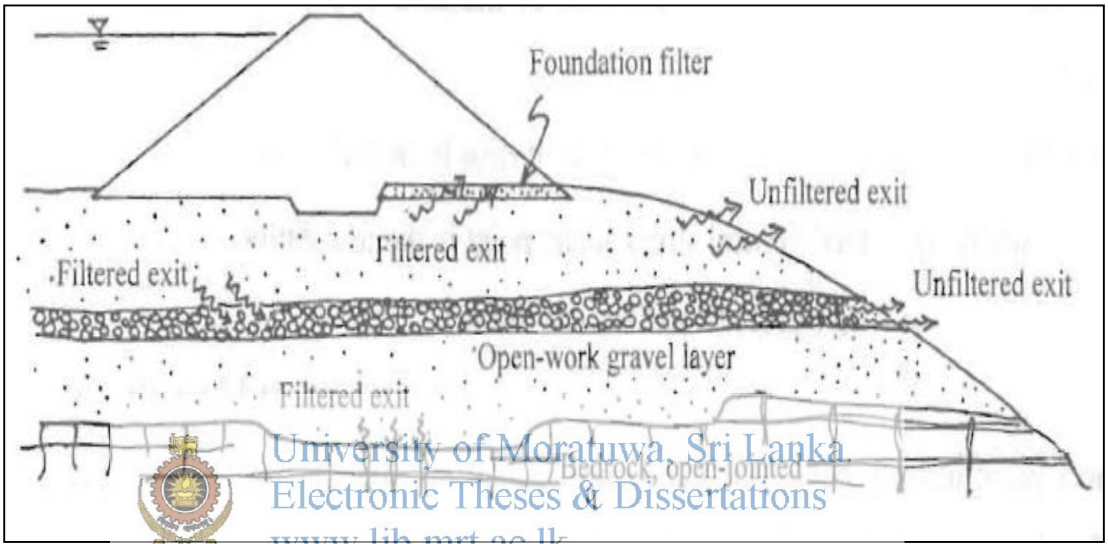


Figure 5.10: Examples of filtered and free exit points for piping through the foundation (Foster. et. al, 1999)

At unfiltered exit points of seepage, there is no potential for filtration and clogging of eroded materials, and removal of eroded materials can continue unrestricted.

It is recommended that given internal erosion has initiated the probability of continuation of erosion be estimated by (Fell. et. al, 2005):

a) Assessing the likelihood that the exit will be filtered or unfiltered exit.
b) Given the exit is unfiltered; the probability of continuation will be 1.0.
c) Given the exit is filtered, assess the filters as described below, and assign the probability of continuation using judgement or simulation method.

The probability of continuation will be the product of the probability of an unfiltered exit and the probability assessed considering the filters.

The judgmental approach for estimating the probability of continuation, given a filtered exit involves (Fell. et. al, 2005):

a) Plotting particle size distributions for the base soil and the filters or transitions which are protecting the base soil. This should be based on all available data. An assessment should be made of the potential for segregation and lapses in construction control, and what effect these could have on the likely range of particle size distributions.
b) Assess the particle size distributions against the no erosion, excessive erosion and continuing erosion criteria developed by Foster (1999), Foster and Fell (1999b, 2001).
c) From this, use judgement to assign a probability that given the soil and filter gradations and other factors such as potential segregation, the filters will be:
   - No erosion (filters finer than no-erosion criteria).
   - Some erosion (filters between no erosion and excessive erosion)
   - Excessive erosion (filters between excessive erosion and continuing erosion)
   - Continuing erosion (filters coarser than continuing erosion criteria)

The probabilities should sum to 1.0 (Fell. et. al, 2005).

Summary results of statistical analysis and proposed criteria of the no erosion boundary of filter tests for the assessment of filters of existing dams (Foster 1999, Foster and Fell 2001) is given in Table 5.7, and criteria for excessive and continuing erosion boundaries is given in Table 5.8.

Filtering action is possible where seepage exits into foundation on drainage system of the dam, ie, seepage into foundation filter, toe drain or relief well system, or where seepage passes through fine grained and coarse grained layers in the foundations. As for piping through the dam, the probability of continuing erosion would be very low probability where the compatibility of adjacent soils falls into the no or some erosion category, and there would only be none or minor erosion loss required for filtration to occur. For the excessive and continuing erosion category, or where the exit is unfiltered, continuing erosion and large erosion losses are more likely (Foster. et. al, 1999).

5.2.1.3.3 *Assessment of likelihood of progression of internal erosion and piping through the foundation*

The piping initiating in the foundation is less likely to progress to failure than piping through the embankment (Foster. et. al, 1999). The progression of piping in the foundation is related to the ability of the foundation soils to support a roof and to the factors influencing pipe enlargement.

Ability to support a roof

The formation of an open pipe through the foundation would be expected to be largely influenced by the foundation soil types, soil stratigraphy and by the presence of geological features such as cemented layers and infilled scour channels. Homogeneous cohesionless sands cannot maintain an unsupported roof and therefore such materials are not susceptible to piping unless they are overlain by an artificial roof, such as the base of a concrete spillway structure, or a cohesive material.

The formation of an open pipe is more likely in the foundation if any of the following features are present (Foster. et. al, 1999):

i. The erodible material is cohesive.

ii. There is cohesive material overlying the erodible material. Examples are layers of clay, cemented soil or rock overlying erodible soil or interbedded cemented and non-cemented layers.

iii. Solution features in rock, for example solution channels or cavities in limestone filled with erodible materials.

iv. The erodible materials are below a rigid structure such as a concrete dam, concrete spillway structure or below an outlet conduit.

Given that cohesive embankment materials can support open pipes, it is feasible that the base of the embankment dam could form a roof if piping developed along the embankment/foundation interface. However, there is no evidence of such occurring in the foundation piping failures.Well graded sandy gravels may be able to support a roof by arching action between the coarse gravel particles. The factors influencing the ability to support a roof of the pipe in the foundation are summarized in Table 5.19.

Table 5.19 : Influence of factors on the likelihood of foundation materials able to support a roof (Foster. et. al, 1999)

Factor	Influence on likelihood of foundation materials supporting a roof of a pipe		
	More likely	Neutral	Less likely
Foundation conditions	Piping through soils with cohesive fines Cohesive layer overlying piped material Piping through solution features in rock Piping below rigid structure (e.g. spillway)	Well graded sand and gravel	Homogeneous, cohesion less sands

Enlargement of pipe

The issues influencing the enlargement of the pipe for piping through the foundation are similar to those for piping through the embankment. These are:

Factors influence on flow limitation;

i. Filtering action
ii. Crack filling action
iii. Flow restriction

Factors influence on rate of erosion;

iv. Erodibility of foundation soils
v. Hydraulic gradient

The limitation of flows is less influential for limiting the enlargement of the pipe in piping through the foundation compared to piping through the embankment. However, the processes of filtering action, crack filling and flow restriction contribute by restricting erosion (Foster. et. al, 1999). Therefore, flow limitation and rate of erosion was considered under restriction of erosion in event tree for internal erosion and piping through foundation.

Table 5.20 : Factors influencing the enlargement of the pipe, piping through the foundation - flow limitation (Foster. et. al, 1999)

Factor	Influence on likelihood of pipe enlargement		
	More likely	Neutral	Less likely
Hydraulic gradient (1)	High	Average	low
Filling of 'cracks' or voids by washing in of embankment or foundation materials	Homogeneous zoning or upstream zone of cohesive materials Low permeability cohesionless foundation layer upstream of the dam	Cohesive layer upstream of the dam (may crack)	Zoned type dam with gravel or rockfill upstream shell High permeability layer upstream of dam
Restriction of flow path	Flow path unrestricted dimensions, or Flow path restricted but large dimensions (e.g. large solution channels in limestone)		Flow path of small restricted width (e.g. Piping through crack in cutoff walls or narrow rock joints)

Note: (1) Even dams with very low overall gradients across the foundation, e.g. 0.05, can experience piping failure.

Table 5.21 : Influence of factors on likelihood of pipe enlargement, piping through the foundation – erodibility (Foster. et. al, 1999)

Factor	Influence on likelihood of pipe enlargement		
	More likely	Neutral	Less likely
Soil type	Very uniform, fine cohesionless sand (PI<6) Well graded cohesionless soil (PI<6)	Well graded material with clay binder (6 < PI < 15)	Plastic clay (PI > 15)
Pinhole Dispersion Test (2)	Dispersive soils, Pinhole Dl, D2.	Potentially dispersive soils, Pinhole PDl, PD2.	Non-dispersive soils, Pinhole ND1, ND2.

Critical shear stress (1)	Soils with $\tau_c < 0.0004$ kN/m^2	Soils with $0.0004 < \tau_c <$ 0.0009 kN/m^2	Soils with $\tau_c > 0.0009$ kN/m^2
Relative density	Loose	Medium dense	Dense
Consistency	Soft	Stiff	Very stiff

Note: (1) After Arulanandan and Perry (1983)
(2) Using Sherard Pinhole Test

5.2.1.3.4 Assessment of likelihood of breach mechanism of internal erosion and piping through the foundation

Same as through embankment, most breach mechanism involved gross enlargement of the pipe and few piping failures have resulted from crest settlement/sinkholes (Foster. et. al, 1999). So we have considered these two mechanisms as critical. The probability is assumed as lump of enlargement of the pipe and crest settlement/sinkholes.

Gross enlargement

The factors are similar to those for piping through the embankment shown in Table 5.12 (Foster. et. al, 1999). However, dam zoning would only be important if the pipe exits through the downstream zone of the dam. Gross enlargement is likely if there is continuing enlargement of the pipe and the roof of the pipe can be supported along its full length. Gross enlargement of the pipe can result in the collapse of the crest or emptying of the reservoir through the pipe. This breach mechanism requires continuing enlargement of the pipe and the pipe through the foundation has to remain open.

Crest Settlement or Sinkhole

Piping through the foundation may lead to the formation of sinkholes or settlements of the crest or abutments of the dam and this can lead to loss of freeboard and overtopping. As for piping through the embankment, influential factors are the crest width and freeboard at the time of the incident, and to a lesser extent, the characteristics of the downstream zone if overtopping does occur (Foster. et. al, 1999). The assessment of breach by sinkhole or crest settlement leading to overtopping is assumed to be similar as for piping through the embankment, Table 5.13.

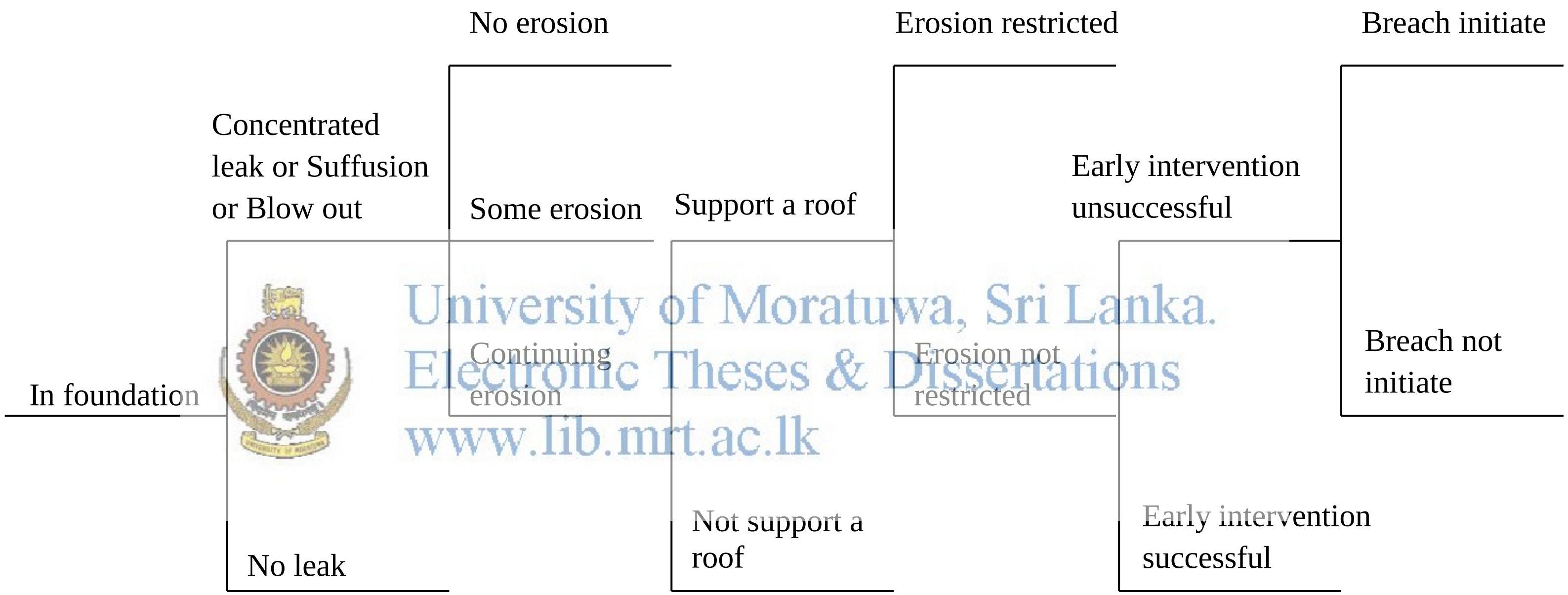


Figure 5.11: Event tree for internal erosion and piping through foundation

5.2.2 Slope instability

Probability of slope failure can be estimated using historical data, mathematical modelling and quantification of expert judgement. Here, the method based on quantification of expert judgement is discussed. When involved with a potentially unstable slope, engineers want to know whether or not the slope will fail. Since there are many uncertainties that affect this determination, the engineer has to settle for estimating the probability of whether the slope will fail.

Figure 5.12 present the relationships between factor of safety and annual probability of failure based on actual engineering projects and developed through quantified expert judgment (Silva. et.al, 2008). This plot is an updated version of the one originally presented by Lambe (1985) and Baecher and Christian (2003) (Silva. et.al, 2008). Figure 5.12 classifies earth structures into four categories, based on the level of engineering, ranges from best Category (I) to poor Category (IV). The level of engineering can be established by examining the practices followed for design, investigation, testing, analyses and documentation, construction, and operation and monitoring. The four categories correspond to the following types of facilities (Silva. et.al, 2008):

i. Category I—facilities designed, built, and operated with state-of-the-practice engineering. Generally these facilities have high failure consequences;
ii. Category II—facilities designed, built, and operated using standard engineering practice. Many ordinary facilities fall into this category;
iii. Category III—facilities without site-specific design and substandard construction or operation. Temporary facilities and those with low failure consequences often fall into this category;
iv. Category IV—facilities with little or no engineering.

The family of curves in Figure 5.12 and the associated Table 5.22 with the four levels of engineering reflect the generally accepted concept that: “A larger factor of safety does not necessarily imply a smaller risk, because its effect can be negated by the presence of larger uncertainties in the design environment” (Silva. et.al, 2008). Four categories of earth structures are described in Table 5.22.

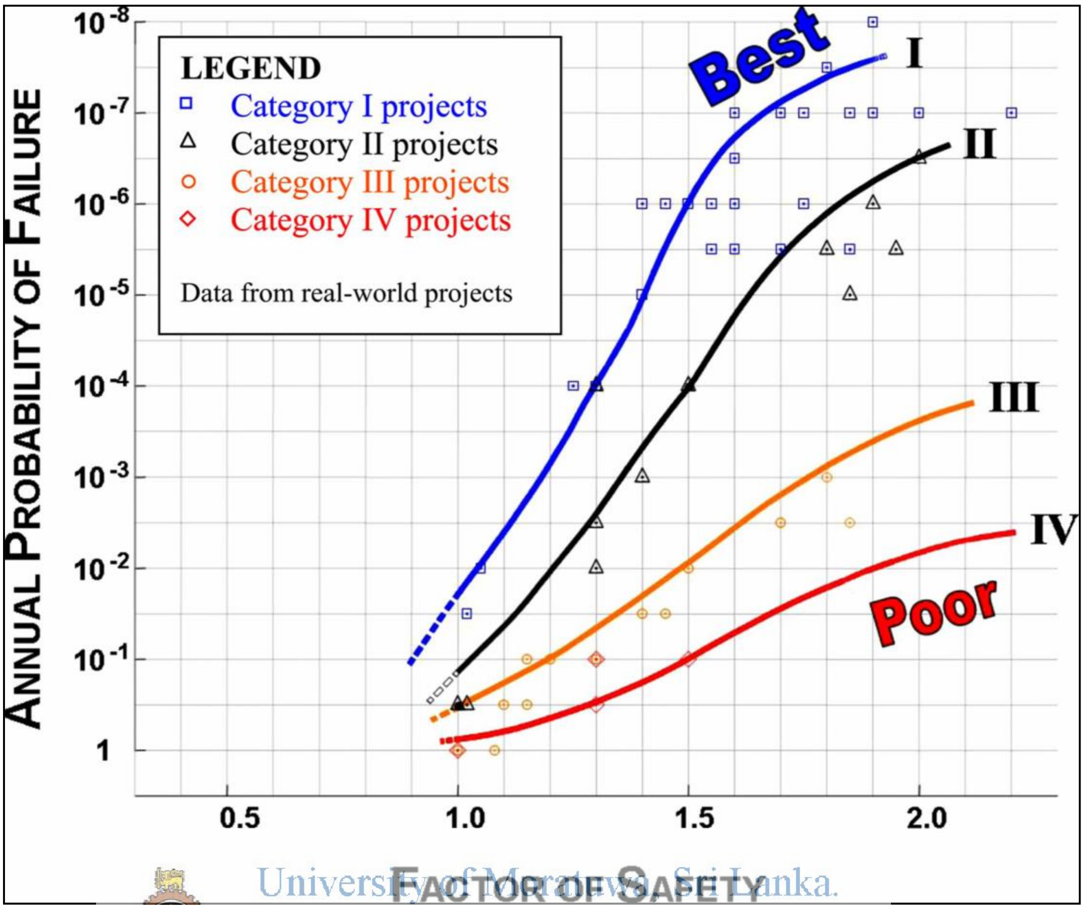


Figure 5.12: Factor of safety versus annual probability of failure

Two frequently mentioned data points served as reference points for the curves in Figure 5.12 (Silva. et.al, 2008):

- (1.5, 0.0001) — Baecher et. al. (1980), Whitman (1984), and Christian et al. (1992) based on historical performance of earth dams designed and constructed with conservative engineering practice; and
- (1.0, 0.5) —Vick (1994) based on the theoretical fact that a normally distributed uncertainty on factor of safety gives a probability of failure of 0.5 at a factor of safety of 1.0.

