NEW ZEALAND
Society on Large Dams

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New Zealand Dam Safety Guidelines 2024

MODULE 3
INVESTIGATION DESIGN AND ANALYSIS

Abstract

Dam safety objectives and principles that are applicable to the investigation, design, construction, commissioning, operation, assessment, rehabilitation, and decommissioning of dams in New Zealand are included in the Parent Document. The Parent Document also includes a glossary of terms used in these Guidelines.

This module provides a framework for the investigation and design of new dams, the analysis of existing dams, and the design of rehabilitation works for existing dams. Some parts of this module may also be applicable to stopbanks (levees). It includes:

- An outline of various dam types and issues that should receive close attention during their design.
- Recommended personnel and quality assurance procedures for the design of new dams and the design of rehabilitation works for existing dams.
- A discussion on the assessment of hazards and threats to the safety of a dam, and recommended performance criteria for flood, earthquake and other hazards.
- An outline of investigation activities that should be addressed during the initial stages of dam design or the initial stages of a dam rehabilitation project.
- Recommended criteria for the design and evaluation of dams and appurtenant structures, and issues that impact on dam safety that should be carefully considered during the design process.
- Recommendations relating to the design and installation of instrumentation systems for the monitoring of dam performance.
- Recommendations relating to design involvement during initial construction, commissioning and rehabilitation, and design documentation.
- Refer also to Module 4 for designer involvement during construction, commissioning and rehabilitation works

A summary of the recommended performance criteria for Low, Medium and High PIC (Potential Impact Classification) dams is presented in Table 1.

Table 1: Recommended design Criteria for Low, Medium and High PIC Dams

<table>
<tr><th rowspan="2">Hazard</th><th rowspan="2">Design Criteria</th><th colspan="3">PIC</th></tr>
<tr><th>Low</th><th>Medium</th><th>High</th></tr>
<tr><td rowspan="4">Wind and Waves</td><td colspan="4">Freeboard for embankment dams may be set as the largest of the following three scenarios, subject to confirmation based on the wider factors described later in Module 3.</td></tr>
<tr><td>Freeboard at Maximum Normal Reservoir Level</td><td colspan="3">Wind set up and wave run up for the highest 10% of waves caused by a sustained wind speed, which is dependent on the fetch, with an AEP[1] of greater than 1 in 100.</td></tr>
<tr><td>Freeboard at Intermediate Flood Levels</td><td colspan="3">Freeboard should be determined so that it has a remote probability of being exceeded by any combination of wind generated waves, wind set up and reservoir level occurring simultaneously.</td></tr>
<tr><td>Freeboard at Maximum Reservoir Level during the Inflow Design Flood (IDF)</td><td colspan="3">Usually the greater of (a) 0.9 m or (b) the sum of the wind set up and wave run up for the highest 10% of waves caused by a sustained wind speed, which is dependent on the fetch, with an AEP[1] of 1 in 10 (with exceptions)[2].</td></tr>
<tr><td>Flood</td><td>Inflow Design Flood (IDF)</td><td>1 in 100 AEP to 1 in 1,000 AEP</td><td>1 in 1,000 AEP to 1 in 10,000 AEP[3]</td><td>1 in 10,000 AEP to PMF[4]</td></tr>
</table>

Earthquake	Operating Basis Earthquake (OBE)	1 in 150 AEP		
	Safety Evaluation Earthquake (SEE)[6]	50th percentile level for the MCE ground motion[5] if developed by a deterministic approach, and if developed by a probabilistic approach then at least a 1 in 500 AEP ground motion but need not exceed the 1 in 1,000 AEP ground motion.	50th percentile level for the MCE ground motion for incremental Potential Loss of Life of 0 and 84th percentile level for incremental Potential Loss of Life of 1 if developed by a deterministic approach and need not exceed the 1 in 2,500 AEP ground motion developed by a probabilistic approach.	84th percentile level for the MCE ground motion if developed by a deterministic approach and need not exceed the 1 in 5,000 AEP ground motion for incremental Potential Loss of Life of 0 or 1 and 1 in 10,000 AEP ground motion for incremental Potential Loss of Life of 2 or more developed by a probabilistic approach.

Notes:
1. AEP is Annual Exceedance Probability
2. Refer to specific guidance on the 0.9 m minimum freeboard criterion in the section on freeboard for embankment dams
3. Recommended IDF is dependent on population at risk (PAR) and Potential Loss of Life.
4. Recommended IDF is dependent on Potential Loss of Life.
5. MCE is Maximum Credible Earthquake
6. Recommendations on SEE design loads are further discussed in section 4.3.4.

Advances in knowledge and techniques continue worldwide and these Guidelines should be interpreted and applied accordingly. New and advanced knowledge and techniques should be validated before they are adopted for use in New Zealand.

This module includes limited discussion on the role of regulators in dam safety. Reference should be made to Module 1 (Legal Requirements) for a more complete description of regulators' roles and responsibilities.

Notice to reader

Although this module is configured to be as self-contained as practicable from a technical standpoint, readers should familiarise themselves with the principles, objectives, and limitations outlined in the Parent Document and Module 1: Legal Requirements before considering the information in this or any other module.

Document history

Release	Date	Released With
Original	May 2015	Parent and all modules
2023	December 2023	Updates to Parent and Modules 1,2 and 5
2024	December 2024	Updates to Parent and all Modules

Contents

1. Introduction

1.1 Principles and objectives

There are a number of natural hazards (e.g., floods and earthquakes) that can affect the safety of a dam, the occurrence of which cannot be controlled by the dam Owner. In addition, threats relating to human activities (e.g., oversights, errors) can affect the safety of a dam, where occurrence can be controlled by the dam Owner. The investigation, design, and rehabilitation of dams should identify and consider all identified hazards and threats to dam safety and be cognisant of the associated consequences of dam failure.

Principle 2 in the Parent Document states that:

All natural hazards, loading conditions, potential failure modes and any other threats to the safe design, construction, commissioning, operation and rehabilitation of a dam should be identified.

Natural hazards that can affect dam safety and therefore need to be considered in the investigation and design of dams include floods, earthquakes, landslides, extreme rainfall and/or wind events, lightning strikes and volcanic activity. Other threats to dam safety that can usually be controlled by the Owner may arise from errors and omissions in design, inadequate specification or supervision of construction, inappropriate operation, inadequate maintenance and inadequate surveillance. Other threats that cannot be controlled but can be addressed by the Owner include vandalism and security breaches.

Principle 3 in the Parent Document states that:

Dams and appurtenant structures should be designed, constructed, commissioned, operated, and rehabilitated in a manner which ensures they meet appropriate performance criteria.

Design criteria should always be commensurate with the consequences of dam failure. In addition, care should always be taken to ensure that the design intent is not adversely affected by subsequent construction, commissioning and operational practices.

The competence and experience of participants in the design, construction and operation of a dam, along with their relationship and continuity through investigation, design, construction and commissioning, are key factors in successful and safe dam ownership throughout the dam's lifespan.

Appropriate funding resources and work programmes should be applied throughout all investigation, design, construction and commissioning activities to ensure all issues critical to dam safety are appropriately considered and addressed. Effective quality assurance is essential to verify the design, to verify the design changes made in response to knowledge gained during design and construction, and to verify that the design intent is achieved during construction.

All designs must include a thorough evaluation of dam safety risks, and the measures to control them, with a focus on providing appropriate resilient backup features and systems to support primary features and systems.

Assessment of potential failure modes is recommended as an effective means to identify dam safety risks. The design of new dams and rehabilitation of existing dams should aim to design out as many potential failure modes as possible.

All designs should consider Safety in Design (Health and Safety at Work Act 2015). Safety in Design can be defined as the integration of hazard identification and risk assessment early in the design process to eliminate or minimise the risks of injury throughout the life of the product/asset being designed so far as is reasonably practical. This includes risks of injury during investigations and construction phases of the lifecycle.

The objectives of this module are to provide guidance for Owners, Designers, and Contractors responsible for the investigation, design, performance monitoring, evaluation, and rehabilitation of dams.

Note: Specific investigation, design, and analysis methods are not addressed in this module; however, a list of reference documents is included to assist the Designer in the selection and completion of specific investigation and design tasks (refer Further Information at the end of this module).

1.2 Design considerations

1.2.1 Design requirements

The Designer's objective is to design a new dam and its appurtenant structures, complete a safety evaluation of an existing dam, or design a dam rehabilitation, in a manner that suitably reflects the characteristics of the site, the loading conditions applicable to the site, the consequences of dam failure, and the function and purpose of the dam.

To obtain a building consent for construction, the Designer must demonstrate that the design has considered all foreseeable hazards at a level appropriate to the consequences of dam failure, and that the hazards are accounted for and their potential effects satisfactorily mitigated. The Designer must also demonstrate that the completed structure will meet durability requirements and achieve, or exceed, the specified intended life for the structure. Note: most dams, once created, have a lifespan well beyond the minimum requirements of the Building Act 2004 and beyond the initial intended life.

Module 1 outlines New Zealand's current legislative requirements included in the Building Act 2004 and Building (Dam Safety) Regulations 2022 for the development and operation of dams. The Health and Safety at Work Act 2015 (HSWA) places obligations on the designer to consider Safety in Design.

The Designer needs to recognise that not all elements of dam structures requiring design are adequately covered by design manuals or standards. The Designer is required to assess all realistic risks by considering all potential failure modes and demonstrate how the design incorporates defensive measures to eliminate or minimise associated risks. For design of rehabilitation works for existing dams, consideration can be given to reducing risk to 'As Low As Reasonably Practicable' (ALARP) (i.e., to the point that further risk reduction is impractical, or its cost is 'grossly disproportionate' to the improvement gained). Refer to Module 7 for more guidance on risk-informed decision making and the use of the ALARP approach.

Attention needs to be directed not only towards extreme event loads of very low probability, that place high structural demands on the dam, but also unforeseen combinations of usual events that could affect dam safety. Consideration needs to be given to how the safety of the dam can be managed in the period following an extreme loading event which may have damaged the dam.

These Guidelines promote the use of robust and resilient features and systems to reduce the risk of dam failure from unexpected and unpredictable events and occurrences.

Resilience in this context is the capacity of the structure or system to withstand changing conditions caused by sudden shocks, gradual stresses and cumulative change, or deterioration. The effects of climate change should be considered when designing for resilience. A focus on resilience does not implying 'gold plating' or gross conservatism, but robust and effective design which assures long-term dam safety and minimises the whole-of-life costs of a dam.

Recognising the uncertainties associated with design parameters, such as the adoption of idealised design loads, the incorporation of resilient features should provide greater confidence in the facility meeting its performance criteria over the whole of its life.

Considerations of resilience may influence the type of dam selected, the dimensions of the dam, and the defensive design features incorporated into the design of the dam and its appurtenant structures and should be considered in the design or rehabilitation of any dam irrespective of its Potential Impact Classification (PIC).

Dams typically have a very long operational life, often well beyond their initial design life. Interventions and upgrades following initial commissioning can be expensive and result in increased dam safety risks during their implementation. Robust design, resilient features, and a whole-of-life approach contribute significantly to ongoing dam safety assurance. The design lives of all components should be considered and elements with shorter design lives than the overall water retaining structure should ideally be readily replaceable without compromising the safety of the dam.

The relationship between the above considerations is shown in Figure 1.1.

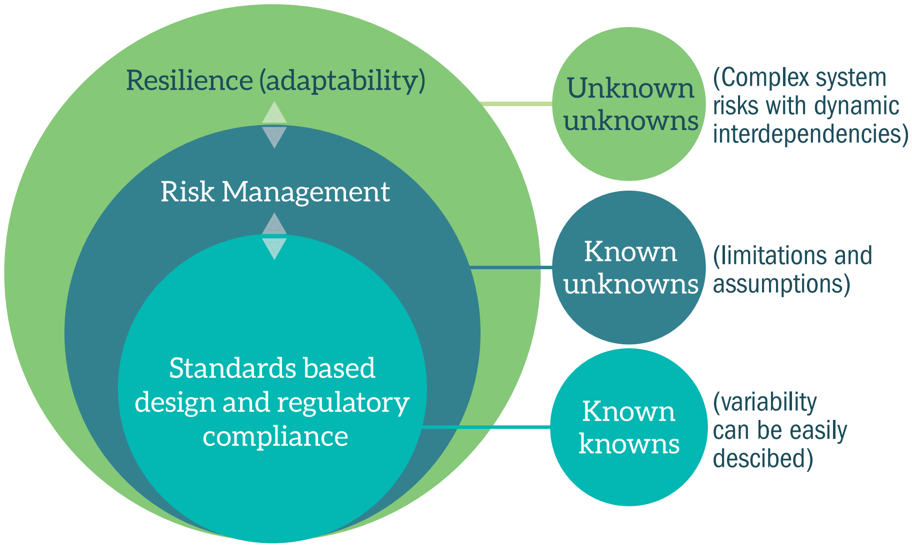

Figure 1.1: Risk and resilience in design

Note: The boundaries in the diagram are not absolute. For example, standards-based design usually incorporates the consideration of risk and resilience.

The context surrounding the physical dam structure needs to be considered in dam design. Dams are generally a component of a larger system (e.g., hydropower, water supply, flood detention, waste management or irrigation) and the dam's role in the overall system is important. The loss of dam operation will affect the system, facility or infrastructure that it supports (e.g., no irrigation supply, reduced power generation, instability in the power transmission system, reduction in water supply, prevention of mining operations) and the design needs to consider the integral role and performance requirements for the dam in the overall system.

During the development of a design the Designer should consider health and safety issues that could arise during construction and operation (Safety in Design). The design should be able to be constructed without introducing health and safety risks that a reasonably skilled Contractor cannot manage. Similarly, design features should not introduce health and safety risks that are unable to be effectively managed by operational personnel during the operational phase of the project.

All those involved in dam design should follow relevant codes of ethical conduct (e.g., Engineering New Zealand, 2016), including a focus on sustainable management of the environment, and a focus on health and safety of all people throughout the whole lifecycle of a dam.

These Guidelines provide guidance for the design of new dams, the evaluation of existing dams, and the rehabilitation of existing dams. The design considerations and design philosophy for the rehabilitation of existing dams are, in general, the same as those for the design of new dams. However, there will be differences in the technical approaches for the assessment of existing dams compared to design of new dams.

The design of new dams should involve the use of modern accepted practices that reflect the latest knowledge and technology. New dams will typically incorporate appropriately conservative and prescriptive design criteria, and/or accepted solutions, with a focus on resiliency and future proofing. In contrast, the assessment of existing dams must acknowledge that the past design of older dams may not align with modern accepted practice. For older dams that do not meet or exceed modern design criteria, or for situations where modern design criteria cannot be practically achieved, the assessment of existing dams may involve evaluation against performance criteria and a risk-informed decision-making approach. Assessment of an existing dam against suitable performance criteria will enable appropriate risk management and prioritisation. Where upgrades or rehabilitation are required, designers should specify works to meet defined acceptable performance criteria, with the overall intent of adapting an existing dam to meet current performance needs given present loading conditions.

This module provides guidance on design criteria and performance criteria across many topics for the investigation, design, and analysis of dams. These criteria are presented in a manner that reflects their most likely application. However, the reader must consider the specific intent and application of any criteria and their relevance to the particular design or assessment task.

The Designer and/or Owner may propose structural measures to reduce the risk of dam failure or non-structural measures to reduce the likelihood of a risk, or the consequences of dam failure. Non-structural measures are more likely to involve other parties and stakeholders, and consultation with affected parties may be necessary during the development of proposed non-structural risk reduction measures. Consultation with affected parties may also be necessary before implementation of proposed structural rehabilitation works. The management of rehabilitation works is addressed in Module 7.

1.2.2 Consequences of failure, PIC and design loads

In New Zealand, the PIC is used to partition dams into broad hazard categories based on consequence assessment.

Many countries use this approach to determine the level of design, construction and operational rigour that should be employed throughout the life of the structure. In similar jurisdictions internationally, it is common that design and assessment criteria require higher levels of dam safety when the consequences of dam failure are greater.

Module 2 provides guidelines for assessing the consequences of dam failure and determining the PIC for a dam. Both 'rainy day' and 'sunny day' failure scenarios should be considered, the consequences of each potential failure scenario should be assessed, and the PIC determination should be based on the scenario that has the most severe consequence.

The resulting PIC for the dam drives the level of the design, construction and operational rigour that should be employed throughout the life of the structure. The design loads for the dam are primarily linked to the PIC for the dam, although the Designer may be able to demonstrate that lesser consequences could relate to a particular load case than the consequences that drove the PIC assessment. Such an example may be a flood detention dam which clearly has a function in an extreme flood event but is very rarely filled and therefore the exposure time to earthquake risk is very small. The premise in this module is that the design loads reflect the PIC of the dam, unless the Designer can demonstrate that a lower design load is appropriate.

Design loading conditions for various dam types are presented and discussed in various guidelines, e.g., ANCOLD guidelines, Canadian Dam Association (CDA, 2013), and various USACE and USBR engineering manuals. It is commonly accepted internationally that two extreme loading conditions do not need to be considered to occur at the same time. However, it is important to establish that the dam would not fail due to the effects of a combination of realistic loading conditions. Therefore, the Designer should consider the effects of one type of hazard occurring closely followed by perhaps a moderate event of the other type of hazard. For example, a dam damaged by a major earthquake may experience a significant aftershock or moderate flood before repairs can be completed. The dam should be able to withstand the second hazard in a damaged condition without failure.

Module 2 notes that separate PICs can be applied to a dam, subsidiary dam, and appurtenant structure if the consequences of their failure are different. A gate and/or valve system (gate and/or valve together with its power supply and control/protection/communication systems) that fulfils a dam safety function, and is installed in a dam, subsidiary dam, or appurtenant structure, is not assigned a PIC; however, the purpose of the gate and/or valve system must be understood and the design solution must ensure that the gate and/or valve system is able to perform its dam safety function. The dam, subsidiary dam, or appurtenant structure must not fail due to a functional failure of the gate and/or valve system.

1.2.3 Potential failure modes

A potential failure mode is a mechanism or set of circumstances that could result in the uncontrolled release of all or part of the contents of a reservoir. Avoidance of a potential dam failure mode, or mitigation to prevent or reduce the likelihood of a potential dam failure mode eventuating, is a cornerstone of effective dam design.

While this has always been an inherent expectation of dam design, formal consideration of potential failure modes is promoted in these Guidelines as an informed approach to reduce the risk of dam failure; particularly those aspects not generally covered by standards-based design. Identified potential failure modes provide valuable information about a dam which should be shared across design, surveillance and monitoring, safety review, and rehabilitation activities. Identifying, describing and evaluating potential site-specific dam failure modes are arguably the most important steps in evaluating the safety of a dam.

1.3 Scope of module

This module provides a framework for the investigation and design of new dams, the evaluation of existing dams, and the design of rehabilitation works for existing dams. It includes:

- An outline of various dam types and issues that should receive close attention during their design.
- Recommended personnel and quality assurance procedures for the design of dams and rehabilitation works.
- A discussion on the assessment of hazards.
- An outline of investigation activities that should be addressed during the initial stages of dam design or the initial stages of a dam rehabilitation project.
- Recommended design criteria for new dams and appurtenant structures, the rehabilitation of existing dams and appurtenant structures, and issues that impact on dam safety and should be carefully considered during the design process.
- Recommendations relating to the design and installation of instrumentation systems for the monitoring of dam performance.
- Recommendations relating to design involvement during construction and commissioning, and design documentation.
- Refer also to Module 4 for designer involvement during construction, commissioning, and rehabilitation works.

A list of reference documents is included at the end of the module to assist Designers in the selection of appropriate analytical methods and the completion of specific design tasks.

2. Dams, dam types and related factors

Dams are mainly used for:

- Water storage (dams) and conveyance (canals) for community water supplies, irrigation, and hydropower generation.
- Flood detention (special purpose dams and stopbanks).
- Storage of mine tailings and other wastes and residues.
- Pollution control and water treatment.
- Recreation.

The purpose, use, and associated life of a dam can have a significant bearing on design requirements and standards. These needs should be appropriately considered for each dam.

Most references tend to approach dam design from the perspective of dams which normally have consistently high reservoir levels. However, loading conditions for flood detention and tailings dams often vary from typical assumptions and warrant a modified approach. By way of example:

- A flood detention dam has short exposure times to high reservoir levels, which usually means that internal piezometric pressures that can develop during short- term high reservoir levels are substantially lower than those in a dam that maintains a high reservoir level. Conversely, a low-level conduit in a flood detention dam passes normal flows at valley floor level at all times, which may involve passing sediment materials with significant erosion potential. Therefore, care needs to be taken in the design to account for the wide variations in reservoir level and the longevity and practical repair of the low-level conduit.
- River and estuarine or coastal flood protection banks, usually called 'stopbanks' in New Zealand (but referred to as levees internationally), are linear embankments that serve a flood routing function for an existing watercourse, or act as a flood barrier at a coastal margin. While the Building Act 2004 specifically states that a dam "does not include a stopbank designed to control floodwaters", Owners and Designers of stopbanks should consider applying these Guidelines to stopbank design. The ICOLD Technical Committee on Levees is intending to publish four ICOLD bulletins that will provide additional guidance.
- A tailings dam typically has a relatively short operating life and is rehabilitated to a 'walk away' situation which is quite different from other dam types. The short operating life may warrant one set of design criteria and the rehabilitation condition will usually require different design criteria. The environmental hazard of potentially toxic materials in the dam structure, or stored upstream of the dam, may warrant a much higher level of material and seepage control than that required for a dam storing chemically neutral materials.

The above examples illustrate the need to identify and understand specific design factors and criteria, and avoid the inappropriate application of textbook designs.

There are various types of embankment and concrete dams which are described in sections 6.5 to 6.11 of this module. Some examples of the significant factors which should be recognised and addressed during the design, construction, evaluation and rehabilitation of the more common dam types are as follows. These lists of factors are not intended to provide an exhaustive checklist of factors to be considered in design. Further details are provided in section 6.

Earthfill dams

- Foundation materials that have low strengths and are prone to instability and excessive settlement (e.g., estuarine deposits and thick deposits of soft and compressible soils).
- Foundations in Karstic areas (areas of irregular limestone in which erosion has produced fissures, sinkholes, underground streams and caverns).
- Open joints in the foundation rock allowing seepage flow under the dam with little head loss and resulting artesian conditions at or downstream of the dam toe.

- The need for foundation treatments at and along the contact with the embankment to ensure embankment materials cannot be eroded into open joints or fractures in the foundation.
- The importance of abutments with smooth profiles and without steps or irregularities that could result in arching or inadequate compaction at the foundation contact, which can encourage the development of preferential paths for seepage and seepage-induced erosion.
- The importance of good foundation preparation and appropriate design of filter and drainage materials for the effective control of seepage.
- The potential for contamination of and/or segregation of filters and drains through poor design, production, or placement techniques.
- The potential for low levels of on-site supervision to result in near-horizontal preferential seepage paths, high seepage flow volumes and velocities, or high internal pressures.
- The vulnerability of certain New Zealand materials, such as loess and central North Island pumiceous deposits, to erosion or piping and the extreme care that needs to be taken in their use.
- The care that needs to be taken in the use of dispersive soils.

Zoned earthfill and rockfill dams, and concrete faced rockfill dams

- The quality of the foundation for seepage control in upstream sloping core, central core or concrete faced rockfill dams requires close attention.
- The foundation must be capable of withstanding high hydraulic gradients.
- The transition materials in a sloping core or central core rockfill dam must be capable of preventing erosion and allowing effective drainage of seepage flows.
- The durability, compressibility and permeability of the rockfill must be sufficient to limit face slab deflections and allow controlled drainage of seepage flows.
- The care that needs to be taken in the design of the joint between the concrete plinth and the concrete face slab in a concrete faced rockfill dam to minimise the potential for deformation and leakage.
- The care that needs to be taken in the design of transition materials downstream of the concrete facing and plinth in a concrete faced rockfill dam to restrict seepage flows through the dam in the event of damage to the concrete face slab.

Concrete gravity, hardfill and roller compacted concrete (RCC) dams

- The care that needs to be taken in the investigation, evaluation and treatment of foundation discontinuities (i.e., joints, shears and faults) that could individually or in combination affect the stability of the dam.
- The use and effectiveness of foundation drains and internal drainage systems for the control of uplift pressures.
- The choice of materials that will not degrade during the long-term life of the structure due to their chemical reactivity or chemical attack from the stored reservoir contents.
- The upstream facing system and lift joint treatments in hardfill and RCC dams for the effective control of seepage.
- The design needs to account for the adverse effects that weak or deformable foundations and uncontrolled heat of hydration may have on concrete stresses within the dam.

Concrete buttress dams

- The care that needs to be taken in the investigation, evaluation and treatment of foundation discontinuities (i.e., joints, shears and faults) that could individually or in combination affect the stability of the dam.
- The foundations need to be able to withstand concentrated buttress forces, particularly under earthquake loading.
- The foundation must be capable of withstanding high hydraulic gradients.
- Temperature effects are also important.

Concrete arch dams

- The care that needs to be taken in the investigation, evaluation and treatment of foundation discontinuities (i.e., joints, shears and faults) that could individually or in combination affect the stability of the dam.
- The care that needs to be taken in determining the effective deformation modulus for the abutments and foundation, as a variation in the modulus will affect the ability of the abutments to support the arch thrusts and result in a redistribution of stresses in the dam and the loads on the abutments.
- The foundations and abutments must be able to carry gravity loads and arch thrusts.
- Arch dam design is highly complex and requires a high-level of specialisation in arch dam analysis.
- The abutments and foundation must be capable of withstanding high hydraulic gradients.
- The care that must be taken in determining piezometric pressures that develop along the planes of potential foundation and abutment failure blocks and wedges, as they can contribute both uplift and driving forces and reduce foundation and abutment stability.
- Temperature effects are also important.

3. Personnel and Quality Assurance

3.1 Introduction

It needs to be appreciated that due to the unique nature of each individual dam, dam engineering is a mixture of science and art. The art of dam engineering is the merging of geology, hydrology, hydraulics, structural, and geotechnical engineering disciplines based on science and experience to deliver safe dam structures.

Regardless of how complete investigation and design methods may be, they cannot replace the application of competence and experience.

Owners responsible for the development or rehabilitation of dams should seek advice on current dam engineering practice to ensure dam safety is not compromised by their decisions. Some Owner organisations with dam portfolios may have in-house personnel with sufficient experience to make sound decisions regarding the appropriate investigation, design, construction, commissioning, rehabilitation and operation of a dam. However, in other cases where in- house personnel knowledge and experience are limited, an Owner should consider appointing an Owner's Engineer to assist in all decision making relating to the development or rehabilitation of a dam.

Owner decisions that significantly enhance the whole-of-life safety of a dam include:

- The appointment of organisations and personnel with sufficient skills, qualifications, and experience for the investigation, design, construction, commissioning, operation, assessment, rehabilitation, and decommissioning of a dam.
- All appointed personnel should be suitably qualified and experienced in the development and operation of dam projects similar to that proposed by the Owner.
- The consistent involvement of the key design personnel throughout the complete design, construction supervision, commissioning, and performance review post commissioning of a dam. The consistent involvement of design personnel during construction and commissioning is strongly recommended to ensure that the design intent is correctly interpreted and proper account is taken of the effects of site conditions that prove to be different from those assumed in the design. Such changes may not be obvious to people other than the key design personnel.
- Quoting from the US Federal Guidelines for Dam Safety (FEMA, 2023) "the design function can never be considered finished as long as the dam remains in place - design involvement should continue throughout construction and operation of the project".
- The engagement of suitably skilled and experienced Peer Reviewers throughout the entire process, including scoping, investigation, design, construction supervision, commissioning, and performance review post commissioning of a dam. This is particularly important for Medium and High PIC dams, as well as high value dams or those with novel design or operational aspects. Peer Review by suitably skilled and experienced senior engineers provides critical oversight and technical advice on the dam development to Owners and key stakeholders. They also provide independent assurance that the design and construction are appropriate and meet acceptable practice for the dam.
- The allocation of sufficient funds and time for the scoping, investigation and design of a dam, or the rehabilitation of an existing dam. Proper investigation, design and rehabilitation require that all issues critical to the whole-of-life safety of the dam are identified, investigated and appropriately resolved. This requires that each stage of the work is thoroughly scoped and executed, with due allowances for dealing with uncertainty.

Owners must obtain appropriate advice and allocate sufficient time and funds to ensure that the completion of these activities is not compromised by inadequate scoping or practices, or timeframes. Where unknowns cannot be appropriately resolved within the initial allocated budget, the Owner must be prepared to allocate additional funds and resources.

The Owner has legal responsibility for the whole-of-life safety of the dam, so inadequate funding for scoping, investigation, design or rehabilitation will reflect directly on the Owner in the event of a dam safety incident or dam failure.

Where design and build contracts are used to procure a combination of designers and constructors, close attention is required to ensure that appropriately experienced dam designers are included in the team and remain engaged throughout the design development and construction.

The following subsections outline appropriate personnel and quality assurance procedures for the investigation and design of Low, Medium, and High PIC dams, and the rehabilitation of existing Low, Medium, and High PIC dams.

3.2 Low PIC dams

The Lead Designer should have had prior experience in the design of dams similar to the proposed dam type. The Lead Designer has an ethical duty to recognise any capability limitations in the design team and seek expert advice to address the capability limitations, and any unforeseen conditions that arise during investigation or construction. Expert advice that should be available includes:

- Specialist hydrological support for the estimation of design inflows.
- Specialist geological/geotechnical support where the dam foundation incorporates unusual features, and/or in the selection of materials for use in the dam body for embankment dam elements.
- Specialist mechanical, electrical and automation support where the project incorporates Dam Safety Critical Systems (i.e., gate and/or valve systems and their associated power and controls systems).

A formal peer review of the investigation and design, by an independent experienced engineer, is unlikely to be required for Low PIC dams, unless unusual conditions are evident.

Regardless of whether an independent peer review is conducted, the Lead Designer should implement an appropriate in-house quality assurance system. This system should regularly review the work to ensure that the investigation and design processes properly address all engineering issues relating to the project, particularly those related to the whole-of-life safety of the dam.

In some cases, it may be appropriate to seek input from in-house or external Technical Specialists to address specific engineering issues, particularly during the initial conceptual phases of a project.

3.3 Medium PIC dams

The Lead Designer should have had prior experience as a Lead Designer for a similar Medium PIC dam, or should have been a major contributor to the design of a similar Medium, or High, PIC dam.

The design team should also include:

- Specialist hydrological support for the estimation of design inflows.
- Specialist geological and geotechnical personnel with appropriate prior experience in the assessment of dam foundations, knowledge of the geology local to the dam site, the assessment of embankment materials, and the use of embankment materials in the design and construction of dams.
- Specialist personnel with prior experience in the design and placement of concrete materials (e.g., concrete mix design, aggregate quality, durability, and temperature control) in the design and construction of dams.
- Specialist personnel with appropriate prior experience in the design and construction of hydraulic structures (e.g., spillways, intake structures).
- Specialist mechanical, electrical and automation support where the project incorporates Dam Safety Critical Systems (i.e., gate and/or valve systems and their associated power and control systems).

A formal peer review of the scoping, investigation, and design, by a suitably competent and experienced independent engineering team, reporting to the Owner, should be completed. The building consent authority may require a peer review report to be included in the building consent application.

The peer review team would normally include a geotechnical engineering specialist. When Dam Safety Critical Systems are involved, a specialist in such systems should also be considered.

Peer Review should include an early initial review, to ensure that the design concept is appropriate for the proposed site, hazards, and available construction materials, as well as regular reviews at appropriate intervals during the investigation, design, and construction.

Formal in-house systems for the planning, checking, and reviewing of all investigation and design work should be in accordance with a quality plan. The quality plan, which will vary in scope and detail depending on the project, should include:

- A detailed investigation and design brief setting out objectives, data sources and assumptions, engineering design criteria, standards, methods of analysis, and the like.
- Description statements of personnel responsibilities and interdisciplinary interfacing.
- A means for handling design changes that arise during design and construction.
- Communication and documentation requirements.

3.4 High PIC dams

The Lead Designer would be expected to have had prior experience as a Lead Designer for a similar High PIC dam or should have been a major contributor to the design of a similar High PIC dam. The design team should also include specialist personnel with appropriate skills, qualifications and prior experience in all areas of investigation and design, including those identified in section 3.3 for a Medium PIC dam.

A formal peer review of the scoping, investigation, design and construction by an independent experienced engineering team should be a mandatory requirement. The Reviewer(s) should have a sound background of experience in the type of dam being designed and constructed. To achieve the most effective and cooperative peer review, some basic tenets should be followed, including:

- Appointing a panel of peer reviewers where the dam includes multiple features that cannot be effectively addressed by a single peer reviewer (e.g., complex geological aspects, dam embankment, spillway and low-level outlet structures, Dam Safety Critical Systems). For major new dams, and complex refurbishment works on existing dams, it may be necessary to engage international specialists on the peer review panel.
- Commencing peer review early in the design process to ensure that the design concept and investigation are appropriate for the proposed site, hazards, and available construction materials and resources.
- Encouraging good and regular communication between the Owner, Reviewer(s) and the Designer.
- Briefing the Reviewer(s) to assess, discuss, and provide formal comment on areas considered to be important for dam safety.
- Ensuring that the Reviewer does not become the ‘Designer’ of the dam by including this principle in the peer review contract, and ensuring all designs are proposed and developed by the Designer.
- Guidance for reviewing the work of another engineer is provided in Engineering New Zealand Practice Note 02 - Peer Review (Engineering New Zealand, 2018).
- Having a mechanism agreed between all parties for resolving any areas where the Reviewer and Designer have strongly differing views and the Reviewer does not support the Designer’s proposals. For example, the Reviewer may consider that an aspect of design is not appropriate or proven, or does not adequately address a particular dam safety risk. Ultimately all differences should be resolved to the Owners satisfaction.
- The design must remain the Designer’s responsibility.

Formal in-house systems for planning, checking, and reviewing all investigation and design work should comply with a quality plan that meets the requirements of an appropriate quality assurance system (e.g., ISO, 2015). The plan should be appropriate for the scale and importance of the project and include the items outlined above for Medium PIC dams, along with additional detail to adequately cover all specialist inputs to the project. Adherence to the quality plan is of prime importance.

3.5 Producer Statements

Refer Module 1 (Legal Requirements).

4. Hazards, threats and performance criteria

4.1 Introduction

There are a number of natural hazards that can affect the safety of a dam. The primary natural hazard during the construction of a new dam, or the rehabilitation of an existing dam, is a flood that exceeds the discharge capacity of the diversion works during initial construction or the available discharge capacity during the completion of a rehabilitation project. Natural hazards during commissioning and operation primarily include floods and earthquakes (ground shaking and fault hazards). Additionally, reservoir related hazards such as landslides, reservoir-induced seismicity, high winds and waves, and seiches developed by fault movement or landslides into the reservoir can affect dam safety. In some areas of New Zealand, volcanic hazards are also present. An overview of regional natural hazard information is provided on the New Zealand Natural Hazards Portal (Natural Hazards Commission Toka Tū Ake, 2024).

In addition to natural hazards, threats relating to human activities can affect the safety of a dam. These threats include human errors, oversights, inadequate supervision, cyber attacks, data breaches, reliability of communication and control systems, security breaches and vandalism. The Owner needs to implement appropriate systems to minimise the consequences arising from such threats.

Owners and Designers have an obligation to identify and assess all natural hazards and threats relating to a proposed development or a proposed rehabilitation project and adopt procedures that reduce these hazards and threats to acceptable levels of risk. In dam engineering this is normally achieved through the classification of a dam, according to the consequences that would result from a dam failure, and the adoption of appropriate design criteria that reflect the consequences. These Guidelines utilise criteria that are closely aligned to those recommended in internationally recognised dam engineering publications. In some cases, consent processes and legal precedents may set design criteria which are more stringent than those recommended in internationally recognised publications. For example, a consent process could result in far more stringent seepage control measures for a tailings dam than for a water supply dam. In addition, minimum design or performance criteria could be set by the Owner's insurers.

In some countries, risk assessment has been utilised as a tool to assist in establishing appropriate design criteria for hazard management (refer Bulletin 130, ICOLD, 2005a).

While risk assessment can be used as a tool to assist in the identification of appropriate design criteria, these Guidelines provide criteria closely aligned to those recommended in ICOLD Bulletins and in countries with similar attitudes to risk as New Zealand (e.g., Canada, USA).

The following sections discuss hazards and threats and include recommended design criteria for the management of hazards at Low, Medium, and High PIC dams.

4.2 Flood hazards

4.2.1 Permanent works

The PIC for a dam and the incremental consequences of a dam failure (i.e., the consequences over and above the pre-breach condition) are the main determinants in selecting the Inflow Design Flood (IDF).

ICOLD Bulletin 82 (ICOLD, 1992a) includes a discussion on the selection of the IDF and lists IDFs adopted by a number of countries for particular dam classifications. Federal Energy Regulatory Commission (FERC, 2016) states that "the PMF should be adopted as the IDF in those situations where consequences attributable to dam failure for flood conditions less than the PMF are unacceptable. The determination of unacceptability is clearly necessary when the area affected is evaluated and indicates there is a potential for loss of human life and extensive property damage".

The selection of design loads for a dam should relate to the life risk exposure resulting from credible failure scenarios. There are two parameters which provide a measure of life risk exposure – Population at Risk (PAR) and Potential Loss of Life. These parameters are discussed in section 2.4.3 of Module 2. The Population at Risk is the easier parameter to quantify whereas the Potential Loss of Life (the number of possible fatalities) resulting from a hypothetical dam failure is more difficult to quantify. Any estimate of Potential Loss of Life will have a high degree of uncertainty. However, there are merits in using both parameters to guide the selection of an IDF.

A two-table approach is recommended for the selection of the minimum IDF. Table 4.1a uses the incremental PAR together with the assessed PIC of a dam to define the recommended minimum IDF as an initial determination. If this produces a minimum IDF which is considered too conservative for the expected life exposure risk for a credible dam failure scenario (for example where the PAR is primarily comprised of itinerant persons), then Table 4.1b can be used. Table 4.1b uses the estimated incremental Potential Loss of Life in conjunction with the assessed PIC of a dam to define a refined minimum IDF.

However, for a High PIC dam, the minimum IDF obtained using Table 4.1a may be less conservative than the IDF obtained from Table 4.1b. In this case, it is recommended that the minimum IDF should be evaluated using both Tables 4.1a and 4.1b and the more conservative IDF adopted.

Application of Table 4.1b will require estimates of Potential Loss of Life to be made using the quantitative methods discussed in section 2.4.3.4 of Module 2 (USBR's RCEM empirical methodology and more advanced modelling tools). In section 2.4.3.4 of Module 2, it is carefully noted that USBR uses "RCEM Potential Loss of Life estimates in support of dam safety risk assessments and not for life loss estimates for PIC assessment purposes". If Table 4.1b is used for the selection of the minimum IDF for a dam based on an estimated Potential Loss of Life obtained by applying the RCEM methodology, then practitioners need to recognise that they are applying the methodology outside of its original purpose.

Table 4.1a: Recommended minimum Inflow Design Floods based on PAR

PIC	Incremental Population at Risk	IDF (1 in T AEP)
Low	No limit	1 in 100 to 1 in 1,000 AEP [1,2]
Medium	No limit	1 in 1,000 to 1 in 10,000 AEP [1,2]
High	0	1 in 10,000 AEP
	1-10	1 in 10,000 AEP to average of 1 in 10,000 AEP and PMF peak discharge [3]
	11-100	Average of 1 in 10,000 AEP and PMF peak discharge, to PMF [3]
	> 100	PMF

Notes:

1. Selected primarily on basis of incremental consequences of dam failure. However, from an asset management perspective, the operational criticality of a dam structure may dictate a higher IDF standard than that required by the PIC rating of the dam.
2. For Low and Medium PIC dams, the IDF should account for potential climate change effects to the end of 2100 for 1 in 100 and 1 in 1,000 AEP estimates of flood magnitude.
3. If the Incremental PAR is between the lower and upper limits, the minimum IDF should be determined on a pro-rata basis between the peak discharge values for the lower and upper IDF limits.

Table 4.1b: Recommended minimum Inflow Design Floods based on Potential Loss of Life

PIC	Incremental Potential Loss of Life [4]	IDF (1 in T AEP)
Low	0	1 in 100 to 1 in 1,000 AEP [1,2]
Medium	0	1 in 1,000 AEP [1,2]
	1	1 in 10,000 AEP [1,2]
High	0-10 persons	1 in 10,000 AEP to PMF [3]
	> 10 persons	PMF

Notes:

1. Selected primarily on basis of incremental consequences of dam failure. However, from an asset management perspective, the operational criticality of a dam structure may dictate a higher IDF standard than that required by the PIC rating of the dam.
2. For Low and Medium PIC dams, the IDF should account for potential climate change effects to the end of 2100 for 1 in 100 and 1 in 1,000 AEP estimates of flood magnitude.
3. If the Incremental Potential Loss of Life is between the lower and upper limits, the minimum IDF should be determined on a pro-rata basis between the peak discharge values for the lower and upper IDF limits.
4. Refer to Module 2 for Potential Loss of Life evaluation procedures.

As an alternative to the prescribed Inflow Design Flood standards set out in Tables 4.1a and 4.1b, a risk-informed approach could be used to determine an appropriate IDF for a new or existing dam. Chapter 6 of the ANCOLD Guidelines on Selection of Acceptable Flood Capacity for Dams (ANCOLD, 2000) describes a risk-informed approach to assist in considering options for acceptable flood capacity for a dam. Further guidance on the use of risk-informed decision making (RIDM) in design is provided in Module 7.

It is recognised that the estimation of extreme flood events exceeding the 1 in 100 AEP event can be difficult. Accepted flood estimation approaches for larger catchments (>10 km^2) in New Zealand include flood frequency analysis for gauged catchments, where a standard probabilistic distribution is fitted to a long-term series of annual flood maxima, and the application of a regional flood frequency method for ungauged catchments using flood estimates scaled from other similar gauged catchments (McKercher and Pearson, 1989). McKerchar and Pearson (1989), NIWA (2016), NIWA (2018a) also provide guidance on the reliability of extrapolated flood estimates based on a limited length of gauged flow record.

Rainfall patterns in New Zealand are highly influenced by orographic (i.e., altitude) effects, resulting in larger catchments often being affected by rainfall gradients. Rainfall/runoff modelling, using rainfall frequency estimates, a rainfall temporal distribution and a uniform rainfall pattern across a catchment as inputs, is adopted for flood estimation for small catchments less than 10 km^2, larger catchments, and where the duration of gauged flow records is inadequate or non-existent. Medium and High PIC dams are more likely to have larger catchment areas, making flood frequency analysis and the regional flood frequency method generally appropriate for flood estimation.

McKerchar and Pearson (1989), NIWA (2016), NIWA (2018a) describe suitable methods for the estimation of design floods for New Zealand catchments. Recommended methods for estimating the IDF for Low, Medium, and High PIC dams are as follows:

- **Low PIC dams:** Flood frequency analysis should be used where adequate data for gauged catchments is available. Alternatively, less rigorous methods for estimating the IDF, such as the rational method in conjunction with a triangular-shaped flood hydrograph, other regional flood estimation approaches, and rainfall/runoff routing, are appropriate.

- **Medium PIC dams**: IDFs should be determined using two or more recognised hydrological methodologies and appropriate judgement. IDFs should generally be determined either by a flood frequency analysis approach where the catchment is gauged and the gauged flow records are adequate, by a regional flood frequency approach where the catchment is ungauged or the gauged flow records are inadequate, or by rainfall/runoff routing. Whatever approach is adopted, the available base data (river/stream flows or rainfall) used to derive the flood estimates means that there will always be a degree of uncertainty in estimates of floods greater than the 1 in 100 AEP flood. In the face of this uncertainty, a conservative approach should be adopted in estimating the IDF.
- **High PIC dams:** IDFs should be estimated from a range of hydrological methods as recommended for Medium PIC dams. PMFs should be determined using the methodologies included in Tomlinson and Thompson (1992) and Campbell et al. (1994) (summarised in NIWA, 2010). Comparisons should be made with flood estimates derived using the methodology included in Griffiths et al. (2014), which estimates extreme rainfall depths. These estimates can then be routed through catchments to determine IDFs, as included in Table 4.1.

Hydrological data records increase over time, enhancing the understanding of hydrological hazards. It is therefore highly likely that the estimated flood values for the return periods listed in Table 4.1 will change with time, and could necessitate future upgrade works to increase the capacity of flood discharge facilities.

For existing dams, the flood discharge facilities should be capable of safely discharging the IDFs listed in Table 4.1. If the flood discharge facilities are unable to safely discharge the recommended IDF, the discharge capacity is deficient and should be addressed. If the upgrade works to address a flood discharge deficiency are limited by practicality or practicable cost, the target flood capacity should be as high as reasonably practicable.

IDFs should always be estimated by hydrologists and engineers with experience in New Zealand hydrological conditions. In addition, appropriate established hydrological methodologies for New Zealand conditions should be employed for the assessment of IDFs up to the Probable Maximum Flood (PMF).

The potential effects of climate change are uncertain. However, these effects should be assessed using the latest guidance from the Ministry for the Environment for non-coastal climate risks (e.g., MfE, 2018; MfE, 2021; Bodeker et al., 2022) when evaluating IDFs up to at least the 1 in 1,000 AEP IDF. Collins (2020) provides some guidance on the likely effect of climate change on mean annual floods across New Zealand although it is highly uncertain whether the mean annual flood trends are also likely to be indicative for extreme floods.

For IDFs more extreme than the 1 in 1,000 AEP flood (excluding the PMF), the uncertainty in any estimate is likely to exceed the uncertainty due to the potential effects of climate change. Both sources of uncertainty should be considered in the selection of a 1 in 10,000 IDF. Estimation of PMFs for NZ conditions follows a prescribed methodology involving rainfall runoff modelling using Probable Maximum Precipitation (PMP) (Tomlinson and Thompson, 1992; Campbell et al., 1994; Thompson and Tomlinson, 1993). PMP estimates should be compared to major storms which have occurred in New Zealand but were not considered within the aforementioned methods. An example of such is the 1981 Kerikeri storm, which, when maximised, has been identified to potentially exceed PMP estimates from Thompson and Tomlinson (1993) by 25% (McKerchar, 2009).

The methodology for incorporating the potential effects of climate change when estimating PMP is an area of active investigation. Until the results of peer-reviewed and verified investigations become available, it is not recommended that PMF estimates should be adjusted for potential climate change effects

As a minimum, for all dams, freeboard sensitivity should be assessed for potential future changes to hydrology, including a range of climate change scenarios and potential land use changes in the upstream catchment. For new dams, the need to future-proof the dam and spillway arrangements for future changes to hydrology is an important design consideration.

If the reservoir is small and provides no opportunities for the storage of flood water during the IDF, the peak flow during the IDF is the only parameter necessary for the design of the permanent flood management facilities.

Where the reservoir is sufficiently large for the storage of flood flows during the IDF, the hydrograph for the IDF should be determined to allow flood routing and the calculation of the peak lake level and outflow from the reservoir for various AEP storms.

A range of storm durations should be considered to determine the critical peak reservoir level. If a rainfall-based approach is used to estimate the IDF, the hyetograph or rainfall pattern could be based on recorded rainfall events or prescribed patterns (e.g., HIRDS). Some level of justification is required for the selected pattern and sensitivity analysis should be considered. If an inflow-based approach is used to estimate the IDF, the IDF can potentially be derived by scaling a representative historic inflow hydrograph calculated from measured reservoir level and outflow data. However, care must be exercised to ensure that the scaled IDF hydrograph reflects the correct inflow volume for the critical storm duration.

The initial reservoir level should typically be taken as the normal operating water level. For many flood detention dams, this will be the normal level of stream flow in the impounded watercourse. However, for those flood detention dams with a permanently impounded reservoir, the initial level should be taken as the normal pond level under dry weather conditions.

Sensitivity analysis could be considered to evaluate the effect of either a double peaked rainfall storm, or two consecutive rainfall storms in rapid succession if such occurrences have been observed in historical rainfall records. The latter situation will be significant if the second rainfall event arrives before the reservoir has been fully drawn down after the occurrence of the first rainfall event.

A complex routing model involving more than one reservoir should be peer reviewed by a suitably experienced Technical Specialist.

Finally, due consideration should always be given to the effects of possible future land use changes on flood magnitudes. For example, deforestation and subdivision development in upstream catchments are likely to result in increased runoff and larger flood events.

Upgrading an existing dam to a PMF design standard-may be difficult to achieve and hard to justify economically. This applies particularly to flood detention dams in a constrained urban setting. A risk-informed approach could be considered in such circumstances as an alternative method for selecting the IDF.

4.2.2 Temporary works

Information on diversion flood frequencies adopted by various countries for the construction of embankment and concrete dams is included in ICOLD Bulletin 48A (ICOLD, 1986a), and a discussion on the selection of an appropriate flood for the sizing of diversion works during construction is included in ICOLD Bulletin 144 (ICOLD, 2010a). There is no universally accepted method for selecting an appropriate flood for the sizing of diversion works during construction and the choice is generally based on the dam site, the dam type, the construction cost, and the consequences if the diversion capacity is exceeded.

The performance criteria for diversion works during the construction of a dam, or the completion of rehabilitation works, should be as follows:

- For new dams the risk (likelihood x consequence) of loss of life during construction, as far as practicable, should be no greater than that over the life of the dam.
- For existing dams where reasonably practicable, the risk (likelihood x consequences) of dam failure should not be increased during the completion of any rehabilitation works.
- The design of any temporary works should include consideration of the PIC for any necessary cofferdams, and the design criteria for the cofferdams should be consistent with their PIC as recommended in section 6.3 of this module.

If there is insufficient upstream storage during construction to attenuate the peak diversion flood flow, the peak flow for the selected diversion flood dictates the required hydraulic capacity of the diversion works. However, if the diversion design relies partly on upstream storage to attenuate the peak flow, the total volume of the selected diversion flood should be estimated, and the diversion design should include consideration of the possibility of the available storage volume being exceeded.

A risk-informed approach provides a useful basis for selecting and sizing flood passage facilities during construction. Risk reduction measures should be implemented where large reductions in risk are available for relatively low expenditure. In some cases, it may be necessary to adopt risk reduction measures that are not justified on economic grounds; however, in all cases, the adverse effects of the risks should be made as low as reasonably practicable irrespective of any absolute criteria. By properly considering potential failure modes and applying 'as low as reasonably practicable' criteria, based at least in part on risk assessment (qualitative, and/or quantitative), useful inputs can be provided for developing a defensible design with an assurance that all reasonably foreseeable failure modes have been identified and adequately addressed.

The most obvious potential failure mode during construction is overtopping of the dam due to the inflow flood exceeding the capacity of the diversion facilities. Clearly the consequences of overtopping an embankment dam during construction would be far greater than those that would result from the overtopping of an equivalent concrete dam. The following subsections provide comment on the diversion works for concrete, embankment and concrete-faced rockfill dams.

Concrete dams

For concrete dams, overtopping during construction would most likely lead to flooding of the work area and possibly some erosion of the dam toe. Provided the risk of toe erosion was not excessive, the main risks would be injuries or loss of life for construction personnel and damage to equipment. Because of the 'block type' construction typically used for concrete dams, the level of potential damage and adverse effects can be managed and kept sufficiently low. Risks to personnel can be mitigated by understanding warning times and implementing appropriate evacuation procedures. However, risks of damage to construction plant and the works, as well as the tolerance for these costs, depend largely on the Owner's risk tolerance and/or insurance considerations.

If personnel and dam safety risks are adequately managed, an inflow event with a return period of 10 years may be appropriate for the sizing of the diversion works.

Embankment dams

Embankment dams are highly unlikely to withstand sustained overtopping; hence the sizing of the diversion works is critical to dam safety during construction. A risk-informed approach should consider the likelihood of overtopping of the partially completed structure (e.g., early in dam construction the consequences of overtopping should be significantly less than those when the dam is nearing completion); the time of exposure; the consequences of dam failure at various stages of construction; and the risks to the construction works, the construction programme, and construction personnel. Quantitative risk assessments, which consider factors such as likelihood, exposure times, and downstream consequences, may be appropriate, particularly for Medium and High PIC dams.

Diversion and spill facilities should not concentrate discharge flows onto the dam body.

The size of the diversion works is critical until such time as the dam crest exceeds the invert levels of the permanent flood discharge facilities by an adequate margin to eliminate the risk of overtopping in a design inflow event. In some cases, temporary spillway facilities at lower levels may be required to reduce the overtopping risk to an acceptable level. Exposure times to flood risk can be considered as part of the risk assessment process. Emergency action plans, with appropriate evacuation procedures, should be considered to assure that the risk to public is appropriately mitigated. Emergency action planning is addressed in Module 6.

If a site-specific risk-informed approach that considers exposure times and the downstream consequences of failure is not completed, the Owner and Designer should consider the following guidelines for diversion capacity:

- For a dam with a Low PIC during construction, an IDF of between a 1 in 10 AEP and a 1 in 50 AEP should be considered, depending on the duration of construction.
- For a dam with a Medium PIC during construction, an IDF of between a 1 in 50 AEP and a 1 in 200 AEP should be considered, depending on the duration of construction.
- For dam with a High PIC during construction, an IDF of between a 1 in 100 AEP and a 1 in 500 AEP should be considered, depending on the duration of construction.

Concrete-faced rockfill dams
Concrete-faced rockfill dams, with well compacted free draining rockfill shoulders, usually have a greater resistance to overtopping than most embankment dams.

Except for overtopping considerations, the guidelines included above for temporary works for embankment dams are also applicable to concrete-faced rockfill dams.

However, added protection to the downstream face of a concrete-faced rockfill dam can reduce the likelihood of an overtopping failure (ANCOLD, 1991a). If it can be demonstrated that proposed protection works on the downstream face will prevent dam failure, then it may be appropriate to include limited overtopping of the dam during the passage of the selected diversion flood event.

4.3 Seismic hazards

Seismic hazards include ground motions, fault displacements, hydrodynamic pressures, liquefaction, landslides (above the reservoir and submarine), rockfalls, seiches (waves caused by fault displacement or strong ground motions in the reservoir), turbidity currents, and tsunamis (including waves generated by landslides into or within the reservoir).

The following subsections outline recommended practices for the selection of appropriate seismic hazard parameters for dams. Ground motions, fault displacements, and liquefaction are discussed.

Landslides and seiches associated with seismic activity are addressed in section 4.5. Consideration of hydrodynamic pressure in the seismic design is required for the design of concrete gravity and buttress dams, arch dams, and appurtenant structure components.

4.3.1 Terminology

A number of terms are commonly used in the assessment of seismic hazards and the definition of seismic performance criteria. They include the Maximum Credible Earthquake (MCE), the Operating Basis Earthquake (OBE), and the Safety Evaluation Earthquake (SEE).

Definitions for each of the terms are presented below and included in the Glossary in the Parent Document of these Guidelines.

- **MCE** – The largest reasonably conceivable earthquake magnitude that is considered possible along a recognised active fault, or fault system, or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. The ground motion affecting a dam site due to an MCE scenario is referred to as the MCE ground motion.
- **SEE** – The earthquake that would result in the most severe ground motion which a dam and appurtenant structures must be able to endure without uncontrolled release of the reservoir.
- **OBE** – The earthquake for which a dam, appurtenant structures and mechanical, electrical, power, control, and communication equipment that fulfils a dam safety critical function is designed to remain operational, with any damage being minor and readily repairable following the event.

4.3.2 Seismic performance criteria

Industry practice for the seismic design and analysis of dams involves consideration of two levels of earthquake: the SEE and the OBE.

- The performance requirement for the SEE is that there is no uncontrolled release of the impounded contents when the dam is subjected to the seismic load imposed by the SEE. Damage to the structure may occur; however, damage must not compromise safe management of the reservoir following the SEE. Minor damage may be acceptable, provided it can be repaired and functionality can be restored quickly, for example, within hours of the event. Safe management of the reservoir requires that dam safety critical equipment associated with the dam and appurtenant structures remains functional following the SEE load case. Definition and classification of dam safety critical equipment is given in section 6.12.

- The performance requirement for the OBE is that the dam and appurtenant structures remain functional and that the resulting damage is minor and easily repairable. Dam safety critical equipment should be operational during and following the OBE load case.

4.3.3 Ground motions for design and evaluation

Ground shaking affects all structures and dam safety critical equipment above and below ground. Ground shaking at elevated structures on dams and features such as steep abutments can be amplified from the ground shaking that occurs at the valley floor.

Measures of ground shaking include peak parameters (i.e., peak ground acceleration, velocity or displacement), response spectra, and time-histories. Acceleration is the most commonly used parameter for ground motions in dam design, while velocity and displacement parameters are used less frequently.

4.3.3.1 Horizontal orientation and directivity

Peak ground motion parameters and response spectra can be defined as the largest in any directions (often called RotD100), the median of all directions (often called RotD50), the largest of two orthogonal as-measured directions, the geometric mean of two orthogonal as-measured directions (often called geomean), or some other orientation definition (Boore, 2010). The RotD50 is an orientation independent combination of the two horizontal component ground-motions. Modern seismic hazard studies provide probabilistic estimates of RotD50 seismic loads in terms of peak ground acceleration and spectral acceleration. These values are readily available for download from the NSHM 2022 (GNS Science, 2022a) and can be adopted for the design and analysis of dams.

Dam sites near major active faults may be subject to the effects of directivity during large earthquakes involving extended rupture of the fault. Directivity refers to the phenomenon where seismic waves are more strongly focused in the direction of the fault rupture. Three main effects of directivity on ground motions include:

- an increase in long-period RotD50 spectral acceleration when rupture occurs toward the site and a decrease when rupture occurs away from the site of interest (typically at periods greater than about 0.5 seconds),
- long-period spectral accelerations often become polarised, where the fault normal spectral accelerations will be greater than the fault parallel spectral accelerations, and
- sites subject to forward directivity will often contain a large pulse in the velocity time histories in the fault normal direction (Donahue et al., 2019). In such cases the fault normal and fault parallel motions should be considered for directivity.

4.3.3.2 Uniform Hazard Spectrum (UHS)

The response spectrum based on the probabilistic approach is a Uniform Hazard Spectrum (UHS). A UHS provides response spectral ordinates (usually accelerations) that each have the same Annual Exceedance Probability (AEP). A UHS is developed by first computing the hazard at a suite of spectral periods using response spectral ground motion models. The hazard is computed independently for each spectral period. For a selected AEP, the spectral ordinate for each spectral period is selected from the hazard curves. These spectral ordinates are then plotted as a function of period to form the UHS. Since the hazard is computed independently for each spectral period, a UHS does not represent the spectrum of any single event. For long return periods, if a real earthquake results in a spectral acceleration equal to the UHS at a given period of interest, then it is less likely that spectral acceleration at other periods will be as high as the UHS.