Table 5.22: Earth structure categories and characteristics (Silva. et.al, 2008)

Level of engineering	Design			Construction	Operation and monitoring
	Investigation	Testing	Analyses and documentation		
I (Best) Facilities with high failure consequences	Evaluate design and performance of nearby structures Analyze historic aerial photographs Locate all nonuniformities (soft, wet, loose, high, or low permeability zones) Determine site geologic history Determine subsoil profile using continuous sampling Obtain undisturbed samples for lab testing of foundation soils Determine field pore pressures	Run lab tests on undisturbed specimens at field conditions Run strength test along field effective and total stress paths Run index field tests (e.g., field vane, cone penetrometer) to detect all soft, wet, loose, high, or low permeability zones Calibrate equipment and sensors prior to testing program	Determine FS using effective stress parameters based on measured data (geometry, strength, pore pressure) for site Consider field stress path in stability determination Prepare flow net for instrumented sections Predict pore pressure and other relevant performance parameters (e.g., stress, deformation, flow rates) for instrumented section Have design report clearly document parameters and analyses used for design No errors or omission Peer review	Full time supervision by qualified engineer Construction control tests by qualified engineers and technicians No errors or omissions Construction report clearly documents construction activities	Complete performance program including comparison between predicted and measured performance (e.g., pore pressure, strength, deformations) No malfunctions (slides, cracks, artesian heads) Continuous maintenance by trained crews
II (Above average) Ordinary facilities	Evaluate design and performance of nearby structures Exploration program tailored to project conditions by qualified engineer	Run standard lab tests on undisturbed specimens Measure pore pressure in strength tests Evaluate differences between laboratory test conditions and field conditions	Determine FS using effective stress parameters and pore pressures Adjust for significant differences between field stress paths and stress path implied in analysis that could affect design	Part-time supervision by qualified engineer No errors or omissions	Periodic inspection by qualified engineer No uncorrected malfunctions Selected field measurements Routine maintenance
III (Average) Unimportant or temporary facilities with low failure consequences	Evaluate performance of nearby Structures Estimate subsoil profile from existing data and borings	Index tests on samples from site	Rational analyses using parameters inferred from index tests	Informal construction supervision	Annual inspection by qualified engineer No field measurements Maintenance limited to emergency repairs
IV (Poor) Little or no engineering	No field investigation	No laboratory tests on samples obtained at the site	Approximate analyses using assumed parameters	No construction supervision by qualified engineer No construction control tests.	Occasional inspection by non-qualified person No field measurements

5.2.3 Embankment overtopping

The probability of failure is calculated from the reservoir level versus AEP(Annual Exceedance Probabilities) and a system response curve, that is, probability of failure versus depth of water over the dam crest, which is developed for that dam. Selection of the response relationship is subjective, with factors such as material type, compaction and inherent susceptibility to erosion influencing the choice.

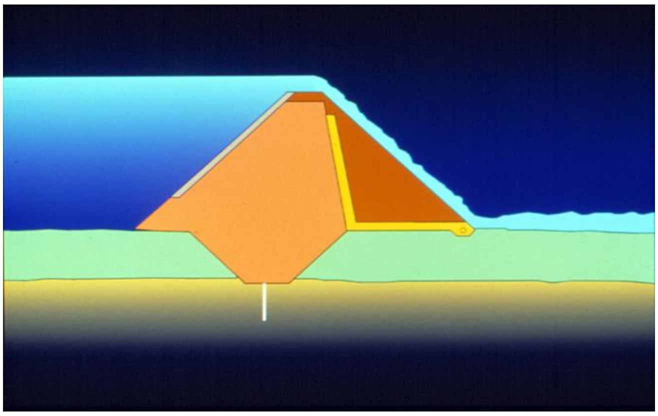


Figure 5.13: Embankment overtopping

Most studies seem to accept that the probability of failure approaches 1.0 when the depth of overtopping is between 0.5m and 1m for a modern compacted rockfill dam or a well-grasses cohesive earthfill dam (ANCOLD, 2003).

5.2.4 Spillway and spillway energy dissipation scour, and overtopping of spillway chute wall

The rate and extent of scour can be based on calculation or hydraulic models, coupled with judgement. References include Pinto (1994) and Van Schalkwyk et al (1994).

Spillway chute walls are often likely to overtop at floods less than the flood to overtop the dam. If the chute is adjacent the embankment, the overtopping can scour the dam and lead to failure. If it is remote from the dam, it is necessary to consider the likelihood that overtopping would undercut the excavation batter, causing a slide that partially or completely block the spillway. The annual probability of the discharge state that causes overtopping of the walls can be estimated from calculation or from physical hydraulic scale models. The scour estimates are usually judgemental (ANCOLD, 2003).

5.3 Combining the Probabilities

In quantitative analysis, annual probability of failure should be estimated from the estimation of probabilities previously made. Here, the estimation of failure per annum by load states is discussed.

5.3.1 Common cause of failures

Common cause failure modes are failure modes that can occur simultaneously at a single dam section due to a single initiating event, and failure modes that can occur simultaneously at multiple sections of a dam due to a single initiating event. The total probability of dam failure is some combination of the probabilities of dam failure that are associated with each of the possible modes. For this case, there is no practicable way of computing the estimated overall probability of failure, given the several individual mode conditional probabilities of failure. Following the theory of uni-modal bound, the bounds can be determined.

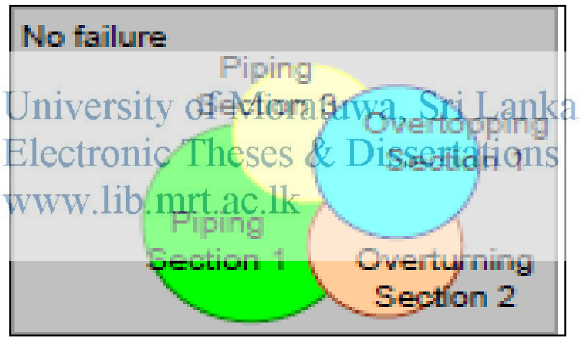


Figure 5.14: Venn diagram for common cause of failure modes

5.3.2 Uni-model bound theorem

The conditional probabilities for the failure modes that are not mutually exclusive can be adjusted for common cause occurrence by using the uni-modal bounds theorem. Following the theory of uni-modal bounds, the bounds are determined as upper bound and lower bound.

The *upper bound* is the union of the events, the several failure modes. From de Morgan's rule, the estimated upper bound conditional probability is;

$P_{UB} = 1- (1-P_1). (1-P_2)....... (1-P_n)$

where,

P_{UB}	= the estimated upper bound conditional probability of failure
P_1 to P_n	=the estimates of the several individual mode conditional probabilities of failure.

This computation must be made on the estimated conditional probabilities of failure before multiplying by the annual probability of the loading scenario (ANCOLD, 2003).

The *lower bound* estimate is the maximum individual conditional probability out of several failure modes.

5.3.3 Combining probabilities of failure modes initiated by flood

The annual probability of occurrence of the load state or scenario needs to be multiplied by the estimated conditional probability of failure, in order to find the annual likelihood of failure for each failure mode. If likelihood of failure is to be aggregated over several failure modes that are not mutually exclusive, it is necessary to apply de Morgan’s rule to compute the estimated upper bound conditional probability before multiplying by the annual likelihood of the load state or scenario(ANCOLD, 2003). It should be noted that the simple addition approximates de Morgan’s rule, if the conditional probabilities are low in value.

Table 5.23: Example computation of combining probabilities of failure modes initiated by flood

Load Scenario	Annual Probability of Flood Scenario	Failure Mode	Conditional Probability of Failure	Conditional Probability of Failure for Flood Scenario	Annual Probability of Failure for Flood Scenario
F1	4.0×10^{-4}	Piping through the embankment	6.5×10^{-1}	7.31×10^{-1} (U) 6.50×10^{-1} (L)	2.92×10^{-4} (U) 2.60×10^{-4} (L)
		Piping through the foundation	5.0×10^{-4}		
		Embankment overtopping	2.3×10^{-1}		

5.3.4 Combining probabilities of failure modes initiated by normal operating load

The annual probability of the maximum reservoir level being in each level state is multiplied by the conditional probabilities of failure, typically found from event trees. Here, the level state affects the conditional probabilities.

Alternative to the above method, for normal operating conditions, it is the reservoir level state that contributes the load state. For normal operating load, the annual probability of failure, found by multiplying the annual probability of initiation and the conditional probability of failure, are weighted by the dimensionless proportion of time that the reservoir is in each level state (ANCOLD, 2003). Here the conditional probabilities are influenced by level state. Since the reservoir level states are mutually exclusive, and exhaustive of the total reservoir level domain, proportion of time that the reservoir is in each level state should sum to 1.0.

Table 5.24: Example computation of combining probabilities of failure modes initiated by flood (ANCOLD, 2003)

(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
Reservoir Level State	Proportion of Time in that state	Initiating Defect	Annual Probability of Initiation	Conditional Probability of Failure	Annual Probability of Failure **(6)=(2)x(4)x(5)**
L1 (close to FSL)	0.089	Rupture seal	1.5×10^{-02}	1.0×10^{-01}	1.3×10^{-04}
		Piping	1.0×10^{-04}	1.0×10^{-02}	8.9×10^{-08}
		Slide	5.0×10^{-06}	Zero	Zero
L2	0.254	Rupture seal	2.3×10^{-04}	2.0×10^{-03}	1.2×10^{-07}
		Piping	4.3×10^{-06}	4.8×10^{-05}	5.3×10^{-11}
		Slide	Zero	Zero	Zero
L3	0.366	Rupture seal	Zero	Zero	Zero
		Piping	Zero	Zero	Zero
		Slide	Zero	Zero	Zero
L4	0.225	Rupture seal	Zero	Zero	Zero
		Piping	Zero	Zero	Zero
		Slide	Zero	Zero	Zero
L5	0.066	Rupture seal	Zero	Zero	Zero
		Piping	Zero	Zero	Zero
		Slide	Zero	Zero	Zero
Total for normal operating load					$\mathbf{1.3 \times 10^{-04}}$

RISK ESTIMATION-ESTIMATION OF CONSEQUENCES

CHAPTER 6

The consequences of failure play a role in assessing the significance of the potential failure mode. Potential consequences resulting from an uncontrolled release of a reservoir have several different dimensions. In addition to the economic losses related to lost project benefits and potentials damage to property in the inundated area, there is a potential for loss of life, alteration of the habitat and environment, social impact on local community and loss of confidence in the dam owner and operators.

The consequences of failure and the circumstances surrounding a failure (advance warning, detection possibilities, impact of the failure, etc.) should be discussed for each potential failure mode during the discussion of the potential failure mode since these factors play a role in assessing the significance of the potential failure mode.

Incremental consequences are defined as the difference in consequences between those due to dam failure, and those due to the same routed through the dam without its failure.

The following sections provide general considerations for estimating the potential magnitudes of uncontrolled outflows, the extent of inundated area, and the resulting potential for loss of life and economic damages.

6.1 Identifying Dam Break Scenarios

Dam-break scenarios which are adequately representative of all of the overall dam failure scenarios should be identified. The term overall dam failure scenario refers to the total suite of states and conditions that defines each dam failure case that is analysed in the study (ANCOLD, 2003).

For example, an overall dam failure scenario could be defined by:

- A loading scenario (such as concurrent reservoir level);
- A dam component (e.g. Embankment);
- A failure mode(e.g.: Downstream slope instability);

- Downstream conditions;
- An exposure scenario.

In quantitative risk analysis, every overall dam failure scenario, of which there may be thousands in complex analyses, has an estimated probability of occurrence with a dam failure, and there is a complementary probability of occurrence of the scenario with no failure. In theory, a set of consequences attached to the "failure" and "no failure" outcomes for every one of these many overall scenarios.

Because of practical considerations of cost and time, only a relative few dam breach, dam – break and consequences analysis are normally undertaken.

6.2 Estimation of the Downstream Inundation Characteristic

For quantitative analyses, undertake breach analyses to estimate the outflow flood hydrograph for each representative dam breach/break scenario, using methods appropriate to the level of detail of the risk analysis.

Route the selected dam–break flood through the downstream channel. Record such out comes as inundation limits, peak flow depths, peak mean velocities and flood wave travel time at representative sections along the channel.

The zone affected by a dam break flood may be defined by experienced judgement as an initial assessment or by inundation mapping for more comprehensive assessments. An inundation map provides a description of the areal extent of flooding which would be produced by a dam-break. It should be plotted on a scaled plan to show the maximum extent of a dam failure flood as it travels downstream, regardless of the time after failure occurred.

Inundation maps may be prepared on a number of different levels depending on the degree of accuracy required and the initial perceived consequences of a hypothetical failure.

ANCOLD guidelines on assessment of the consequences of dam failure (ANCOLD, 2000b) summarised three levels of determination of inundation zones and these are:

- Method 1- Approximate Determination;
- Method 2- Semi-empirical Determination;

- Method 3- Dam Break Analysis.

6.2.1 Approximate determination

This involves a windscreen inspection of both sides of the valley downstream. Any dwellings (both occupied and unoccupied) as well as any infrastructure such as bridges, roads, railway lines, power lines within a height above the stream bed of between 1/3 and ½ of the dam height should be noted and located on the largest scale map of the area available (ANCOLD, 2000b).

A line should then be drawn on the map starting at the 1/3 to ½ height of the dam and extending downstream roughly parallel to the slope of the valley (ANCOLD, 2000b).

It should be noted that these results indicate the entire inundation zone including the pre existing flood conditions. This method should never be used by itself if the end result of the assessed consequence would result in major upgrades to the dam.

6.2.2 Semi empirical determination

If the first method is not sufficient to accurate to determine whether residences and people are at risk from a dam-break flood and the hazard rating cannot be easily assessed. This method assesses the approximate depths of a dam –break flood.

The information required for this exercise includes the dam's characteristics, (storage capacity, height, catchment area), the valley's downstream characteristics, width and slope, and the estimated dam breach development.

ANCOLD guidelines on assessment of the consequences of dam failure (ANCOLD, 2000b) suggest that if a semi empirical method such as this is to be used, it would be advisable to carry out a rough survey of the floor level of any residences that could be affected. Such a survey need not be any more accurate than -/+ 0.5 m vertically and -/+ 5 m horizontally.

Survey data should be relative to the creek bed (a channel occupied by a stream) at the cross section under consideration. The distance at the sections downstream of the dam should also be determined to within -/+ 50 m.

When sufficient depths at downstream sections have been determined the results should be plotted on the largest scale maps available. Interpolation between "calculated points" should be based on the prevailing topography and contours.

6.2.3 Dam break analysis

In the detailed risk assessment, dam break analysis is required. Such a study is a site specific extension of Method 2 and requires detailed extensive accurate surveys of the downstream valley. Such survey should locate all dwellings which are thought to be at risk. In this case the analyst should err on the conservative side and include dwellings thought to be on or just above the failure flood envelop boundaries.

Cross sections should be taken at all locations where there are dwelling as well as at sufficient other locations, including hydraulic controls such as bridges, weirs, waterfalls, etc, to allow a reasonable model to be set up.

As a guide, sections should be taken at about the following intervals for dams of the following storage size (ANCOLD, 2000b):

Table 6.1: Intervals between sections for different storages (ANCOLD, 2000b)

Storage ML	Intervals Between Sections (Total Distance)
20000	1 Kilometre (up to 60 kilometres)
2000	0.5 to 1 Kilometre (up to 20 kilometres)
200	Not greater than 0.5 Kilometre (up to 60 kilometres)

Total distance in parentheses in the table above are based on actual dam break studies indicating the distances downstream where the incremental effects of the dam break flood becomes relatively small. Care should be taken to treat each case as site specific, particularly in cases where the downstream valley is confined and narrow for great distances. In these cases, it has been found that the dam break flood does not dissipate quickly and greater distances downstream may need to be considered, especially where there are dwellings at risk.

The cross section should extend for at least half the vertical height of the dam above the stream bed at each location. This height may be decreased at a greater distance downstream of the dam. Cross sections should be generally being accurate to within +/- 0.1 m vertically and +/- 0.5 m horizontally (ANCOLD, 2000b).

The output from a dam break analysis should include the following:

1. Hydrograph at each section (Flow versus time).
2. Depth at each section at appropriate time intervals.
3. Velocities at each section at time intervals.
4. Flood peak arrival times at each section.
5. The first rise in water level at each section.
6. Recession time of the dam break flood.

This information should be summarised in tables and plotted on the largest available scale map. Suitable map scales have been found to be 1 in 4000 with contours at 2 metre based on judgement and guided by the prevailing topography and contour data.

Computer programs that can carry out dam break analysis include:

- BOSS FLOODWAV, International NWS DAMBRK (Version 3.0)
- Danish Hydraulics Institute.....MIKE 11

It is generally thought that a dam break analysis will provide results which are at best accurate to +/- 1 m vertically.

6.3 Estimation of Life Safety Consequences

In quantitative risk analysis estimation of life safety consequences can be divided into two steps as follows (ANCOLD, 2003):

- Loss of life (LOL) for both the "failure" and "no failure" cases, for each overall failure scenario (needed to estimate societal life safety risks);
- Conditional probability of fatality for the person or group most at risk, given dam failure, for each overall failure scenario and the complementary "no failure" case (needed to estimate individual risk to life)

Note that the currently available empirical models developed for estimating LOL due to dam-break are not suitable for estimating LOL for the case without dam failure.

6.3.1 Estimating loss of life

To estimate the number of life loss, the model of Graham (1999) is considered the most suitable of the empirical approaches.

The Graham Method estimates loss of life based on data taken from every documented U.S. dam failure that resulted in more than 50 fatalities and every documented dam failure that occurred after 1960 resulting in at least one fatality. Graham found that loss of life resulting from dam failure is highly influenced by three factors: (1) the number of people occupying the dam failure floodplain; (2) the amount of warning that is provided to the people exposed to dangerous flooding; and (3) the severity of the flooding. The method proposed by Graham is composed of seven steps given below:

Step 1 - Determine dam failure scenarios to evaluate.

Step 2 - Determine time categories for which loss of life estimates are needed.

Step 3 - Determine when dam failure warnings would be initiated.

Step 4 - Determine area flooded for each dam failure scenario.

Step 5 - Estimate the number of people at risk for each dam failure scenario and time category.