4.3.3.3 Conditional Mean Spectra (CMS)

To remove the inherent conservatism of UHS, the concept of a Conditional Mean Spectra (CMS) was introduced by Baker and Cornell (2006). This allows for the creation of a response spectrum that consists of the mean values of the spectrum at all periods, conditioned on the spectral acceleration from a UHS at a specific period of interest (such as the fundamental period of a dam) being reached. The CMS concept also can be applied to deterministic seismic hazard analysis when using a percentile greater than 50%. Both UHS and CMS are considered acceptable, but Designers should be familiar with the limitations of the different approaches. Additional judgement on the period(s) of interest is required in the application of CMS to consider the vibration modes that would be contributing to the responses. For example, continuum or distributed systems such as dams do not exhibit a marked response at their fundamental period of vibration. These systems respond over a broad range of frequencies and thus, the UHS is generally more appropriate for these systems.

4.3.4 Selection of seismic design criteria

ICOLD Bulletin 148 (ICOLD, 2016) also provides guidelines for the selection of parameters to be used in the seismic design, analysis, and safety evaluation of new or existing dams and their appurtenant structures. Recommended seismic design criteria for dams are as follows:

4.3.4.1 Operating Basis Earthquake (OBE)

Traditionally, the OBE represented ground motions with an annual exceedance probability (AEP) of about 1 in 150. Some Owners may wish to adopt a higher standard for the OBE (e.g., an AEP of 1 in 500) to reflect the value of the asset or its importance in providing a service. Owners should also consider other regulated requirements for essential infrastructure. Based on the results of a risk assessment, damage to flood detention dams may be considered acceptable for the OBE.

4.3.4.2 Safety Evaluation Earthquake (SEE)

The SEE represents the maximum level of ground motion that a dam should withstand without an uncontrolled release of the reservoir. Modern practice allows the SEE ground motion parameters to be developed by either a deterministic or a probabilistic approach, or consideration of both approaches. ICOLD Bulletin 148 (ICOLD, 2016) and Mejia et al. (2001) provide recommended SEE ground motion parameters for the design and analysis of dams. Both references consider the PIC or hazard rating of the dam as a basis for setting recommended levels for the design earthquake. The recommended basis for developing minimum SEE design loads is detailed in Table 4.2. These Guidelines provide AEP ground motions at the mean value. The recommended minimum SEE ground motion depend on the PIC and Potential Loss of Life. The dependency on Potential Loss of Life for Medium and High PIC dams is consistent with the principal dam safety objective of ensuring life safety.

The AEP criteria for selection of seismic design loads do not align with those for the Inflow Design Flood (Table 4.1). The AEP for the IDF is generally lower than that for the SEE for Medium and High PIC dams. Justification for this is:

1. Precedent with other Guidelines: Other dam design guidelines don't adopt the same AEP for IDFs and SEEs for similar PIC, PAR and Potential Loss of Life. This includes CDA (2013), ANCOLD (2000 and 2019a), and ICOLD Bulletins 170 (ICOLD, 2018a), Bulletin 148 (ICOLD, 2016) and Bulletin 194 (ICOLD, 2022a).
2. Dam failure Statistics: Bulletin 188 preprint (ICOLD, 2019) provides an update on dam incident and failures statistics as of 2019. Of the 291 dam failures studied, 110 occurred under normal operation, 132 under flood conditions, and only 7 were due to earthquakes. Case histories indicate that flood is the leading cause of failure.
3. Probable maximum values: There is no consistent AEP for the PMF but it is generally considered to be in a range of 1 in 100,000 to 1 in 10,000,000 in New Zealand. The definition of an active fault is a fault with a recurrence interval less than 125,000 years which is greater than the AEP of the PMF, although in more tectonically active areas it is much lower (less than 5,000 years). Adopting an AEP of 1 in 10,000 for the SEE and considering the MCE (associated with the multiple rupture scenarios) with a probability of occurrence greater than 1 in 100,000 can be considered close to the probable maximum value with engineering significance for seismic design.

Table 4.2 Recommended minimum SEE design loads

PIC	Incremental Potential Loss of Life	SEE
Low	0	50th Percentile of MCE ground motion, and at least a 1 in 500 AEP ground motion but need not exceed 1 in 1,000 AEP ground motion [1, 2, 4]
Medium	0	50th Percentile of MCE ground motion [1, 4] but need not exceed the 1 in 2,500 AEP ground motion
	1	84th Percentile of MCE ground motion [1, 4] but need not exceed the 1 in 2,500 AEP ground motion
High	0	84th Percentile of MCE ground motion but need not exceed the 1 in 5,000 AEP ground motion [1, 3, 4, 5]
	1	
	2 or more	84th Percentile of MCE ground motion but need not exceed the 1 in 10,000 AEP ground motion [4]

Notes:

1. The SEE design loads should not be lower than the lower-bound spectrum calculated based on NSHM22 for Auckland.
2. The AEP of the selected ground motion or percentile level of MCE ground motion is selected primarily on basis of incremental consequences of dam failure.
3. Adopting a lower SEE design load for High PIC dams with PLL of o or 1 is generally consistent with the recommendations in ANCOLD, CDA and ICOLD guidelines which allow higher AEP for dams with PLL of 0 or 1.
4. If the SEE design spectrum is defined via the deterministic approach, the envelope of the ground motion spectra associated with different MCE scenarios should be adopted.
5. If the PLL is 0 or 1, a dam could still have a High PIC where a catastrophic damage level is assessed for one or more of the critical or major infrastructure, cultural and natural environment categories (refer to Module 2).

Owners of the dam may choose to design or evaluate their dams for ground motions higher than the recommended design level. The decision to adopt a higher criterion could be based on the following reasons:

- The functional requirements.
- The risk rating of the completed dam and reservoir.
- The consequences of underestimating or overestimating the risk.
- The Owner's attitude towards dam safety risks.
- Where there is a significant 'step change' in response and safety with a small incremental change in seismic loading (e.g., liquefaction of foundations or embankment).
- The insurance requirements.

In some cases, the Owner may choose to design the dam based on the highest design level of ground motions regardless of the current assessed PIC. This consideration is due to the potential substantial costs of future remediation if downstream consequences change.

Generally, flood detention dams that do not have a permanent storage volume can be designed assuming Low PIC for earthquake loading conditions given that it is highly unlikely for a SEE to coincide with a large flood. Most flood detention dams are small earthfill embankments that inherently exhibit good resilience to earthquake loads. However, for flood detention dams assessed as Medium or High PIC under 'rainy day' failure scenarios, consideration should be given to the risks associated with a large flood occurring after the SEE but before any damage associated with the SEE has been repaired. These risks depend on the estimated damage from the design earthquake, the dam's performance in a subsequent flood if no repair or only partial repair is undertaken, the resources and time available to undertake repairs following the design earthquake, and the likelihood of a large flood occurring during this period. It is important to note that resources for repairs could be stretched following the design earthquake. To mitigate these risks, it may be appropriate in some cases to design for a higher earthquake loading. Alternatively, post-SEE damage risk mitigation could involve modifying the dam to prevent it from storing a significant volume of water in a subsequent flood event (e.g., by removing a section of the dam or opening a spillway gate or fuse plug).

4.3.4.3 Deterministic approach vs. probabilistic approach

There are two approaches for estimating seismic hazard: deterministic and probabilistic. Each approach has its limitations. The probabilistic approach is favoured by many seismic hazard experts because it provides a uniform basis for evaluating the hazard and yields more consistent results. It considers the sources of all possible earthquakes that may affect a site, resulting in a Uniform Hazard Spectra (UHS) where all ordinates have the same probability of exceedance. The adoption of the UHS for design can be conservative, depending on how it is used. Scenario events can be developed by disaggregation of the results from a probabilistic seismic hazard analysis. Probabilistic methods that consider background sources are best suited for dam sites located away from known active faults. Conversely, a deterministic approach is typically used when active faults are located nearby, as it represents a specific earthquake scenario. The deterministic approach can sometimes result in higher or lower design ground motion estimates compared to probabilistic methods.

Determining whether a probabilistic or deterministic approach is more appropriate for a given dam site requires judgement. Designers may opt for the higher of the deterministic or probabilistic ground motion estimates where in doubt. In the cases where deterministic estimates are significantly lower than probabilistic estimates, it is recommended to limit the reduction in seismic demand parameters to not less than 80% of the higher estimates. This aligns with the limits recommended in ASCE (2022), and concepts within the US Army Corp of Engineers Engineer Regulation 1110-2-1806 (USACE, 2024) and NZTA Bridge Manual (NZTA, 2022).

4.3.4.4 Lower-bound SEE design loads

The lower-bound SEE design loads are introduced to account for unrecognised seismic sources. This will primarily affect Low, Medium and some High PIC dams in areas of low seismicity, such as Northland, Auckland, and the northern and south-western parts of Waikato. The lower-bound design loads apply whenever the probabilistic and deterministic ground motion estimates are less severe than the ground motion resulting from the minimum earthquake. The adopted SEE design loads for dams should not be less than the uniform hazard spectra for Auckland (-36.87, 174.77) based on the NSHM 2022 with the AEP as given below:

- Low PIC: 1 in 1,000
- Medium PIC: 1 in 2,500
- High PIC: 1 in 5,000.

The Low and High PIC lower-bound earthquake spectra are similar to the expected 50th and 84th percentile spectra associated with a magnitude 6.5 earthquake with a closest distance to the rupture surface of 15 km, respectively. This approach aligns with the concepts within NZS 1170.5 (Standards New Zealand, 2004) where a minimum level of seismic loading is adopted.

4.3.5 Methods for estimating ground motion parameters

GNS has published the National Seismic Hazard Model (NSHM) for New Zealand in September 2022 (Gerstenberger et al., 2022). The probabilistic estimates on 0.1-degree intervals in latitude and longitude, are available on the GNS NSHM web-based portal (GNS Science, 2022a), which also links to additional supporting publications. These Guidelines refer to the NSHM 2022, noting that, like all hazard models, the NSHM will undergo periodic updates as the understanding of seismic hazard improves. Owners should work with practitioners to ensure that designs and assessments are based on the most up to date version of the NSHM. Recommended methods for estimating the SEE and OBE for Low, Medium, and High PIC dams are as follows:

4.3.5.1 Low PIC dams

Simplified seismic design will normally be sufficient for Low PIC structures. Detailed site-specific seismic hazard studies are not required.

The GNS NSHM web-based portal provides probabilistic estimates in terms of peak ground acceleration and spectral acceleration, which can be adopted for the design and analysis of Low PIC dams and associated appurtenant structures. The estimates depend on site-soil conditions which are represented by V_{S30} (the average shear wave velocity in the top 30 metres). Estimates of VS30 should consider the uncertainty in the assessment of shear wave velocity profile. V_{S30} may be based on site data where available, site-specific direct measurements, or be inferred from other parameters (e.g., SPT values). Site-specific measurements for V_{S30} are not required for all Low PIC dams. The GNS NSHM web-based portal also includes disaggregation (breakdown of earthquake events by magnitude and distance that contribute to the seismic hazard). This can be used to derive design earthquake magnitude for seismic analyses (e.g., liquefaction and deformations).

MBIE and NZGS published earthquake geotechnical engineering guidelines that provide guidance on three methods that may be used for estimating seismic hazard parameters (Module 1, MBIE and NZGS, 2021). The MBIE and NZGS geotechnical guidelines specifically refer to the NZSOLD New Zealand Dam Safety Guidelines as a document that provides more specific guidance for specialist structures that should, in general, take precedence over the MBIE and NZGS (2021) guidance for these structures.

Deterministic estimates associated with the MCE will require assessment to identify earthquake scenarios, with a specific combination of magnitude, distance, and tectonic type, that may have a significant impact to the site and use of appropriate ground motion characterisation models. The rupture scenarios considered in NSHM 2022, including multiple rupture scenarios, can be obtained from the GNS NSHM Rupture Map (**nshm.gns.cri.nz/RuptureMap**). Deterministic estimates of seismic hazard are not required for the design and evaluation of Low PIC dams, except for the case that a more realistic selection of seismic parameters is needed.

4.3.5.2 Medium PIC dams

For most Medium PIC dams, published data, e.g., NSHM 2022, can generally be used to obtain probabilistic estimates of seismic hazard for design and analysis.

Deterministic estimates of seismic hazard associated with the MCE will require examination of published geological maps and fault databases to identify earthquake scenarios that may have a significant impact on the site. If embankment fill materials or foundations could soften when subjected to strong earthquake ground motions (e.g., cyclic softening or potentially liquefiable soils), or there are weak foundations, the recommended approach detailed below for High PIC dams should be employed. Where complex analyses are used to determine the seismic performance of a dam, the selection of acceleration time history records will require the completion of site-specific seismic hazard assessments as described below for a High PIC dam.

4.3.5.3 High PIC dams

A site-specific seismic hazard assessment should be completed by an appropriate Technical Specialist, using both deterministic and probabilistic analyses. The design ground motions for the SEE and OBE should be selected based on the results of the analyses and should include response spectra and possibly acceleration time histories, depending on the analysis method. Uncertainties and site amplification effects should be addressed. Epistemic uncertainties associated with earthquake sources and ground motion characterisation models should be considered. Site-specific shear wave velocity (V_S) profiles should be determined for dam design if the dam is founded on soils or weathered rocks. In the cases of dam design with foundation undercut, investigations may need to be deeper than 30 m to obtain appropriate V_{S30} to represent the conditions assumed in analysis. If the dam is founded on competent rock, a conventional assumption of V_{S30} = 750 m/s can be applied.

The site-specific seismic hazard assessment requires investigations of the regional tectonic framework, regional and local historical seismicity, potential seismic source, and the relationship between event magnitude, region, site characteristics, and fault characteristics versus the ground motion for a given seismic source. The reliability of the results depends largely on the experience of the investigators in synthesising and interpreting various types of geological, seismological, and geophysical data. Disaggregation of a probabilistic analysis can help inform which magnitudes, distances, sources, and other potential event conditions are contributing to the hazard. It is sometimes helpful to review the disaggregation results to provide additional insight to define the earthquake scenarios considered in the deterministic analysis. The earthquake magnitude used for deterministic analysis requires considerable judgement based on the available data. This magnitude may be different from the maximum magnitude considered in probabilistic analyses. Methods for the determination of magnitude include adopting the mean characteristic magnitude based on a full rupture of the fault, by adopting weighted average magnitude based on a logic-tree approach, or by adopting the most likely fault rupture scenario for the faults that are segmented (which may include a multi-segment rupture across more than one fault). The earthquake scenarios considered in the NSHM (GNS Science, 2022a) including multiple rupture scenarios, have rates of occurrence in the range from 10^{-2} (1in 100) to 10^{-20} per year (**nshm.gns.cri.nz/RuptureMap**). Earthquake scenarios with rate of occurrence less than 10^{-5} per year are considered to have limited engineering significance for the design and evaluation of dams and appurtenant structures and are not required to be considered in the deterministic approach.

Recommendations for the selection and scaling of acceleration time histories are included in Whittaker et al. (2011) and Chopra (2020). Selecting and scaling time history records that are appropriate for the specific site and structure being analysed requires the expertise of an experienced specialist. Duration of ground motion shaking is also a critical parameter that needs to be considered in time history selection, particularly for embankments and other nonlinear analyses.

The current acceptable procedures to develop or modify time histories for the purpose of dam analysis with the goal of matching the selected target response spectrum include amplitude scaling and spectral matching. The amplitude scaling method starts by selecting multiple sets of candidate recorded ground motions. These are subsequently scaled in such a way that the average of their spectral response ordinates, or the envelope of spectral response ordinates, matches the target spectrum over the specified period range of interest. This method has the advantage that it preserves the variability of response between various ground motions. The spectral matching method starts with the selection of a suitable set of candidate time history recordings. The method then proceeds to adjust each component iteratively, using either a time-domain or a frequency-domain procedure. Because the spectral accelerations fit the target response spectrum more closely, the median dam response is more efficiently determined with fewer time histories. The downside of this approach is that the inherent variability between ground motions is reduced, and some natural characteristics of the ground motion may be lost. It is important that this method be applied by an appropriate Technical Specialist, particularly when being used for nonlinear analysis, to ensure that the duration of selected time histories is not significantly altered. Careful consideration must be given when evaluating the near-fault pulse characteristics of ground motions.

4.3.6 Non-linear time history analysis

In addition to the PIC for the dam, the type of dam, and its potential modes of failure, the proposed seismic analysis methodology can also influence the selection of appropriate ground motion parameters.

Analyses for dams can vary from simplified to more elaborate numerical modelling procedures, such as the finite element or finite difference methods. Peak ground motion parameters and response spectra will be sufficient if simplified evaluation procedures are adopted. Dynamic finite element response analyses may be performed using either response spectra or acceleration time histories where linear elastic behaviour is expected. Time histories are required when nonlinear behaviour is expected. Both horizontal and vertical components of ground motion need to be considered for concrete dams and for embankment dams with very steep slopes. Vertical accelerations can be equal to or greater than horizontal accelerations when close to the earthquake source, so specialist advice is recommended in determining these parameters. Damping rates for concrete dams are usually in the range of 3% to 10% but for embankment dams are usually in the range of 5% to 20%. The effect of damping can also be addressed by selecting appropriate constitutive material models for the analysis of an embankment dam.

ICOLD Bulletin 148 (ICOLD, 2016) provides more detail on issues that should be considered in the selection of seismic parameters for the analysis and design of embankment and concrete dams. Time history records can be either actual earthquakes at another location in a similar tectonic setting (subsequently scaled to the site peak or more commonly spectral accelerations) or generated synthetically using specialist software. The source earthquakes should be of similar magnitude and distance from the source to represent the energy anticipated for the SEE. There are many methods for selecting and scaling time histories and for matching the design spectrum. Advice from a Technical Specialist is recommended. The number of time histories appropriate for analysis depends on whether the time histories are scaled or spectrally matched, whether the analysis is linear or nonlinear, as well as the intended use of the time history analyses (e.g., to assess acceptability of response, the median response, or the variability of response). The minimum number of time histories that should be considered is three. In this case the performance should be assessed based on the maximum response of the three ground motions. Some guidance documents recommend adopting the use of between 5-to-11-time histories and assessing performance based on median response. FEMA P-2082 (FEMA, 2020) provides a commentary on how to select an appropriate number of ground motions to evaluate the probability of acceptable performance. When the primary goal of numerical modelling for the SEE is to assess the containment of a dam's contents, the Designer should evaluate response values exceeding the median. Understanding the range of possible outcomes and their implications for dam safety is essential, particularly in preventing any release of contents.

4.3.7 Aftershock considerations

Ground motions from the SEE may lead to cracking, increased seepage, and reduced strength in dams. Aftershocks should be anticipated following a major earthquake. For high value assets and for all High PIC dams, the site-specific seismic hazard assessment should include the estimation of aftershock parameters, enabling the assessment of dam stability following a main shock and aftershock sequence. There is no industry consensus on the method to assess aftershocks. For the purposes of dam safety assessments, at least one aftershock of one magnitude less than the SEE should be anticipated within one day of the SEE. Further aftershocks may be expected in the following days, weeks and months following the SEE. The characteristics of the aftershock earthquake sequence, including design magnitude, distance and tectonic type, depend on the site-specific SEE fault characteristic. In the case where the SEE is based on a probabilistic approach, these characteristics can be selected based on review of the results of disaggregation of the analysis.

Repeated aftershocks can result in cumulative damage and reductions in dam stability. The Owner and Designer should consider how the safety of the dam will be managed in the period through the aftershocks until repairs can be completed. For soils susceptible to liquefaction, behaviour and response can vary significantly depending on the rate of dissipation of earthquake generated excess pore pressures. The Designer will need to take this into account when assessing the response to aftershocks.

4.3.8 Fault displacements

There is no universally accepted definition of an active fault. The New Zealand Community Fault Model (GNS Science, 2022b) states that *a fault is classified as active if there is evidence for ground-surface displacement/ deformation in the past 125,000 years* (i.e., since the peak sea-level associated with the last interglacial period, marine isotope stage 5e). A different age criterion is used for the Taupo Rift, which is evolving so rapidly (especially narrowing its locus of activity) that faults there are classified as active if there is evidence for displacement/deformation in the past 25,000 years (i.e., the age of the widespread Kawakawa tephra, and associated sedimentary deposits, that provide a useful regional timeline in landscapes of the Taupo Rift.)

Displacements associated with an active fault located beneath a dam can result in damage to the dam and the development of potential seepage pathways. Fault displacement in the reservoir can result in the loss of freeboard and the generation of a displacement wave (i.e., reservoir tsunami). Fault geometry or orientation and sense of movement may also result in general landform deformation.

Active faults that can result in displacement beneath a dam can include primary active faults and secondary active faults. Primary active faults are faults that have seismogenic potential (i.e., they are sources of earthquakes). Secondary active faults are faults that move in sympathy or because of movement on a nearby primary active fault. Movements will be much less than on a primary active fault, but the displacement can still be sufficient to require consideration in the design of a dam.

For engineering design purposes, it is generally not considered necessary to design for fault displacement where the annual probability of fault displacement is below a certain threshold. For dam design in New Zealand, it is recommended that the threshold for design be based primarily on the PIC of the dam.

4.3.8.1 Criteria for evaluating design fault displacement

Available guidance generally calls for the design fault surface displacement for the SEE to be defined consistently with the design ground motions (e.g., Mejia et al., 2002; Mejia et al., 2005). Recommendations for evaluating the design fault displacements have been proposed by Mejia (2023). They are primarily deterministic, based on the MCE for the site, but include an upper limit of fault displacement based on AEP. The recommended criteria for evaluating design fault displacements are summarised below:

- **Low PIC dams:** Fault displacements associated with the MCE should be based on median (50th percentile) deterministic estimates, but the displacements need not exceed the value associated with a 1 in 1,000 AEP event determined from a probabilistic fault displacement hazard analysis.
- **Medium PIC dams:** Fault displacements associated with the MCE should be between the 50th and 84th percentile deterministic estimates, but the displacements need not exceed the value associated with a 1 in 2,500 AEP event determined from a probabilistic fault displacement hazard analysis. If the deterministic approach is used, the Designer needs to consider the PAR, Potential Loss of Life, and consequences of failure in determining the appropriate percentile deterministic estimate of fault displacement and the fault recurrence interval threshold level. For dams at the lower end of the Medium PIC classification, 50th percentile estimates associated with active faults should be considered. For dams at the upper end of the Medium PIC classification, 84th percentile estimates associated with active faults should be considered.
- **High PIC dams:** Fault displacements associated with the MCE should be based on 84th percentile estimates, but the displacements need not exceed the value associated with a 1 in 10,000 AEP event determined from a probabilistic fault displacement hazard analysis.

For Low and most Medium PIC dams it is generally unnecessary to conduct a probabilistic fault displacement hazard analysis. For design purposes, the deterministic estimates associated with the MCE can be adopted. Foundation fault displacement need not be considered for the OBE.

The return period of fault surface displacement is typically at least a few hundred years, except on some of the most active faults. Foundation fault rupture is rarely associated with the OBE, for which an AEP of 1 in 150 is usually adopted. However, for some projects, the return period selected for the OBE may be comparable to the recurrence interval of foundation fault surface rupture. In such cases, foundation fault rupture needs to be considered as part of the loading associated with the OBE. When OBE loading includes fault surface rupture, it may be necessary to accept damage during the OBE as part of the performance criteria for dam design. The New Zealand Community Fault Model (GNS Science, 2022b) also includes the linear map traces of the capable faults. Capable faults are defined as "*the faults that are considered potentially capable of producing large earthquakes (>Mw 6), even if they have not been proven to be active*". The criteria for inclusion of capable faults have not been formally standardised across the New Zealand region; thus, inclusion has been based on a case-by-case assessment. It is prudent to consider fault displacements associated with capable faults in the risk assessment for a High PIC dam.

A special case involves non-seismogenic faults that can induce foundation fault displacement in response to a large earthquake on a nearby fault that does not underlie the dam. Design criteria specific to secondary movement on the fault underlying the site should account for the potential of sympathetic fault rupture. The secondary fault displacement specified for design should have confidence or probability-of-occurrence levels comparable to those discussed above for primary fault displacement.

4.3.8.2 Methods for estimating fault displacement parameters

For Low and Medium PIC dams, the locations of active faults can generally be obtained from the New Zealand Community Fault Model (GNS Science, 2022b), the GNS Active Fault Database, published geological maps, and territorial authority hazard maps. These resources are continually updated, and it is essential for the Designer to verify the currency and accuracy of the information being used in the design process. Estimates of active primary fault displacement and earthquake magnitude for Low and Medium PIC dams can be obtained using empirical scaling relationships recently updated for New Zealand as part of the NSHM 2022 by Stirling et al. (2024). Estimates of secondary fault displacements for strike-slip faults can be obtained using the empirical relationships developed by Petersen et al. (2011).

For High PIC dams the characterisation of design fault displacement should be based on knowledge of the regional and site-specific geology and a careful study of any foundation faults. Site-specific studies should be undertaken to determine the potential for active faulting and to quantify fault hazards (i.e., direction, magnitude, and recurrence of future faulting). Estimates of primary and secondary fault displacements should be based on site-specific studies undertaken by professionals with appropriate skills (e.g., paleoseismology, seismotectonics, geodetics). If possible, a predictive relationship should be established between a secondary fault and its primary active fault. Features on a dam site formed during earlier tectonic periods may exhibit little or no evidence of sympathetic movement in response to displacements on nearby active faults.

4.3.8.3 Design considerations

Mejia (2023) provides recommendations for design considerations regarding fault displacements. Preferably, dams should not be located across or immediately adjacent to an active primary fault; however, in the New Zealand geological context, avoiding such faults can be challenging. In some cases, existing dams may be found to be located on either primary or secondary active faults after construction. The proximity of an active fault to a potential dam site does not necessarily warrant the abandonment of the site. In these cases, design features that maximise resilience should be incorporated to adequately withstand the anticipated fault movements, while also accounting for uncertainties in the fault displacement history.

If there is sufficient evidence that an active fault is located directly beneath a dam, the dam should be capable of safely accommodating the estimated potential fault displacement without an uncontrolled release of the impounded contents. For embankment dams, Mejia (2013) states that recent practice has been to provide filter zones with thicknesses of at least 1.5 times the expected filter shear offsets corresponding to the design fault displacements. It is advisable to apply a similar factor to estimates of fault displacement when designing other dam safety critical elements of a dam.

4.3.9 Liquefaction and lateral spreading

It has long been recognised that loose saturated sands, silty sands, and gravelly sands in a dam foundation, as well as inadequately compacted sands and silts in embankments or tailings dams, are susceptible to liquefaction. The liquefaction of such deposits can lead to loss of shear strength, potentially initiating dam failure. Loss of strength under earthquake loads can also occur in more cohesive soils, such as silty clays and clayey sands, which should also be evaluated.

Where such deposits are present in a dam foundation, or are proposed to be utilised for dam construction, their susceptibility to liquefaction should be assessed. Investigations of liquefaction potential usually comprise field investigations, including cone penetration tests and shear wave velocity measurement; laboratory testing, including particle size distributions and Atterberg Limits; and sometimes cyclic testing. Fell et al. (2015) includes a simplified method for assessing the liquefaction resistance of soil deposits. MBIE and NZGS Earthquake Geotechnical Engineering Practice Module 3 (MBIE and NZGS, 2021) provides guidance on identification, assessment and mitigation of liquefaction hazards. In addition, there are advanced numerical effective stress methods for analysing the dynamic response of soils with liquefaction potential; however, such analyses should only be undertaken by Technical Specialists. 'Clay-like' soils may significantly soften and fail under cyclic loading but do not exhibit typical liquefaction features. Appendix A of ICOLD Bulletin 194 (ICOLD, 2022a) and National Academics Sciences Engineering Medicine (NASEM, 2021) provide a comprehensive discussion on the shear strength and deformation behaviour of soils that may significantly soften during an earthquake. The state of art is evolving, and Designers should determine the relevance and currency of methods before applying them to specific situations.

Typically, methods for assessing liquefaction potential are intended for application to level ground. The presence of an embankment results in additional shear stresses in both the underlying ground and the embankment itself, which need to be considered when evaluating liquefaction potential. Guidance on how to account for these factors is provided by Idriss and Boulanger (2014). These effects can also be addressed through advanced numerical effective stress analyses.

Where liquefaction is possible, post-earthquake stability analyses should be completed, using liquefied or residual strengths of the liquefied materials, to review the stability of the dam following the earthquake.

The consequences of liquefaction can be significant and so a conservative design approach is necessary. The design objective should be to achieve a foundation or embankment with an extremely low potential for liquefaction, ensuring that earthquake induced displacements remain acceptably small. Displacements can be predicted through simplified analyses using empirical regression models, sliding block analyses using analytically-based regression models, and nonlinear deformation analyses. Useful discussion on these analytical approaches is provided by USSD (2022) and further discussed in section 6.5.4. If the predicted displacement is acceptably small, it is recommended to check that the post-earthquake Factor of Safety - using residual strengths of the liquefied materials - meets the criteria of greater than or equal to 1.2. Lower factors of safety may be acceptable depending on the confidence in the accuracy of the residual strength. Useful discussion is provided in ICOLD Bulletin 194 (ICOLD 2022a).

To guard against liquefaction or strength loss during or following earthquake shaking, it is good practice to:

- Either remove all loose foundation materials and replace them with highly compacted materials, or densify the loose materials by vibroflotation or other appropriate foundation improvement techniques.
- Thoroughly compact all zones of embankment dams.
- Avoid using fill materials that are prone to generating significant pore pressures during strong shaking, particularly in the upstream shoulders and cores of embankment dams, as well as below the phreatic surface in the downstream shoulders of embankment and tailings dams.

Fill reinforcement measures, such as geogrid, may be appropriate in certain circumstances; however, it is essential to carefully consider the potential for concentrated seepage and/or other potential failure modes.

Significant damage to Low PIC dams due to liquefaction may be deemed acceptable, provided that an uncontrolled release of impounded water does not pose a risk of life loss, and the cost of ground improvement measures is disproportionately high relative to the benefits gained from mitigation.

4.4 Volcanic hazards

Volcanic activity in New Zealand within the last 500 years has been restricted to the North Island. The most recent eruptions include Rangitoto in Auckland (1400); White Island (ongoing); Tarawera south-east of Rotorua (1886); Taranaki (1655); and Ruapehu (1995/96), Ngauruhoe (1974/75), and Tongariro (2012) in the Central Volcanic Plateau.

Those volcanic hazards most relevant to the engineering and operation of dams include pyroclastic flows, lava flows, lahars, lateral blasts and ash falls. Dams situated in close proximity to the above volcanoes could be directly affected by pyroclastic flows, lava flows, lahars and lateral blasts. Dams distant from the volcanoes could be affected by ash falls, lahars and floating pumice deposits. Ash falls are unlikely to directly affect the safety of a dam, although they could affect power supplies, communication systems and control equipment for the operation of gates and/or valves that fulfil dam safety functions. Pumice deposits could blanket lakes and block intake structures. Pyroclastic flows and lahars could result in extreme flood flows that exceed the discharge capacity of spillway facilities and overtop downstream dams. Similarly, extreme flood flows released from failures of upstream volcanic debris dams could release sufficient flood flows to overtop downstream dams.

Regional councils and unitary authorities have a statutory responsibility to identify and assess natural hazards in their regions. Information on natural hazards is collated on the Natural Hazards Portal (Natural Hazards Commission Toka Tū Ake, 2024) for all regions. Specific information on volcanic hazards is provided for the Auckland, Waikato, Bay of Plenty, and Taranaki regions. The available information on volcanic hazards should be considered in any proposal to develop a dam within close proximity of a known volcano. In some cases, it may be necessary to complete a more detailed study to better understand volcanic hazards and their possible effects on the safety of an existing or a proposed dam. Probabilistic volcanic hazard quantification is currently in its infancy so resilient design features should be considered, especially for vulnerable equipment that fulfils a dam safety function.

4.5 Reservoir hazards

There is often a tendency to focus primarily on the dam itself, with less attention given to the reservoir. For Low PIC dams, the assessment of reservoir effects on dam safety is generally based on judgement. However, for Low PIC dams where reservoir effects could affect dam safety, as well as for all higher PIC dams, consideration should be given to the effects of landslides, reservoir induced seismicity, high winds and waves, and seiches generated by strong ground motions and/or fault displacement.

4.5.1 Landslides

The following should be considered:

- Whether there is any part of the reservoir perimeter (e.g., a narrow ridge) which may be more likely to fail than the closure dam.
- Whether there is any potential for landslide generated waves (seiches) to affect communities adjacent to the reservoir.
- Whether any existing landslides may reactivate or new landslides may develop under any of the possible reservoir conditions, to the extent that the dam could be overtopped and/or the reservoir or upstream tributaries blocked.
- Whether reservoir operation could result in toe erosion adjacent to dormant or potential landslide areas.
- Whether any of the reservoir surrounds in the proximity of the spillway and/or low-level outlet facilities may fail and block the facilities or impair their functions.
- What management regime should be implemented to prevent sediment or debris from affecting the performance of spillway and/or low-level outlet facilities.
- What operational requirements should be implemented to ensure the stability of dormant and potential landslide areas are not adversely affected by reservoir drawdown.
- What management regime should be implemented to monitor the performance of known landslides and any completed remedial works during dam commissioning and operation.

ICOLD Bulletin 124 (ICOLD, 2002a) provides guidelines for the investigation and management of reservoir landslides, comments on possible risk mitigation measures, and discusses requirements and methods for the ongoing monitoring of reservoir landslide performance.

4.5.2 Reservoir Triggered Seismicity (RTS)

Reservoir Triggered Seismicity (RTS) is an increase in seismic activity following the formation of a reservoir.

RTS is relatively uncommon but can occur (e.g., Benmore dam in New Zealand, 1965, and Oroville dam in California, 1975). Where RTS has occurred, the earthquake ground motions have been typically less than the SEE for the dam.

ICOLD Bulletin 137 (ICOLD, 2011a) notes that dams and appurtenant structures that have been correctly designed for seismic loads are protected against RTS; however, existing structures and facilities in the vicinity of a proposed reservoir could be susceptible to RTS as the resulting seismic loads could be larger than those assumed in their design.

4.5.3 Reservoir seiches

Reservoir seiches, generated by landslides into the reservoir, strong ground motions and/or fault displacement, have the potential to overtop dams and affect dam safety. While concrete dams can usually accommodate some overtopping without serious damage, embankment dams have a limited ability to withstand overtopping and large reservoir seiches could result in sufficient overtopping to initiate a dam failure.

Significant reservoir seiches can occur if the natural frequency of the reservoir is at or close to resonance with the dominant frequency of the earthquake waves affecting the site. In most cases normal freeboard provisions should be sufficient to safely accommodate seiches generated by strong ground motions; however, where a Medium or High PIC dam is located close to an active fault, the potential for seiche waves to overtop the dam crest and initiate a dam failure should be assessed.

Reservoir seiches can also be generated by fault displacement beneath a reservoir (e.g., Webby et al., 2007) and, if a sufficient volume of the reservoir is uplifted by the fault displacement, they can result in sufficient overtopping to initiate a dam failure. Where an active fault crosses the floor of a reservoir impounded by a Medium or High PIC dam, the potential for reservoir seiches to initiate a dam failure should be assessed. Assessments should consider the potential changes in the reservoir volume capacity resulting from the fault displacements. In some cases, if the estimated fault displacement is significant, it may be necessary to develop a hydrodynamic model for the reservoir to analyse the seiche effects and provide wave characteristics for evaluating wave run-up and the potential effects of any overtopping flows.

4.5.4 Freeboard

4.5.4.1 Overview

Freeboard is specified to prevent dam failure due to high reservoir levels relative to the dam crest. For the purposes of this section, ‘high reservoir levels’ refers to both the still water level plus dynamic water surface effects like wind set up, wave run up, and seiches.

(a) The following factors may be relevant in a freeboard assessment:
(b) Reservoir still water level across design loading conditions, including normal operation and floods.
(c) Wind set up and wave run up (refer methods for calculation later in this section).
(d) Malfunction of spillways and outlets, including gates and/or valves.
(e) Operational errors.
(f) Debris blockage of spillways and/or outlets.
(g) Seiches caused by landslides into the reservoir, earthquake-shaking, and/or fault displacements.
(h) Future changes in flood estimates i.e., due to changes in the catchment, climate change, or advances in methodology.
(i) Future changes in upstream or downstream consequences i.e., for controlled spillways and outlets, downstream development could impose constraints on releases, and upstream development could impose constraints on flood rise.
(j) Additional static or seismic settlement, deformation, and/or slope movements beyond what has been allowed for separately, say as camber.
(k) Security from human damage to the dam crest.
(l) Ability of the dam to tolerate water levels close to or above the dam crest, including the ability to safely withstand a degree of overtopping where relevant. This ability differs with the dam type and specific dam.
(m) Uncertainty in any of the factors above, including uncertainty in flood estimates and routing assumptions.
(n) Potential Impact Classification.

A range of scenarios should be developed based on reasonably probable combinations of the factors above. The minimum freeboard should be selected so that high reservoir levels do not threaten the safety of the dam for the critical scenario. High reservoir levels should also avoid compromising safe access for critical operational activities.

The appropriate scenarios will vary based on site-specific considerations. The intent is not that every conceivable adverse factor is combined in a scenario because some factors may be highly improbable. Consideration should be given to factors that are reasonably likely to occur concurrently. For example, a landslide into the reservoir may be more likely to be triggered by heavy rainfall and elevated reservoir levels, potentially coinciding with a flood event.

The level of assessment of each of the factors ((a) to (m)) above can range from cursory judgement through to detailed analysis, as per robustness and sensitivity studies described in USBR (2022) DS14 section 2.4.2.3, or the risk-informed approaches described in USBR and USACE (2019) Section D-3, ANCOLD (2000), and Module 7 of these Guidelines. Regardless of the level of detail, the rationale should be documented for how each factor has been considered, how scenarios have been identified, and how the critical scenario has been determined.

4.5.4.2 Wind and waves

As per factor (b) above, the effects of wind set up of the reservoir adjacent to the upstream face of a dam, and wave run up on the upstream face of a dam, should be considered in setting the freeboard requirements for a dam. Wind speed, wind direction, and fetch length are the predominant factors in establishing wind set up. Wave height, wavelength, and the physical characteristics (slope, roughness) of the upstream slope of the dam are the predominant factors in establishing wave run up.

Methods for the determination of wind set up and wave run up are included in Fell et al. (2015), USACE (2003), and EurOtop (2016). Regional wind speeds can be found in Standards New Zealand (2021). It is important to note that information provided in terms of 0.2s wind gust speeds must be transformed to sustained wind speeds for assessment purposes.

4.5.4.3 Dam type

Refer also to guidance on freeboard for specific dam types later in Module 3 (sections 6.5 to 6.11).

4.6 Threats and other hazards

Threats that can affect dam safety can be grouped under internal threats (e.g., errors/omissions in design, construction defects, inappropriate operation and lack of maintenance) and external threats (e.g., vandalism and terrorism). Other concerns, such as the safety of the public and the health and safety of personnel during construction and operation, do not usually affect dam safety.

Threats associated with errors/omissions in design, construction defects, inappropriate operation, and the lack of maintenance can be minimised by identifying the threats, adopting appropriate quality assurance systems during design and construction, and preparing and implementing proper procedures for operation, maintenance and testing of the facilities. Quality assurance systems for design and construction are discussed in section 3 of this module and section 2.8 of Module 4. The preparation and implementation of proper procedures for operation, maintenance and testing are discussed in Module 5.

With recent trends in the adoption of automatic and remote systems for the normal operation of dam facilities, and the associated absence of on-site personnel, it is important that appropriate systems are put in place to minimise the potential for unauthorised operation of facilities that are critical to dam safety.

This may never be an issue for a Low PIC dam located on private property and operated by the dam Owner. However, Medium and High PIC dams that are located in areas accessible to the public are prone to vandalism. Appropriate security measures should be installed to minimise the potential for vandalism and the unauthorised operation of equipment that fulfils dam safety functions. Appropriate security measures can include the installation of barrier fences and security cameras, the enclosure of control systems critical to dam safety in secure buildings, the use of authorised access cards, and the installation of intruder alarm systems.

Dams and their associated facilities should incorporate appropriate systems to protect people from hazards associated with their operation. In some cases, few protection systems may be necessary (e.g., a Low PIC dam located on private property and operated by the dam Owner) while in other cases, where a dam and its associated facilities are accessible by the public, multiple protection systems may be necessary for public safety. Public safety is discussed in more detail in Module 7.

Typical areas that warrant attention include:

- Reservoir areas immediately upstream of intake facilities (e.g., powerhouse, spillway and penstock intake structures).
- Gate and stoplog shafts.
- Gate or valve operation areas.
- Spillway channels and discharge areas.
- Steep and slippery canal side slopes.
- Tailings discharge facilities.

Proper protection can only be provided by identifying potential hazards, evaluating the risks, mitigating or controlling hazards through the installation of appropriate protection systems (e.g., warning signs, lake booms, fences, handrails, sirens), and ensuring that the operating personnel are aware of the hazards and their responsibilities for the proper management of the hazards.

5. Investigations and data assembly

5.1 Introduction

All investigations and data assembly for the design must be conducted to a level which is commensurate with the complexity of the dam site, the contemplated dam design or rehabilitation works, and the commercial value of the dam. Areas normally requiring investigation or measurement relative to dam safety are:

- Topography.
- Flood hydrology.
- Regional and site-specific geology.
- Seismic hazards (ground motions and fault displacement).
- Foundation characteristics.
- Construction materials.

Section 4.2 of this module provides guidelines for the assessment of flood hazards and the selection of appropriate flood magnitudes for the design and analysis of flood management facilities for Low, Medium, and High PIC dams. Similarly, section 4.3 of this module provides guidance for the assessment of seismic hazards and the selection of appropriate seismic design parameters for the design and analysis of Low, Medium, and High PIC dams. The following subsections provide guidelines for planning and managing an investigation programme and outline additional investigation activities which should be completed to support the design or rehabilitation of Low, Medium, and High PIC dams.

Several of the references, particularly ANCOLD (2020), ICOLD Bulletin 129 (ICOLD, 2005b) and Fell et al. (2015), listed at the end of this module provide detailed guidance on investigation techniques. Many more detailed references can be found in the literature, including technical books, papers, and conference proceedings. As such, the following subsections include little comment on investigation techniques. Additionally, since the focus of these Guidelines is on dam safety, the subsections include little comment on the scope of any social, community, or environmental studies that may need to be incorporated into an investigation programme. Owners are responsible for ensuring that all obligations related to dam ownership and operation are fulfilled.

5.2 Planning and managing an investigation programme

Most investigation programmes are completed in a series of separate stages with the following objectives:

- A pre-feasibility investigation – to identify possible dam sites and dam types, or possible options for rehabilitation of the dam, and obtain sufficient information for the planning of a feasibility investigation.
- A feasibility investigation – to identify a preferred dam site and dam type, or a preferred option for rehabilitation of the dam, confirm the technical feasibility of the preferred solution, and estimate the cost of project development.
- A design investigation – to address any outstanding issues raised in the feasibility investigation and address any additional questions that are raised during the detailed design and construction of the dam or rehabilitation works.

Many historical dam failures can be attributed to a lack of understanding of how the dam site reacts to the construction of the dam and the formation of the reservoir. It is therefore most important that investigation programmes are carefully resourced, planned and managed to address all unknowns that could affect dam safety. As outlined in section 3.1, important requirements for all investigation programmes include the use of appropriately qualified and experienced personnel, as well as the allocation of sufficient funds and time to ensure successful completion. A team approach is essential, and all personnel (project manager, engineering geologists, engineers, and Technical Specialists) should be selected for their technical skills and willingness to work in a team environment. For success and the avoidance of rework it is also most important that the lead Designer, or a senior member of the design team, is responsible for planning and managing the investigation programme.

Investigation programmes should be completed in a progressive fashion to ensure all unknowns are properly identified and addressed. An appropriate process for the completion of an investigation programme is outlined in Figure 5.1. This process includes:

- Definition of the investigation objectives. Objectives will vary depending on the anticipated dam type, or the proposed rehabilitation, and the investigation stage.
- Collection and assessment of existing information and the identification of information gaps. For a new greenfield site there may be little existing information available, and many information gaps may be identified. Alternatively, for the investigation of a site previously considered for development or a proposed rehabilitation project, the existing information may be comprehensive, and few information gaps may be identified.
- Planning an investigation programme to address the identified information gaps. For a new dam, regional studies are often necessary to identify features within close proximity to the site that could affect the feasibility of its development (e.g., major fault systems and landslides), and site-specific studies are necessary to characterise the foundation and identify potential sources of construction materials. While an investigation programme for a major rehabilitation project could require the completion of regional studies, a programme for a limited rehabilitation project would normally be dominated by targeted site-specific studies to establish existing conditions.
- Implementing the investigation programme, reviewing the results as they become available and, if necessary, initiating additional investigation work. Although the scope of investigation work required to address uncertainties typically decreases over time, investigation activities often remain necessary during construction. In some cases, investigation activities may also be required following commissioning.

It is critical that all investigation results, interpretations, and conclusions during all stages of an investigation programme are thoroughly documented. Ideally, a report should be prepared at the completion of each stage of an investigation programme to record the objectives, the work completed, the interpretations of the work completed, and any uncertainties that should be addressed through the completion of further investigations.

Investigation methods such as test pits and drillholes are invasive by nature and have the potential to introduce dam safety defects if they are not properly completed or sealed/rehabilitated. For example, drilling in an embankment dam should only be carried out using dry drilling techniques, drilling in the cores of embankment dams should exclude the use of water and high-pressure air, and foundation drilling beneath an operational dam should include protection systems to avoid hole blowout if a high pressure zone is encountered in the foundation. The locations of all test pits and drillholes should be accurately recorded and shown on plans, and they should be sealed/ rehabilitated either as part of the investigation program or during construction.

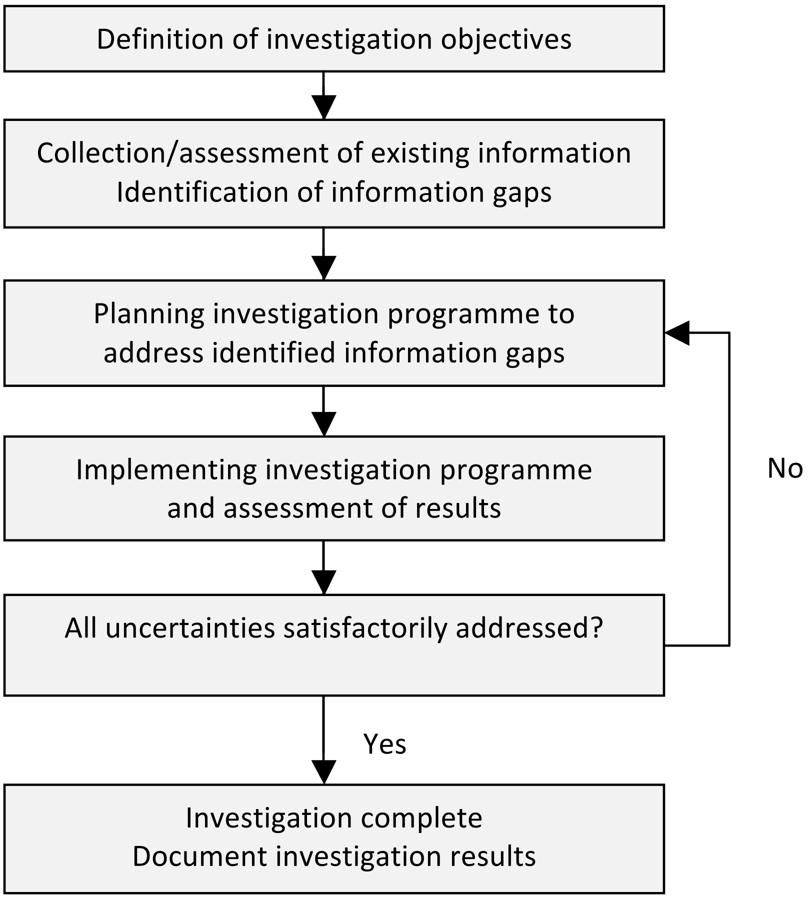

Figure 5.1: Progressive investigation programme

5.3 Topography

Topographical maps are an essential requirement for the investigation and design of any dam and usually include:

- Regional maps at a 1:50,000 scale and with a 20 m contour interval, which are available from Land Information New Zealand (LINZ).
- Site-specific maps to suit the site conditions which are typically at scales between 1:2,000 and 1:50 and with 2 m to 1 m contour intervals. Site-specific topographical mapping should be produced during the initial stages of an investigation programme using ground survey, photogrammetry, or remote sensing technology that measures distance by illuminating a target with a laser and analysing the reflected light (e.g., LiDAR). The quality of maps produced by photogrammetry and remote sensing techniques can be affected by vegetation coverage and, in some instances, manual ground surveys will be necessary to give the required resolution.

All site-specific mapping should be completed in relation to a regional coordinate system and datum, and all features recorded during an investigation (e.g., geological features, drillholes) should be located and levelled to the same coordinate system and datum.

5.4 Geology and foundations

Regional and site-specific geological studies should be completed for all dams, regardless of their PIC. The extent of the studies necessary will vary according to the quality of the existing information (e.g., regional geological maps, aerial photographs), the complexity of the site, composition of foundation, the type of dam, and the dam's PIC.

Dam design, and rehabilitation works when they are related to the foundation, are heavily influenced by the characteristics of the foundation materials. It is therefore important to understand the stratigraphy and the extent to which the materials are weathered or erodible, the strength and stiffness of the materials, the permeability of the materials and whether they incorporate potential leakage paths, and the joints and whether they are oriented in a manner that could contribute to foundation instability. The geomorphological context of the site should be understood to account for ongoing geological processes that may interact with the dam, its foundation, and/or reservoir margins across the lifecycle of the dam.

Recommended minimum initial investigation requirements for the design of Low, Medium, and High PIC dams are detailed below. Further guidance is available from the referenced documentation including Fell et al. (2015), ANCOLD (2020), and MBIE and NZGS (2021) guidelines for geotechnical investigations. In all cases, if the recommended minimum requirements identify any potential difficulties with the site (e.g., existing signs of slope instability, adversely orientated joints, weak or karstic foundation materials extending beyond the depths of the completed field work, the presence of volcanic ash or potentially liquefiable materials), additional investigation work should be completed.

- **Low PIC dams**: Investigation activities should include examination of published geological maps, canvassing of local knowledge, inspection of the dam site and reservoir area for signs of surface instability, faults, dormant or ancient landslides and other adverse geological features (particularly if they form the abutments), and the completion of appropriate and limited shallow investigations (such as hand auger holes and test pits/ trenches) over the dam footprint. The need for specific geotechnical site investigations and their extent should be confirmed by the Designer to suit the structures, expected geology, and geotechnical ground conditions at the site. Drilling may be considered appropriate for some sites. Where potentially liquefiable soils are present, specific site investigations (such as CPT or SPT) to inform liquefaction triggering analyses may be appropriate. Designers should determine whether testing of the foundation materials should also be completed to determine foundation characteristics and the extent of any necessary foundation excavation. Depending on the nature of the materials, these could include water contents, in situ strengths, Atterberg limits, dispersivity tests, gradation and permeability.
- **Medium PIC dams**: Geological and foundation investigation programmes should be sufficient to permit rational design of the dam and should include:
  - All of the items listed above for Low PIC dams.
  - Air photo interpretation and an appraisal of the regional geology.
  - Engineering geological mapping and interpretation of geological structures and defects in the dam site area.
  - Sufficient test pits and drillholes, with in- situ permeability testing, to characterise the foundation. Fell et al. (2015) and ANCOLD (2022) includes discussion on the applicability and limitations of various site investigation methods.
  - Additional in-situ and laboratory testing (e.g., shear strength, consolidation) to determine the characteristics of the foundation materials, as appropriate to the site conditions.
- **High PIC dams**: Investigation activities should be similar to those outlined above for Medium PIC dams but should be more comprehensive and should be focussed on key issues identified by Technical Specialists. Additional investigation activities often include:
  - Excavation and logging of shafts and drives.
  - Drilling and monitoring of groundwater observation wells.
  - Seismic testing to confirm design VS30 values.
  - Downhole electronic logging and core orientation.
  - Geophysical logging of the subsurface foundation materials.
  - Large scale in-situ tests (e.g., plate bearing tests, shear tests).
  - Reactivity of concrete aggregates.
  - Reservoir slope stability investigations (e.g., drilling, groundwater observation, deformation surveys).

5.5 Construction materials

The identification and investigation of potential sources of construction materials are key components of any investigation programme for a new dam and can also be important for the rehabilitation of an existing dam. Haul distances between the borrow areas and dam site, the characteristics of the available materials (e.g., suitability for concrete or embankment construction, material quality, variability in the borrow area), and the scope of the completed investigation programme can all significantly affect the dam type and the final cost of a dam or rehabilitated dam.

The scope of borrow material investigation will vary with the dam type, the characteristics of the rehabilitation project and the stage of the investigation. In comparison to a concrete dam and concrete appurtenant structures, which require durable, fine and coarse, non-reactive concrete aggregates, an embankment dam requires a wide range of materials including earthfill (core and shoulder materials), rockfill, transition, filter, and drainage materials. The Designer should therefore be looking for materials that:

- Have sufficient strength when placed in the dam.
- Do not deteriorate during placement, unless this is a desired characteristic that can be achieved with an appropriate level of quality assurance.
- Do not have high rates of weathering where weathering could compromise the design performance or function.
- Do not have expansive properties (e.g., alkali-silica reactive properties in concrete aggregates).
- Do not have dispersive characteristics unsuitable for low-permeability zones (e.g., cores of embankment dams, or liners of canals).
- Have fines contents appropriate for their purpose (e.g., low permeability core, filter function, drainage function).
- Have appropriate plasticity for use in specific zones of embankment dams.
  - Low plasticity materials can be used in dam cores but materials with some plasticity should be used if they are available.
  - Filter and drainage materials should comprise non-plastic granular soils that cannot hold an open crack.
- Are not gap-graded.
- Are not prone to segregation.

A pre-feasibility investigation should include the completion of sufficient geological mapping to identify potential borrow material sources and enable the scoping of a later feasibility investigation. In comparison, a feasibility investigation for any dam project, including a dam rehabilitation project, should include sufficient work to:

- Identify preferred borrow areas.
- Prove that sufficient volumes of the material are available from the preferred borrow areas.
- Establish that the preferred materials are suitable for their intended design use.
- Determine the likely processing requirements and construction methods necessary for the construction of the dam or rehabilitation project.
- Select the appropriate dam type(s), or rehabilitation works, with respect to the foundation and available construction materials.
- Provide assurance that the materials will meet the design specification.

To satisfactorily address the above questions, a feasibility investigation normally includes:

- An exploration programme (test pits, shafts and/ or boreholes) to log the available borrow materials, recover samples for laboratory testing, and enable the estimation of borrow area volumes.
- A laboratory testing programme to establish the characteristics of the materials and the suitability of the materials for their intended use. Laboratory testing requirements will vary according to the dam type and borrow material, but would typically include:

  - Gradation, water content, Atterberg limit, compaction, permeability and strength tests for fine grained embankment materials. Dispersion tests and shrink/swell tests may also be necessary in some instances.
  - Gradation, permeability, soundness, cohesion and durability tests for transition, filter, and drainage materials (FEMA, 2011).
  - Collapsibility tests to evaluate the cracking and self-healing potential of compacted filter materials (FEMA, 2011; Soroush et al. 2012).
  - Gradation tests for rip-rap and rockfill materials.
  - Gradation, specific gravity and absorption, abrasion, soundness, durability and alkali-aggregate tests for fine and coarse concrete aggregates.
  - Pumice content for volcanic soils.
  - Petrographic analysis to assess the suitability of rip- rap, rockfill, transition, filter and drainage materials, and concrete aggregates.
  - Where geosynthetic materials are incorporated in an embankment design, it may be appropriate to complete a series of laboratory tests on the proposed geosynthetic material (e.g., shear strength, peel strength, permeability, filtration compatibility, hydraulic transmissivity).
- Construction trials to demonstrate the applicability of materials or construction methods, for example:
  - Concrete mix design.
  - Embankment trials to determine appropriate fill placement, conditioning, and compaction methods; to determine the properties of the placed fill; and to confirm quality control methods.
  - Grout mix design tests.
- An assessment of the investigation results to confirm the suitability of the materials for dam construction, establish likely processing requirements, and estimate the costs of embankment or concrete placement.

6. Design considerations

6.1 Introduction

Performance criteria to demonstrate the required levels of dam safety are typically established through a standards-based approach.

While the standards-based approach does not directly account for uncertainties in loads and the ability of a dam to resist the loads or function as a system, it acknowledges uncertainties through the use of factors of safety and the completion of parametric sensitivity studies. This approach has been very successful and is widely accepted by the dam engineering profession worldwide. A standards-based approach is generally adopted in these Guidelines for the setting of dam design criteria and the evaluation of dam performance.

While the standards-based approach is adopted by these Guidelines, a risk-informed approach may be helpful in some instances to validate the design and provide an enhanced understanding of residual risks where appropriate data is available. In addition, a risk-informed approach may be helpful to compare the relative merits of alternative design solutions for rehabilitation projects.

A risk-informed approach can be used to understand the contributors to dam failure risks, the relative levels of risk from each of the various elements of the dam and how these risks might be mitigated.

It should also be noted that New Zealand does not have 'tolerable risk' criteria that have been accepted by New Zealand government, regulators, industry, or the public. However, the Building (Dam Safety) Regulations 2022 and Building Act 2004 provide definitions and criteria for flood prone, earthquake prone, and dangerous dams which provides some guidance on intolerable risk for dams.

An outline of the two approaches is as follows:

- Established design practice is based on the standards- based approach. It utilises design criteria largely based on deterministic concepts of reliability, typically because it is relatively straightforward and uses numerical measures of performance such as safety factors. The actual probability of failure cannot be explicitly evaluated using a deterministic approach, and the risks are managed implicitly through the adoption of a PIC for the dam, the selection of an appropriate Inflow Design Flood (IDF) and Safety Evaluation Earthquake (SEE) for the PIC of the dam, and the application of appropriate factors of safety or performance parameters (e.g., deformation) during the design process. It is common practice to select an IDF and SEE with lower annual exceedance probabilities for higher PIC dams to reduce the levels of risk where the consequences of failure are high.
- A risk-informed approach uses estimates of risk (calculated as probability multiplied by consequence) from potential dam failure scenarios or adverse events as indicators of achieved dam safety levels. These estimates can be compared with specific dam safety goals also expressed in probabilistic terms. Risk assessment can be a complex process, often following a Potential Failure Modes Assessment (PFMA), applying Failure Modes and Effects Analysis (FMEA) methodologies (refer to section 6.2.2) and requiring the guidance and input of numbers of Technical Specialists. In most instances concerning dams in New Zealand, risk assessment can only be done qualitatively or semi-quantitatively, as for many aspects of dams there is insufficient information to allow risks to be robustly assessed quantitatively. However, a risk-informed approach can enhance the understanding of potential failure modes and adverse consequences, highlight the greatest contributors to risk, and provide insights into possible means to reduce risk and add resilience, both in initial design, and the safety evaluation and management of existing dams.

PFMA and risk assessment have proven to be very valuable to many organisations in determining the necessity for, and most effective means of, rehabilitating existing dams.