Step 6 - Apply empirically based equations or methods for estimating the number of fatalities.

Step 7 - Evaluate uncertainty.

6.3.1.1 Determine dam failure scenarios to evaluate

A determination needs to be made regarding the failure modes to evaluate. Determination of dam break failure scenarios are discussed under section 6.1.

6.3.1.2 Determine time categories for which loss of life estimates are needed

The number of people at risk downstream from some dams is influenced by seasonality or day of week factors. The number of time categories (season, day of week, etc.) evaluated should display the varying usage of flood plain and corresponding number of people at risk. Since time of day can influence both when a warning is initiated as well as the number of people at risk, each study should include day category and nigh category for each dam failure scenarios evaluated.

6.3.1.3 Determine when dam failure warnings would be initiated

In general, warning time as it relates to dam failure is the time period from when communication warning of a dam failure or impending dam failure reaches a specific PAR (Population at Risk) to when the breach flood arrives at the location of the specific PAR. In other words, it is the amount of time for people to evacuate a breach flood zone after they receive notification of a dam failure and before the flood failure wave arrives. However, warning time depends on many factors. For instance, warning time may be zero or very short for a PAR in the downstream flood area near an unattended dam in a remote location where knowledge of a dam breach is not known until it reaches or is very near the affected PAR. On the other hand, warning time may be relatively long in a densely populated area below a dam that has a sophisticated early warning system where warning can come directly from the warning system and also from personal communication among the affected residents.

In order to help define when warning time likely would be given, Graham developed a table that offers various scenarios to choose the start of the warning time period. Graham broke the decision factors down to five options. It is up to the user to decide which of the options best fits the dam in question. The five factors are as follows:

1. Dam type (earthfill)
2. Cause of failure (overtopping, piping, or seismic).
3. Special considerations that include drainage area size, immediate failure or delayed failure.

4. Time of day when failure occurs.
5. Observers at the dam – if the dam is attended, there are “many observers” at the dam; if the dam is unattended; there are “no observers” at the dam.

Graham clearly points out that each dam and scenario has to be evaluated individually and special circumstances need to be taken into consideration.

6.3.1.4 Determine area flooded for each dam failure scenario

Determination of area flooded for each dam failure scenario is discussed under section 6.2.

6.3.1.5 Estimate the number of people at risk for each dam failure scenario and time category

For each failure scenario and time category, number of people at risk should be determined. Population at risk (PAR) is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning (Graham, 1999). The number of people at risk varies throughout the day. The PAR will likely vary depending upon the time of year, day of week and time of day during which the failure occurs. Utilize census data, field trips, aerial photo graphs, telephone interviews, topographic maps and other sources that would provide a realistic estimate of floodplain occupancy and usage.

According to ANCOLD guidelines on risk assessment, PAR can be estimated on the basis of (ANCOLD, 2003);

- Field inspection;
- Interviews with local inhabitants, council and business personals;
- Flood inundation mapping;
- Aerial photography;
- Geographic information system.

Table 6.2: Guidance for estimating when dam failure warnings would be initiated (Earth Fill Dams) (Graham, 1999)

Dam type	Cause of failure	Special consideration	Time of failure	When would dam failure warning be initiated?	
				Many observers at dam	No observers at dam
Earthfill	Overtopping	Drainage area at dam less than $100mi^2$ ($260km^2$)	Day	0.25 hrs. before dam failure	0.25 hrs. after fw reaches populated area
		Drainage area at dam less than $100mi^2$ ($260km^2$)	Night	0.25 hrs. after dam failure	1.0 hrs. after fw reaches populated area
		Drainage area at dam more than $100mi^2$ ($260km^2$)	Day	2 hrs. before dam failure	1 hr before dam failure
		Drainage area at dam more than $100mi^2$ ($260km^2$)	Night	1 to 2 hrs. before dam failure	0 to1 hr before dam failure
	Piping (full reservoir, normal weather)		Day	1 hrs. before dam failure	0.25 hrs. after fw reaches populated area
			Night	0.5 hrs. after dam failure	1.0 hrs. after fw reaches populated area
	Seismic	Immediate failure	Day	0.25 hrs. after dam failure	0.25 hrs. after fw reaches populated area
			Night	0.5 hrs. after dam failure	1.0 hrs. after fw reaches populated area
		Delayed failure	Day	2 hrs. before dam failure	0.5 hrs. before fw reaches populated area
			Night	2 hrs. before dam failure	0.5 hrs. before fw reaches populated area

Note: “many observers at dam” means that a dam tender lives on high ground and within site of the dam or dam is visible from the homes of many people or the dam crest use as a heavily use roadway. These dams are typically in urban areas. “No observation at dam” means that there is no dam tender at dam, the dam is out of site of nearly all homes and there is no roadway on the dam crest. These dams are usually in remote areas. The abbreviation “fw” stands for floodwater.

6.3.1.6 Apply empirically based equations or methods for estimating the number of fatalities

The fatality rates are obtained based on the flood severity, amount of warning and a measure of whether people understand the severity of the flooding. Graham developed a new way of looking at the severity of flooding based on data from previous dam failures. Severity relates to the force of the flood and the ability of humans to survive the flood. Flooding danger to humans is dependent on the depth and velocity of water. The flood severity categories are as follows (Graham, 1999):

Low Severity

If a flood is classified as having low severity, it has the ability to wash buildings off their foundations. However, it also implies the building remains relatively intact and humans within the building have a reasonable chance for survival.

Medium Severity

Medium severity applies to floods that destroy homes but certain features like trees or mangled homes remain in the flooded area where people can seek refuge.

High Severity

This is the least likely form of severity, but certainly the deadliest. High severity refers to a flood of such magnitude that the flooded zone is swept clean to the ground and nothing remains. This type of flooding has occurred only a few times in recorded history, but it has the potential for happening in certain geographical areas. In Graham's study, data for high severity failures was not well represented and guidance is not given for estimating loss of life for this case.

In determining LOL, Graham suggests using one of three categories for warning time. The warning time categories are as follows:

- **No warning** means the media or official sources issue no warning in the particular area prior to the flood water arrival; only the possible sight or sound of the approaching flooding serves as a warning.
- **Some warning** means officials or the media begin warning in the particular area 15 to 60 minutes before floodwater arrival. Some people will learn of the flooding indirectly when contacted by friends, neighbours or relatives.
- **Adequate warning** means officials or the media begins warning in the particular area more than 60 minutes before the floodwater arrives. Some people will learn

of the flooding indirectly when contacted by friends, neighbours or relatives. It also considers a population's understanding of a flood's severity that affects the ability of those affected by the flood to evacuate. The following discussion focuses on Graham's definitions of the three categories of severity and how it relates to loss of life, and the two categories of flood severity understanding.

Flood severity understanding is the last factor related to severity that has an impact on the ultimate estimation of LOL. The relative understanding of the flood severity is a function of the distance or time from the dam failure or the source and origination of flooding. The farther one is from the source of the flooding, the greater the likelihood that the warning will be precise and accurate. This is because people have seen the flooding in upstream areas, they understand the damage potential of the flooding and the warnings are adjusted to reflect the actual danger. Similarly, the people receiving the warning should obtain a better understanding of the danger to which they are exposed (Graham, 1999).

A warning of potential flooding, before it actually occurs, may not be understood by the warning issuers and would therefore be difficult to describe. Recipients of this warning will therefore not get an accurate picture of the flooding about to occur and may not evacuate at all or not as quickly as they should. This factor will come into consideration only when there is some or adequate warning. The flood severity understanding categories are as follows (Graham, 1999):

1) Vague Understanding of Flood Severity means that the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.
2) Precise Understanding of Flood Severity means that the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others."

In determining whether flooding is low severity or medium severity, use low severity if most of the structures will be exposed to depths of less than 10 feet and medium severity if most of the structures will be exposed to depths of 10 feet or more (Graham,1999). (Note that low severity flooding can be quite deadly to people attempting to drive vehicles.).Use high flood severity only for locations flooded by the near instantaneous failure of a concrete dam, or an earthfill dam that turns into "jello" and goes out in seconds rather than minutes or hours (Graham, 1999).

Graham suggests another method that can be used to separate low severity flooding from medium severity flooding by use of the parameter DV where:

$DV = (Q_{df} - Q_{2.33})/Wd_f$

where,

Q_{df} - the discharge at a particular site caused by dam failure.

$Q_{2.33}$ -the mean annual discharge at the same site. This discharge can be easily estimated and it is an indicator of the safe channel capacity. As discharges increase above this value, there is a greater chance that it will cause overbank flooding.

Wd_f - the maximum width of flooding caused by dam failure at the same site.

The units of DV are d^2/s or depth (D) time's velocity (V). Graham suggests low flood severity should be assumed, in general, when DV is less than 50 ft^2/s (4.6 m^2/s). Medium flood severity should be assumed, in general, when DV is more than this value.

Graham summarized his findings in a table that lists recommended fatality rates for estimating loss of life (LOL) for dam failures. Fatalities can be estimated using Table 6.3.

Table 6.3: Recommended fatality rates for estimating loss of life resulting from dam failure (Graham, 1999)

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested range
HIGH	No warning	Not applicable	0.75	0.30 to 1.00
	15 to 60	Vague	Use the values shown above and apply to the number of people who remain in the dam failure flood plain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	More than 60	Vague		
		precise		
MEDIUM	No warning	Not applicable	0.15	0.03 to 0.35
	15 to 60	Vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	More than 60	Vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	No warning	Not applicable	0.01	0.0 to 0.02
	15 to 60	Vague	0.007	0.0 to .015
		precise	0.002	0.0 to 0.004
	More than 60	Vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

6.3.1.7 Evaluate uncertainty

Graham (1999), suggest various types of uncertainty that can influence loss of life estimation as follows;

- Step 1 of this procedure suggests that separate loss of life estimates be developed for each dam failure scenarios. Various dam failure scenarios will result in different downstream flooding and therefore, it will result in differences in number of people at risk as well as the severity of the flooding.

- Step 2 of this procedure suggests once again that separate loss of life estimates be developed for various possible combinations. The time at which warning is initiated and the number of people at risk may depend upon the time at which failure occurs. For example, Night time failures have been shown to be deadlier than daytime failures. Weekday daytime failures that affect residential areas could have lower fatality rates than during weekends. Other factors such as time of year could introduce uncertainty in the estimates determined by this method.

- Step 3 and Table 6.2 provide guidance on when warning would be initiated. Other warning scenarios may be equally or more likely. Uncertainty associated with the warning initiation can be evaluated by varying the assumptions regarding when a warning would be initiated.

- The last type of uncertainty is associated with the inability to precisely determine the fatality rate. Uncertainty in the type of force area, warning times, and other factors in the estimation process could lead to wide variability in fatality rates. Some of the factors that are contributed to life loss are not captured in the categories shown in
- Table 6.**3**. Some possible ways of handling this uncertainty would be to 1) use the range of fatality rate shown in
- Table 6.**3**, 2) when the flooding in particular area falls between two categories, the loss of life estimates can be developed using the fatality rates and range of rates from all categories touched by the event.

6.4 Estimation of the Monetary Loss Consequences – Economic and Financial

In quantitative analysis, for each selected dam-break scenario, the direct and indirect monetary losses and damages should be estimated for both “failure” and “no failure” cases. The direct losses can be estimated from flood inundation mapping, aerial photography, GIS, field inspections and interview and available data. Some of the direct losses are;

- Destruction of part or the entire dam;
- Destruction of, or damage to, residential, commercial, industrial and agricultural properties;
- Destruction of, or damage to, infrastructure such as pipelines, power lines and telephone system;
- Destruction of crops, fences and farm machinery;
- Drowning of stock;
- Land rendered unproductive.

The estimation of indirect losses can be complex and difficult, especially for widespread impacts in a closely developed region, when regional economic modelling may be only means of obtaining a reliable estimate. Typical indirect losses are;

- Cost of emergency response and temporary care;
- Cost of alternative accommodation;
- Lost industrial production due to loss of power;
- Lost agricultural production due to loss of irrigation water;
- Loss of revenue to power, water and telephone providers;
- Loss of wages to workers temporarily out of work.

It is usual to distinguish between economic losses which affect the society at large and financial losses which directly affect the owner’s business.

RISK ESTIMATION-REPORTING THE RISK

CHAPTER 7

7.1 Estimation of Probability of the Overall Dam Failure Scenario

The estimation of probability of overall dam failure scenarios should consider the downstream conditions. The overall dam failure scenarios are comprised of two key sub- scenarios;

- The states and conditions that contribute to the dam failure mechanism leading to breach of the dam - the scenario "at the dam";
- The states and conditions that exist in the dam-break affected zone - the "downstream" scenario.

Probability of the overall dam failure scenario is estimated by multiplying the probability of the dam failure by the exposure factor – dam failure at a time when that exposure scenario applies.

Annual Probability of the overall dam failure scenario = *Annual Probability of failure* × *Exposure factor*

In each failure scenario, there is the corresponding "no failure" scenario, in which all states and conditions are the same, except that the dam does not fail. The annual probability of the "no failure" scenario is 1.0 minus the annual probability of the failure scenario.

7.1.1 Exposure factor

Population at risk and their vulnerability vary according to time of day, day of week and season of the year, as a minimum. This fact gives rise to the concept of exposure scenario and exposure factor (ANCOLD, 2003).

ANCOLD guidelines on risk assessment (ANCOLD, 2003), summarizes the exposure factors which can be highly variable, as illustrated by the following typical values:

- Residents of nursing home in dam-break zone – 1.0 (there continuously);
- Residents who lives in dam-break zone, but work elsewhere – 0.70 (away from home 10 hours per day for 5 days a week);
- Staff of nursing home who live outside dam-break zone – 0.24 (at nursing home 8 hours per day for 5 days a week);
- Staff at a tourist park shop in the dam-break zone – 0.24 (at the shop for 8 hours per day, 5 days a week);
- School children who live outside dam-break zone, the school being in the zone – 0.18 (at school 6 hours a day, 5 days a week);
- Tourist who visit the tourist park in the dam-break zone for two hours a month – 0.0027;
- Overseas visitors, who visit the park in the dam-break zone for two hours in a lifetime – 0.000003.

Such exposure factors directly affects the risk imposed on particular individuals (individual risk), but only affects the estimation of LOL if they change the PAR at particular times. For example, if there are a number of large schools in the dam-break zone, the population at risk may reduce significantly outside school hours. Other kind of exposure factors can be defined by proper judgement.

7.2 Estimation of Risks

Risk is the measure of the probability and severity of an adverse effect to life, health, property, or the environment. In the general case, risk is estimated by the combined impact of all triplets of scenario, probability of occurrence and the associated consequences. The annualised risk can be estimated as the product of the probability of overall failure scenario and the consequences. The following sections discuss about estimating life safety risks and monetary risks.

7.2.1 Estimation of life safety risks

The primary outcome of a quantitative risk analysis is a series of estimated probability of failure and estimated consequences pairs, one pair for each specified overall failure scenario. These are termed “f,N” pairs (for risk to life), and should be reported to a decision maker, since they represent potential dam failure outcomes. Risk to life can be reported as, the individual risk to life for the person or group most at risk and societal risk to life.

7.2.1.1 Individual risk of life

The assessment of individual risk to life is based on the person or group most at risk. Usually it is a small group, such as the occupants of a single house or a small hamlet, because it is not practicable to say that any one member of that group bears a higher risk than the any other member. All members are taken to bear the same risk and it is this risk that is computed as the individual risk. It is necessary to identify this group.

For each failure scenario, the contribution to individual risk is computed as the product of (ANCOLD, 2003):

- The annual probability of the overall dam failure scenario; and
- The conditional probability of fatality, given dam failure.

Aggregating individual risk components, contributed by each of the failure scenarios, requires care. ANCOLD guidelines on risk assessment (ANCOLD, 2003) suggest the following guiding principles:

- It is always acceptable to add results from mutually exclusive states (each of a series of flood states – or earthquake states – are mutually exclusive; flood, earthquake and normal operating conditions states are taken as mutually exclusive);
- For states that are not mutually exclusive (often the several failure modes, and the several dam components), simple addition can give erroneous results unless the estimated conditional probability of dam failure is low (say, less than 0.01).

7.2.1.2 Societal risk

The relevant outputs of a risk assessment are “f,N” pairs.

where,

“f” – Estimated probability of occurrence of each overall failure scenario

“N” – Corresponding estimated number of lives that would be lost

These pairs show a decision maker the failure scenarios that could occur, the likelihood that they will occur, and the best estimate of loss of life if they do occur. It is helpful to report them in both tabular and graphical format. ANCOLD guidelines on risk assessment (ANCOLD, 2003) suggest that on a graph, the convention is to have "f" on the vertical axis using a log scale, and "N" on the horizontal axis using a log scale. The pairs will plot as a cloud of points, generally with no pattern to them.

Expected value of life loss (lives per annum) is the product of "f" and "N". The product "f ×N" aggregated over all scenarios, is often given as the correct measure of risk, but in reality is a special case of the general definition of risk.

ANOCOLD guidelines on risk assessment (ANCOLD, 2003), prefers the way of presenting the societal risk is the use of F – N plots, where "F" is the complementary cumulative distribution function, the estimated annual probability of a failure expected to result in the loss of "N" or more lives.

If there are a number of scenarios with the same "N" value, it is necessary to aggregate the annual probability of failure scenario ("f") for those scenarios, before computing the complementary cumulative distribution function (the "F" values). If the scenarios are not mutually exclusive, the aggregation is computed as the union of events, using de Morgan's rule.

7.2.2 Estimation monetary loss risks (economic and financial)

Reporting of monetary risks in quantitative studies follows the same principles as those for computation of life safety risks, with "N", the estimated number of lives lost, replaced by the estimated monetary loss. There are similar concerns with the expected value of monetary loss as there are with expected value of life loss.

Where failure initiators or modes are not mutually exclusive, report either the upper and lower bounds in annual probability of failure scenario "f", or just the upper bound, according to the purpose of study.