An introduction to risk assessment for dam safety management and an outline of the risk assessment process is included in ICOLD Bulletin 130 (ICOLD, 2005a). The Bulletin highlights that "the profession has yet to come to an accepted position on the role and usefulness of risk assessment as an aid to dam safety management".

All designs should conform to established engineering principles for the safety of engineered systems and be assessed for the recommended dam safety performance criteria included in these Guidelines. When the recommended design criteria in these Guidelines are not met, a potential dam safety deficiency may be present, which requires evaluation and possibly rehabilitation to restore an appropriate level of dam safety.

Risk assessment may be an appropriate and acceptable means to identify and communicate the dam condition, effective risk reduction measures, and an appropriate time frame for the implementation of the risk reduction measures.

Note that the consideration of the safety of a dam following a damaging load event such as an earthquake or extreme flood will likely require a detailed review of its design.

In the design of a new dam or the safety evaluation of an existing dam it is recommended that Owners and Designers consider safety evaluation load scenarios. This involves consideration of the circumstances likely to follow a load event that will damage the structure(s), particularly for dams in cascade systems. For example, following a damaging earthquake, in addition to aftershocks, it is likely that the spillway may have to be used to draw the reservoir down, and/or to continue to pass inflows for months or years after the initial event until the dam is made operable or repaired. The safety evaluation of the spillway and its Dam Safety Critical Systems (DSCS) should consider this demand.

Following an extreme flood when damage is possible to the spillway and/or stilling basin, the reservoir is likely to be full, and so the spillway will see further spilling in the weeks and months following the initial event.

Post-earthquake criteria provided in these Guidelines are only for the temporary condition, immediately post-earthquake, until satisfactory repairs have been completed and acceptable normal operating dam safety criteria have been restored. Any necessary interim risk reduction measures and the time to implement effective repairs should be agreed between the dam Owner, the regional authority, and the affected stakeholders.

Much of dam design relates to achieving appropriate physical arrangements for the various components and careful detailing to account for the resulting hydraulic and seepage forces. Such details are included in recognised texts, technical papers, and ICOLD bulletins and are beyond the scope of these Guidelines. However, all designs should give due attention to important dam safety considerations including the following:

- Wherever it is practical and economic, secondary lines of defence should be incorporated within design arrangements.
- Possible changes in material characteristics or the inadequate performance of critical design elements within the expected life of the dam (e.g., physical degradation of materials, drain blockages).
- Shapes and dimensions to avoid excessive stresses and provide structural resilience to unexpected events.
- Ready access for future maintenance or repair.
- Health and safety during construction and operation.

While not directly related to the safety of a dam, all designs should also give due attention to control measures that minimise the risks to health and safety throughout the life of the structure. As detailed in section 1.2, Safety in Design is a requirement of the HSWA. Design solutions should provide safe access for operational personnel and protect operational personnel and the public from hazards associated with the operation of the dam and its associated hydraulic structures.

The following three sections discuss topics related to design methods, temporary works, and foundations and abutments that can affect the safety of any dam. The subsequent sections outline potential failure modes, loads and loading conditions, and recommended performance criteria for embankment dams, concrete dams, tailings dams, and appurtenant structures. The guidelines presented are applicable to the design of new dams, the evaluation of existing dams, and the design of rehabilitation works for existing dams. Important design issues that can affect dam safety are included for each dam type.

6.2 Design methods

6.2.1 Analysis techniques

A detailed discussion on design methods is beyond the scope of these Guidelines. Designers are referred to ICOLD bulletins and other references listed at the end of this Module.

The available literature addresses design methods that range from simplified analysis techniques, such as a rigid body stability analysis, to more sophisticated finite element analysis techniques. The selection of the appropriate analysis method for the design of a dam, the analysis of an existing dam, or the rehabilitation of an existing dam should take into consideration the dam type and the PIC of the dam, and the ability of the analysis method to evaluate the safety of the dam against its potential failure modes.

In many cases, a simplified approach is appropriate for the initial development of a design concept. Later in the design process, more sophisticated techniques are necessary to obtain an improved understanding of dam behaviour. Ultimately, the Designer must be able to demonstrate that the dam will meet appropriate performance criteria and that the requirements of the Building Code (structure, durability, and other criteria as applicable) will be satisfied.

General guidelines for the selection of design methods for embankment design, according to the PIC of the dam, are as follows:

- **Low PIC dams:** Precedent or empirically based design methods may be acceptable where the dam is less than 10 m in height and the dam type and foundation incorporate no unusual characteristics. Alternatively, rational design methods that consider material properties and currently accepted safety factors within the dam engineering profession should be adopted.
  - For Low PIC dams where empirical or semi-empirical methods are utilised for the design of the dam, the design should be suitably conservative in recognition of the uncertainties inherent in empirical design. In addition, for a Low PIC dam where full time on-site supervision is not in place during construction, the design should be suitably conservative in areas where the potential for poor construction and associated adverse effects are the greatest (e.g., selection, placement, and compaction of embankment materials adjacent to concrete structures).
- **Medium PIC dams:** Precedent or empirically based design methods may be acceptable where the dam is less than 10 m in height and the dam type and foundation incorporate no unusual characteristics. In addition, if precedent or empirically based design methods are adopted, the proportions and details for the dam should be conservative. Otherwise, rational design methods based on material properties and currently accepted factors of safety in the dam engineering profession should be adopted.
- **High PIC dams:** Rational design methods based on material properties and currently accepted factors of safety in the dam engineering profession should be adopted for all dams. Design methods should be comprehensive and reflect nationally and internationally accepted practice. The Designer should develop the design progressing from simplified analysis techniques to more sophisticated analysis techniques, using the preceding steps as validation of each following stage. An independent check of analyses may be required to validate the design.

For the design of both Low and Medium PIC dams, the Owners and Designers should remain cognisant of the fact that over the lifetime of the dam, downstream development may mean that the failure consequences of the dam change, and changing understanding of loads may mean that the safety evaluation loads and scenarios change. Consequently, the design of a dam should not hinder future rehabilitation or improvements of the dam to meet updated safety evaluation criteria, nor should it preclude management measures aimed at mitigating potential failure consequences.

6.2.2 Potential failure modes

The identification and assessment of potential failure modes for a dam (new or existing) is an essential component of dam design. The assessment of potential failure modes should be included in the design phase to ensure appropriate defensive measures are incorporated in design to prevent initiation and development of failure modes, and to inform the appropriate instrumentation, surveillance, and monitoring requirements and procedures for the dam.

Identification of potential failure modes can be achieved through the completion of a Failure Modes and Effects Analysis (FMEA). FMEAs originated in the petrochemical and power industries and are commonly utilised in the dam industry, nationally and internationally, to identify inherent dam-specific and site-specific credible potential failure modes (and therefore key vulnerabilities) for a dam. By leveraging the findings and understandings developed from the completion of FMEAs, dam designs can be enhanced to minimise the potential for development of identified potential failure modes, thereby reducing risk and increasing resilience.

Potential failure modes for a dam can often be difficult to identify and evaluate. In particular, subtle geological features that can have an important influence on the safety of a dam can be difficult to identify (e.g., isolated lenses of openwork gravels beneath the core of a dam, or potential shear planes in the foundation of aging tailings dams). However, historical dam failures (ICOLD, 1973; USSD, 1994; ASDSO, 2024) and analyses of historical embankment dam failures by internal erosion and piping (e.g., Fell et al., 2015) do provide useful information.

Module 5 provides guidance on the use of FMEA in the operations phase for existing Medium and High PIC dams.

6.3 Temporary works

Typically, the Contractor designs all temporary works necessary for the construction of the dam. This usually includes all formwork and falsework, worker safety protection, dewatering facilities, and can include cofferdams and diversion works.

Any engineering design work completed by the Contractor for the permanent works, or temporary works that could affect the quality of the permanent works or are incorporated within the permanent works (e.g., diversion facilities), must satisfy design criteria provided by the Designer and be approved by the Designer. Furthermore, the decommissioning of any temporary works that could affect the quality of the permanent works must also be approved by the Designer. Contractors often engage specialist design consultants to supplement their in-house skills and assist with such design tasks.

The design of diversion works and temporary works can influence the arrangement of the permanent works. It is critically important to clearly define where design responsibility lies for the various components of temporary works and permanent works.

The Designer should specify the parameters for diversion during construction (i.e., diversion facilities, their capacity, and their associated cofferdams), taking into account the Owner’s risk tolerance, the recommendations included in section 4.2.2, and public safety.

Diversion arrangements during construction should be carefully considered in relation to the potential for floods to outflank the diversion facilities and the consequences that such an event could have on dam construction and people, property, and the environment downstream of the dam. As discussed in section 4.2.2, the potential for overtopping of the dam during construction may be high while the dam is low but the consequences may only be minor. Conversely, the consequences may be significant as the dam reaches full height, but the potential for dam overtopping may be low due to the short time exposure and the upstream storage available for routing of the flood event. The design for any cofferdams should reflect their PIC.

The Contractor should propose final diversion details for approval by the Designer, based on risk allocation set out in the contract documents. Further guidance on construction processes is provided in Module 4.

6.4 Foundations and abutments

6.4.1 Foundation defects

Foundation defects can affect the integrity and stability of any dam type. Untreated foundation defects have contributed to many dam failures around the world.

The foundation for any dam must fulfil the following five functions:

- It must provide stability.
- It must provide sufficient stiffness to ensure deformations are within acceptable limits.
- It must control and limit seepage flows and uplift/ piezometric pressures beneath the dam.
- It must prevent the transportation of dam materials through the foundation.
- It must not degrade over time.

If any one of the above functions is only marginally satisfied, the safety of the dam may be less than envisaged. Any concerns that arise in relation to the above functions should be addressed by appropriate foundation engineering.

At some dam sites, geological conditions are reasonably straightforward and all of the above functions are readily satisfied. At other dam sites, geological conditions are complex and many defects may not become apparent until foundation excavation gets underway. The challenge is to keep the uncertainties within acceptable limits; however, there are some geological environments that require more care during investigation, design, and construction. They include:

- Clean coarse sands, gravels, and cobbles (open work deposits, including paleochannels) which could provide a pathway for preferential seepage, foundation piping, or the piping of embankment materials into the foundation.
- Loose silt or sand deposits that are potentially liquefiable.
- Infilled joints that could be eroded out and provide the potential for high seepage flows or the erosion of embankment materials.
- Interbedded soil deposits (fine against coarse) that could provide the potential for particle migration and erosion in the foundation.
- Weak strata, interbeds, and seams with low strengths that could result in potential sliding failure surfaces within the foundation.
- Highly compressible and/or dispersive soils (including loess) which could result in collapse and differential settlements, and cracking or foundation erosion.
- Volcanic deposits whose engineering properties can vary enormously over short distances. Lava flows can be underlain by beds of breccia, scoria or sand with high permeabilities and low resistance to erosion. Sites where tuffs, lahar deposits and agglomerates are present often incorporate low density and low strength materials.
- Karst features, such as caves and sinkholes, can lead to significant seepage losses and the formation of additional sinkholes following impoundment, due to the erosion of infilling or overlying materials.
- Persistent sub-horizontal joint sets that control the shear strength at the dam/foundation interface or within the dam foundation.
- Faults and other major discontinuities may incorporate low strength materials and, if unfavourably orientated, can adversely impact dam stability.
- Active faults (primary and secondary) that can result in displacements beneath a dam and the initiation of internal erosion, increased uplift pressures and reductions in dam stability.
- Landslides or unstable rock abutments that may require substantial remedial works to protect the long-term integrity of the abutments.

6.4.2 Foundation treatments

Foundations for dams require some treatment to satisfy the requirements of stability, deformation and water-tightness. Generally, the scope of any foundation treatment depends on the type of dam, the PIC of the dam, and the characteristics of the foundation materials. For a Low PIC embankment dam on a soil foundation, it may be sufficient to remove only the overlying organic materials. In contrast, for a Medium or High PIC concrete gravity dam on a rock foundation, it may be necessary to remove all overlying materials to achieve suitable founding conditions. Additionally, further treatment may be required to address rock defects or to complete a programme of consolidation and curtain grouting to achieve suitable rock quality.

ICOLD Bulletin 129 (ICOLD, 2005b) provides a detailed account of foundation treatment methods which are grouped into excavation and surface treatment, treatment by sealing measures, treatment by drainage measures, and treatment by strengthening measures. In addition, ICOLD Bulletin 88 (ICOLD, 1993a) provides a detailed account of the investigation, design, and treatment of rock foundations.

Excavation and surface treatment involves the removal of all undesirable materials necessary to achieve a foundation that satisfies - or can be treated to achieve - the requirements of stability, deformation, and water tightness. This necessitates the following for specific dam types:

6.4.2.1 Zoned embankment dam

The removal of all erodible, weak, unstable, liquefiable, compressible, or loose materials, along with the treatment of any rock defects, is essential to achieve a uniformly varying foundation and abutment profile. This ensures a tight bond between the core material and its foundation, while also providing adequate defence against the development of preferential seepage erosion pathways that could transport embankment materials along the foundation and abutment contacts.

Foundation shaping to remove steps or prominent features that could result in areas of low stress and initiate settlement cracking in the core.

If it is uneconomic to remove liquefiable materials, they must be stabilised by special ground improvement works. Impervious foundation materials beneath the dam's drainage features which would prevent proper functioning of the feature must be removed. If necessary, graded filters should be installed to prevent the erosion of embankment materials into the foundation and foundation materials into the embankment.

6.4.2.2 Concrete faced rockfill dam

Seepage paths beneath the plinths (upstream toe slabs) are short, resulting in high hydraulic gradients. It is therefore critical that excavation and surface treatments minimise the potential for erosion or piping in the foundation beneath the plinth. Excavation methods should be selected to minimise the potential for foundation damage. Foundation clean up should be completed to a standard that ensures a well bonded contact between the concrete and foundation rock.

Except for a short distance downstream of the plinth, where soil and soft weathered rock should be removed if filter and transition materials are installed between the face slab and downstream rockfill, the foundation beneath the downstream rockfill typically requires only the removal of surface deposits to expose the points of hard in situ rock. If the foundations are weathered, then the section downstream of the concrete plinth could be founded on material which is more prone to erosion and piping. In this case, filters and seepage control measures will need to be more extensive. Gravel deposits are often left in place as they frequently have a higher modulus of compressibility than well compacted rockfill.

6.4.2.3 Concrete gravity dam

Apart from low head structures which may be built on suitable overburden materials but require special treatments for the control of seepage flows, all concrete gravity dam foundations should be cleaned down to reasonably uniform surfaces of competent rock.

Foundation defects such as weathered zones, fault zones, and weak seams should be excavated to appropriate depths and backfilled with concrete. In some cases where prominent defect zones containing erodible material are present, it may be necessary to excavate upstream and downstream cutoff shafts and backfill them with concrete. In other cases, where the global stability of concrete monoliths is adversely affected by unfavourably orientated weak foundation seams that form blocks or wedges (e.g., bedding surfaces, joints, fault and shear surfaces), it may be necessary to remove additional material or construct shear keys to achieve adequate margins of stability. Drainage is recognised as one of the most effective foundation treatment procedures available. Drainage of discontinuities associated with foundation blocks and wedges can reduce or remove both hydraulic driving pressures and uplift pressures on resisting planes, thereby achieving more suitable founding conditions.

6.4.2.4 Concrete arch dam

Excavation and surface treatment requirements for arch dams are more demanding than for concrete gravity dams. The requirements for a stable, symmetrical, and uniform foundation often necessitate deep excavations, particularly in the abutments, and extensive foundation treatment works.

Sealing measures are often specified at dam sites to reduce seepage flows through dam foundations (to reduce water loss), to prevent foundation erosion and, in conjunction with drainage systems, to reduce uplift pressures beneath dams. The most appropriate treatment method will depend on the nature of the foundation material. Treatment methods include:

- The excavation of a shallow cutoff trench to a lower impermeable layer, directly beneath the core of a zoned embankment dam, and backfilling the cutoff trench with core material.
- Curtain grouting or consolidation grouting to form a curtain or blanket of grouted rock.
- Grout curtains should be designed for the site-specific geological conditions. Only stable (low bleed) grouts should be used. Triple row grout curtains are preferred with holes oriented to intersect prominent joint sets (e.g., vertical holes are inappropriate when treating vertical defects). Designers should specify the spacing and orientation of grout holes, the grout curtain depth, the grout mix parameters, the target permeability measured in Lugeons, and the quality control requirements.
- Grouting techniques should be appropriate for the geological conditions and the specified grouting performance requirements.
- The excavation and construction of a deep cutoff wall to a lower impermeable layer directly beneath the dam or in the abutments. Available cutoff walls include diaphragm walls and slurry walls which are excavated and backfilled with various materials (e.g., plastic concrete, cement/bentonite slurry), walls constructed from contiguous and interlocking concrete-filled piles (secant pile walls), and relatively thin walls formed by a descending vibrating beam and then backfilled with mortar during its withdrawal. Cutoff walls can be constructed through overburden and through rock foundations.
- The in-place mixing of overburden materials with a cementitious binder.
- The construction of a blanket of impermeable material immediately upstream of the dam.

Drainage measures are provided for the control of seepage through and beneath dams, and the reduction of uplift pressures beneath dams. In rock foundations, these objectives are often achieved by the construction of an upstream grout curtain and the later drilling of a fan of drainage holes from one or more galleries or the foundation surfaces immediately downstream of the dam. In overburden materials, drainage objectives are usually achieved through the construction of drainage blankets, toe drains, and/or relief wells.

Strengthening measures can sometimes be necessary to improve the characteristics of a dam foundation. The strengthening of a rock foundation to provide a more homogeneous material and minimise deformations under dam loadings can be achieved through the grouting of discontinuities, and the excavation, concrete backfilling, and grouting of weak zones of foundation material. Where a dam is founded directly on overburden materials, the important structural design consideration is bearing capacity. For cohesive materials, the bearing capacity can be improved by drainage measures to enhance the consolidation process. For cohesionless materials, a number of ground improvement techniques are available that include static (pre-loading) or dynamic compaction from the ground surface, vibrocompaction, blasting through a grid of boreholes, compaction grouting using a low fluidity grout to displace the borehole walls into the overburden material, and deep mixing and jet grouting to provide a series of column inclusions within the overburden material. Methods such as stone columns that provide densification through the addition of materials (especially porous materials), or direct compaction (with drainage provisions) such as dynamic compaction with wick drains, have proven to be more effective than soil replacement methods.

Detailed descriptions of the above foundation treatment methods and their applications are included in ICOLD Bulletins 88 and 129 (ICOLD, 1993a; ICOLD 2005b). Bruce (2013) provides a compilation of current practice in the wide range of techniques for dam foundation engineering. Weaver and Bruce (2007) provide considerable detail on contemporary drilling and grouting for dam foundations.

The design of foundation engineering works must account for performance criteria, constructability, health and safety, and quality assurance needs to demonstrate that the feature will perform its intended function and provide an adequate service life.

6.5 Embankment dams

6.5.1 Introduction

Embankment dams are grouped according to the types of material used in their construction. They commonly include:

- Homogeneous earthfill dams that are constructed from a single material except for a pervious zone which is placed beneath the downstream shoulder or at the downstream toe.
- Homogeneous earthfill dams that incorporate additional features such as:
  - an upstream geomembrane liner as an impermeable barrier (refer section 6.7), or
  - a concrete core wall as an impermeable barrier,
  - filter and drainage materials (such as a chimney drain linked to a pervious downstream blanket at the foundation contact) to maintain the downstream shoulder in a dry condition and to control and discharge seepage flows.
- Zoned earthfill dams that normally incorporate a low permeability core material, higher permeability shoulder materials, and filter and drainage materials for the control and discharge of seepage flows.
- Zoned earth and rockfill dams that normally incorporate a low permeability core material, rockfill shoulder materials, and filter and drainage materials for the control and discharge of seepage flows.
- Concrete-faced rockfill dams with an upstream concrete facing and a rockfill or gravel embankment (refer section 6.6).
- Rockfill dams with a central impervious core of earth, asphalt or concrete.

The following subsections discuss potential failure modes for embankment dams, loading conditions which must be taken into account during their design, evaluation, and rehabilitation, and recommended performance criteria for embankment dams. Defensive design details that are important to dam safety are also discussed. Concrete- faced rockfill dams and geomembrane–lined embankment dams are discussed specifically in sections 6.6 and 6.7.

6.5.2 Potential failure modes

Section 6.2.2 provides additional information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams. Some guidance on potential failure modes for embankment dams is provided below.

It is not possible to provide a complete listing of potential failure modes for embankment dams, as each dam is unique, incorporating different materials, foundation conditions, design configurations, and specific design details. It is therefore most important that the identification of potential failure modes for a dam is based on site-specific conditions and the specific characteristics of the dam.

The examination of potential failure modes should be carried out for dam safety evaluations to assist in the determination of any necessary rehabilitation works. They are also a valuable tool for use during the design of new dams and the rehabilitation of existing dams to ensure that potential dam vulnerabilities are addressed, and risk reduction measures are incorporated as appropriate.

6.5.2.1 Deformation and internal erosion mechanisms

Embankment dams can be vulnerable to, and should therefore be designed against:

- Deformation and consequent loss of freeboard, and/ or increase in seepage, caused by processes such as:
  - Liquefaction or strain softening in the dam and foundations.
  - Slope instability due to insufficient shear strength in embankment materials.
- Internal deterioration through internal erosion processes (e.g., piping). Internal erosion can involve one or more of the following processes:
  - Concentrated leak erosion - refers to the erosion that occurs along a concentrated seepage path in soils capable of sustaining an open crack or gap, or within the interconnecting voids of a continuous permeable zone. Erosion occurs along the sides of the crack or gap, or within the voids, where the shear stress of the seepage flow exceeds the critical shear stress of the soil particles.
  - Backward erosion – The detachment of soil particles when seepage exits to a free unfiltered surface such as the ground surface downstream of a soil foundation, the downstream face of a homogeneous dam, or a coarse rockfill zone immediately downstream of a fine grained core material. Erosion starts at the exit point and a continuous passage is developed by backward erosion when the seepage gradient exceeds the critical hydraulic gradient of the soil (i.e., the point at which the seepage force overcomes the downward gravitational force on the soil particles, resulting in erosion of the particles).
  - Internal instability (e.g., suffusion) – Internal instability describes the inability of a soil to prevent the loss of its own small particles in the presence of seepage forces. This process involves selective erosion of finer particles from the matrix of coarser particles, in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton of coarse material. This process can occur via seepage through the main dam body and/or foundation of a dam which is not protected by an adequate filter. Internal instability is typically of most concern in granular materials, particularly gap-graded and widely-graded soils with low fines contents.
  - Contact erosion – Selective erosion of fine particles along a contact surface between a fine soil and a coarse soil, caused by flow passing through the coarse soil (e.g., flow occurring along a contact surface between silt and gravel sized materials). It relates only to conditions where the flow in the coarser layer is parallel to the interface between the coarse and fine layer.

In the backward erosion process, detached particles are carried away by the seepage flow and the process gradually works its way towards the upstream side of the embankment or its foundation until a continuous pipe is formed. There are generally two forms of backward erosion:

- Backward erosion piping, in which erosion begins at a free (or inadequately filtered) surface and progresses in a backward (upstream) direction. The roof or sides of the erosion ‘pipe’ are formed by a cohesive soil within the embankment or foundation.
- Global backward erosion (GBE), where an erosion ‘pipe’ forms due to progressive detachment of particles at the downstream extent of an unfiltered (or inadequately filtered) fill zone. Progression of erosion may be assisted by gravity, resulting in vertical or near-vertical erosion features. GBE can occur in widely-graded silt/sand/gravel soils and - unlike backward erosion piping - does not require roof or side support from cohesive soil. Preferential erosion of finer particles within a soil may result in a ‘washed out’ pipe-like erosion feature comprising a zone of coarser particles.

It is important to consider that internal erosion failure modes can involve multiple erosion mechanisms, e.g., internal instability (suffusion) within a fill or foundation soil could alter the material such that it results in contact erosion or filter incompatibility between material zones. Erosion mechanisms can vary spatially (within a dam or soil zone or unit) and temporally (with time) and some soils may exhibit ‘metastable’ behaviour, where particle erosion is intermittent (often triggered by changes in loading conditions).

6.5.2.2 Vulnerabilities in embankment dams

Flaws in an embankment dam that would be considered vulnerabilities in an FMEA include:

- Cracks caused by differential settlement or hydraulic fracture.
- Irregularities or steps in the abutment or foundation profile.
- Desiccation cracks near the crest due to drying, shrinkage, or freeze-thaw effects.
- Gaps or cracking adjacent to spillway walls or conduits.
- Poorly graded materials or segregation, which can give rise to coarse zones susceptible to high seepage flows and the migration of fines.
- Poorly compacted layers which can give rise to interconnected voids or a gap, with potential for wetting induced collapse.
- Poor or uneven compaction around instruments situated within the dam fill.
- Poor compaction at interfaces between separate zones of a dam.
- Horizontal low-permeability layers (construction horizons), particularly those resulting from wet or freezing conditions during construction.
- The lack of sealing or inadequate protection of joints in the core/foundation or core/abutment contact areas.
- The lack of appropriate filters.
- Inadequate drainage provisions.
- Relic defects in soil foundations.
- Infilled defects in rock foundations.
- Dam or foundation soils susceptible to liquefaction.
- High foundation permeabilities that enable the development of artesian pressures, raising the risk of potential 'blowouts' (e.g., fluidisation or heave) at the dam toe.

6.5.2.3 Cracking mechanisms

Potential cracks in an embankment dam that would be considered vulnerabilities are shown in Figure 6.1 and the influence of various factors on the likelihood of cracking occurring are listed in Table 6.1. Transverse cracks are especially hazardous to water- retaining embankments because they present an open pathway across the embankment that can potentially quickly erode and downcut, leading to a breach. ICOLD (2017) provides details of typical cracking scenarios for earth dams. Fong and Bennett (1995) report transverse cracks are more prone to occur near the abutments of embankment dams. Swaisgood (1998) reports that they particularly tend to occur where abutments are steeply sloping and stiffer than the embankment. Transverse cracking in embankment dams is also possible where differential settlements occur across steps in foundations or rigid structures.

Embankment cracking can occur due to differential settlement, deformation, arching, drying or desiccation, hydraulic fracture, or other stress concentrations within fill soils. Earthquake-induced cracking and deformations due to liquefaction, lateral spread, and volumetric strain are covered in more detail in section 6.5.4.2. Seismically induced cracking is often similar in pattern and location to cracking caused by differential settlement under static loading (Sherard, 1973, and Mejia, 2024). An understanding of cracking under static loads will therefore inform the nature of potential earthquake-induced cracking. In addition to factors outlined in Table 6.1, further guidance on the susceptibility of embankments to cracking is provided by Fell et al. (2015), ICOLD (2017, 2022b), Mejia and Dawson (2019), and Mejia (2024). There are no precise and accurate methods to determine likely crack depth and width for any specific dam or loading scenario; however, empirical screening methods such as those detailed in Fell et al. (2015) are based on a review of the literature on observed cracking and may provide initial guidance.

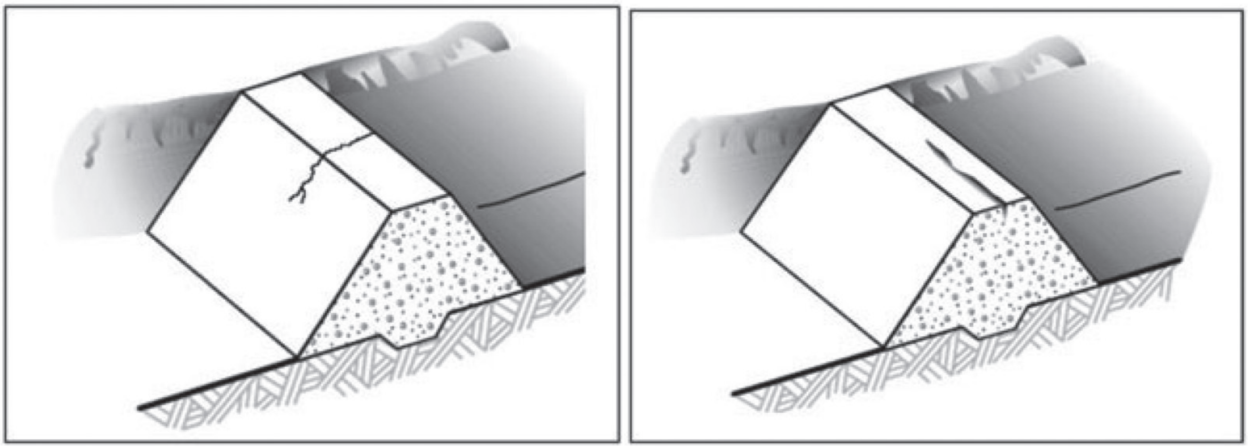

Figure 6.1: Transverse and longitudinal cracking (from FEMA, 2016).

Table 6.1: Influence of factors on the likelihood of cracking or hydraulic fracturing (after Foster and Fell, 2001).

Factor	Influence on likelihood of cracking or hydraulic fracturing		
	More likely	Neutral	Less likely
Overall abutment profile	Deep and narrow valley. Abrupt changes in abutment profile, continuous across core. Near vertical abutment slopes.	Reasonably uniform slopes and moderate steepness (e.g., 0.25H:1V to 0.5H:1V	Uniform abutment profile, or large scale slope modification. Flat abutment slopes (>0.5H:1V)
Small scale irregularities in abutment profile	Steps, benches, depressions in rock foundation, particularly if continuous across width of core (e.g., haul road, grouting platform 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 soils.	Soil foundation, gradual variation in depth.	Low compressible soil foundation. No soil in foundation.
Core characteristics	Narrow core, H/W>2, particularly core with vertical sides. Core material less stiff than shell material. Central core. Brittle core materials.	Average core width, 2<H/W<1 Core and shell materials equivalent stiffness.	Wide core, H/W<1 Core material stiffer than shell material. Upstream sloping core.
Closure section (during construction)	River diversion through closure section in dam, or new fill placed a long time after original construction.		No closure section (river diversion through outlet conduit or tunnel).