7.3 Uncertainty in the Risks

In quantitative analysis, uncertainty of both probability and consequences should be reported. This need to report uncertainty may be less critical for studies that simply aim to rank the relative risk. In risk analysis the estimates of probability incorporate many of the uncertainties. The additional areas of uncertainty that need to be considered are;

- Conditional probabilities of dam failure, given a natural event load are uncertain;
- The AEP values of flood are highly uncertain for such reasons as limited periods of flood records, and the extrapolations, beyond experience, that are necessary to estimate the AEP of extreme event;
- Dam-break and inundation modelling is uncertain for a whole range of reasons, including the uniqueness of each breach situation, difficulty of predicting downstream and tributary concurrent stream flow, inaccuracies in topographic models, inability to accurately reflect the highly variable hydraulic resistance properties of stream channels and errors in the flood routing models;
- Estimation of the rupee value of direct and indirect losses is the well known uncertainty of loss valuation.
- Estimation of life safety consequences is highly uncertain.
- Stage damage models for buildings and other structures are uncertain, because of the limited databases on which these are based, the unique resistance properties of each structure and the unpredictable nature of dam failure floods, especially in steep confined valleys, where large debris mats and temporary debris dam formation can be expected;

In consequence analysis, uncertainty needs to be dealt separately for each type of consequence. The resulting best estimates of risk analysis outcomes will generally not be the same as, and can be significantly different from, those obtained from calculations that use only best estimate inputs without uncertainty analysis.

RISK EVALUATION & REDUCTION

CHAPTER 8

8.1 Risk Evaluation

The determination of tolerable levels of risk is fraught with difficulty. If the dams cannot be made absolutely risk free, then we need to know the tolerable risks. Risk assessment typically requires tolerable risk policies and criteria. It is the responsibility of the dam owner to ensure that the policies and criteria are set, and to endorse them. The dam owner needs to decide what risks are tolerable.

Four conditions need to be met for a risk to be deemed tolerable, as below (ANCOLD, 2003);

1. We can live with the risk so as to secure certain benefits;
2. The risk is within a range of risk that we do not regard as negligible or as something we might ignore;
3. We need to keep the risk under review;
4. We need to reduce the risk still further if and as we can satisfy ALARP.

8.1.1 Life safety risks

The tolerable risk to life should be identified if the dams are not absolutely risk free. In any particular case, all three guidelines on tolerability of life safety risk are to be satisfied; that is (ANCOLD, 2003):

- The individual risk guideline;
- The societal risk guidelines;
- The ALARP (As Low as Reasonably Practicable) requirement.

The first two guidelines are applied to establish ceilings or limit of tolerability above which risks should be regarded as unacceptable in all but most exceptional circumstances. The decision that there are exceptional circumstances that justify risks higher than the limit is to be made by government or its regulators, and normally be based on the benefits to society of facility, despite its risks.

8.1.1.1 Individual Risk

For individuals within an age bracket, there is a wide distribution of mortality risk according to health, occupation, recreational pursuits, and lifestyle habits, but it is the average background risk for that age group that is the guide to tolerability of risks from a facility, such as a dam. We have considered the tolerable risk criteria accepted by ANCOLD as suitable for ancient Sri Lankan earth dams. Under exceptional circumstance the value tolerable risk criteria can be modified.

For existing dams ANCOLD guidelines proposed that (ANCOLD, 2003);

- The limit for individual risk to the person or group, which is most at risk, is $\mathbf{10^{-4}}$ per annum, except in exceptional circumstance;
- The risks are to be lower than the limits of tolerability to an extent (between the limit value and broadly acceptable value level) determined in accordance with the ALARP principle.
- The average/broadly acceptable individual risk to the person or group is 10^{-6} per annum.

The quantification "except in exceptional circumstances" requires explanation. The discretion to decide that circumstances are exceptional should not reside with the owner, but should be a matter for government, or for a dam safety regulator acting on behalf of government. The decision would normally be based on the benefit to society of the facility, despised its risks.

For risks within the broadly acceptable region –nobody worries too much about further risk reduction unless there are some obvious low cost improvements that could be made.

8.1.1.1 Societal risk

ANCOLD guidelines on risk assessment (ANCOLD, 2003) propose that for existing dams, a societal risk that is higher than the limit curve, shown on Figure 8.**1** below, is unacceptable, except in exceptional circumstances. Also the risks are to be lower than the limits of tolerability to an extent determined in accordance with the ALARP principle.

The horizontal truncations of Figure 8.1 is without precedent, but represent ANCOLD's present judgement of the lowest risks that can be realistically assured in light of (ANCOLD, 2003);

- Present knowledge and dam's technology
- Methods available to estimate the risks

In the case of ancient earth dams, they were built long ago using very poor technology. Whilst some aspect of technology can be improved, it is simply impracticable to bring such dams fully up the safety levels of a well design and constructed modern dam. The choice is to either accept the horizontal truncation or to abandon the dam. Since dams are of significant benefit to society, it is considered that the horizontal truncation is justified.

For societal risk, the New South Wales Dam Safety Committee has adopted a negligible level, which is two orders lower than (one hundredth of) the limit of tolerability. The DSC regards the negligible level of risk as usually acceptably low. Here, the negligible level adapted by NSWDSC has been included in to the "revised ANCOLD societal risk guidelines for earth dams". So it can be taken that the risk is negligible if it is two orders lower than the limit of tolerability. ALARP should be satisfied for risk in between the limit value and negligible value.

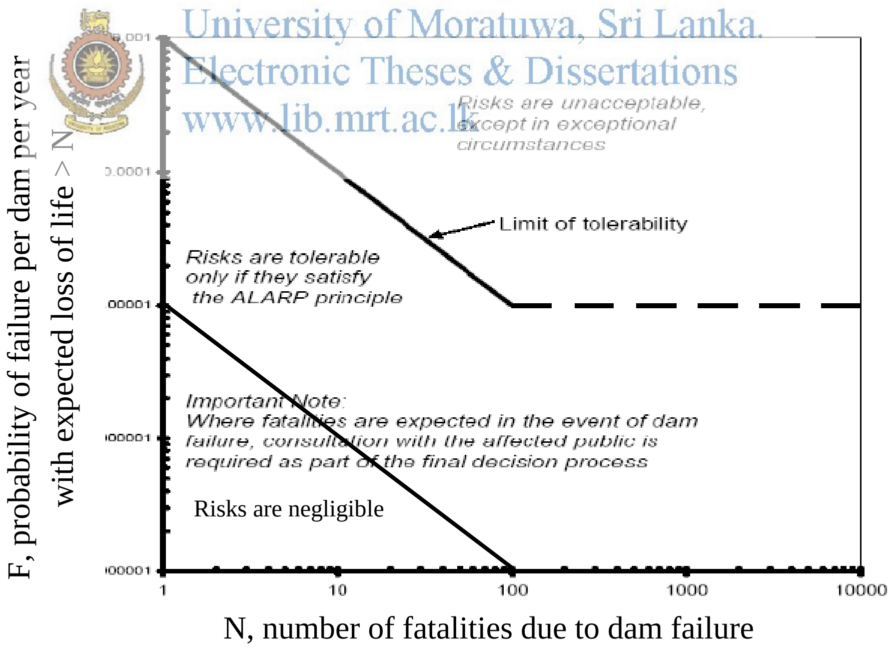


Figure 8.1: Revised ANCOLD societal risk guideline for existing dams (ANCOLD, 2003) with included negligible level.

8.1.1.2 ALARP (As Low As Reasonably Practicable) principle

The term ALARP arises from UK legislation, particularly the Health and Safety at Work etc. Act 1974, which requires "Provision and maintenance of plant and systems of work that are, as far as is reasonably practicable, safe and without risks to health". ALARP is the key determinant of tolerable risk. Determining that ALARP is satisfied is a matter for judgement by the dam owner, subject to any regulatory requirements that must be met.

Some statements of the ALARP principle are (ANCOLD, 2003):

- Risk is tolerable only if risk reduction is impracticable or if its cost is grossly disproportionate (not equal) to the improvement gained (Health and Safety Executive, 1992);
- Residual risk is tolerable only if further risk reduction is impracticable or requires action that is grossly disproportionate in time, trouble and effort to the reduction in risk achieved (HSE, 1999a).

The ALARP test requires consideration of possibilities for risk reduction. In order to satisfy the test, the dam owner needs to demonstrate gross disproportion between (ANCOLD, 2003):

- The sacrifice – the money, time and trouble required to implement risk reduction measures

  and
- The reduction in risk that would be achieved by those measures

There is no “formula” by which to decide that risks are ALARP. The owner needs to reach a judgement that sacrifice is grossly disproportionate to the reduction in risk that would be achieved. Some points that are relevant in making a judgement on whether risks are ALARP, based on practices elsewhere, are;

- Cost-to-save-a-statistical-life (CSSL) is a consideration for life safety risk;
- Whether good practice is met is a consideration;
- The level of the existing risk is the consideration;
- Societal concerns may be a consideration;
- Affordability is not a consideration for life safety risks;
- Duration that the risk applies may not be a consideration for life safety risks in some circumstance.

Bowles and his co-workers (Bowles, 2001a) have promoted the cost-effectiveness measure, cost-to-save-a-statistical-life (CSSL), as a guide to satisfaction of ALARP.

The "adjusted" CSSL is calculated as follows (ANCOLD, 2003):

$$CSSL(A) = \frac{C_A - (E[R:e] - E[R:pr]) - ([O:e] - [O:pr])}{E[L:e] - E[L:pr]}$$

Where,

CSSL (A) = adjusted CSSL, with the condition that a negative value is taken as zero

C_A = annualised cost of implementing risk reduction measures, dollars per annum

E [R: e] = existing expected value of risk cost (failure probability times monetary losses to the owner) for existing dam, dollars per annum

E [R: pr] = expected value of risk cost post-risk reduction, dollars per annum

E [L: e] = expected value of life loss for existing dam, lives per annum

E [L: pr] = expected value of life loss post-risk reduction, lives per annum

O: e = existing operating costs per annum

O: pr = post-risk reduction operating costs per annum.

In the above equation the values are in dollars, so they need to be modified to Sri Lankan rupees. The case in favour of satisfying the ALARP test strengthens progressively as CSSL value increases.

Table 8.1: Tentative guidance on ALARP justification for risks just below the limit of tolerability (ANCOLD, 2003)

ALARP Justification Rating	Range of Cost-per-statistical-life saved (A$M/life)	
	Greater than or equal to	Less than
Very Strong	Zero	5
Strong	5	20
Moderate	20	100
Poor	100	

Table 8.2: Tentative guidance on ALARP justification for risks just above the broadly acceptable risk (ANCOLD, 2003)

ALARP Justification Rating	Range of Cost-per-statistical-life saved (A$M/life)	
	Greater than or equal to	Less than
Very Strong	Zero	1.5
Strong	1.5	6
Moderate	6	30
Poor	30	

8.2 Risk Reduction Options

Formulation of options for risk reduction is a task for the risk analysis team. Options are classified as structural or non-structural. An option may be a package of measures (structural, non-structural or both). Options refers to the total package of measures that are intended to ultimately achieve tolerable risks, without regard to the period over which they would be implemented, the sequence in which they would be implemented or the details of the implementation program (ANCOLD, 2003).

The full range of possible options needs to be considered, some typical generic classes of options are (ANCOLD, 2003):

- Full physical upgrade of the dam (reducing the probability of failure);
- Physical upgrade of the dam to a base safety condition (reducing probability of failure and reducing incremental consequences to a tolerable level);
- Enhanced monitoring and surveillance procedures;
- Removing or re-locating the population and improvements, which are at risk (reducing the consequences of dam failure);
- Improved flood warning and evacuation planning (reducing the expected loss of life and minor property losses);
- Operating restrictions;
- Enhanced security measures – security checks of personnel, data protection, protection of monitoring equipment, transmission system and computer systems, security breach response plans;
- De-commissioning of the dam (removing any potential for dam failure).

Often it will be possible to achieve a significant reduction in risk through interim measures, which can be implemented quickly and will improve safety while studies of permanent risk reduction measures continue. Flood warning and evacuation planning are often implemented as an interim measure, through interim structural measures are also sometimes available. It is usual to examine the do nothing option as the datum against which other options are measured (ANCOLD, 2003).

8.2.1 The “sacrifice” in implementing each risk reduction option

Implementation of a risk reduction measure has a range of adverse consequences, which together constitute the sacrifice to be made by the owner. This sacrifice is of key importance in reaching a conclusion as to whether post-implementation risk would be ALARP (ANCOLD, 2003).

Typical elements of the sacrifice include (ANCOLD, 2003):

- The monetary cost of implementing the risk reduction option (probability of 1.0 – certain, subject only to the uncertainty of the estimate);
- Ongoing economic loss (in some cases, such as those cases where operating restrictions are part of the risk reduction option);
- Increased operating cost (a reduction in operating costs is a benefits that is to be offset against the sacrifice);
- Personal stress and effort by those involved with implementation;

There may also be something to be given up by others or society at large, in order to implement the risk reduction. For example (ANCOLD, 2003):

- Social harmony (discord is likely in the case of controversial options);
- Adverse environmental impacts (in some cases).

Implementation of a risk reduction measure can give rise to new risks, such as (ANCOLD, 2003):

- Risk of death or injury to construction workers (there is a risk);
- A temporary increase in risk of failure during construction (in some cases – it is not always possible to avoid a temporary increase in risk);
- The creation of a new, though lesser, ongoing risk of failure (it is sometimes impossible to avoid the creation of some new risk).

Such risks are not part of the sacrifice, but should be taken in to account as offsets against the main risk reduction. The sacrifice needs to be fully identified, analysed and described by the risk analysis team for each risk reduction option.

8.2.2 Select the preferred implementation strategy and program

Formulate a strategy and program for safety improvements, over dams of a portfolio, components of dams or failure modes, following these principles (ANCOLD, 2003):

- Priority (the order in which risk reduction measures are to be implemented) – give priority to the highest risks. Give priority to life safety risks over other risks;

- Urgency (how soon the measures should be implemented) – make urgency proportional to the extent by which risks exceed tolerable risks, as determined by tolerable risk policies or criteria. Risks higher than the limit of tolerability, in particular life safety risks, need to be reduced to a level below the limit as soon as practicable;
- Progressive improvement – plan for studies or safety improvements in stages, if that would achieve the best outcomes in reducing risk for the available resources.

Consider the possibility of interim risk reduction measures that can be implemented quickly to provide reasonable protection to community and business interests whilst planning for long-term risk reduction is undertaken. For small risk reduction projects, the planning for implementation may be straightforward. For larger projects, particularly those involving a package of diverse measures to be implemented over a period of years, and possibly over a portfolio of dams also, systematic planning is needed to identify a risk reduction pathway that will best meet safety goals (ANCOLD, 2003).

Where quantitative estimates of risks have been generated as part of the risk assessment process, one approach to identifying a risk reduction pathway involves tabulating each measures, its cost and the risk reduction it achieves. This can be done for both life safety and monetary loss risks (ANCOLD, 2003).

In staging of risk reduction options, a balance needs to be struck between (ANCOLD, 2003):

- Achieving the maximum rate of risk reduction for the available rate of recourse input;
- The need for practical and cost effective construction packages.

QUANTITATIVE RISK ASSESSMENT OF NACHCHADUWA DAM: A CASE STUDY

CHAPTER 9

9.1 Introduction

The Nachchaduwa dam is owned by the Irrigation Department (ID) and is situated some 15 km south east of Anuradhapura. It is an ancient tank built to supply the city tanks, and was restored about one hundred years ago in 1906 and improved in 1917. It breached during 1957 storm and was again damaged following the 1978 cyclone. The layout of this scheme is comprehensive, with earthfill and gravity dams, gated and ungated spillways and three sluices.

In the historical view, the Nachchaduwa tank is attributed to King Moggallana II (535-555 A.D), great tank builder and identified as Pattapasana Vapi of old. Later it is reported as being repaired and restored by Vijeyabahu (1055-1110 A.D). Nachchaduwa dam is built across Malwathu Oya and its tributary Maminiya Oya. There are four medium tanks and hundreds of minor tanks in the catchment. The area consists of jungle, paddy fields, hamlets and chena with moderate slope.

Here we have selected the initial level risk assessment, considering the available data and time constrain. Therefore the downstream dam (dams which get supply from Nachchaduwa dam) failures are not considered in this study.

9.2 Inspection of Dam and Inundation Area

Nachchaduwa dam is believed to be an essentially homogenous earthfill dam with associated spillway and sluice structures. There is no known zoning of fill materials. Originally the dam is being constructed based on the bund of an ancient tank which has been restored various times to give the present dam.

The upstream slope of initially 1(v):2.5(h) is generally protected with riprap. The slope is grassed above the riprap line. There is significant tree growth along the dam crest and upstream slope; many trees are of large size and considerable age.

The dam crest is around 3.5 to 4 m wide and carries an unsurfaced road on the centreline. The profile is reasonably regular and no major settlement was identified.

The downstream slope is initially given as 1(v):2(h). Upper areas of the slope are generally grassed but in the toe zone there is a considerable number of large trees. There are areas which are heavily settled and eroded by pedestrian and animals crossing the slope. There are three piezometers located downstream of the dam.

The dam is with gated and ungated spillways and three sluices. The main ungated spillway (length 142 m) is a mass concrete structure buttresses build on a massive outcrop of rock in the bed of the river. Adjacent to the ungated weir, at its left-hand end, is the gated auxiliary spillway built with concrete with six vertical gates, each of 2.77 m (w) ×2.29 m (h).

At the right–hand end of the main spillway, a three bay masonry faced sluice to supply water to the low-level Nuwarawewa transfer canal is constructed. The sluice is called “Right bank sluice”. An abandoned central sluice at ch 01+250 m is an old sluice and has been plugged with concrete at the upstream end. At the left abutment of the dam, a two gated concrete/natural stone (masonry) outlet which discharges into a pond from which high and low level canal are fed. The right bank high level sluice is a recently build sluice and consist of 8 gates to discharge water to the Nuwarawewa feeder canal. The sluice is situated several kilometres north of the dam, along the main road to Anurathapura.

9.2.1 Tank data

Crest level	: 104.32 m MSL
Full supply level	: 101.68 m MSL
Crest width	: 3.5 – 4.0 m
Upstream slope	: 1 (v): 2.5 (h)
Downstream slope	: 1 (v): 2 (h)
Length	: 1650 m
Nature	: Earth fill
Fill material	: Clayey sand

9.3 Identifying the Hazards

According to the developed guidelines the following hazards are categorized as obvious hazards for Nachchaduwa dam:

- Normal operating load;
- Flood load.

9.4 Identifying the Failure Modes

The failure modes are classified by hazard situation as given in the previous section.

9.4.1 Comprehensive facility review (CFR) – identifying different failure modes

From the comprehensive facility review, the positive and adverse factors for the following failure modes are identified.

Failure modes identified under normal operating load

1 Seepage water through the embankment into the foundation carrying embankment materials into joints and fractures of the foundation rock:

Positive Factors Making Failure Mode Less Likely:

- Piezometers indicate low pressures in the embankment;
- Embankment material is sandy, not so easily eroded in to the rock and self healing;
- Fracture of the rock may be too small to accept the embankment materials;
- Rock is not very fractured or jointed;
- No seepage seen exiting from rock.

Adverse Factors Making Failure Mode More Likely:

- No filters within the embankment;
- Peizometers have not been in place for very long;
- Seepage is evident at the downstream toe.

2 Seepage water through the embankment carrying embankment materials to the downstream face of the dam:

Positive Factors Making Failure Mode Less Likely:

- Piezometers indicate low pressures in the embankment;

- Embankment material is sandy, not so easily eroded in to the rock and self healing;
- The animal burrow holes were not deep and showed no seepage;
- No crack evident in the embankment;
- Seepage reduces when the reservoir is less than 21 feet deep;
- Embankment is well tested at full reservoir;
- No evidence of sediment transport with the seepage flow through the embankment.

Adverse Factors Making Failure Mode More Likely:

- No filters within the embankment;
- Peizometers have not been in place for very long;
- Trees exists on the embankment and in the downstream toe area;
- Seepage is evident at lower elevations of the downstream embankment slope;
- Upstream slope had burrows and ant hills;
- Downstream slope has a few animals (wild pigs) burrows;
- High reservoir levels occur annually and last 6 months;
- Downstream slope was uneven and has some shallow slope failures.

3 Seepage water through the weathered foundation (overlying soils) carrying foundation materials to an exit downstream of the dam:

Positive Factors Making Failure Mode Less Likely:

- There is no seepage exiting when the reservoir water is low, even though water is over the foundation;
- Eroded materials are coarse and not highly erodible. Seepage failure would take a long time;
- Eroded material not likely to sustain a roof;
- Dam has existed for a very long time without the full development of this failure mode.

Adverse Factors Making Failure Mode More Likely:

- Foundations conditions not well known;
- Substantial seepage is evident at the downstream toe;

- Seepage was large and pressurized at the downstream toe at station 1+545, carrying sandy sediment;
- Sand boils at station 0+700 m. significant quantity of flow and eroding foundation materials.

4 Seepage water through the fractured rock at the embankment contact picks up embankment material and carries it to a downstream exit:

Positive Factors Making Failure Mode Less Likely:

- There is no seepage exiting when the reservoir water is low, even though water is over the foundation;
- No seepage failure of the dam for over 1,000 years;
- Seepage rate has been steady for the last 5 years;
- Rock is not very fractured or jointed;
- No seepage seen exiting from rock. Seepage flows around or above the rock.

Adverse Factors Making Failure Mode More Likely:

- Foundations conditions not well known;
- Seepage is evident at the downstream toe, especially where there are rock outcrops;
- No foundation treatment;
- We do not know the history of seepage prior to 2002;
- Tree roots could open up the fractures and joints of the rocks.

5 Seepage along the outside walls of the old abandoned sluice structure:

Positive Factors Making Failure Mode Less Likely:

- No seepage is obvious exiting along the outside of the conduit;
- No sediment deposits at the end of the conduit.

Adverse Factors Making Failure Mode More Likely:

- The conduit of the structure is unknown;
- Structure may collapse;
- Bottom of the conduit may form a roof and erosion could occur from beneath;
- Low stress conditions may exist along the outside of the side walls.

6 Seepage along the outside of the walls of the old, abandoned sluice structure exiting into the conduit:

Positive Factors Making Failure Mode Less Likely:

- Water observed at the end of the conduit could be due to an inadequate upstream plug (poor seal);
- Slope over the conduit has appeared irregular for a long time;
- End of conduit can be observed.

Adverse Factors Making Failure Mode More Likely:

- Seepage has been seen exiting from the conduit outlet;
- Mortar is very old. Mortar probably has cracks and is continuing to deteriorate;
- Conduit could collapse;
- Conduit, founded on soil, likely has settled and has open joints;
- Transverse depressions above the conduit could indicate the erosion of materials associated with the conduit. This type of settlement was evident over the left outlet that failed;
- Seepage may occur through the brick walls;
- Conduit is close to the reservoir where full hydrostatic head applies.

7 Seepage adjacent or beneath the emergency gated spillway structure:

Positive Factors Making Failure Mode Less Likely:

- Backside of the wall is uneven so the wall and soil have a good contact;
- No evident of seepage between the wall and the soil backfill;
- No seepage exiting at the downstream end of the wall;
- Seepage beneath the gates is probably flowing through rock.

Adverse Factors Making Failure Mode More Likely:

- Seepage is evident on the left side beneath the elevation of the gates.

8 Wind generated waves erode the riprap and underlying embankment. Scour holes occur in the upstream slope, over steepening the slope. Slope failures progress through the crest and beneath the dam:

Positive Factors Making Failure Mode Less Likely:

- Failure takes a long time to develop.

Adverse Factors Making Failure Mode More Likely:

- Monsoon season is a difficult time to repair the riprap;
- Funds are not available to repair the slope;
- Riprap design is not adequate for the wave attack. Bedding is not compatible with the riprap;
- A few places of erosion were observed on the upstream slope up to 1 m in depth that is over steepened. Over steepened slopes are vertical in some areas.

9 Downstream slope becomes unstable and slips, resulting in an over steepened downstream slope. Slope instability progresses upstream, involves the dam crest and encounters the reservoir. The reservoir flows through the slide area and forms a breach, releasing the reservoir:

Positive Factors Making Failure Mode Less Likely:

- Progression to the reservoir is unlikely;
- Rain could be causing the slope instability;
- Slips observed were fairly shallow and contained to within the embankment.

Adverse Factors Making Failure Mode More Likely:

- Scarps and slumps are evident on the downstream slope;
- Some of the downstream slope is steep;
- Embankment may be composed of clay or week materials in some areas;
- Some areas of the embankment may not have been compacted very well;
- Given slope instability has occurred, some of the embankment is at residual shear strength.

Failure modes identified under extreme flood load

1 Extreme flood event overtops the embankment:

Positive Factors Making Failure Mode Less Likely:

- Grass cover protect the downstream slope;
- Breaching section and gates are available to pass large floods.

Adverse Factors Making Failure Mode More Likely:

- Around the 1000-yr flood event overtops the embankment;
- Crest is not paved;
- No erosion protection on the downstream slope except for some vegetation;
- Crest elevation has some minor variations;
- Embankment is composed of a sandy, erodible material;
- Gate operation is difficult in the emergency spillway, especially during a flood event;
- Small dams upstream could fail and add to the inflows during extreme flood events such as what occurred in 1957;
- Hesitation may occur to operate the gates because of downstream consequences resulting from high spillway flow;
- Trees on the dam could concentrate erosive overtopping flows;
- Trees or other debris could possibly block the debris structure beneath the footbridge or a gate.

2 Increase likelihood of seepage failure during a flood event:

Positive Factors Making Failure Mode Less Likely:

- Animal burrows were not numerous.

Adverse Factors Making Failure Mode More Likely:

- Upstream slope had burrows and ant hills.

3 Flow over the ogee spillway undermine the spillway structure:

Positive Factors Making Failure Mode Less Likely:

- Rock is competent. Some depressions exist, but are in rock and have stabilized.

Adverse Factors Making Failure Mode More Likely:

- Leakage through the structure is evident.

9.4.2 Failure modes included in to the study

The following failure modes are included in the study, considering the data gathered from CFR:

Normal operating load:

- Internal erosion and piping through the embankment – in the dam;
- Internal erosion and piping through the embankment – along and into the conduit;
- Internal erosion and piping through the weathered foundation;
- Downstream slope instability.

Flood load:

- Embankment overtopping;
- Internal erosion and piping through the embankment – in the dam;
- Internal erosion and piping through the embankment – along and into the conduit;
- Internal erosion and piping through the weathered foundation.

Other failure modes are excluded from the studies, considering the data collected from the CFR and the current condition of Nachchaduwa dam.

9.5 Evaluating the Load States

A representative critical load state from the normal operating load and flood load was selected for the analysis. The loading states selected under normal operating load and extreme flood load are discussed in the following sections.

9.5.1 Normal operating load

Since this is an initial level risk assessment, it was assumed that the reservoir is always at Full Supply Level (FSL). So the probability of loading state is taken as 1.0 under normal operating load.

9.5.2 Extreme flood load

The extreme flood level was assumed as 103.6 m MSL with the Annual Exceedance Probability (AEP) of 1:1000.

9.6 Estimation of Probabilities

9.6.1 Internal erosion and piping

9.6.1.1 Probability of failure under normal operating load

9.6.1.1.1 Internal erosion and piping through the embankment – in the dam

According to the site investigation report of Nachchaduwa dam prepared under the proposed dam safety and water resources planning project, the permeability value of the fill material is in the range of 10^{-2} to 10^{-3} m/s. But this permeability range is high for an earth dam. Since the fill material is clayey sand, it was assumed an average permeability value of 10^{-7} m/s in the analysis.

Initiation

Nachchaduwa dam is believed to be an essentially homogenous earthfill dam and there is no known zoning of fill materials. The fill material is a compacted clayey sand underling by small amount the residual formation.

Cracking or wetting induced collapse susceptibility of core materials

The dam was constructed nearly 1500 years ago and the embankment was compacted using the elephants. So the embankment can be considered as poorly compacted with less than 95% stranded relative compaction. Also the compaction water content can be taken as the dry of standard optimum water content (approx. OWC – 3%). In terms of soil type, since it is clayey sand, it comes under medium plasticity clay fines. According to Table 5.3, the likelihood of cracking or wetting induced collapse susceptibility of the embankment fill material is high.

Hydraulic fracture

The upstream slope has some burrows, ant hills and steep slope at few locations. The downstream slope is uneven and has some shallow slope failures. The overall abutment profile is relatively a flat slope with some irregularities. The dam was constructed on a rock foundation; therefore it doesn't have much soil in the foundation. So there is no chance for differential foundation settlement. Hence, according to Table 5.4, likelihood of low stress conditions is low.

High permeability zone

The quality of the construction might be less compared to the present technology. There is no chance for any engineering supervision during the construction. As discussed under transverse crack, the embankment can be considered as poorly compacted with less than 95% stranded relative compaction. Also the compaction water content can be taken as the dry of standard optimum water content (approx. OWC – 3%). There is no instrumentation in the embankment for very long. Trees and animal burrows exist on the embankment and downstream of the toe. Hence, according to Table 5.5, likelihood of high permeable zone present within the embankment is high.

Suffusion

The fill material is clayey sand the particle size distribution doesn't comes under well graded or poorly graded range. Since the fill material is clayey sand, the permeability of the fill material is assumed as 10^{-7} m/s. As discussed under transverse crack, the embankment can be considered as poorly compacted with less than 95% stranded relative compaction. Hence, according to Table 5.6, the likelihood of initiation by suffusion is slightly above average.

Considering all the above conditions, according to Table 5.**2**, the probability of initiation is taken as 0.2.

Continuation

Continuation of internal erosion is mainly depending on the filter criteria. The Nachchaduwa dam is a homogeneous earthfill dam and there are no filters within the embankment. So the likelihood of continuation is very high.

Considering all the above conditions, according to Table 5.**2**, the probability of continuing erosion is taken as 0.9.

Ability to support a roof

The most important factor influencing the ability of a material to support a roof is the fines content. The embankment fill material is of clayey sand with approximately 20% fine content. Also the degree of saturation of the soil can be taken as partially saturated. Hence, according to Table 5.**9**, the ability to support a roof is high.

Considering all the above conditions, regarding to Table 5.**2** the probability of ability to support a roof is taken as 0.5.

Limitation of flow

There are no filters within the embankment. There is no known zoning of fill materials, therefore the ability to Fill the cracks by washing in of material from upstream and to restrict the flow are very low. Hence, according to Table 5.10, the likelihood of pipe enlargement is nearly high.

Considering all the above conditions, according to Table 5.**2**, the probability of inability to limit the flow is taken as 0.6.

Erodibility

The plasticity index of clayey sand is nearly 15. So it can be considered as plastic clay. As we discussed under transverse crack, the embankment can be considered as poorly compacted with less than 95% standard relative compaction. Also the compaction water content can be taken as in the dry of standard optimum water content (approx. OWC – 3%). Using the geo-slope model, it was identified that the hydraulic gradient across the embankment is low. Hence, according to Table 5.11, the chances of filling materials being eroded are close to average.

Considering all the above conditions, according to Table 5.**2**, the probability of soil erodibility is taken as 0.009.

Early intervention

In Nachchaduwa dam leakage is generally accessible on downstream slope. The access to the location is available and the dam is being monitor by officials. There are few pizometers installed in the dam for monitoring purposes. But there is no known instrumentation, other than piezometers in the dam. In terms of stability, there is no crack evident in the embankment. Also if there is a piping failure, they can be identified earlier from the downstream toe. Apart from these, the embankment is of 1650 m long. So this may cause some difficulties to monitor. Considering all the above factors, the likelihood of early intervention is high.

Considering all the above conditions, according to Table 5.**2**, the probability of early intervention is taken as 0.5.

Breach Mechanism

Nachchaduwa dam is believed to be an essentially homogenous earthfill dam with no known zoning of fill materials. The storage volume is comparatively large. Hence, according to Table 5.12, the likelihood of breaching of the dam by gross enlargement is high.

The freeboard during the full supply level is nearly 3m. The crest is around 3.5 to 4 m wide. Hence, according to Table 5.13, the likelihood of breaching of the dam by Sinkhole or crest settlement is low. Even though the likelihood of breaching by sinkhole or crest settlement is low, the likelihood of breaching by gross enlargement is high. So the dam is likely to breach by gross enlargement.

Considering all the above conditions, according to Table 5.**2**, the probability of formation of breach mechanism is taken as 0.4.

Conditional probability of failure by internal erosion and piping through embankment-in dam is calculated as 9.7×10^{-5}.

Refer to Figure 9.1 for the event tree and conditional probability of failure for internal erosion and piping through embankment- in dam.

9.6.1.1.2 Internal erosion and piping through the embankment – along and into the conduit

Initiation

The chances of initiation of piping along and into the conduit are high in dams less than 30 m height (Foster et al, 1999). The maximum height of the Nachchaduwa dam above the foundation is 10. 7 m. Also there is no known zoning within the embankment. Since, there is no engineering supervision during the construction, the embankment can be considered as poorly compacted.

The conduit is made of masonry brick. There are some cracks generated in the conduit. Also the conduit is founded on soil, likely has settled and has open joints. The trenches are of medium depth. There are transverse depressions above the conduit which could indicate the erosion of materials associated with the conduit. The mortar is very old and probably has cracks and is continuing to deteriorate. Hence, according to Table 5.14, the likelihood of concentrated leak associated with a conduit is high.

Considering all the above conditions, according to Table 5.**2**, the probability of initiation is taken as 0.5.

Continuation

There are no filters within the embankment. Since the conduit is located within the embankment there is no filtering action around the conduit. So the likelihood of continuation of piping along the conduit is very high.

The water observed at the end of the conduit doesn't have any eroded materials. So it was assumed that the erosion comes under some erosion category. So the likelihood of continuation of erosion into the conduit is low.

Eventhough the likelihood of continuing erosion into the conduit is low; the likelihood of continuing erosion along the conduit is very high.

So it was taken that the likelihood of continuing erosion along or into the conduit as high.

Considering all the above conditions, according to Table 5.**2**, the probability of continuing erosion is taken as 0.99.

Progression

The factors influencing the likelihood of progression of piping along and into conduit are same as piping through the dam embankment. But the likelihood of erosion developing beyond the initiation stage is greater than without a conduit. However the progression of piping may be limited or slowed due to the limited width of the open joint or crack. Filtering of the embankment materials against the crack, particularly if the crack is narrow and the embankment materials well graded may prevent the continuation of piping.

Considering all the above conditions, according to Table 5.**2**, the probability of ability to support a roof is taken as 0.5.

Considering all the above conditions, according to Table 5.**2**, the probability of inability to limit the flow is taken as 0.65.

The compaction along the conduit might be influenced by the soil conduit interaction. So, according to Table 5.**2**, the probability of soil erodibility is taken as 0.02.

Early intervention

The factors influencing on the likelihood of early intervention are same as for piping through dam embankment. So the likelihood of early intervention is high.

Considering all the above conditions, according to Table 5.**2**, the probability of early intervention is taken as 0.5.

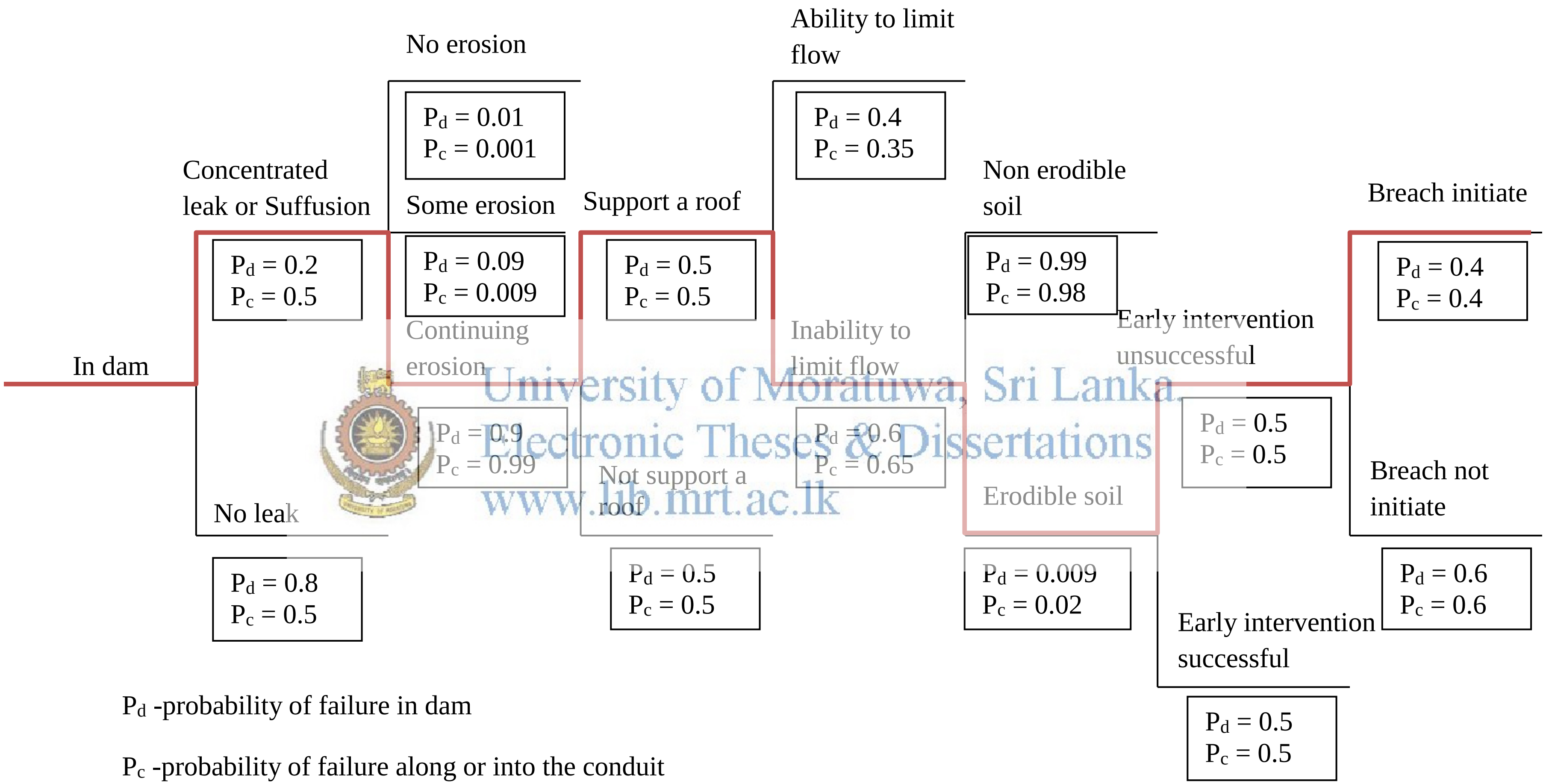
Breach mechanism

The factors influencing the likelihood of breaching due piping along and into conduit are same as piping through the dam embankment.