Cracking can also occur at interfaces between embankments and spillway walls, conduits, and other rigid structures that are located adjacent to, beneath, or passing through embankment dams. These cracks create an opening and/or loose materials resulting in a preferential seepage erosion pathway. Inappropriate details at embankment/structure interfaces, the lack of filter and drainage protection, and low stresses associated with arching of embankment fills across the tops of conduits or other material interfaces, can initiate cracking and the erosion of embankment materials. Many embankment dam failures have been influenced by inappropriate design details, and inadequate filter and drainage protection adjacent to conduits.

6.5.2.4 Common potential failure modes for embankment dams

As described above, the identification of potential failure modes for a dam should be based on site-specific conditions and the specific characteristics of the dam. However, as an aid, Table 6.2 outlines the mechanisms responsible for common potential failure modes associated with embankment dams and their foundations.

Table 6.2: Initiating mechanism for potential failure modes for embankment dams

Initiating mechanism	Common causes
Overtopping	Insufficient freeboard to accommodate storms and flood events
Internal erosion of embankment materials	Presence of defect or crack, cohesionless core material or core material with a Plasticity Index less than 7, dispersive soils, lack of adequate filter protection
Internal instability (e.g., suffusion) of embankment materials	Widely-graded and gap-graded granular (cohesionless) materials with low fines content and a Plasticity Index less than 7.
Internal erosion of embankment materials into foundation materials	Open joints at interfaces, lack of adequate filter protection, lack of or inappropriate foundation treatment
Internal erosion of foundation materials	Foundation material has a Plasticity Index less than 7, dispersive foundation materials, lack of or inappropriate foundation treatment
Instability of downstream shoulder	Weak foundation, weak shallow seam in foundation, poor conditioning and compaction, lack of effective drainage and saturation of downstream shoulder, insufficient shear strength, strong earthquake shaking
Instability of upstream shoulder	Weak foundation, poor conditioning and compaction, rapid drawdown of reservoir, insufficient shear strength, strong earthquake shaking
Loss of freeboard, overtopping and subsequent erosion	Insufficient freeboard to accommodate foundation and embankment settlement, settlement following seismic loading, liquefaction of embankment and/or foundation materials, seiches generated by earthquakes, uplift of the reservoir due to fault displacement, reservoir landslides
Erosion along embankment/structure interfaces	Inappropriate design details, lack of filter and drainage protection, poor compaction adjacent to structure

6.5.3 Loading conditions

Loading conditions for the design and rehabilitation of embankment dams are presented and discussed in various ANCOLD guidelines, Canadian Dam Association (CDA, 2013), and various USACE and USBR engineering manuals.

Loading conditions that should be considered in the design or rehabilitation of an embankment dam are:

- Normal loading conditions.
- Unusual loading conditions.
- Extreme loading conditions.

Normal loading conditions are those which the dam is expected to continuously withstand during normal operation. Examples include steady state seepage and embankment stability with normal maximum reservoir elevation, and embankment stability with no reservoir for a flood detention dam.

Unusual loading conditions occur on an infrequent basis. Examples include the end of construction condition where high pore water pressures can exist in core and foundation materials, severe wave action, rapid drawdown of the reservoir, and the OBE. Minor damage, such as crest settlement and minor shallow or surface cracking, is acceptable; however, the dam should continue to behave in a satisfactory and safe manner.

Extreme loads are those associated with low probability events which, if they were to occur, would be considered a severe test of a dam's performance and would require diligent visual inspection and observation, and a readiness to respond to a dam safety incident or emergency. Examples include floods at or above the IDF, earthquakes at or near the SEE, and the post-SEE loading condition. Significant damage to the structure is possible and major repairs may be required; however, the damage must not result in an uncontrolled loss of the reservoir.

6.5.4 Stability and deformation performance criteria

Potential stability failures for embankment dams under different loading conditions should be assessed in terms of minimum factors of safety.

6.5.4.1 Static assessment

For embankment dams, the dam, foundation, and abutments must be stable during construction and under all operating conditions, including full or partial drawdown. Recommended minimum factors of safety for limit equilibrium stability studies under static loading conditions are listed in Table 6.3. The recommended factors of safety reflect those adopted by the CDA. In addition, for static loading conditions, they are similar to those adopted by the US Bureau of Reclamation and the US Corps of Engineers, and those recommended by Fell et al. (2015).

Table 6.3: Recommended minimum Factors of Safety for slope stability – static assessment

Loading condition	Slope	Minimum Factor of Safety[1,2,4]
End of construction before reservoir filling	Upstream and downstream	1.3
Long-term (steady state seepage, normal reservoir level)	Downstream	1.5
Full or partial rapid drawdown	Upstream	1.2 to 1.3[3,5]

Notes:

1. The Factor of Safety is a representation of the factor required to reduce operational shear strength parameters, or increase the loading, in order to bring a potential sliding mass into a state of limit equilibrium, using generally accepted methods of analysis.
2. Higher factors of safety may be necessary if there are high levels of uncertainty in the inputs to the stability analysis.
3. Higher factors of safety may be required if drawdown occurs relatively frequently during normal operation.
4. The above factors of safety are appropriate for the design of new dams on high strength foundations with low permeability zones constructed of soil which is not strain weakening, using reasonable conservative shear strengths and pore pressures developed from extensive geotechnical investigations of borrow areas, laboratory testing and analysis of the results. Fell et al. (2015) provides guidance for adjusting the above minimum factors of safety for other conditions such as an existing dam, soil or weak rock foundation materials, strain weakening soils, and limited strength investigation and testing.
5. Detention dams that retain water intermittently should only be in this Factor of Safety range if:

- The failure surface is shallow and would not threaten the integrity of the detention dam when holding water, or
- The phreatic surface estimated during water retention conditions is supported by rigorous transient analysis based on laboratory or field permeability testing and allows for worst case soil moisture conditions (considering infiltration and evapotranspiration) in the embankment, or;
- The phreatic surface estimated during water retention conditions assumes worst case (i.e., lowest Factor of Safety) saturation conditions.

If these conditions are not met, then a minimum static Factor of Safety of 1.5 should be targeted when the detention dam is retaining water, even if a steady state is not reached

Static assessments should reflect an appropriate understanding of the dam structure. It is important to understand design details, as described in section 6.5.5, given that slope and global stability assessments will be influenced by embankment detailing. Specifically, static stability and deformation assessments should include relevant details of the embankment materials; crest; filter, transition, and seepage control zones; drainage provisions; conduits, penetrations, and inclusions; and interfaces.

6.5.4.2 Seismic assessment

A dam may be damaged during an earthquake, but it must be able to safely contain the reservoir contents in its post-earthquake condition. Earthquake damage could include crest settlement and lateral spreading, longitudinal or transverse cracking, separation or cracking at the boundary of embankment and concrete structures, and/or slope movements on the upstream or downstream face of a dam. Crest settlement must not result in the reservoir overtopping the crest of the dam, and slope movements must not result in the loss of freeboard or the loss of support to the core or upstream water retaining membrane. It is important to check if slope movements or dam deformations will impose loads on outlet structures, conduits or other appurtenant structures within or near the dam footprint. The need for adding filter and crack stopper zones should be considered to protect areas where cracking or separation may occur between the embankment and concrete structures. Cracking or separation should be limited to the depth above the full supply level. If this limit is exceeded, immediate intervention is necessary to protect against seepage erosion that could lead to breach of the dam.

Earthquake-induced deformations: A variety of methods are available to evaluate the earthquake-induced deformation of embankment dams, and they may be grouped into the following three types of analyses: (1) simplified analyses using empirically-based regression models, (2) sliding block analyses using analytically-based regression models, and (3) nonlinear deformation analyses. Further discussion is given below:

1. **Simplified analyses using empirically-based regression models**
   Empirical methods are generally based on historical data and can be used to estimate earthquake induced embankment deformation (usually crest settlements). Examples of empirically based regression models are Swaisgood (1998, 2003) and Pells and Fell (2002, 2003).
2. **Sliding block analyses using analytically-based regression models**
   Sliding block analyses assume that deformations of a dam result from sliding of a mass or block of the dam by yielding along the base of the block. Bray et al. (2018) and Bray and Macedo (2019) provide simplified methods for estimating shear-induced deformation that accounts for embankment properties, including shear strength and geometry, by taking yield acceleration into account. These are simplified methods that use analytically-based regression models. These procedures consider both the ground motions from shallow crustal and subduction earthquakes and thus should be given preference over generalised procedures.
   The Designer should determine the yield acceleration of a potential failure mass to assess the resultant permanent displacements by undertaking a limit-equilibrium analysis. Yield acceleration refers to the analytically determined acceleration applied to a potential failure mass, indicating an instantaneous Factor of Safety against instability of 1.0. If the spectral acceleration for the design loading condition - accounting for structural amplification - is greater than the calculated yield acceleration, it implies that some displacement will occur each time the yield acceleration is exceeded during the earthquake. The Designer should then assess the cumulative displacement and determine whether the dam will continue to retain its contents in its deformed state. Embankments with short natural periods in the order of 0.1 second (e.g., low height and stiff cross sections), are likely to experience near-resonant response and high spectral accelerations at the embankment crest. The sliding block methods using analytically-based regression models for evaluating embankment seismic response and permanent displacement are appropriate for most applications.
3. **Nonlinear deformation analyses**
   Linear and non-linear dynamic analysis methods are normally only utilised for High PIC dams where stability and deformation studies indicate marginal safety or material degradation, or where the dynamic response of the dam is not readily estimated. Guidance on these analyses is provided in USSD (2022).

Liquefaction

Soil liquefaction within dam or its foundation is a critical phenomenon that can significantly influence both the magnitude and pattern of earthquake-induced displacements. Thus, potential for liquefaction should be addressed regardless of the analysis approach. Simplified deformation analysis methods are generally unsuitable when liquefaction is anticipated. Therefore, it is essential to assess both the potential for liquefaction and its extent to validate the applicability of these methods. The impacts of material strength-loss on overall embankment stability are often assessed through limit-equilibrium analyses in which the yield acceleration is determined using a reduced strength. Nonlinear deformation analyses using soil constitutive models that directly simulate anticipated changes in soil behaviour are commonly used to evaluate the impacts of strength loss on embankment stability and deformation. These analyses can capture changes in material response and strength, as well as the impacts of those changes on embankment deformation.

In assessing the capability of an embankment dam or foundation to resist earthquake motions, the potential for liquefaction must be addressed. Where possible, liquefiable materials should be avoided or removed, or foundation improvement undertaken. If liquefaction is possible, the post-liquefaction static stability of the dam should be evaluated using the estimated residual strength of the liquefied soil. This evaluation should consider both scenarios: with and without remedial measures, to ensure dam failure does not occur.

Volumetric strains

Seismic deformations in embankment dams are associated with the plastic shear and volumetric strains induced in the dam and foundation materials during earthquake shaking, as well as the volumetric strains resulting from reconsolidation after earthquake shaking. During shaking, shear strains are the main component of seismic deformations under undrained conditions. Volumetric strains in saturated liquefiable materials (e.g., sand, gravel and rockfill embankment fills and foundations) can be significant due to drained or partially drained conditions during earthquake and post-earthquake reconsolidation. Volumetric strains are generally small in dense or cohesive soils, even if unsaturated, because these materials are not susceptible to compression (i.e., densification) under vibratory loading, and in fact may dilate under shear. Volumetric strains during shaking can be significant in unsaturated soils susceptible to densification under vibratory loading such as dry loose-to-medium-dense sands, gravels, and rockfill. Volumetric strains during shaking may also be significant in saturated rockfill as these materials can exhibit drained behaviour during the shaking because of their large permeability.

Given that volumetric strains can significantly contribute to the seismic deformations of embankment dams, they must be included in the evaluation of a dam's seismic performance. Simplified analysis using the Bray et al. (2018) and Bray and Macedo (2019) procedures accounts for shear deformations only, and evaluation of dam seismic deformations using these approaches must be supplemented with a separate assessment of potential volumetric compression. Depending on the soil constitutive models used for analysis, nonlinear deformation analyses can directly include shear as well as volumetric strains and deformations. The capabilities and limitations of any constitutive model should be thoroughly understood before the model is used to estimate volumetric deformations, as many models fail to address key aspects of seismically-induced volumetric strains.

Procedures commonly used to estimate volumetric strains of unsaturated sands (seismic compression) are generally based on studies of settlements under cyclic loading in the laboratory, such as Tokimatsu and Seed (1987), Duku et al. (2008), and Lasley et al. (2016). Post-liquefaction reconsolidation settlements of sands are often estimated using similar types of data (e.g., Ishihara and Yoshimine, 1992; Wu et al., 2003; Idriss and Boulanger, 2008). It is occasionally assumed that the above studies and procedures can be used to approximate volumetric strains in cohesionless soils other than sands, such as gravelly sands and sandy gravels. However, studies of gravelly soils (e.g., Kokusho, 2007) indicate that the deformation behaviour of such soils can be significantly different than that of sands. Additional research and data are needed to facilitate the evaluation of volumetric strains and deformations in well-graded and coarse-grained soils.

Embankment cracking

Cracking of an embankment can occur due to deformation or differential settlement (see above: 'Earthquake-induced deformations', 'Liquefaction' and 'Volumetric strains') or in the absence of deformation due to brittle behaviour under shaking loads, or other stress concentrations within fill soils. Seismically-induced longitudinal cracking is most likely occur along the crest and upper faces of the embankment. There is limited case history knowledge of transverse cracking in embankment dams, but the examples identified tend to occur at higher accelerations or are directly related to foundation shape discontinuities. Most documented cases of embankment dams subjected to earthquakes with a Moment Magnitude (Mw) >6.75 and a peak ground acceleration (PGA) > 0.3 g report transverse cracking in addition to longitudinal cracking.

Procedures for the evaluation of earthquake-induced cracking range from simplified indirect evaluation using empirical methods through to complex direct evaluation by detailed numerical analyses. Mejia and Dawson (2019) provide an overview of evaluation procedures and proposed approaches.

Designers should adopt a practical approach, selecting appropriate analytical complexity and effort based on the importance and characteristics of a dam project. Regardless of analytical complexity, the evaluation of settlement, deformation, and cracking potential should be interpreted in the context of site-specific dam zonation (e.g., material properties, design intent, filter provisions and zone geometries).

In addition to the assessment of earthquake-induced deformations, screening-level methodologies that can be used by Designers to estimate cracking include:

- Fong and Bennett (1995) provide plots of normalised settlement versus crack depth which can be used to estimate crack depths.
- Pells and Fell (2002, 2003) plot embankment dam cases for which both longitudinal and transverse cracking occurred, along with cases where only longitudinal cracks were reported.
- Fell et al. (2015) describe a simplified methodology for estimating settlement and cracking in embankment dams subjected to seismic shaking. The methodology uses research by Pells and Fell (2002, 2003) and Swaisgood (1998), developed from the historical seismic performance of embankment dams, and includes plots of damage contours versus earthquake magnitude and peak ground acceleration for earthfill and rockfill dams.

Caution is essential when using screening techniques, particularly when ground motion parameters for a dam (e.g., magnitude and/or PGA) exceed the range of empirical data used to develop these techniques. Extrapolating beyond the limits of empirical datasets is not recommended.

Slope Stability Requirements

Recommended minimum requirements for seismic stability are listed in Table 6.4. The recommended factors of safety for post-earthquake loading conditions align with those adopted by ANCOLD (2019a) and ICOLD Bulletin 194 (ICOLD, 2022a).

Table 6.4: Recommended minimum requirements for slope stability– seismic assessment

Loading condition	Slope	Minimum Factor of Safety or acceptable deformation
OBE (consider embankment response)	Upstream and downstream	Generally 1.0. Minor deformations are acceptable provided the dam remains functional and the resulting damage is easily repairable
SEE (consider embankment response)	Upstream and downstream	Deformations are acceptable provided they do not lead to an uncontrolled release of the impounded contents
Post-earthquake	Upstream and downstream	1.0 to 1.2 (1)
1. As in ANCOLD (2019a) and ICOLD Bulletin 194 (ICOLD, 2022a)		

Other considerations
Designers should familiarise themselves with recent case-studies on the seismic performance of dams, particularly where indirect methodologies can be retrospectively applied and verified against the observed performance of dams in the New Zealand context. Morris et al. (2013) and Amigh et al. (2017) provide recent case studies of settlement and cracking in New Zealand dams.

Design methodologies that are empirically derived are often based on limited databases of dams that have been damaged by earthquakes. With advances in research, existing sources may be revised or new methodologies developed. Therefore, it is important for the Designer to stay informed of the latest techniques and methodologies for estimating seismic-induced deformation in embankments.

For detailed numerical analyses, the Designer should use simplified methodologies and relevant case studies as validation checks.

The assessed damage (cracking and settlement) for the SEE should incorporate some margin to provide assurance that an uncontrolled release of the reservoir cannot be initiated. The Designer should consider the dam fundamental period in response to ground motions and case studies of dam performance. The Designer should analyse the post-SEE condition of the dam to obtain an understanding of the state of the dam before analysing the aftershock loading condition. Damage may result in pore pressure changes within the embankment that have an adverse effect on material properties and dam stability. Estimates of any strength reductions that result from the main shock should be incorporated in the aftershock analysis and again in the final post-earthquake analysis. If it is envisaged that the reservoir will not be drawn down immediately following the SEE, then the aftershock loading conditions should be analysed with the dam in a medium to long-term post-earthquake seepage (pore pressure) condition.

As stated in section 6.1, the post-earthquake criteria are only applicable for the temporary condition, immediately post-earthquake, until satisfactory repairs have been completed and acceptable normal operating dam safety criteria have been restored. During this period, interim risk reduction measures may be necessary to provide an acceptable level of dam safety until repairs have been completed.

If cracking is so extensive that subsequent leakage saturates the embankment, it could exit the downstream slope leading to destabilising forces at the face, with resultant slope instability or unravelling of the shoulder material. Embankment resistance to leakage instability is predominantly a function of the downstream slope angle, the mean particle diameter of the shoulder material, and the leakage discharge exiting the downstream face and toe.

6.5.5 Design details

In addition to meeting the above performance criteria, successful embankment dam design relies on the adoption of good defensive design details. These are addressed in a range of ICOLD bulletins and include:

- Providing ample freeboard and appropriate crest details.
- Using the best available materials in the more critical areas of the embankment.
- Providing well designed and constructed filter and transition zones to ensure compatibility between adjacent materials.
- Providing ample drainage zones for the interception and control of seepage flows.
- Providing good design details (e.g., flaring or widening the filter and transition zones) at all interfaces between the embankment and its foundation, and at all interfaces between the embankment and concrete structures (e.g., spillways and diversion culverts).
- Providing adequate protection against erosion by wave action and runoff.

6.5.5.1 Freeboard

The recommendations in the general section on freeboard for all dams, earlier in Module 3, are applicable for embankment dams, subject to the additional guidance in this section.

Freeboard is especially critical for embankment dams, which are generally more vulnerable than other types of dams to overtopping type failures. Unless robust analysis demonstrates that exceedance does not endanger the dam, reservoir levels - including still water and dynamic set up, run up, and seiches - should be kept:

- Below the level of the top of the low permeability core.
- Below the crest of the dam.

The earlier section on freeboard recommends consideration of scenarios based on a range of factors reasonably likely to occur in combination. For embankment dams, Table 1 in the Abstract of Module 3 provides three scenarios that typically should be included, covering: (i) reservoir still water level, and (ii) wind set up and wave run up. For a specific dam and site, it may be appropriate to consider further scenarios in addition to the three in Table 1. It also may be appropriate to modify the three scenarios to allow for further factors (as listed in section 4.5.4 on freeboard) that are reasonably likely to occur at the same time as the specified wind, wave, and flood levels.

Further to the details in Table 1, a minimum freeboard of 0.9 m is identified as part (a) of the third freeboard scenario in the table. The minimum 0.9 m freeboard is a USBR historic standard for embankment dams. For New Zealand dams that deviate from the typical dam in the USBR portfolio, a freeboard smaller than 0.9 m may be acceptable after assessing the wind set up and wave run up as per part (b) of the third freeboard scenario in combination with the IDF peak reservoir level, plus any other factors (per earlier section 4.5.4 on freeboard) that are reasonably likely to occur at the same time. However, justification for the departure from the 0.9 m minimum freeboard must be documented in the design report.

6.5.5.2 Crest details

Crest details for an embankment dam should be designed to:

- Provide a suitable width for construction and, if appropriate, a suitable width for a permanent access road. Unless a permanent access road is needed, a crest width of 6 m should be sufficient for most dams. To enable access for maintenance, crest widths should never be less than 4 m.
- Provide a suitable level of protection against internal erosion and piping. During extreme flood conditions the reservoir level and wind set up (often excluding waves subject to permeability of materials) should not exceed the top of the low permeability core (unless analysis indicates exceedance does not endanger the dam as noted above). In addition, any filter and transition materials should extend to the top of the core material, especially where there is risk of cracking due to desiccation or freezing.

6.5.5.3 Embankment materials

At many sites there is a shortage of good quality, naturally occurring materials and in many cases processing is necessary to obtain suitable materials for construction. At the outset, it is important to establish the characteristics of the naturally-occurring materials and where they could be best utilised in the construction of the embankment. For example:

- There may be insufficient plastic core material for the construction of the dam core and, as such, it may be prudent to utilise the plastic material in the more critical areas (e.g., adjacent to the core/foundation interface and all core/concrete interfaces).
- The scarcity of a suitable plastic core material may necessitate the use of a core material with little plasticity (e.g., a silty sandy gravel) and the adoption of a wider core along with enhanced filter protection.
- The naturally-occurring material may be abundant but variable and it may be appropriate to utilise the material with less fines and more variability in the downstream shoulder of the dam.
- The available rockfill may break down during compaction, necessitating the installation of additional filter and drainage materials to achieve a drained downstream shoulder.
- The naturally occurring alluvial materials may require a prohibitive amount of processing to produce the specified filter and drainage materials. Good control of filter and drainage materials is critical, and it will often be more economical to import suitable materials to the site.

The construction specification should include material grading envelopes, filter compatibility and internal stability requirements, target moisture contents, compaction requirements, quality control tests, and quality assurance requirements. Trial processing and embankment trials are recommended to establish final specification parameters. Strict adherence to the compaction specification is necessary to avoid the presence of crushed layers that adversely affect permeability contrasts, and poorly compacted layers that encourage embankment settlement and cracking. Dam cores susceptible to desiccation cracking should be protected from drying out during any construction shutdown and capped at their crests to minimise the potential for desiccation cracking during their operational lives. Cracking of embankments is discussed in section 6.5.2 and section 6.5.4.

The potential for internal instability or suffusion of a soil is governed by material, stress, and hydraulic factors. Internal instability is a concern for widely-graded and gap-graded granular materials, particularly those with low fines contents. Given the prevalence of widely-graded dam fill and foundation materials in New Zealand, an assessment of internal instability potential could be necessary for core, shoulder, filter, or foundation materials.

There are no generalised methods for accurately predicting whether a potentially-susceptible material will be internally unstable in a given application. Fell et al. (2015) provide guidance on the assessment of internal instability. In high-consequence applications, Fell et al. (2015) recommend laboratory tests of soil samples under simulated field conditions.

For the purposes of screening-level assessments, various geometric (particle-size) methods have been proposed to assess material susceptibility to internal instability. While not a definitive predictor of internal instability, these empirical screening tools were developed from laboratory tests for particle migration. Published studies that provide empirical data from laboratory tests for internal instability include:

- Kenney and Lau (1985, 1986), who recommend stability limits based on the slope of the finer end of the grading curve to assess the internal stability of granular soils. These limits are derived from laboratory tests on sandy gravels.
- Li and Fannin (2008) adapts the Kenney and Lau (1985, 1986) method to incorporate the Kezdi (1979) split-gradation method.
- Burenkova (1993) proposes stability thresholds defined by the slopes between characteristic particles sizes of grading curves. These thresholds are derived from laboratory tests on sand-gravel mixtures with undefined fines content (inferred to be less than 10% fines).
- The use of characteristic particle sizes serves as the basis for subsequent recommendations by Wan and Fell (2008) and Fell and Fry (2013) regarding silt-sand-gravel materials.
- Douglas et al. (2019) report laboratory tests for seepage-induced instability in a range of soils; however, a 'stable' bound is not defined. Like all laboratory studies, the interpretation of this data requires careful consideration.

All empirically derived screening techniques must be applied with reference to soil gradation, plasticity, and mineralogy, as well as specimen reconstitution and testing arrangements (including confinement and hydraulic conditions). Each gradation-based method should only be applied to assess soil types that are the same as the soil type used in testing to derive the corresponding empirical threshold. Care should be taken to ensure grading similarity throughout the entirety of the grading curve, particularly where characteristic particle sizes are used to assess material susceptibility. Discussions on the applicability of gradation-based methods are presented by Rönnqvist et al. (2014), Fell et al. (2015), Crawford-Flett and Haskell (2016) and Rönnqvist et al. (2017). Specific laboratory tests are recommended for applications involving soils that indicate potential susceptibility to internal instability.

6.5.5.4 Filter, transition and drainage zones

Seepage through embankment dams must be managed to prevent erosion and degradation of dam components. Filters and drainage zones should be provided where the shoulder material is coarse in relation to the core, should be considered essential where the core incorporates dispersive soils, and should be provided around culverts, conduits, and any penetrations through the dam. Filter, transition and drainage zones must be designed to ensure compatibility between adjacent materials and provide sufficient drainage capacity to safely accommodate the anticipated seepage flows under all loading conditions, including the post-earthquake condition.

Filters can be divided into critical and non-critical filters. Critical filters are those that are critical to the control of internal erosion in a dam and, as such, they should be designed and constructed to meet stringent, no-erosion filter criteria. Non-critical filters are those that can be readily repaired if erosion occurs. Examples of critical filters in Figure 6.2 are 'g' and 'h' (if there is potential for the erosion of embankment material into the foundation), and examples of non-critical filters are 'a' and 'c'. Although not shown in Figure 6.2, filters just downstream of the core around conduits passing through a dam and around any other penetrations through a dam are also critical filters.

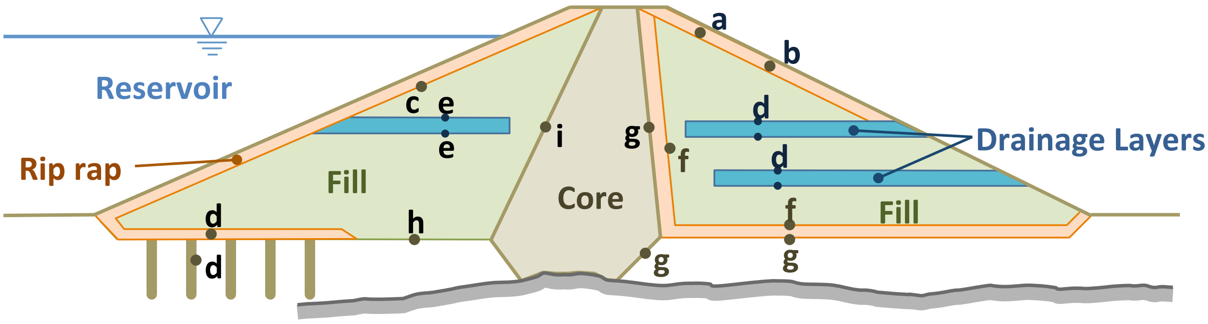

Figure 6.2: Critical and non-critical filters (redrafted from Fell et al. 2015).

6.5.5.5 Critical filters

A properly designed filter will block the movement of soils eroded from a crack and prevent subsequent erosion. Figure 6.3 schematically demonstrates how an eroded fine material is caught at the filter face, and how high hydraulic gradients between the water in the crack and the adjacent filter can result in a widening of the eroded material on the filter until the gradient is reduced. Upstream filters can also have a function of 'crack stopping' to fill an open crack and prevent further erosion.

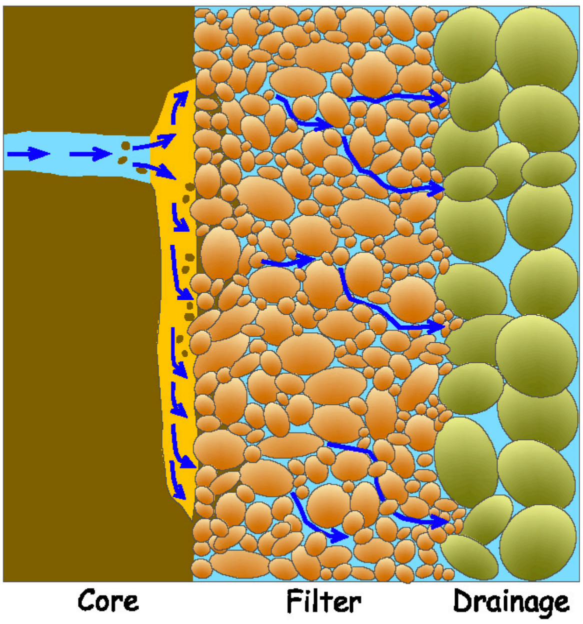

Figure 6.3: Schematic of an effective filter

Filter applications, design considerations, and construction requirements are provided in FEMA (2011) and Fell et al. (2015). Both FEMA (2011) and Fell et al. (2015) outline accepted methods for the design of filters, review factors that affect filter design and performance, and recommend methods for the design of critical filters. ICOLD Bulletin 164 Volumes 1 and 2 (ICOLD, 2017 and ICOLD, 2022) includes a detailed account of internal erosion processes and provides guidelines for the engineering assessment of the vulnerability of a dam to failure or damage by internal erosion. It also includes a discussion on design methods for critical filters and comments on the appropriateness of the methods.