Considering all the above conditions, according to Table 5.**2**, the probability of formation of breach mechanism is taken as 0.4.

Conditional probability of failure by internal erosion and piping through embankment-along or into conduit is calculated as 6.44×10^{-4}.

Refer to Figure 9.1 for the event tree and conditional probability of failure for internal erosion and piping through embankment- along or into conduit.



P_d -probability of failure in dam

P_c -probability of failure along or into the conduit

Conditional Probability of failure for internal erosion and piping through embankment – in dam $= 9.70\text{x}10^{-5}$

Conditional Probability of failure for internal erosion and piping through embankment – along or into conduit $= 6.44\text{x}10^{-4}$

Figure 9.1: Event tree for internal erosion and piping through embankment

9.6.1.1.3 Internal erosion and piping through the foundation

Initiation

Nachchaduwa dam is founded on a bed rock. The rock is of igneous type and there are no cracks present in the foundation. Few geological features like joints are present, but they are not critical as to develop piping. The permeability of the rock foundation is very low compared to the embankment fill material. Also the dam has existed for a very long time without the full development of this failure mode. Hence, according to Table 5.16, the likelihood of developing a concentrated seepage path through the foundation is very low.

The Nachchaduwa dam is founded on bed rock with high density. The permeability of the foundation is very low. So there is no chance for initiation of piping by suffusion.

The foundation at downstream toe has only low permeable layer. The sand boils at station 0 +700 m with significant quantity of flow and eroding materials. The factor of safety (F_u) for effective stress condition is calculated as 2.46. Hence, according to Table 5.18, the likelihood of initiation by blow out is comparatively low.

Considering all the above conditions, according to Table 5.**2**, the probability of initiation is taken as 0.001.

Continuation

In Nachchaduwa dam, the exit is unfiltered. So the likelihood of continuation of piping is high.

Considering all the above conditions, according to Table 5.**2**, the probability of continuing erosion is taken as 0.5.

Ability to support a roof

The foundation is of bed rock and there is no chance of piping trough the solution features in rock. So the ability to support the roof is very low.

Considering all the above conditions, according to Table 5.**2**, the probability of ability to support the roof is taken as 0.0005.

Restriction of erosion

The hydraulic gradient across the foundation is very low. But, the dam doesn't have any zoning and the dam foundation is of rock. Hence, with regarding to Table 5.20, the likelihood of limitation of flow is nearly average. So the likelihood of pipe enlargement is average.

The foundation is made with rock of high density and stiffness. Hence, according to Table 5.21, the likelihood of erodibility is very low.

Considering all the above conditions, according to Table 5.**2**, the probability of inability to restrict the erosion is taken as 0.004.

Early intervention

The factors influencing on the likelihood of early intervention are same as for piping through dam embankment. So the likelihood of early intervention is high.

Considering all the above conditions, according to Table 5.**2**, the probability of early intervention is taken as 0.5.

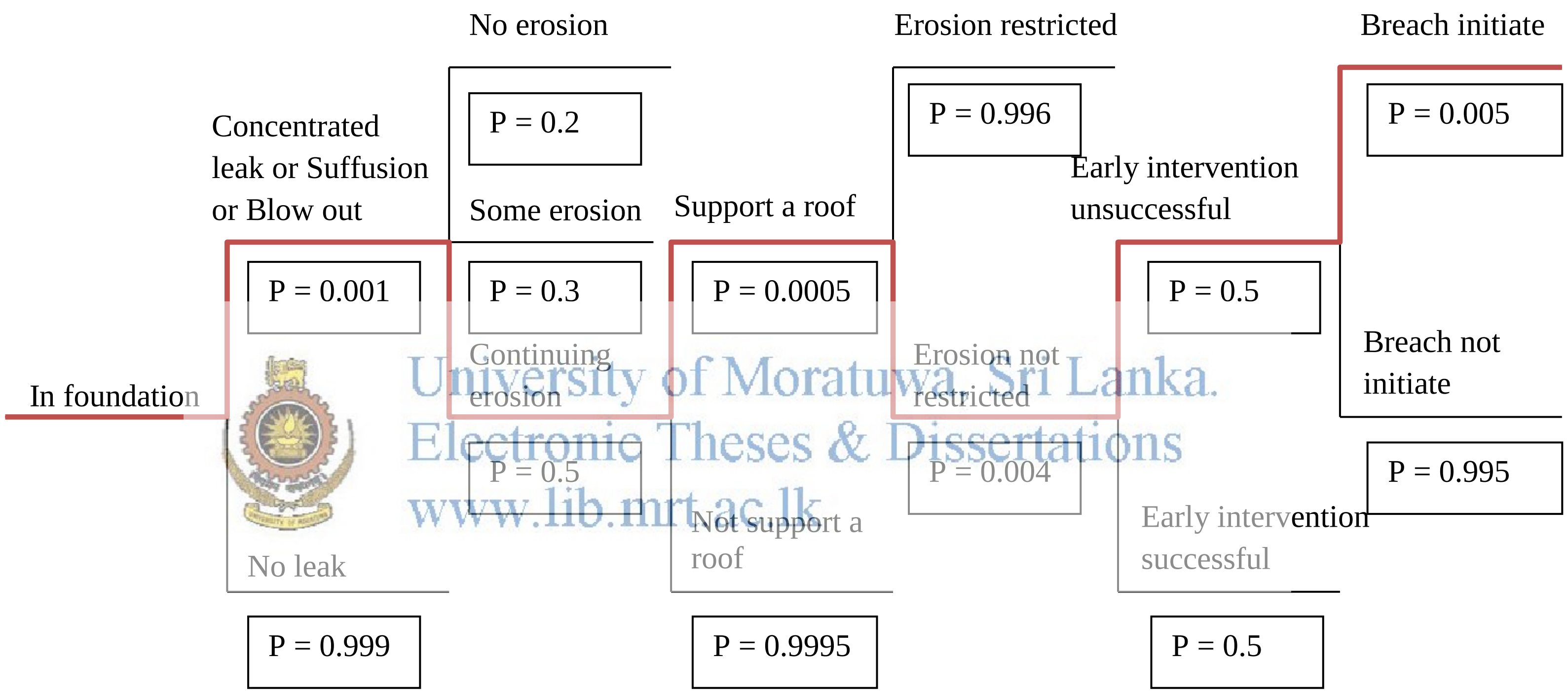
Breach mechanism

Nachchaduwa is a homogeneous dam with large storage volume. Gross enlargement is likely if there is continuing enlargement of the pipe and the roof of the pipe can be supported along its full length. Gross enlargement of the pipe can result in the collapse of the crest or emptying of the reservoir through the pipe. This breach mechanism requires continuing enlargement of the pipe and the pipe through the foundation has to remain open. Since the rock foundation is without any cracks or large opening. The likelihood of gross enlargement is low.

As for piping through the embankment, influential factors are the crest width and freeboard at the time of the incident, and to a lesser extent, the characteristics of the downstream zone if overtopping does occur. The rock foundation is without any cracks or large opening. So the likelihood of breaching of the dam by Sinkhole or crest settlement is low.

Considering all the above conditions, according to Table 5.**2**, the probability of formation of breach mechanism is taken as 0.005.

Therefore, the conditional probability of failure by internal erosion and piping through embankment-along or into the conduit is calculated as 2.5×10^{-12}. Refer to Figure 9.2 for the event tree and conditional probability of failure for internal erosion and piping through foundation.



Conditional Probability of failure for internal erosion and piping through the foundation = 2.50×10^{-12}

Figure 9.2: Event tree for internal erosion and piping through foundation

9.6.1.2 Probability of failure under extreme flood load

The factors influencing on the likelihood of internal erosion and piping is same for both normal operating load and extreme flood load. But the reservoir level chances under normal operating load and flood loading condition. Reservoir water level is recognized as an important factor on the likelihood of a concentrated leak forming, pipe enlargement and of the formation of a breach mechanism.

So it was assumed that the likelihood of initiation, pipe enlargement and the formation of breach mechanism under extreme flood loading, increase by the percentage given below:

- Initiation -30 %
- Pipe enlargement -20 %
- Formation of breach mechanism -50 %

Table 9.1: Probability values for internal erosion and piping through the embankment under extreme flood loading

Events	Probability values for piping trough the embankment	
	In the dam	Along or into the conduit
Initiation	0.26	0.65
Continuation	0.9	0.99
Ability to support the roof	0.5	0.5
Inability to limit the flow	0.72	0.78
Soil erodibility	0.009	0.02
Unsuccessful early intervention	0.6	0.6
Formation of breach mechanism	0.6	0.6
	2.724×10^{-4}	1.807×10^{-3}

Table 9.2: Probability values for internal erosion and piping through the foundation under extreme flood loading

Events	Probability values for piping trough foundation
Initiation	0.0013
Continuation	0.5
Ability to support the roof	0.0005
Inability to restrict the erosion	0.0048
Unsuccessful early intervention	0.6
Formation of breach mechanism	0.0075
	7.02×10^{-12}

9.6.2 Downstream slope instability

Nachchaduwa is an ancient dam built around 1500 years ago, the quality of the construction might be less compared to the present technology. There is no chance for any engineering supervision during the construction. Hence, regarding to Table 5.22, it was assumed that the Nachchaduwa dam to be under category III. From the geo-slope model, the factor of safety of downstream slope stability under full supply level is estimated at 1.8.

Hence, according to Figure 5.12, the probability of failure by slope instability is calculated as 7.5×10^{-4}.

9.6.3 Embankment overtopping

The assumed high flood level is 103.6 m MSL. But the average embankment crest elevation is at 104.32 m MSL. Therefore, there won't be any overtopping failure under this flood loading. So the probability of failure is zero.

9.6.4 Combining probabilities of failure modes initiated by normal operating load

Table 9.3: Combined probabilities of failure under normal operating load

Annual probability of reservoir level state (1)	Failure Mode (2)	Conditional probability of failure (3)	Annual probability of failure (4) = (1) x (3)
1.0	Piping through the embankment- in dam	9.7×10^{-5}	9.7×10^{-5}
	Piping through the embankment-along or into the conduit	6.44×10^{-4}	6.44×10^{-4}
	Piping through the foundation	2.5×10^{-12}	2.5×10^{-12}
	Downstream slope instability	7.5×10^{-4}	7.5×10^{-4}
Total for normal operating conditions			1.49×10^{-3}

Since these failure modes occurs at previously experienced water levels (except on first filling), the four modes of failures are taken to be mutually exclusive.

9.6.5 Combining probabilities of failure modes initiated by extreme flood load

Table 9.4: Combined probabilities of failure under extreme flood load

Annual probability of Extreme flood load (1)	Failure Mode (2)	Conditional probability of failure (3)	Conditional probability of failure for flood load (4)	Annual probability of failure for flood load (5) = (1) x (4)
0.001	Piping through the embankment- in dam	2.724×10^{-4}	2.079×10^{-3} (U) 1.807×10^{-3} (L)	2.079×10^{-6}(U) 1.807×10^{-6} (L)
	Piping through the embankment- along or into the conduit	1.807×10^{-3}		
	Piping through the foundation	7.02×10^{-12}		
	Embankment overtopping	zero		
Total for flood conditions				2.079×10^{-6}(U) 1.807×10^{-6} (L)

Note: U- Upper bound, L- Lower bound

9.7 Estimating the Consequences

In this case study the load states under full supply level and expected extreme flood was included. Also the probabilities and the consequences are calculated for the whole dam embankment, without considering different components. Here, in this case study we have given priority to the life safety consequences.

9.7.1 Estimating the life safety consequences

According to the information provided by resident engineer, the inundation area under "no failure" condition doesn't have any population. There are only paddy fields in this area. So the population at risk is zero under "no failure" condition. Therefore the incremental consequences are equal to the failure consequences. In the following section, consequences under failure condition are estimated.

9.7.1.1 Estimating the loss of life

The loss of life is estimated using the method proposed by Graham (1999). The different steps proposed by Graham are discussed below.

Step 1 - Determine dam failure scenarios to evaluate

In this case study, four failure modes under normal operating load and four failures under extreme flood load were included. The whole dam embankment was considered as one component and the probabilities are estimated under critical condition. Here, it was assumed that the dam break flood conditions same for all the locations by considering a constant downstream condition. The included failure scenarios are given in Table 9.5.

Table 9.5: Included dam failure scenarios

Load Scenario	Dam Component	Failure Mode	Exposure Scenario
Normal operating load at full supply level	Embankment	Piping through the embankment- in dam	Residents of Anuradhapura district
		Piping through the embankment-along or into the conduit	
		Piping through the foundation	
		Downstream slope instability	
Natural extreme flood load	Embankment	Piping through the embankment- in dam	
		Piping through the embankment-along or into the conduit	
		Piping through the foundation	
		Embankment overtopping	

Step 2 - Determine time categories for which loss of life estimates are needed

In this case study, both day and night category is considered and the most critical category out of day and night are included in the study. According to Table 6.**2**, night time without any observers is taken as the critical category.

Step 3 - Determine when dam failure warnings would be initiated

Regarding to Table 6.**2** and the experienced judgement, the warning times for different failure scenarios are determined as in table.

Table 9.6: Warning time for different failure scenarios

Load Scenario	Dam Component	Failure Mode	Warning Time
Normal operating load at full supply level	Embankment	Piping through the embankment- in dam	1.0 hours. after flood water reaches populated area
		Piping through the embankment-along or into the conduit	
		Piping through the foundation	
		Downstream slope instability	1.0 hours. after flood water reaches populated area
Natural extreme flood load	Embankment	Piping through the embankment- in dam	More than 1.0 hrs. after flood water reaches populated area
		Piping through the embankment-along or into the conduit	
		Piping through the foundation	
		Embankment overtopping	1.0 hours. after flood water reaches populated area

Step 4 - Determine area flooded for each dam failure scenario

Since this is an initial level risk assessment, it was assumed that Anuradhapura town, Nachchaduwa division and 25% of Mahavilachchiya division will be flooded under the entire failure scenarios. If proper estimation to be made, it is required to carry out dam-break analysis for each failure scenarios.

Step 5 - Estimate the number of people at risk for each dam failure scenario and time category

In this case study the most critical time category is considered and assumed that the area flooded is be same for all the failure modes included in the study. According to the available data, the population of the selected areas are as follow:

- Anuradhapura town – 40000

- Nachchaduwa division – 25464
- Mahavilachchiya division – 22258

According to the above information, the population at risk in the dam-break zone is estimated at 71029.

Step 6 - Estimating the number of fatalities

The factors influencing on the estimation of fatalities are flood severity, amount of warning and a measure of whether people understand the severity of the flooding. The fatality rate was estimated in accordance with Table 6.3.

The flood severity is assumed as low for normal operating conditions and medium for extreme flood conditions, considering the distance to the populated area, dam height and reservoir water level during the failure.

The warning time is taken as the most critical range. The warning time for all the failure scenarios comes under no warning category as per the assumptions. Therefore, the understanding of the severity of flood is not applicable.

The following table shows the estimated fatality rates for different failure scenarios included in the study.

Table 9.7: Fatality rate for different failure scenarios

Load Scenario	Failure Mode	Flood Severity	Amount of Warning	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)
Normal operating load at full supply level	Piping through the embankment- in dam	Low	No warning	Not applicable	0.0075
	Piping through the embankment-along or into the conduit	Low			0.0075
	Piping through the foundation	Low			0.0075
	Downstream slope instability	Low	No warning		0.01
Natural extreme flood load	Piping through the embankment- in dam	Medium	No warning		0.04
	Piping through the embankment-along or into the conduit	Medium			0.04
	Piping through the foundation	Medium			0.035

Table 9.8: Number of life loss for different failure scenarios

Load Scenario	Failure Mode	Fatality Rate (Fraction of people at risk expected to die)	Number of Life Loss
Normal operating load at full supply level	Piping through the embankment- in dam	0.0075	533
	Piping through the embankment-along or into the conduit	0.0075	533
	Piping through the foundation	0.0075	533
	Downstream slope instability	0.01	711
Natural extreme flood load	Piping through the embankment- in dam	0.04	2842
	Piping through the embankment-along or into the conduit	0.04	2842
	Piping through the foundation	0.035	2487

Step 7 - Evaluate uncertainty

- Here it was assumed that the constant downstream condition for all the failure scenarios. But in real conditions, dam-break flood will be different for each failure modes. So the dam-break analysis should be done for each failure modes in detailed assessments.
- In dam failure scenarios we have considered only the residents of downstream area. But the exposure factor will change for different group of people based on their time of exposure to the dam break flood. So the different group of people should be considered separately in the estimation of life safety consequences.
- In this case study, it was considered that the flood severity will be constant for all the downstream areas. But in real conditions, the height, velocity, and severity of the flood will change with different downstream conditions, depending on the distance, geological features, etc. So in detailed assessment these conditions should be properly analysed.
- In terms of warning time, different time categories should be considered. The warning time will change according to the time category and the condition of observation.
- The fatality rate and number of life loss are estimated based on assumptions. But for detailed assessments the factors influencing on the fatality rate should be properly identified from proper analysis.