The recommended methods included in FEMA (2011) or Fell et al. (2015) should be adopted for the design of any filter, transition or drainage zones.

The following general criteria apply to filters:

- The granular filter should be non-plastic and highly unlikely to hold an open crack.
- The filter should be designed to meet 'no-erosion' criteria as described in Fell et al. (2015). In High PIC applications where a critical filter protects dispersive base soil, the designer should review current literature to verify that the 'no erosion' criterion is appropriate for the particular application. Specific guidance on filter design for dispersive base soils is provided by Delgado-Ramos and Escuder (2006) and Vakili et al. (2018) among others.
- The filter should be sufficiently permeable for the seepage flow to pass without significant build up of pore pressure. The grading of the filter should have ≤2% (or at most 5%) fines passing the 0.075 mm sieve.
- The fines content of filter materials should be tested using the wet sieving method in accordance with NZS 4402:1986 Part 2.8.1 or equivalent (Standards New Zealand, 1986). Dry sieve methods are unsuitable due to their systematic underestimation of fines contents.
- Gap graded filters and gradations prone to segregation, degradation, or internal instability should be avoided.
- Filter and drainage zones must be sufficiently wide to adequately perform their filtering and drainage functions, to minimise the potential for the introduction of construction related defects (e.g., horizontal offsets, segregation), and to remain effective following any differential movements during construction or displacements following an earthquake.
  - For filters upstream and downstream of a dam core the specified width should reflect the proposed construction method (e.g., end dumping from a truck, the use of a spreader box).
  - For horizontal filters the specified thickness should be sufficient to minimise the potential for continuous coarse zones through the filter material. Fell et al. (2015) recommend that the specified thickness should be no less than 20 times the maximum particle size of the filter material.
- To ensure the full height of the core is protected against internal erosion, critical filters downstream of the core should extend over the full height of the core.
- Filter placement methods should minimise the potential for segregation and contamination.
- Filter compaction levels should be dense enough to be dilative but not so dense as to become brittle and crack prone.

FEMA (2011) describes laboratory testing for particle retention and material quality in filter applications.

Horizontal drains also need to be filter compatible. If large discharges are expected then a coarse drain, protected by filters above and below the drain (i.e., three layers), is recommended.

6.5.5.6 Non-critical filters

Non-critical filters, such as those upstream of a dam's core - where the filter is not exposed to the risk of high exit gradients in the event of core cracking - and those situated beneath upstream rip rap protection where some damage may be tolerable, are sometimes designed to lesser standards than critical filters. However, if proper protection is required, the filters should be designed as critical filters.

Fell et al. (2015) includes recommendations for the design of non-critical filters and highlights specific applications where it is appropriate to adopt the design philosophy recommended for critical filters. In many cases, a well graded gravelly sand, with a maximum particle size of 75 mm and less than 5% fines, should be suitable.

6.5.5.7 Assessing filter interfaces and internal erosion vulnerabilities in existing dams

Fell et al. (2015) and ICOLD (2017) provide guidelines for the engineering assessment of the vulnerability of an existing dam to internal erosion. These publications include the identification of potential internal erosion failure modes, screening of the potential failure modes according to particular dam, foundation, and concrete structure characteristics, identification of those potential failure modes that are more likely to occur, and analysis of the more likely potential failure modes to determine whether internal erosion could initiate, continue, and progress. Figure 6.4 provides examples of possible locations where internal erosion can be initiated in embankment dams.

In addition to the locations illustrated in Figure 6.4, it is noted that embedded instruments, abutment steps or other irregularities are locations where internal erosion (seepage erosion) can be initiated in an embankment dam.

In addition, it is noted that the pressures along a foundation seepage path (detail 6 in Figure 6.4) can be significantly greater than the pore pressures in the dam, especially beneath and downstream of the core. In such instances, the gradient is from the foundation into the dam and the potential for the erosion of foundation materials into the downstream shell or a drainage blanket should be considered.

It is also important to recognize the dominant internal erosion mechanism as either (1) seepage erosion (also known as scour or tractive force erosion) where the seepage flow path is open (e.g., a crack or separation along a wall or conduit) or (2) backward erosion piping where the seepage flow path is opened by erosion of particles starting from the downstream exit and progressing by pipe formation to the reservoir.

Once the pipe and flow pathway is fully developed, the erosion mechanism becomes seepage erosion along the sidewalls of the pipe. In Figure 6.4 pathways 1 to 3 are typically seepage erosion pathways while pathways 4 to7 are typically backward erosion piping pathways. However, the two processes can sometimes operate in tandem.

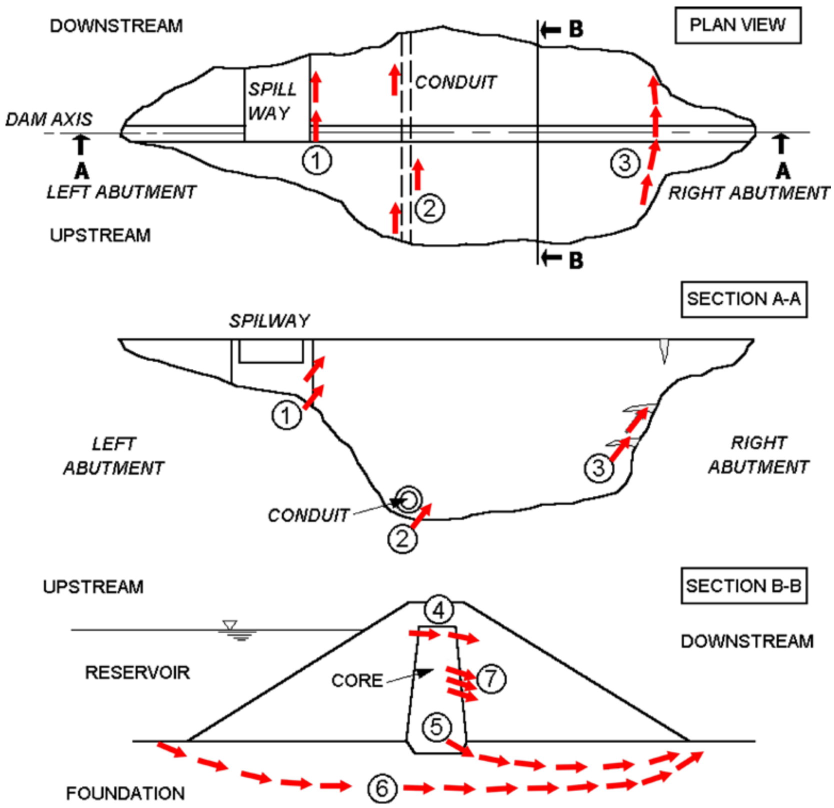

Figure 6.4: Possible locations for the initiation of internal erosion (Fell and Fry, 2007).

Analysis of the seepage erosion and backward erosion piping related potential failure modes should include a review of the following:

- Initiation: Will erosion of the material (considering its relative erodibility) be initiated under the existing seepage gradients? Is there a design or construction flaw, or a reason for erosion to commence (e.g., flood level above the top of the core)?
- Continuation: Is there an unfiltered or poorly filtered exit? Will the filters, transition zones or downstream zones prevent erosion continuing? The gradation/s should be assessed against criteria for no-erosion, some erosion, excessive erosion, and continuing erosion.
- Progression: Is the seepage path open or will a developing pipe stay open? Will the upstream material fill the crack or limit the seepage flow? Will the critical gradient or velocity be reached for erosion to progress?

- Intervention: Will there be sufficient warning to intervene? Can the reservoir be drawn down in sufficient time to prevent a failure? Do we have ready access to suitable Technical Specialists, materials and equipment to avert a failure?

If the filter protection systems within an existing dam are insufficient to resist the more likely potential failure modes, it will be necessary to consider the need for remedial works to reduce the potential for failure to occur. 'No-erosion' filters are clearly acceptable, but filters that fit the 'continuing erosion' category will probably necessitate the completion of remedial works. For filters that fit the 'some erosion' or 'excessive erosion' categories, a qualitative assessment of the risk should be completed to establish whether remedial works are necessary. As described in section 6.5.2, internal erosion processes can involve multiple erosion mechanisms and can vary spatially and temporally. A quantitative risk assessment of internal erosion is complex and should only be carried out by Technical Specialists experienced in the technique.

Further discussion on the assessment of existing dams that do not satisfy modern design criteria is provided by Foster and Fell (2001), Fell et al. (2015), Rönnqvist and Viklander (2016), and Foster et al. (2018).

6.5.5.8 Geotextiles in lieu of granular filters

ICOLD Bulletins 55 and 95 (ICOLD, 1986b; ICOLD,1994), and Fell et al. (2015), provide guidelines for the use of geotextile fabrics in embankment dams. The following recommendations generally reflect the guidelines:

- Geotextiles should primarily be used where they can be readily exposed, repaired or replaced, or where they can provide temporary control of seepage flows that have the potential to transport materials. For example, geotextiles installed beneath wave protection layers and within toe drains can be inspected, repaired or replaced if required.
- Geotextiles should not be used as critical filters (i.e., in a configuration where they serve as the sole defence against dam failure). As such, they should not be used in lieu of sand/gravel filters for the control of internal erosion within, or beneath, an embankment dam.

Giroud (1997) includes comments on geotextile filter design and installation.

6.5.5.9 Protection around conduits

A conduit or pipe through an embankment dam is a common location for the initiation of internal erosion, particularly for existing dams that do not include good filter protection. Inappropriate conduit details, the lack of filter and drainage protection, and low stresses associated with arching of embankment fills across the tops of conduits can initiate the erosion of embankment materials. Erosion can progress through the loss of material into the conduit, erosion along the outside of the conduit, and water losses from the conduit. Many embankment dams have failed through inappropriate conduit design, and inadequate filter and drainage protection adjacent to conduits. FEMA (2005) and Fell et al. (2015) provide recommended practices, which include:

- Where practicable, avoid placing conduits through the dam or across soil and erodible rock foundations. Instead, consider the use of tunnels in the abutments for larger dams or placing the conduit in a trench excavated into non- erodible rock, which is then backfilled with concrete up to the dam foundation surface.
- Where conduits or other penetrations pass through existing dams, or where they must be designed to pass through embankments - as will be the case for most smaller dams and many larger dams on soil foundations - filters should be installed immediately downstream of the core and should completely surround the conduit. Alternatively for Low PIC dams where filters are not provided immediately downstream of the core as part of the design cross section, a filter diaphragm should be placed around the conduit and suitable drainage should be provided for the controlled discharge of seepage flows from the filter diaphragm to the toe of the dam.
- Conduits should be continuously supported on a concrete bedding which extends, as a minimum, up to the centreline of the conduit and has outside slopes no steeper than 1 (horizontal) to 8 (vertical).
- To enable good compaction adjacent to the conduit, cutoff collars should not be used. Additionally, the soil being compacted adjacent to the conduit should be wet of optimum moisture content.
- Cast in situ conduits should have sides no steeper than 1 (horizontal) to 8 (vertical) and should be constructed in trenches sufficiently wide to enable proper compaction of the backfill material. The side slopes should be no steeper than 1 (horizontal) to 1 (vertical) to minimise the potential for arching across the top of the culvert.

- Care should be taken to ensure no desiccation cracking is left in place at the base or adjacent to the sides of a trench excavated for a conduit.
- Care should be taken to ensure the conduit joint details will prevent the erosion of backfill materials into the conduit and prevent the leakage of water out of the conduit.
- No un-encased metal conduits should be used unless they are separated from the embankment fill by an air space (e.g., within a larger conduit that provides ready access for maintenance and repairs). Metal conduits which are concrete encased and are not separated from the embankment fill by an air space should be continuously welded, and the encasement should be reinforced to carry all static and dynamic loads without any contribution from the metal conduit.

Where any of the above recommended practices have not been followed, the completion of a risk assessment may assist in establishing whether a potential deficiency needs to be addressed.

6.5.5.10 Interfaces between embankments and concrete structures or abutments

Interfaces between embankment dams and concrete structures are potential sources of internal erosion. All concrete surfaces adjacent to embankment materials, particularly core materials, should be smooth and free of construction defects (e.g., horizontal offsets along construction joints), and should incorporate slopes no steeper than 1 (horizontal) in 8 (vertical) to encourage positive contact pressures along the interface.

Filter and drainage materials should always be provided for the control of seepage flows along such interfaces and, where filter and drainage materials are included within embankments, some consideration should be given to flaring the core material and widening the downstream filters in the vicinity of the interfaces.

Many of the features outlined above for interfaces between embankment dams and concrete structures are also applicable to dam abutment contact surfaces. Fell et al. (2015) provides further guidance on design of abutment interfaces.

6.5.5.11 Drainage pipes

Drainage pipes should only be utilised in areas where they are readily accessible for maintenance or replacement (e.g., in toe drains). The Designer should specify corrugated smooth wall pipe rated for the embankment loads, with the perforation size based on the filter grading.

6.5.5.12 Toe drainage capacity

High drainage capacity at the downstream toe can be provided by a toe drain system or by a zone of materials with a suitable grading to withstand the predicted flow.

This might be achieved through coarse free draining fill at the dam toe or a partial height toe buttress (often referred to as a Swedish berm). Scandinavian researchers developed empirical methods for the design of drainage buttresses to prevent toe unravelling based on large scale tests. Bartsch and Nilsson (2007) provide an empirical relationship between the mean rock particle size (D50), the downstream slope of the rockfill, and the unit discharge flow based on these test results.

6.5.5.13 Surface erosion

The upstream slopes of embankment dams and their abutments require protection against erosion by wave action. ICOLD Bulletin 91 (ICOLD, 1993b) provides a detailed discussion on loads that need to be considered and design criteria that should be adopted for the design of upstream slope protection systems including dumped rip-rap, hand placed rip-rap, soil cement facings, concrete paving and precast concrete blocks, bituminous concrete linings, gabions and reno-mattresses, steel and timber facings, and roller compacted concrete facings. The design methodologies included in ICOLD Bulletin 91 should be adopted for the design of upstream slope protection systems.

Quarried and dumped angular rip-rap, where it can be economically obtained, is the preferred material for the protection of upstream slopes because of its flexibility and its thickness. The integrity of other materials such as rounded river cobbles and boulders can be affected by wave action and embankment settlement, particularly if they are placed over a geosynthetic fabric. Such materials should only be used where quarried rockfill cannot be economically obtained and where the materials can be readily inspected, repaired or replaced.

To ensure the protection of underlying materials, rip-rap should be well graded and durable, and should extend a sufficient distance down slope to protect the underlying material from wave action at the minimum reservoir operation level. Additional protection may be necessary below the level of the rip-rap if there is the potential for initial reservoir filling to erode the embankment material and undercut the rip-rap.

Downstream slopes should also be protected from erosion where they are constructed from materials other than rockfill. Design details to minimise the potential for surface erosion on the downstream slopes of dams include:

- The placement of a protective layer of rockfill, or topsoil and grass.
- The provision of berms to limit the distance over which runoff can concentrate.
- The provision of berm drains for the interception and controlled discharge of runoff to the dam toe.
- The provision of lined open drains along the abutment contacts.

6.6 Concrete-faced rockfill dams

6.6.1 Introduction

Concrete-faced rockfill dams (CFRDs) are a form of embankment dam that rely on the upstream concrete face slab for water retention. Asphaltic concrete has been used as a variant to conventional concrete in some dams in the world.

Typical CFRD design will have a concrete plinth, founded on competent rock at the upstream toe of the dam, to connect the face slab to the foundation. This is a critical element of the dam.

The following subsections discuss potential failure modes for CFRD dams, loading conditions which must be taken into account during their design, evaluation and rehabilitation, and recommended performance criteria for CFRD dams. Defensive design details that are important to dam safety are also discussed.

ICOLD Bulletin 141 (ICOLD 2011b) and Fell et al. (2015) provide detailed accounts of the design and construction features of CFRDs and their performance.

6.6.2 Potential failure modes

Refer also to section 6.2.2 which provides additional information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams, and section 6.5.2 which provides guidance on potential failure modes for embankment dams.

As described above, the identification of potential failure modes for a dam should be based on site-specific conditions and the specific characteristics of the dam. However, as an aid, Table 6.5 outlines the mechanisms responsible for common potential failure modes associated with CFRD dams and their foundations.

Table 6.5: Initiating mechanism for potential failure modes for concrete-faced rockfill dams (CFRD)

Initiating mechanism	Common causes
Overtopping	Insufficient freeboard to accommodate storms and flood events
Excessive leakage and unravelling of downstream shoulder	Settlement, defect or crack in facing slab or plinth, shoulder material not coarse enough to withstand leakage discharge
Internal erosion of embankment materials	Defect or crack in the facing slab, lack of adequate filter protection, high fines content in embankment fill
Internal erosion of foundation materials	Foundation material has a Plasticity Index less than 7, dispersive foundation materials, lack of or inappropriate foundation treatment, high gradient around plinth
Instability of downstream shoulder	Defect or crack in facing slab, weak shallow seam in foundation, lack of effective drainage and saturation of downstream shoulder, insufficient shear strength, strong earthquake shaking
Instability of upstream shoulder (sliding failure involving/ disrupting the face slab)	Rapid drawdown of reservoir, insufficient drainage, strong earthquake shaking
Loss of freeboard, overtopping and subsequent erosion	Insufficient freeboard to accommodate foundation and embankment settlement, settlement following seismic loading and/or foundation materials, seiches generated by earthquakes, uplift of the reservoir due to fault displacement, reservoir landslides
Loss of freeboard, overtopping and subsequent erosion	Insufficient freeboard to accommodate foundation and embankment settlement, settlement following seismic loading, liquefaction of embankment and/or foundation materials, seiches generated by earthquakes, uplift of the reservoir due to fault displacement, reservoir landslides

Further guidance on potential failure modes and initiating mechanisms in CFRDs can be found in the literature. Fell et al. (2015) outlines a framework for assessing potential failure modes for a CFRD. Rogers et al. (2010) describes the failure of the Taum Sauk reservoir in the USA and the combination of factors leading to this failure. Pinto and Marques (1998) discuss incidents of cracked upstream face slabs due to high stresses attributed, in part, to the valley shape. Wieland (2009) provides descriptions of damage to CFRDs from earthquakes but notes that there are few observations of dam responses to strong earthquakes.

6.6.3 Loading conditions

The loading conditions that should be considered in the design or rehabilitation of a CFRD are as stated for an embankment dam in section 6.5.3.

6.6.4 Stability and deformation performance criteria

The dam, foundation, and abutments must be stable during construction and under all operating conditions, including full or partial drawdown. Normally, a competent rock foundation of high strength and stiffness is necessary for CFRDs. Recommended minimum factors of safety for limit equilibrium stability studies, for static and seismic loading conditions, are the same as for embankment dams and are listed in Table 6.3 and Table 6.4. The comments and guidelines relating to embankment stability, deformation, and post-earthquake performance in section 6.5.4 are also applicable to CFRDs.

Settlement may occur under static conditions or as a result of earthquake shaking. The potential for settlement should be assessed and consequent effects should be addressed during the design, including:

- The loss of freeboard and the risk of overtopping.
- The extent of damage to the upstream face slab.

The performance of the upstream face slab is critical to dam performance as extensive cracking could result in sufficient leakage to threaten the stability of the dam. In addition, a stable well-founded plinth and stable supporting backfill are critical to the performance of the face slab. An inadequate transition between the concrete face slab and rockfill can restrict flow into the rockfill if cracking or joint opening occurs in the slab. Poorly graded materials with the potential for internal instability could result in increased leakage and unravelling or instability of the downstream shoulder. The design process should include an assessment of the potential settlement of the dam, the potential for cracking or joint opening in the concrete face slab, and sufficient seepage and stability analyses to demonstrate that the embankment has adequate reserves of stability.

6.6.5 Design details

In addition to meeting the above performance criteria, successful CFRD design relies on the adoption of good defensive design details, such as:

- Adopting an appropriate location and orientation for the dam to reduce the risk of foundation displacement damaging the plinth.
- Founding the plinth on competent rock.
- Providing an adequate filter zone behind the plinth.
- Providing sufficient slab thickness for the anticipated loads.
- Providing ample freeboard and appropriate crest details.
- Providing appropriate material zoning and well-designed filter and drainage zones behind the face slab, using the best available materials, to ensure compatibility between adjacent materials and enable free drainage of leakage through the concrete facing.
- Adopting stringent compaction standards to minimise potential embankment settlements.
- Locating facing slab joints to account for non-uniform deformations of the supporting rockfill, providing good joint and waterstop design details to lower the risk of a major leakage through a joint, and selecting a slab width and a jointing system that accommodates the reversible nature of seismic responses to strong ground shaking and temperature variations.
- Shaping the foundation beneath the plinth to avoid high stress concentrations.

6.6.5.1 Location and orientation of the dam

If the concrete plinth crosses or rests on any foundation features that could displace under reservoir loading or in an earthquake, the dam should be oriented to minimise the offset implications for the plinth. In the vicinity of such foundation features, specific details to reduce damage to the plinth should be assessed and filters with dimensions at least 1.5 times the expected offset should be placed behind the perimetric joint between the face slab and the plinth.

An upstream impermeable blanket, with appropriate filter layers, should also be considered to cover the plinth and perimetric joint in the location of the foundation feature.

6.6.5.2 Slab thickness

The upstream face slab is a stiff element that relies on rockfill support. Any loss of support will result in slab cracking and leakage; therefore, the slab thickness must be sufficient to accommodate robust joint details. Consideration should also be given to situations where the valley shape could introduce high compressive stresses in the face slab. Problems have occurred with high CFRDs in narrow canyons, where the dam height and crest length have had roughly equal dimensions.

The face slab will develop high in-plane stresses from the cross-valley component of earthquake ground motions, and the potential for shear failure and spalling needs to be addressed in the slab design.

6.6.5.3 Freeboard and crest details

The recommendations on freeboard for all dams (section 4.5.4) and freeboard for embankment dams (section 6.5.5.1) are also applicable for CFRDs.

Crest structures, including wave walls, require careful detailing to accommodate:

- The predicted settlement without compromising the watertightness of the joints.
- The different response characteristics, in comparison to those experienced by the main rockfill embankment, during strong earthquake shaking.

6.6.5.4 Material zoning

Filters are required beneath the face slab and immediately downstream of the plinth to restrict flow into the rockfill in the event of cracking or joint openings in the face slab. Filters also help limit deformation of the slab at the perimetric joint and restrict flow into the embankment or foundation if the perimetric joint opens.

The gradations of the embankment materials must be internally stable and the embankment zones should increase in coarseness towards the downstream face and toe. Where embankment materials break down under compaction and result in materials with high proportions of sand and silt, the resulting fill may not be free draining. In such cases, filter and drainage layers must be provided beneath the face slab and along the foundation contact to ensure the controlled collection and drainage of leakage to the dam toe.

6.6.5.5 Compaction standards

Earlier CFRDs were constructed of dumped rockfill. This is no longer recommended due to the effects of excessive settlement on the concrete face slab and other rigid structures.

The long-term settlements of well compacted rockfill can be expected to be in the range of 0.1 to 0.2% of the embankment height. Strong ground motions during earthquakes will produce greater settlements.

6.6.5.6 Facing slab joints

The spacing of vertical joints in the face slab should consider the predicted embankment settlement under all loading conditions. Generally, more joints and narrower slab widths are recommended to provide more articulation of the slab. Joints should also be located and detailed above features likely to initiate differential settlement (e.g., steps in the foundation).

Shear keys and durable waterstops that can sustain some movement are recommended at the perimetric joint and at all vertical joints. The joint dimensions need to account for the reversible nature of the embankment dam response to earthquake ground motions.

6.6.5.7 Foundation and abutment shaping

Prominent features (steps or irregularities) in the foundation or abutments should be removed to reduce the likelihood of differential settlement. Normally, a competent rock foundation of high strength and stiffness is necessary for CFRDs. As stated earlier, narrow valleys can result in high compressive forces in the upstream face slab.

6.7 Geomembrane-lined embankment dams

6.7.1 Introduction

Impermeable geomembrane liners are often used as the watertight barrier for small embankment dams, water storage ponds and associated spillways. They are also employed in remediation of concrete faced dams or canals, and ponds containing liquids, tailings, sludge or industrial wastes that may pose a potential environmental risk. Many of the design requirements for embankment dams and CFRDs and their foundations apply, particularly with respect to settlement, slope deformation, and withstanding leakage.

Various different types of geomembrane materials have been used as impervious layers in dams and canals. ICOLD Bulletin 135 (ICOLD, 2010b) lists 10 different polymers used as geomembranes in more than 240 large dams around the world (refer Table 6.6) and provides guidelines for the design of geomembrane sealing systems for embankment dams.

The use of polyvinyl chloride (PVC) membranes accounts for 65% of the installations cited. Of these, roughly equal numbers are installed as covered and exposed membranes. By contrast, the other two popular membranes (linear low density polyethylene – LLDPE and high density polyethylene – HDPE) are typically installed in covered arrangements in higher risk situations to provide protection against environmental conditions and mechanical damage or vandalism.

It is noteworthy, however, that there are many applications of exposed linings, particularly HDPE.

Table 6.6: Use of geomembranes in dams (from ICOLD Bulletin 135, ICOLD, 2010a)

Material	Abbreviation	Total No. of Dams			Total
		Exposed	Covered	Unknown	
Polyvinyl Chloride – Plasticised	PVC-P	80	73	3	156
Linear Low Density Polyethylene	LLDPE	0	29	1	30
High Density Polyethylene	HDPE	3	12	1	16
Butyl rubber	IIR	5	4	2	11
Polyisobutylene	PIB				
Ethylene-propylene-diene monomer	EPDM				
Chlorosulfonated polyethylene	CSPE	3	5	1	9
Geotextiles impregnated with polymers	In situ membrane	2	7	0	9
Polyolefin	PP	3	3	0	6
Chlorinated polyethylene	CPE	0	3	0	3

The following subsections discuss potential failure modes for geomembrane-lined embankment dams, loading conditions which must be taken into account during their design, evaluation, and rehabilitation, and recommended performance criteria for geomembrane- lined embankment dams. Defensive design details that are important to dam safety are also discussed.

6.7.2 Potential failure modes

As stated previously, the identification of potential failure modes for a dam should be based on site- specific conditions and the specific characteristics of the dam. Section 6.2.2 provides general information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams, and section 6.5.2 provides guidance on potential failure modes for embankment dams.

The more common potential failure modes for geomembrane-lined embankment dams are quite similar to those for CFRDs and the potential failure modes listed in Table 6.5 are applicable. However, additional potential failure modes for new and existing geomembrane-lined dams include:

- The potential for deterioration that has occurred or could occur over time.
- The potential for poor construction practice in placement, interconnections and sealing, particularly at pipe penetrations through the liner.
- The potential for damage of the upstream face liner due to impact or drag forces from floating debris.
- Uplift of the geomembrane under high wind situations that can result in damage to the liner.
- Uplift of the liner due to seepage beneath the geomembrane. This can occur due to high groundwater levels under normal or extreme conditions. There is greater potential for uplift under reservoir drawdown conditions.
- Seepage through the subgrade and beneath the geomembrane resulting in erosion channels beneath, or transportation of fines accumulating beneath, the geomembrane at the base. Both can result in overstressing and damage to the geomembrane or lead to exposure of gravels that could puncture the liner.

6.7.3 Loading conditions

The loading conditions that should be considered in the design, evaluation or rehabilitation of a geomembrane-lined embankment dam is as stated for an embankment dam in section 6.5.3.

6.7.4 Stability and deformation performance criteria

The dam, foundation, and abutments must be stable during construction and under all operating conditions, including full or partial drawdown. Recommended minimum factors of safety for limit equilibrium stability studies, for static and seismic loading conditions, are the same as for embankment dams (the membrane provides no resistance) and are listed in Tables 6.3 and 6.4. The comments and guidelines relating to embankment stability, deformation, and post- earthquake performance in section 6.5.4 are also applicable to geomembrane-lined embankment dams. However, the membrane does provide a potential failure surface and could create a location for pressure build up greater than that for a geometrically similar dam with no membrane.

Settlement may occur under static conditions or because of earthquake shaking. The potential for settlement should be assessed and consequent effects that should be addressed during the design include:

- The loss of freeboard and the potential for overtopping.
- The extent of any embankment cracking and whether the geomembrane can continue to function as a water retaining element.

The installation and resulting performance of the geomembrane is critical to the performance of the dam as openings, tears or joint failures could result in sufficient leakage to threaten the stability of the dam. Inadequate supporting and drainage layers between the geomembrane and embankment fill may lead to saturation and instability of the embankment. Furthermore, poorly graded embankment materials with the potential for internal instability could result in increased leakage and unravelling or instability of the downstream shoulder. The design process should include an assessment of the potential for damage to the geomembrane lining, as well as sufficient seepage evaluations, internal erosion assessments, and stability analyses to demonstrate that the embankment has adequate reserves of stability.