9.8 Estimation of Risk

In this case study, the life safety risks is estimated in terms of individual risk and societal risk. Here, the residents of the dam-break zone were considered for the risk estimation and exposure factor is taken as 1.0.

Table 9.9: Annual probability of overall dam failure scenarios

Load Scenario	Failure Mode	Annual Probability of Failure	Exposure Factor	Annual Probability of Overall Dam Failure Scenario
Normal operating load at full supply level	Piping through the embankment- in dam	9.7×10^{-5}	1.0	9.7×10^{-5}
	Piping through the embankment-along or into the conduit	6.44×10^{-4}	1.0	6.44×10^{-4}
	Piping through the foundation	2.5×10^{-12}	1.0	2.5×10^{-12}
	Downstream slope instability	7.5×10^{-4}	1.0	7.5×10^{-4}
Natural extreme flood load	Piping through the embankment- in dam	2.724×10^{-7}	1.0	2.724×10^{-7}
	Piping through the embankment-along or into the conduit	1.807×10^{-6}	1.0	8.124×10^{-7}
	Piping through the foundation	7.02×10^{-15}	1.0	7.02×10^{-15}

Here, it was reported the risk to life as, the individual risk to life for the person or group most at risk and societal risk to life. The following sections show the tabulated results of individual risk f life and societal risk of life.

9.8.1 Individual risk of life

In this case study, the individual risk of life was calculated for the residents of the dam-break zone. Here it was assumed that all the residents in the dam-break zone bear the same amount of risk. Since the conditional probabilities are low, the individual risks of life contributed by each of the failure scenarios are aggregated by simple addition. The individual risk of life for the residents of the dam-break zone, under each of the identified failure scenarios are given in Table 9.10.

Table 9.10: Individual risk for different failure scenarios

Load Scenario	Failure Mode	Annual Probability of Overall Dam Failure Scenario	Conditional Probability of Fatality	Individual Risk
Normal operating load at full supply level	Piping through the embankment- in dam	9.7×10^{-5}	0.0075	7.27×10^{-7}
	Piping through the embankment-along or into the conduit	6.44×10^{-4}	0.0075	4.83×10^{-6}
	Piping through the foundation	2.5×10^{-12}	0.0075	1.88×10^{-14}
	Downstream slope instability	7.5×10^{-4}	0.01	7.50×10^{-6}
Natural extreme flood load	Piping through the embankment- in dam	2.724×10^{-7}	0.04	1.09×10^{-8}
	Piping through the embankment-along or into the conduit	1.807×10^{-6}	0.04	7.23×10^{-8}
	Piping through the foundation	7.02×10^{-15}	0.035	2.46×10^{-16}

9.8.2 Societal risk

The societal risks are reported as an “F-N” plot. There are number of scenarios with the same “N” value, so the annual probability of failure scenario (“f”) for those scenarios was aggregated, before computing the complementary cumulative distribution function (the “F” values). The Calculated “f,N” and “F,N” pairs for identified overall failure scenarios are given in Table 9.11.

Table 9.11: Cumulative distribution function and number of life loss

<table>
<tr><th>Failure Mode</th><th>Annual Probability of Overall Dam Failure Scenario (f)</th><th>Number of Life Loss (N)</th><th>Aggregated “f”</th><th>Cumulative Probability Function F (>=N)</th></tr>
<tr><td>Piping through the foundation (NOL)</td><td>2.5×10^{-12}</td><td>533</td><td rowspan="3">7.41×10^{-4}</td><td rowspan="3">1.49×10^{-3}</td></tr>
<tr><td>Piping through the embankment- in dam (NOL)</td><td>9.7×10^{-5}</td><td>533</td></tr>
<tr><td>Piping through the embankment-along or into the conduit (NOL)</td><td>6.44×10^{-4}</td><td>533</td></tr>
<tr><td>Downstream slope instability (NOL)</td><td>7.5×10^{-4}</td><td>711</td><td>7.5×10^{-4}</td><td>7.52×10^{-4}</td></tr>
<tr><td>Piping through the foundation (EFL)</td><td>7.02×10^{-15}</td><td>2487</td><td>7.02×10^{-15}</td><td>2.079×10^{-6}</td></tr>
<tr><td>Piping through the embankment- in dam (EFL)</td><td>2.724×10^{-7}</td><td>2842</td><td rowspan="2">2.079×10^{-6}</td><td rowspan="2">2.079×10^{-6}</td></tr>
<tr><td>Piping through the embankment-along or into the conduit (EFL)</td><td>1.807×10^{-6}</td><td>2842</td></tr>
</table>

Note: NOL – Normal operating load
EFL – Extreme flood load

9.9 Risk Evaluation

In the following sections the tolerability of the life safety risk has been tabulated. Determining that ALARP is satisfied is a matter for judgement by the dam owner, subject to any regulatory requirements that must be met. In case of societal risk, the F-N lines method was followed with included negligible level of risk. If the risks are under broadly acceptable level, then there is no need for any risk reduction measures.

9.9.1 Evaluating the individual risk of life

The tolerability of individual risk of life for residents of Anurathapura area is given in Table 9.12.

Table 9.12: Tolerability of individual risk

Load Scenario	Failure Mode	Individual Risk	Tolerability of Risk
Normal operating load at full supply level	Piping through the embankment- in dam	7.27×10^{-7}	Broadly acceptable level
	Piping through the embankment-along or into the conduit	4.83×10^{-6}	Acceptable if ALARP is satisfied
	Piping through the foundation	1.88×10^{-14}	Broadly acceptable level
	Downstream slope instability	7.50×10^{-6}	Acceptable if ALARP is satisfied
Natural extreme flood load	Piping through the embankment- in dam	1.09×10^{-8}	Broadly acceptable level
	Piping through the embankment-along or into the conduit	7.23×10^{-8}	Broadly acceptable level
	Piping through the foundation	2.46×10^{-16}	Broadly acceptable level

According to the evaluation results, the individual risks of life for both, piping through the embankment – in dam and piping through the foundation, under normal operating load are within the broadly acceptable level, while piping through the embankment- along or into conduit and downstream slope instability, under normal operating load are acceptable only if they satisfy the ALARP. Determining that ALARP is satisfied is a matter for judgement by the dam owner, subject to any regulatory requirements that must be met. In case of extreme flood load, all the included piping failure modes are under broadly acceptable level.

9.9.2 Evaluating the societal risk of life

The tolerability of societal risk of life, evaluated based on "F-N" plot method is given in Table 9.13.

Table 9.13: Tolerability of societal risk

Failure Mode	Number of Life Loss (N)	Aggregated "f"	Cumulative Probability Function F (>=N)	Tolerability of Risk
Piping through the foundation (NOL)	533			
Piping through the embankment- in dam (NOL)	533			
Piping through the embankment-along or into the conduit (NOL)	533	$7.41\text{x}10^{-4}$	$1.49\text{x}10^{-3}$	Risk are unacceptable
Downstream slope instability (NOL)	711	$7.5\text{x}10^{-4}$	$7.52\text{x}10^{-4}$	Risk are unacceptable
Piping through the foundation (EFL)	2487	$7.02\text{x}10^{-15}$	$2.079\text{x}10^{-6}$	Risk are tolerable if they satisfy the ALARP
Piping through the embankment- in dam (EFL)	2842			
Piping through the embankment-along or into the conduit (EFL)	2842	$2.079\text{x}10^{-6}$	$2.079\text{x}10^{-6}$	Risk are tolerable if they satisfy the ALARP

Note: NOL – Normal operating load
EFL – Extreme flood load

According to the evaluation results, the failure modes included under normal operating load state are unacceptable, while the failure modes included under natural extreme flood are acceptable if they satisfy ALARP. If they don't satisfy the ALARP then proper risk reduction measures should be implemented. Since, the risks under normal operating load state are greater than the limit in the "F-N" plot, there is an indicated need for risk reduction.

9.10 Summary of the Analysis Results

According to the analysis results, conditional probability of failure for internal erosion and piping failure modes under extreme flood load state are higher than the normal loading condition, because of the influence of water level on, initiation, pipe enlargement and formation of breach mechanism. On the other hand, the conditional probability of failure for internal erosion and piping through the embankment – along or in to conduit is comparatively higher than the internal erosion and piping through embankment – in dam. The maximum conditional probability of failure is estimated at 1.807×10^{-3} for internal erosion and piping through the embankment – along or in to conduit, under extreme flood load.

In the estimation of annual probability of failure, the failure modes included under normal operating load state were considered as "mutually exclusive", while the failure modes included under extreme flood load state were considered as "not mutually exclusive". Therefore, conditional probabilities for extreme flood load state were estimated using uni–model bound theorem. The total annual probability of failure under normal operating load state and extreme flood load state are estimated at 1.49×10^{-3} and 2.079×10^{-6} (U), 1.807×10^{-6} (L) respectively.

Here, the annual probability of failure under extreme flood load state is lesser than the annual probability of failure under normal operating load. Eventhough the conditional probabilities of failure are higher under extreme flood load state, the annual probabilities of failure are higher under normal operating load, because of the influence of annual probability of load states.

The estimated number of life loss is higher for internal erosion and piping through the embankment with the value of 2842. According to the estimated individual risk of life, downstream slope instability under normal operating load state is the most critical failure mode with the value of 7.50×10^{-6}. In terms of societal risk of life, the failure modes included under normal operating load state are unacceptable, while the failure modes included under natural extreme flood are acceptable if they satisfy ALARP.

According to the above analysis results, the failure mode with the highest annual probability of failure doesn't have the highest risk of life, since the estimation of risk is influenced by both probability of failure and consequences. The above results can be used to rank the risk. In the above analysis the consequences were estimated mostly based on assumptions. Therefore, a detailed risk assessment should be done for different load scenarios with a proper dam break analysis, in order to get more accurate results.

TETON DAM FAILURE CASE STUDY

CHAPTER 10

10.1 Introduction

The Teton Dam was situated on the Teton River, three miles northeast of Newdale, Idaho. It was designed to provide recreation, flood control, power generation, and irrigation for over 40,000 hectares (100,000 acres) of farmland. The Office of Design and Construction, U.S. Bureau of Reclamation (USBR), at the Denver Federal Center, designed the dam and the construction contract was awarded to the team of Morrison-Knudsen-Kiewit in December of 1971.

The Teton dam and reservoir were the principal features of the Teton basin project, a multipurpose project, which when completed was to serve the objectives of flood control, power generation, recreation, and supplemental irrigation water supply for large amount of farm land. It was an earth fill dam that had 405 ft (122 m) high creating 17 miles (27.4 km) long reservoir with a 436 Mega yard^3 (333 Mm^3) capacity. The construction work commenced in June 1972 and the dam was completed and first filling started in November 1975 (Sasiharan, 2003).

The earliest use of dams was probably irrigation. Teton Dam is a large earthen dam in eastern Idaho that failed during initial filling of the reservoir on June 5, 1976 killed fourteen people and caused hundreds of millions of dollars in property damage downstream.

Solava et al (2003) summarized that on June 3, 1976 several small seepages were noticed in the north abutment wall. Pictures were taken and these leaks were reported to the Bureau of Reclamation. This led to more frequent inspections of the dam. It was now to be inspected daily, and readings were to be taken twice weekly instead of once a week. On June 4, 1976 wetness was noticed in the right abutment and small springs were beginning to appear (Independent Panel, 1976).

On June 5, 1976 the first major leak was noticed between 7:30 and 8:00 a.m. The leak was flowing at about 500 to 800 liters per second from rock in the right abutment. By 9:00 a.m. the flow had increased to 1,100 to 1,400 liters per second and seepage had been observed about 40 meters below the crest of the dam (Arthur, 1977). At 11:00

a.m. a whirlpool was observed in the reservoir directly upstream from the dam and four bulldozers were sent to try to push riprap into the sinkhole near the dam crest (Independent Panel, 1976).

Two of the bulldozers were swallowed up by the rapidly expanding hole, and the operators were pulled to safety by ropes tied around their waists (Teton Dam Flood @ 2002). Between 11:15 and 11:30 a.m. a 6 by 6 meter chunk of dam fell into the whirlpool and within minutes the entire dam collapsed (Independent Panel, 1976).

At 10:30 a.m. dispatchers at the Fremont and Madison County Sheriffs' offices were notified that the dam was failing. An estimated 300 million cubic meters of water (80 billion gallons) headed down the Upper Snake River Valley. The towns in its path included Wilford, Sugar City, Rexburg, and Roberts.

10.2 Inspection of Dam

The following data of Teton dam were obtained from the thesis on "the failure of Teton dam – a new theory based on "state based soil mechanics"" by Sasiharan (Sasiharan, 2003).

Based on the site conditions, the final design cross-section of the Teton dam at the river valley and the right abutment were as shown in Figure 10.1 and Figure 10.2 respectively. The dam was conservatively designed to have a wide impervious core with a head to width ratio of about 1.5 in the upstream and 1 in the downstream .The impervious core (Zone-1) of the dam consisted of clayey silts of Aeolian origin with low plasticity (PI ~ 4) and USCS classification of CL- ML and it was supported by upstream and downstream shells (Zone-2) consisting mainly of sand, gravel and cobbles. As per the design and specifications Zone-1 material was placed at an average water content of 1.0% dry of optimum and compacted to a maximum dry density of 98-102 % of the Standard Proctor test. Similarly the support zone (Zone-2) (chimney filter/drain) was compacted to a high relative density of the order of 65-70 % (IRG, 1980).

In the main section of the dam, the impervious core was extended through the foundation alluvium by means of a 30.5 m deep cut-off trench backfilled with silt. On the abutments above El.5100, a similar section was adopted but key trenches with a base width of 9.1 m and sides slopes 0.5 on 1 were excavated through the upper 21.3

m of permeable rock and backfilled with clayey silt material used in the core of the dam.

Downstream of the core was a drainage zone of selected sand and gravels (Zone-2). However, no transition zone was provided between the core and the sand and gravel, nor between the impervious core and the riverbed alluvium or between key trench fill and the bed rock walls on the downstream side of the key trench. The core material in the key trench was placed directly against the rock using special compaction of a 0.6 m wide zone of core material placed at water content above optimum. Compaction of this zone was by hand-operated compactors or rubber-tired equipment.

In addition, the design required the joints encountered in the bottom of the key trench be treated by cleaning and low-pressure grouting. A grout curtain was also installed along the full length of the dam. Lines of barrier holes intended to prevent excessive flow of grout from the main grout curtain were installed on 6.1 m centres 3 m, upstream and downstream of the main grout curtain. To prevent seepage, the key trenches and grout curtain were continued well beyond the ends of the embankment, the curtain extending 30.5 m into the right abutment and 152.4 m into the left abutment (H.B. Seed, 1987).

According to Solava et al (2003), there were not enough instruments in the dam to provide adequate information about changing conditions of the embankment and abutments The soil material that formed the impervious core of the dam (Zone 1) was derived from Aeolian deposits and consisted of uniform clayey silt, 88 percent passing through #200 sieve and about 13% of clay fraction (<2 micron) and USCS classification of CL- ML (Sasiharan, 2003).

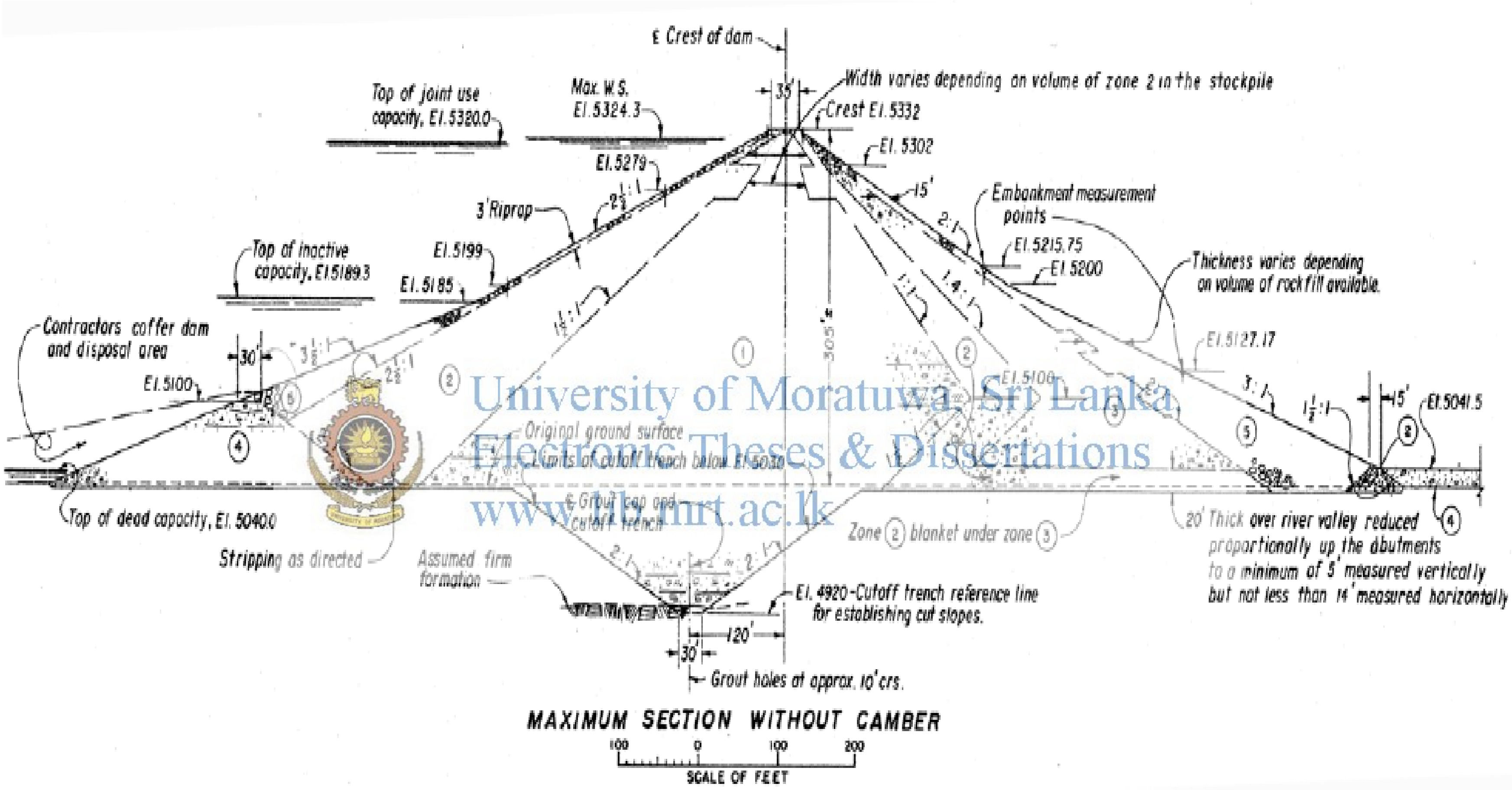


Figure 10.1: Design cross section of the dam at river valley section (IP, 1976) (Sasiharan, 2003)

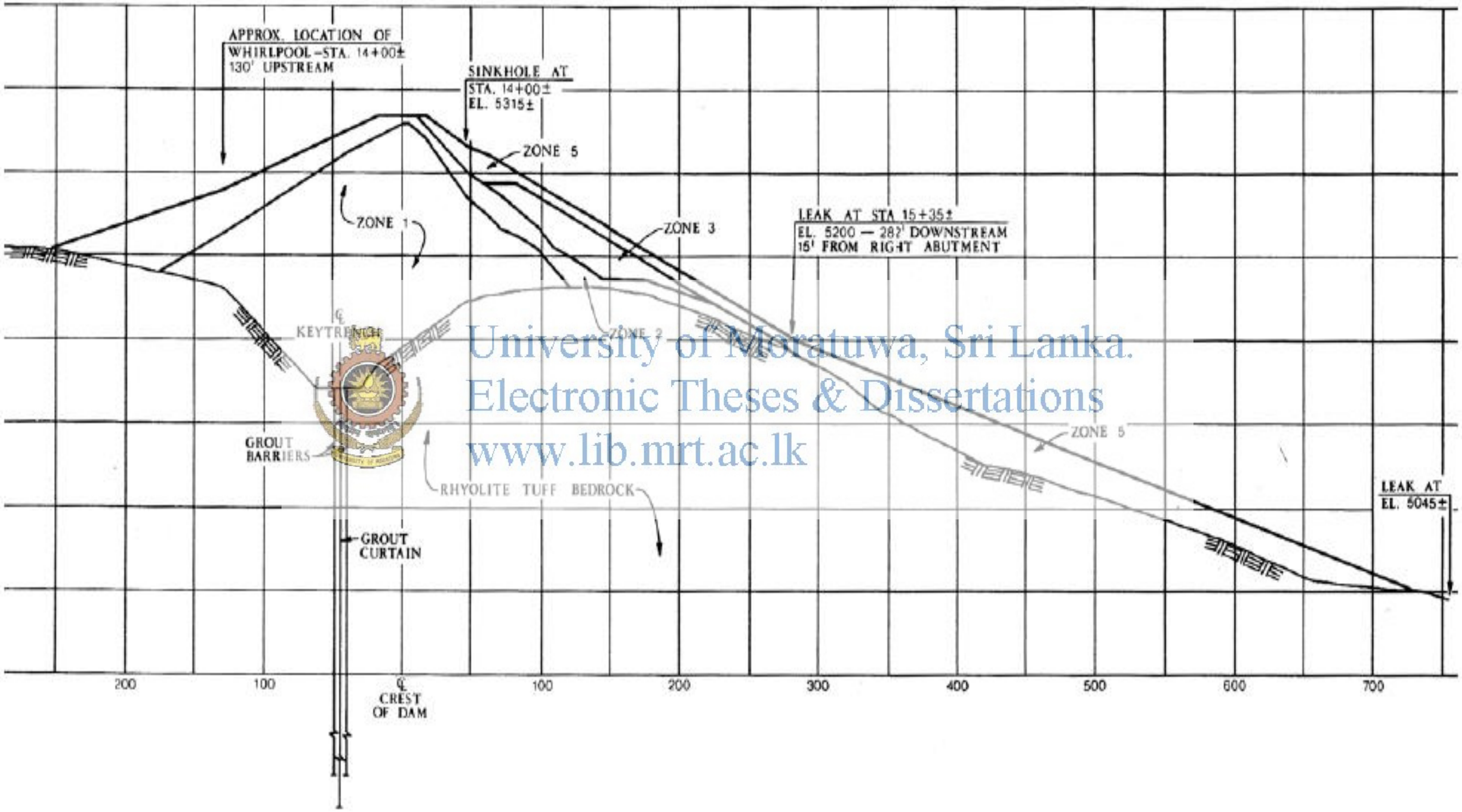


Figure 10.2: Cross section of the dam at the right abutment (IP, 1976) (Sasiharan, 2003)

10.3 Hazard Identification

At the time of failure the reservoir elevation was 1,616 meters and was filling at a rate of 1 meter per day. At full capacity the water surface elevation would have been 1,621.5 meters (Bureau of Reclamation, 1983) (Solava et al, 2003).

10.4 Failure Mode Identification

Solava et al (2003) summarized, piping as the most probable cause of the failure, and then focused its efforts on determining how the piping started. Two mechanisms were possible. The first was the flow of water under highly erodible and unprotected fill, through joints in unsealed rock beneath the grout cap, and development of an erosion tunnel. The second was "cracking caused by differential strains or hydraulic fracturing of the core material." The Panel was unable to determine whether one or the other mechanism occurred, or a combination.

Here, internal erosion and piping through embankment-in dam is included in the analysis in order to check the possibility of such failure. In the following section, the conditional probability of failure for the above failure mode is estimated. Also the possible mechanisms for initiation were identified for piping failure.

10.5 Estimation of Probability of Failure for Internal Erosion and Piping Through the Embankment – in Dam

Initiation

Cracking or wetting induced collapse susceptibility of core materials

The dam was conservatively designed to have a wide impervious core with a head to width ratio of about 1.5 in the upstream and 1 in the downstream. As per the design and specifications Zone-1 material was placed at an average water content of 1.0% dry of optimum and compacted to a maximum dry density of 98-102 % of the Standard Proctor test. The impervious core (Zone-1) of the dam consisted of clayey silts of Aeolian origin with low plasticity (PI ~ 4) and USCS classification of CL-ML. According to Table 5.3, the likelihood of wetting induced collapse susceptibility of core material is low and likelihood of cracking of core material is high. Here for cracking, relative compaction is not a major factor.

Hydraulic fracture

The overall abutment profile is relatively a flat slope with care full slope modification. Solava et al (2003) summarized that there was no evidence of differential foundation settlement contributing to the failure. The dam was conservatively designed to have a wide impervious core with H/W < 1. The dam had a central core and the core materials are stiffer than shell materials. Apart from these the dam failed during the first filling. Hence, according to Table 5.4, likelihood hydraulic fracturing of the core material is low.

High permeability zone

As per the design and specifications Zone-1 material was placed at an average water content of 1.0% dry of optimum and compacted to a maximum dry density of 98-102 % of the Standard Proctor test. There was no instrumentation in the embankment for very long. The dam was constructed with good engineering supervision According to the soil classification done by Sasiharan (2003), the core material has uniform clayey silt. Hence, according to Table 5.5, likelihood of high permeable zone present within the embankment is low.

Suffusion

The dam core consisted of uniform clayey silt, 88 percent passing through #200 sieve and about 13% of clay fraction. So, the particle size distribution doesn't come under well graded or poorly graded range. The core was compacted to a maximum dry density of 98-102 % of the Standard Proctor test. The permeability of the core material is low. Hence, according to Table 5.6, the likelihood of initiation by suffusion is low.

Considering the possibility of all mechanism, likelihood of initiation by cracking of core material is higher than other mechanisms. So the piping is likely to initiate by cracking of core materials.

Considering all the above conditions, according to Table 5.**2**, the probability of initiation is taken as 0.6.

Continuation

Continuation of internal erosion is mainly depending on the filter criteria. The fine content of core material is 88%, so it comes under base soil group 1. For the core material, D85 is nearly equal to 0.075 mm. The zone 2 filter is consisting mainly of sand, gravel and cobbles. According to available data D15 of filter is less than 9 x D85 of core material. Hence, according to Table 5.**7**, filter is finer than the no erosion boundary. So the likelihood of continuation is low.

Considering all the above conditions, according to Table 5.2, the probability of continuing erosion is taken as 0.001.

Ability to support a roof

The fine content of core material is 88% and core materials are well compacted. The dam is failed during first filling, so it's partially saturated. But the fine content of zone 2 material is less than 15%. Hence, according to Table 5.**9**, the ability to support a roof is high.

Considering all the above conditions, regarding to Table 5.**2** the probability of ability to support a roof is taken as 0.6.

Limitation of flow

The filter is capable of restricting erosion. The zone upstream of the core (zone2) is consisting mainly of cohesionless soils and permeability can be assumed as medium to high range. Hence, according to Table 5.10, the likelihood of pipe enlargement is very low.

Considering all the above conditions, according to Table 5.**2**, the probability of inability to limit the flow is taken as 0.0005.

Erodibility

The core consisted of clayey silts of Aeolian origin with low plasticity (PI ~ 4). The core was compacted to a maximum dry density of 98-102 % of the Standard Proctor test. As per the design and specifications Zone-1 material was placed at an average water content of 1.0% dry of optimum. Hydraulic gradient is taken as average. Hence, according to Table 5.11, the chances of filling materials being eroded are in average range.

Considering all the above conditions, according to Table 5.**2**, the probability of soil erodibility is taken as 0.04.

Early intervention

In Teton dam leakage is generally accessible on downstream slope. The access to the location is available and the dam is being monitor by officials. There were not enough instruments in the dam to provide adequate information about changing conditions of the embankment and abutments. In terms of stability, there is no crack evident in the embankment. Apart from these, the embankment is very long. So this may have caused some difficulties to monitor. Considering all the above factors, the likelihood of early intervention is high.

Considering all the above conditions, according to Table 5.**2**, the probability of early intervention is taken as 0.4.

Breach Mechanism

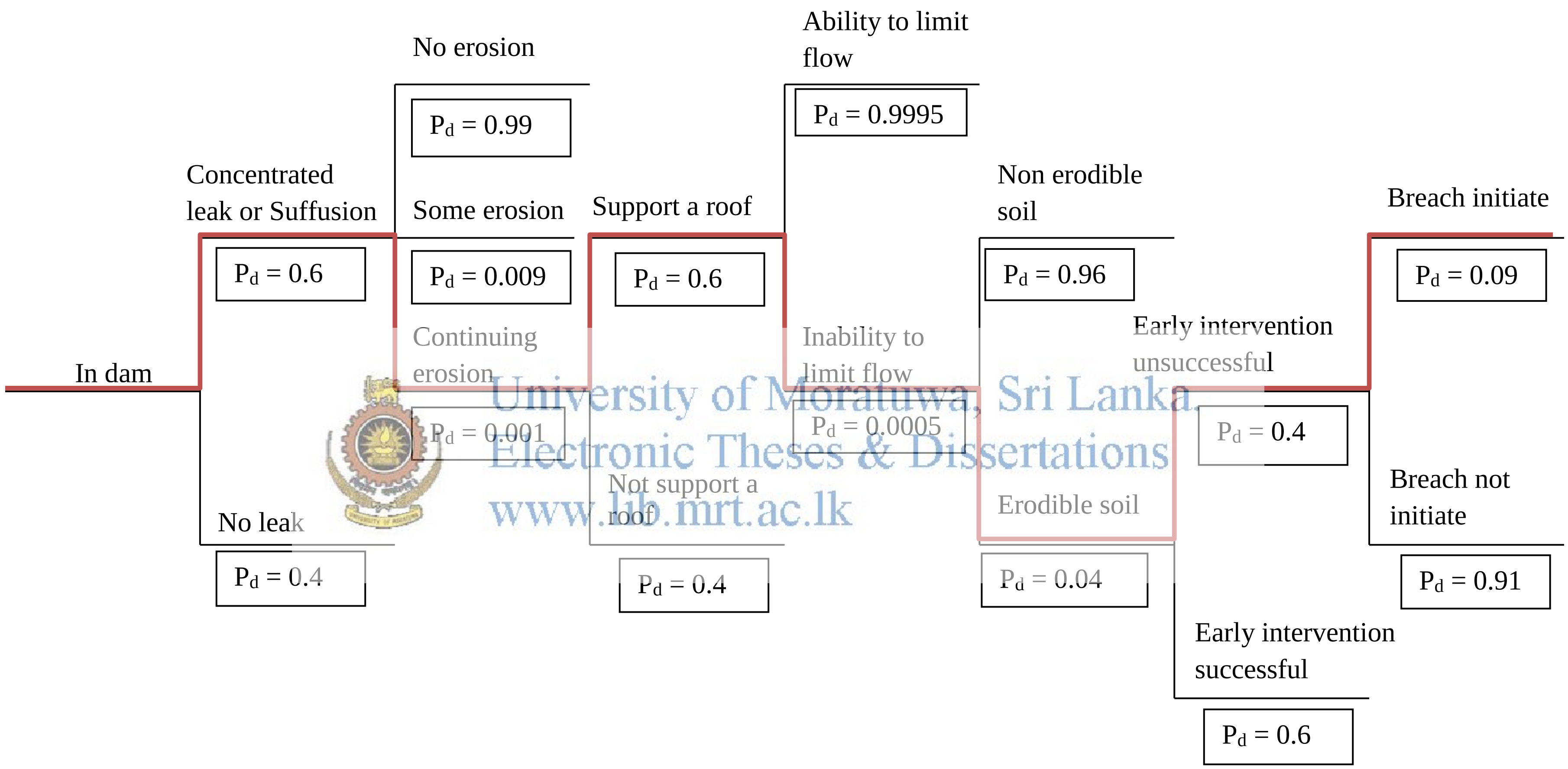
Teton is a zone type dam with downstream zone consisting mainly of sand, gravel and cobbles. The storage volume is large. Hence, according to Table 5.12, the likelihood of breaching of the dam by gross enlargement is nearly high.

The freeboard during failure is greater than 4m. The crest is around 10 m wide. Hence, according to Table 5.13, the likelihood of breaching of the dam by Sinkhole or crest settlement is low. Even though the likelihood of breaching by sinkhole or crest settlement is low, the likelihood of breaching by gross enlargement is nearly high. So the dam is likely to breach by gross enlargement.

Considering all the above conditions, according to Table 5.**2**, the probability of formation of breach mechanism is taken as 0.09.

Conditional probability of failure by internal erosion and piping through embankment-in dam is calculated as 2.6×10^{-10}.

The conditional probability of failure for each branches of the event tree is shown in Figure 10.3.



P_d -probability of failure in dam

Conditional Probability of failure for internal erosion and piping through embankment – in dam = 2.60×10^{-10}

Figure 10.3: Event tree for internal erosion and piping through embankment – in dam

According to the analysis results, the probability of internal erosion and piping trough the embankment – in dam is low. Hence, it cannot be the cause of failure. Apart from these, the analysis shows that the initiation of internal erosion and piping due to cracking of core is high. So the dam may have failed due to,

- Internal erosion and piping stating from cracking of core of the embankment and continuing into the foundation

  or

- Internal erosion and piping stating from cracking of core of the embankment and continuing through key trench.

CONCLUSION

CHAPTER 11

In this report, the quantitative risk assessment framework for safety evaluation of earth dams is developed based on the condition of earth dams in Sri Lanka. Here the methods that are applicable to earth dams using available data and proper investigation has been discussed. It is a task of the risk assessment team to decide on the level of risk assessment.

Here, the earthquake loading is considered as less obvious, based on the Sri Lanka's earthquake history records. Since the embankment instability and loss of free board is mainly occurs under earthquake loading, it has been omitted from discussion.

When using the "verbal descriptors" to estimate the probabilities, engineering judgement should be taken with care. Otherwise it would result in over estimation or under estimation of the probabilities. Here, the internal erosion and piping from embankment to foundation is not discussed, since it less likely to occur in earth dams compared to other failure modes.

When estimating the probability of failure for slope instability, small slope failure results, which are caused by undulations in the dam slope were ignored. This is because they may cause comparatively a minimal effect on the earth dam. So we have considered the critical slope failure condition with minimum factor of safety.

For initial level studies, the conservative assumption that the reservoir is always full under normal operating conditions analyses may be reasonable in some cases, but this position should not be taken without consideration of how representative it is of the annual operating cycle for the reservoir.

Here, in terms of flood loading, only the natural extreme flood was included for the case study of Nachchaduwa dam. In detailed studies, other scenarios such as flooding due to, upstream dam failure and wind effect also should be considered. In this case study, only one loading state is selected for each loading domains and it should be modified with number of loading states for detailed studies.

Most of the Sri Lankan dams are interconnected and failure of an upstream dam may cause other dams failure. However, the failure of upstream dams should not be considered as loading conditions in a risk analysis. The risk of multiple dam failures/incident should be addressed by assigning the cause of failure to the most upstream dam failure and including the resulting dam failures as consequences for that dam.

In the case study of Nachchaduwa dam, priority was given to the life safety consequences. The monetary losses are also should be estimated for a detailed risk assessment, because they affect the economical and financial condition of the country. Consequences other than life safety and monetary losses are not included in the guidelines, considering the minimal influence on countries development.

In estimation of life safety consequences, population at risk should be estimated with care, because it is likely to vary depending upon the time of year, day of week and time of day during which the failure occurs.

For Nachchaduwa dam the annual probability of failure for normal operating load state is higher than extreme flood load state. This is because the annual probability of the normal operating load is much higher than the annual probability of extreme flood load.

From the case study of Nachchaduwa dam, individual risks of life are under broadly acceptable level for most of the failure scenarios, except, piping through the embankment – along or into conduit and downstream slope instability. Also, societal risks of life are unacceptable under all four failure scenarios considered under normal operating load, while the societal risks of life under extreme flood load need to satisfy ALARP.

In the failure case study of Teton dam, probability of failure for internal erosion and piping through the embankment-in dam was estimated, in order to verify the possibility of that failure mode for the dam failure. According to the analysis results the conditional probability of the above failure mode is low and hence it cannot be the cause of failure. So, other failure modes need to be checked in order to find out the real cause of failure. Apart from these, the analysis shows that the chances of piping initiated due to cracking of core materials are high. So, there is a chance for internal erosion and piping, from embankment into foundation and from embankment into key trenches, to be the cause of failure.

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