6.7.5 Design details

In addition to meeting the above performance criteria, successful design relies on the adoption of good defensive design details, such as:

- Specifying a geomembrane suitable for the environmental conditions and the required life expectancy.
- Specifying appropriate subgrade preparation for the geomembrane.
- Installation of subsurface drainage to intercept and manage seepage beneath the geomembrane. This can take the form of subsoil drains comprising drainage materials and perforated pipes with filter protection depending on the particle size of the subgrade, or geocomposite drains or mats.
- Installation of leakage detection beneath the geomembrane. Subsurface drainage to control groundwater can partially fulfil this role. However, for higher risk situations, geocomposite drains or mats can be considered. In high-risk situations a composite liner system with a low permeability clay or geosynthetic clay liner (GCL) beneath the geomembrane can be considered.
- Surface water management to divert runoff from upslope from entering the anchor trench.
- Specifying requirements for joint performance and leakage testing for the chosen geomembrane.
- Providing suitable anchorage and fastenings for the geomembrane.
- Providing robust and effective connections and seals at all structure penetrations.
- Providing ballast where needed to counteract excess groundwater pressures during operation or dewatering.
- Preventing wind uplift damage during construction and at low reservoir levels.
- Providing appropriate protection against waves, and protection against UV light damage and vandalism.
- Providing ample freeboard and appropriate crest details.
- Adopting stringent compaction standards to minimise potential embankment settlements.

- Providing appropriate material zoning and well- designed filter and drainage zones to ensure compatibility between adjacent materials and to allow the free drainage of leakage through the geomembrane. This is of particular importance at pipe penetrations.
- Considering secondary lines of defence at areas vulnerable to cracking.

6.7.5.1 Selection of geomembrane

ICOLD Bulletin 135 (ICOLD, 2010a) provides a detailed account of geomembrane materials, their behaviour and ageing characteristics, and recommended quality control systems that should be adopted during their manufacture and installation. Table 6.7 has been prepared from information included in the bulletin and provides a summary of the oldest geomembrane installations at dams by geomembrane type, as at 2010.

Table 6.7: Geomembrane longevity in dams (ICOLD, 2010a)

Material	Abbreviation	Oldest installation (years)	
		Exposed	Covered
Polyvinylchloride – Plasticised	PVC-P	1974	1960
Linear Low Density Polyethylene	LLDPE	-	1970
High Density Polyethylene	HDPE	1994	1978
Butyl rubber	IIR	1982	1959
Polyisobutylene	PIB		
Ethylene-propylene-diene monomer	EPDM		
Chlorosulfonated polyethylene	CSPE	1981	1986
Geotextiles impregnated with polymers	In situ membrane	-	-
Polyolefin	PP	1995	2000
Chlorinated polyethylene	CPE	-	1970

Fourie et al. (2010) comments on the advantages and disadvantages of the three most commonly used geomembranes – HDPE, LLDPE and PVC. It should be noted however that, because the PVC formulation can be enhanced with additives such as plasticisers, stabilisers and ageing retardants, it can be specifically formulated to meet different service conditions such as resistance to UV, specific contaminants and very low temperature environments.

The following physical/mechanical properties should be considered during the selection of a geomembrane for a particular application:

- Burst strength.
- Stress/strain characteristic.
- Puncture resistance.
- Tear strength.
- The interface shear strength between the geomembrane system and the embankment slope.
- Elongation/expansion and contraction characteristics.
- The jointing system.
- Durability.
- Permeability.

The ability of a liner to span a transverse crack is the primary defence against embankment failure due to transverse cracking. Giroud et al. (2013) includes an approach to determine the behaviour of a geomembrane liner if a crack develops in the subgrade. The approach takes into account the pressure applied by the reservoir, the tensile stress-strain behaviour of the geomembrane, and the interface friction between the geomembrane and supporting material. The tension in the geomembrane is the result of two mechanisms: (i) the shear stresses applied to the lower face of the geomembrane by the supporting material during the opening of the crack, and (ii) the strain of the geomembrane as it deflects over the open crack. The Designer needs to understand the uncertainty related to estimation of the crack width and, as a minimum, should apply a factor of 1.5 to the estimated crack width when determining an appropriate geomembrane solution.

Geomembranes may require reinforcing to provide sufficient strength to span cracks. Geotextiles may also be necessary to protect geomembranes from damage.

6.7.5.2 Subgrade preparation

All surfaces supporting the geomembrane should be smooth and free of angular rocks, roots and debris. The embankment surface should be proof rolled to identify inappropriate subgrade conditions, and voids created from the removal of soft materials, angular rocks and debris should be filled with fine material and proof rolled a second time.

The construction specification should define key subgrade quality requirements such as the maximum acceptable undulation, the maximum rock fragment size and the avoidance of puddles.

6.7.5.3 Joint performance and leakage testing

The integrity of the joints between geomembrane panels is critical to the watertightness of the geomembrane. There are various jointing methods available and each is dependent on the properties of the geomembrane and the requirement for quality assured integrity. All three common geomembranes (HDPE, LLDPE and PVC) are usually joined using double track automatic welding. Single track manual welding is usually only applied in areas that are inaccessible by the automatic welding machine.

Leakage testing of joints should be included in the construction specification to provide quality assurance.

6.7.5.4 Anchorage and fastenings.

Geomembranes require anchorage at the top of embankments, and within or at foundation and abutment contacts. Frequently used anchorages at the top of an embankment slope have included a simple run-out length of geomembrane beneath a soil cover, and embedment of the geomembrane in a trench which is backfilled with an appropriate material. Anchor trenches should be backfilled with low permeability fill when the underlying soils on which the geomembrane is laid are potentially erodible or dispersive. Stainless steel fixings should be used for fastenings where geomembranes abut pipes, embankment penetrations and concrete structures.

6.7.5.5 Groundwater pressures

The potential for groundwater pressures beneath geomembranes to exceed reservoir pressures should be assessed. If excess groundwater pressures are possible during operation or dewatering, ballast should be provided to prevent uplift of the geomembrane.

A geomembrane-lined canal that passes through cuts where high groundwater levels are present is an example where ballast may be necessary to prevent the uplift of the geomembrane.

6.7.5.6 Wind uplift

Ballast and fixings should be provided if large areas of a geomembrane lining are exposed to wind uplift. This can occur during construction and during operation with low reservoir levels.

6.7.5.7 Wave protection and protection from UV light damage and vandalism.

Repeated wave action can deform an exposed geomembrane, leading to fatigue effects on particular material properties and generating uplift tensile stresses due to suction. Waves may also move cover materials, loosen anchorages and damage the supporting layer beneath the geomembrane.

The Designer needs to assess the vulnerability of the proposed geomembrane solution to wave action and, if necessary, provide an appropriate protection layer and improved geomembrane anchorages.

Some geomembranes may require cover to prevent degradation by UV sunlight and damage by vandalism. The temperature effects on any exposed membrane should be assessed.

6.7.5.8 Freeboard and crest details

The recommendations on freeboard for all dams (section 4.5.4) and freeboard for embankment dams (section 6.5.5.1) are also applicable for geomembrane-lined embankment dams.

The crest width and crest details should be designed to accommodate any necessary anchorage zones and provide ready access to the geomembrane system for maintenance and repair.

6.7.5.9 Compaction standards

Compaction standards are important for all embankment dams. For geomembrane-lined embankment dams, excessive settlement from inadequate compaction will result in tensions within the geomembrane lining system.

6.7.5.10 Material zoning

The gradations of the embankment materials must be internally stable and the embankment zones should increase in coarseness towards the downstream face and toe.

The subgrade material beneath the geomembrane should provide good support and restrict flow into the embankment if tears or leakages develop in the geomembrane. Filter and drainage zones should be designed to ensure compatibility between adjacent materials and control drainage of any leakage flows through the embankment.

6.7.5.11 Secondary lines of defence at areas vulnerable to cracking.

A geomembrane should be designed to span estimated crack widths. However, given the uncertainties associated with the estimation of crack widths, a defensive design approach should be adopted in areas where cracking could occur and/or be more extensive than envisaged. For example:

- A thicker geomembrane layer, a reinforcing layer, or a double layer of geomembrane could be installed at critical locations.
- A leakage buttress could be installed at critical locations for the management of discharges from a damaged geomembrane.

6.8 Concrete gravity and buttress dams

6.8.1 Introduction

Concrete gravity and buttress dams are grouped according to the types of material used in their construction and how they achieve their strength and stability. They commonly include:

- Conventional concrete gravity dams (refer Figure 6.5) are constructed from conventional concrete and rely on the shearing resistance developed at their base due to their weight (hence the term 'gravity'). Their design depends on minimising uplift beneath the dam and ensuring the integrity of the foundation to resist the imposed load from the reservoir. Guidelines for the design of concrete gravity dams are provided in FERC (2016) and USACE (1995).

Figure 6.5: Concrete gravity dam – Clyde Dam (provided by Contact Energy)

- Roller compacted concrete (RCC) dams (refer Figure 6.6) are constructed from zero slump concrete using traditional earth placing methods and similarly rely on the shearing resistance developed at their base due to their weight, less uplift beneath the dam. RCC dams also rely on the integrity of the foundation to resist the imposed reservoir load. Hansen and Reinhardt (1991) provide an overview of design considerations for RCC dams.

Figure 6.6: RCC dam – Horseshoe Bend Dam (provided by Pioneer Generation)

- Hardfill dams (refer Figure 6.7) are constructed from a cemented sand and gravel and incorporate a facing of concrete for water tightness and erosion protection. As for gravity dams, they rely on their weight and the integrity of the foundation to resist the imposed reservoir load. Japanese Dam Engineering Center (2007) provides guidelines for the design and construction of cement sand and gravel (CSG) dams. The ICOLD Cemented Material Dams Technical Committee and others are preparing guidance on design of hardfill dams.

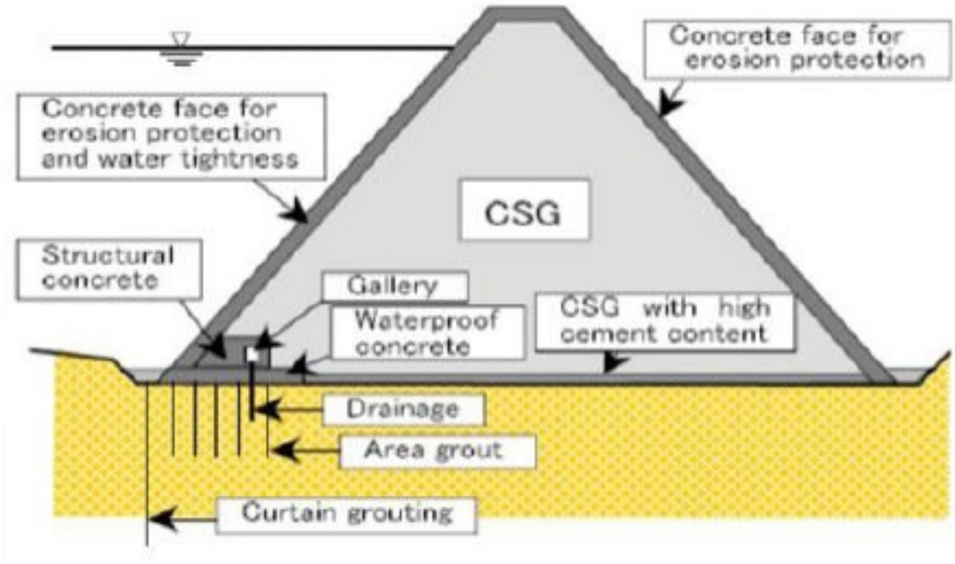

Figure 6.7: Hardfill dam (CSG is Cement, Sand and Gravel)

- Concrete buttress dams (refer Figure 6.8) incorporate an upstream concrete face supported at intervals by a series of support buttresses. They rely on their weight, the structural strength of face and foundation slabs, and the integrity of the foundation to resist the imposed reservoir load. The shearing resistance to resist the reservoir load is developed primarily along the buttresses, as opposed to beneath the entire base for concrete gravity dams.

Figure 6.8: Concrete buttress dam – Branch Dam (provided by Manawa Energy)

- Multiple arch buttress dams incorporate a series of concrete arches supported by, and constructed integrally with, equally spaced triangular shaped buttresses. They have much in common with concrete buttress and concrete arch dams; however, no multiple arch buttress dams are believed to be in service in New Zealand.

The following subsections discuss potential failure modes for concrete gravity and buttress dams; loads and loading conditions which must be taken into account during their design, evaluation, and rehabilitation; and recommended performance criteria for concrete gravity and buttress dams. Defensive design details that are important to dam safety are also discussed.

6.8.2 Potential failure modes

Section 6.2.2 provides general information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams. The general comments on the identification and evaluation of potential failure modes included in section 6.5.2 for embankment dams are also relevant to concrete gravity and buttress dams.

Fell et al. (2015) includes some statistics on dam failures which highlight that failure rates between 1900 and 1975 for concrete gravity and concrete buttress dams were 0.3% and 2.6%, respectively, of dams built. These figures can be compared with the failure rate quoted for embankment dams, over the same time period, which was 1.2% of dams built. The available information demonstrates that the safety of concrete buttress dams between 1900 and 1975 was significantly less favourable than the safety of all other dam types over the same time period.

With respect to the relatively high failure rate of buttress dams, in comparison to that for concrete gravity dams, it is noted that concrete gravity dams are far more robust than the relatively thin slabs and narrow buttresses that support buttress dams. Furthermore, a localised flaw in the dam or foundation of one of the slab and buttress elements could lead to failure. In contrast, a flaw of similar magnitude beneath or within a concrete gravity dam would likely go unnoticed, as the load that could not be supported by that area of the dam would be easily transferred to adjacent monoliths.

It is also well recognized that most concrete gravity dam failures relate to foundation issues. Douglas et al. (1998) reported that for 10 concrete gravity dam failures, six were related to foundation issues, one was related to a structural issue, one was related to an appurtenant structure, and two were classified as unknown. Following the publication of their report, the Camara Dam in Brazil, which was an RCC gravity dam, failed due to foundation problems. Thus, the available evidence shows that seven of eleven concrete gravity dam failures were foundation related.

Hansen and Nuss (2011) summarise the seismic performance of concrete gravity and buttress dams that have experienced earthquakes with peak ground accelerations greater than 0.3 g. Observations from the research indicated that:

- Well-constructed RCC dams performed no differently in earthquakes to dams built of conventionally placed mass concrete.
- Seismic ground motions can be amplified significantly at the dam crest, with an excess of 2 g being recorded at some dams. Damage to equipment and buildings from high accelerations at the dam crests was evident and highlighted the need for careful consideration of amplified ground motions during design.
- Rock slope failures from abutments and onto access roads caused significant damage and delayed access to some sites.
- Cracking in dams occurred at changes in geometry, highlighting the importance of avoiding the inclusion of such features wherever possible.
- Concrete buttress dams developed horizontal cracks at high elevations where there were significant changes in structure stiffness.
- Foundation fault displacement can result in severe damage if fault movement mitigation features have not been included in the design. Hansen and Nuss (2011) noted that in the one example of a foundation fault displacement, the discharge from the reservoir was limited by the size of the gap created by the block displacement.

Hansen and Nuss (2011) note that a number of features contributed to the lack of complete failures at concrete gravity and buttress dams subjected to large earthquake loads. The features included insufficient durations of strong ground motions, the natural frequency of the dam not matching the frequency of the earthquake, load redistribution in the structure, three dimensional effects, and increases in the tensile strength of the concrete during dynamic loading. These Guidelines therefore promote the adoption of dam design features that provide increased resilience.

The mechanisms associated with common potential failure modes for concrete gravity and buttress dams and their foundations are outlined in Table 6.8.

Table 6.8: Initiating mechanism for potential failure modes for concrete gravity and buttress dams

Initiating mechanism	Common causes
Sliding along concrete lift joints in the dam, or cracked surfaces in the dam	Poor lift joint bonding, high uplift pressures, insufficient shear strength
Structural failure	Deterioration in concrete quality, overstressing of buttresses during cross valley seismic loads
Sliding along the concrete/foundation interface, or planes of weakness in the dam foundation	High uplift pressures, insufficient shear strength, inappropriate foundation treatment
Piping of foundation materials	High gradients through foundation, lack of or inappropriate foundation treatment

6.8.3 Loading conditions

Loading conditions for the design and rehabilitation of concrete dams are presented and discussed in various ANCOLD guidelines, CDA (2013), and various USACE and USBR engineering manuals.

Loading conditions that should be considered in the design, evaluation, or rehabilitation of concrete gravity and buttress dams, along with the general performance criteria for each loading condition, are similar to those outlined in section 6.5.3 for an embankment dam. Comments on each loading condition and examples of each loading condition for concrete gravity and buttress dams are provided below.

- During normal loading conditions, the behaviour of the dam should remain in the linearly elastic range. In a rigid body analysis, the normal loading condition should include the consideration of dead loads, hydrostatic loads (headwater and tailwater), loads imposed by silt deposition upstream of the dam and backfill materials adjacent to the dam, internal and external uplift pressures, and temperature effects.
- During unusual loading conditions, minor non- linear behaviour of the dam is acceptable; however, any necessary repairs should be minor. Any rigid body or cracked base analysis should include the consideration of all the loads outlined above for the normal loading condition in combination with the OBE, including hydrodynamic loads, and in combination with an appropriate reduction in the efficiency of underdrains (if present).
- During extreme loading conditions, non-linear behaviour of the dam is acceptable provided the overall performance criterion of safely retaining the reservoir is met. Any rigid body or cracked base analysis should include the consideration of all the loads outlined above for the normal loading condition in combination with the IDF and in combination with the SEE including hydrodynamic loads. The dam-foundation interaction should also be considered. The analysis should also address the post-SEE condition, taking into account an appropriate reduction in the efficiency of underdrains (if present), the loss of any cohesion at the dam/foundation interface, an appropriate reduction in the friction angle at the dam/foundation interface, and any increase in the horizontal load that could result from liquefaction of silts deposited immediately upstream of the dam.
- Where tensile stresses are computed at the foundation interface and at unbonded lift surfaces in an existing dam it should be assumed that a crack or joint opening will develop in the non-compression area. The stress and stability calculations must be iterated to determine the extent of the non-compression, recognizing increased uplift due to reservoir pressure intrusion into the zone of non-compression. The modified uplift profiles for a cracked base with the compression not extending beyond drains, and with the zero compression extending beyond the line of drains are given in Figures 3.2 and 3.3 of ANCOLD (2013).

6.8.4 Stability and structural performance criteria

As for embankment dams, potential stability failures for concrete gravity and buttress dams under different loading conditions are usually assessed in terms of minimum factors of safety. The factors of safety generally relate to sliding stability and compressive stresses at the dam toe.

The primary performance indicator for concrete gravity and buttress dams is their stability against a sliding failure, either along the dam/foundation contact or along a plane of weakness in the dam or dam foundation. Recommended minimum sliding factors of safety (defined as the available shear strength divided by the net driving force) for limited equilibrium stability analysis of concrete gravity and buttress dams are provided in Table 6.9. The tabulated figures and accompanying notes generally reflect the material included in ANCOLD (2013) and are similar to those included in CDA (2013). Additional information relating to failure surfaces, concrete strengths, dam/ foundation interface strengths, and dam foundation strengths is provided by ANCOLD (2013). Note that the recommended factors of safety relate to sliding failures along the dam/foundation contact, along a plane of weakness in the dam, or along a joint set or other plane of weakness in the dam foundation. Guidance on geotechnical investigations to ascertain the required parameters for the analyses is provided by ANCOLD (2020).

Table 6.9: Recommended minimum sliding Factors of Safety for concrete gravity and buttress dams

Loading condition	Minimum sliding Factor of Safety		
	Friction and cohesion present		Friction only present
	Not well defined[1,2,6]	Well defined[1,2,6]	Well defined[2,6,9]
Normal	3.0[4]	2.0[4]	1.5[4]
Unusual	2.0[4]	1.5[4]	1.3[4]
Extreme – Flood	1.5[4]	1.3[4]	1.1[4]
Extreme – Earthquake	(note 10)		
Post-earthquake		1.2 (note 11)	

Notes on the use of this table:

1. Given the significant impact a very small amount of cohesion can have on shear resistance, the recommended minimum sliding factors of safety for friction and cohesion should be used with extreme caution.

For stability within the body of the dam and at the dam/foundation interface:

2. 'Well defined' means that a sufficient number of tests have been completed to define the strength parameters with reasonable certainty (i.e., the assumed strength parameters should be exceeded by 80% of the test results from a test regime involving an appropriate number of tests).
3. For the friction only condition ANCOLD (2013) assumes residual friction. ANCOLD (2013) Appendix A has a significant discussion on selection of strength parameters and acceptance criteria for use in sliding stability Factors of Safety. They offer an alternative Table 2A recommended from FERC (2016) for safety factors if cohesion is not relied on for stability. ANCOLD (2020) also has commentary on requirements to satisfy "well defined strengths". In some cases Ø for concrete could be significantly lower than 45o (e.g. cracked surfaces, open joints, lift joints incorporating a cement slurry bond layer). In such cases and in all cases where friction alone controls the stability of the dam, Ø should be determined from laboratory tests.
4. The minimum sliding factors for friction and cohesion assume that the sliding surface will pass through intact concrete or well-prepared construction joints.
5. In assessing the strength of the dam/foundation interface consideration needs to be given to the thoroughness of the foundation clean-up and whether the strengths of parts of the foundation below the interface may control the stability. For the 'well defined' case for existing dams, strength tests should be carried out on core samples taken from the interface zone (typically a few metres below the interface surface).

For stability within the foundation of the dam:

6. 'Well defined' means that there is good exposure of the foundation material at the dam site and that there is sufficient reliable data to establish the existence and persistence of foundation discontinuities (e.g., faults, joints, bedding plane shears) and define the foundation strength parameters with reasonable certainty (i.e., the assumed strength parameters should be exceeded by 80% of the test results from a test regime involving an appropriate number of tests).
7. Recommended shear strength parameters for foundation discontinuities, for incorporation in stability assessments, are listed in the following table from ANCOLD (2013):

Discontinuity type	Typical feature	Strength parameters (refer note a)
Clean discontinuity (no previous displacement)	Clean joint Bedding plane	f(øb, i) c'=0
Thick infilled discontinuity (no previous displacement)	Infilled joint Infilled bedding plane	f(ø) of infill material (note b) c'=0
Discontinuity with previous displacement	Shears Faults	f(ør) of infilling and f(i) of wall rock c'=0
Multiple discontinuity	Highly jointed rock mass	mb, s, a, σc

Notes:

a. Strength parameters:

f	function of
c'	cohesion (at zero normal stress)
øb	basic friction angle (for wet surfaces)
ør	'effective' residual friction angle
i	average roughness angle
mb, s, a	Hoek-Brown Criterion (Hoek and Brown, 1997) parameters
σc	uniaxial compressive strength of intact rock

b. Test to be carried out on remoulded samples; ø to be based on peak strength under drained conditions; c' to be neglected.

8. The minimum sliding factors assume the adoption of reasonably conservative strengths. For the friction only condition the adopted strength should be at or near the lower bound of good quality test data.
9. Where weak surfaces (e.g., bedding plane shears) are present in the foundation, the actual strength will usually be the frictional strength. In such situations the criteria should be met using the frictional strength.

For stability within the body of the dam, at the dam/foundation interface, and within the foundation:

10. The earthquake load case is used to determine the post-earthquake condition of the dam. In line with US recommendations (e.g., FERC, 2016) a minimum Factor of Safety is not given. If sliding assessments indicate displacement, then the Designer needs to consider the amount of displacement that has occurred along the surface analysed whether it is at the dam/foundation contact, in the dam or in the foundation.
11. For the post-earthquake load case the minimum sliding factor should not be less than 1.2 for the friction only 'well defined' case. If the sliding factor falls below 1.1 there is a high likelihood of failure given that the friction only strength condition has been reached. In such a case the dam should be remediated as a matter of urgency to meet the minimum sliding factor recommended for the 'normal' loading condition.

If a cracked base analysis indicates unacceptable results, the stability analysis can be completed under the assumption that the base cracks completely through. This approach will help determine whether the dam remains stable under the limit case of complete cracking.

Other performance indicators include the position of the resultant force, tensile stresses at the dam heel, and compressive stresses at the dam toe. Recommended criteria for the position of the resultant force and maximum compressive stresses for rigid body analysis of concrete gravity and buttress dams are provided in Tables 6.10 and 6.11. The recommended criteria reflect those included in CDA (2013) and USACE (1995).

Table 6.10: Recommended position of the force resultant for concrete gravity and buttress dams

Loading condition	Position of the force resultant
Normal	Preferably within the middle third of the base (i.e.100% compression). For existing dams, it may be acceptable to allow a small percentage of the base to be under zero compression if all other performance criteria are met [1]
Unusual	75% of the base should be in compression and all other performance criteria should be met.
Extreme	Within the base and all other performance criteria should be met [2]

Notes:

1. It is important that all possible failure modes are addressed under a potential cracked base scenario wherever tensile stresses are present at the dam/foundation contact or along a plane of weakness in the dam foundation.
2. Rocking may occur under extreme earthquake loads and some permanent displacement could result. The Designer needs to determine whether this occurs and, if it does, demonstrate that the reservoir is not released and that post-earthquake stability is adequate.

Table 6.11: Recommended maximum stresses for concrete gravity and buttress dams

Loading condition	Normal compressive stress
Normal	$<0.3f'_c$
Unusual	$<0.5f'_c$
Extreme - Flood	$<0.5f'_c$
Extreme – Earthquake	$<0.9f'_c$
Post-earthquake	$<0.5f'_c$

Notes on the use of this table:

1. In addition to the above, the maximum foundation bearing pressure should be less than the allowable bearing pressure, as determined by an appropriately qualified Technical Specialist, for normal and unusual loads, and less than 1.33 times the allowable bearing pressure for extreme loads.
2. Within the body of a dam, tensile stresses during normal loading conditions may be acceptable so long as the limits of $0.1f'_c$ and $0.05f'_c$ (where f'_c is the compressive strength of concrete) within the concrete mass and at lift joints, respectively, are not exceeded and all other performance criteria are met. Tensile stresses during earthquake loading conditions may be acceptable so long as they do not exceed 1.5 times the above limits and all other performance criteria are met.
3. In the absence of specific testing, tensile strengths at the dam/foundation interface, and along defects in the foundation, should be assumed to be zero.

Relatively long and straight concrete gravity and buttress dams can be analysed as idealised two- dimensional elements. Concrete gravity dams that are built in a narrow canyon, or which are curved in plan and rely on abutments to transfer loads when cantilever capacities are exceeded, should be analysed in three dimensions. Likewise, blocks or wedges in the foundations or beneath the abutments of concrete gravity dams should be analysed in three dimensions.

As stated for embankment dams, higher sliding factors of safety and lower compressive stresses than those listed in the above tables may be necessary if there are high levels of uncertainty in the inputs to the rigid body analyses, particularly in the strength of foundation materials.

At large seismic loads, rigid body stability analyses are likely to result in factors of safety <1.0, indicating that the applied loads exceed the resisting loads. In such cases, analyses that allow the determination of the amount of displacement that occurs during the earthquake loading should be completed. The Designer should then use the resulting information to assess whether or not the dam can safely retain the reservoir during, and immediately following, the earthquake event and determine its post-earthquake condition for incorporation in an analysis of its stability during an aftershock.

Dam block displacements may occur during a large earthquake. If linear elastic analyses indicate that little or no displacements occur, then those analyses can be considered appropriate. However, if the displacements are large enough to result in material property changes, non-linear analyses should be completed. Both analyses should be completed using a number of time history records to effectively analyse dam performance.

The results of non-linear analyses should be validated against the results of alternative analyses, including pseudo-static and linear elastic analyses. In addition, the sensitivity of the results to variations in the assumed model parameters should be assessed. The non-linear analyses should generally consider the potential for dam-foundation interaction and hydro-dynamic loads resulting from dam-reservoir interaction.

As stated in section 6.1, the post-earthquake criteria are only applicable for the temporary condition, immediately post-earthquake, until satisfactory repairs have been completed and acceptable normal operating dam safety criteria have been restored. During this period, interim risk reduction measures may be necessary to provide an acceptable level of dam safety until satisfactory repairs have been completed.

6.8.5 Design details

There are a number of design details for concrete gravity and buttress dams that can affect dam safety. They include:

- The geometry of the dam.
- The treatment of foundation defects.
- Shear transfer between dam blocks.
- The design of the upstream face to reduce the likelihood of tensile cracking and seepage.
- Providing suitable drainage facilities for the control of uplift pressures.
- Providing sufficient freeboard and appropriate crest details.
- Horizontal lift joints.
- Construction, contraction, expansion, and isolation joints.
- The design of surfaces for high velocity flow.
- The concrete mix design including the compressive strength, tensile strength, and modulus of elasticity of the concrete.
- The use of post-tensioned anchors to enhance the stability of dams.

6.8.5.1 Dam geometry

Concrete gravity dams are usually constructed in discrete dam blocks. While there may be construction expediencies that make this advantageous, the primary reason for discrete blocks is to provide contraction joints for thermal expansion and contraction during construction and in long-term operation. In conventional mass concrete gravity dams, the transverse joints are constructed at formed faces; however, in RCC dams the joints are typically created as the RCC is placed.

Experience gained from RCC dams constructed without transverse joints demonstrates that transverse cracking will occur, often induced at undesirable locations due to foundation or other irregularities. In uniform RCC dams, where there are no irregularities, transverse joints have been observed to form at about 15 m intervals. When joint control is exercised, joints are typically constructed at 20 m to 30 m intervals; however, a thermal assessment of the structure should be completed to determine the ideal spacing.

Sliding stability requirements and the available shear strength at the foundation interface are likely to dominate the base dimensions. High tensile stress zones should be avoided wherever possible and, if they are unable to be avoided, the dam shape should be adjusted to reduce the tensile stresses as much as reasonably practicable. Cracks should be expected to propagate from high tensile stress zones and, as such, the following should be considered carefully during dam design:

- Geometric modification of the heel of a dam to reduce tensile stresses, or the incorporation of higher strength concrete in the heel of a dam, to increase the tensile capacity of the concrete. Higher strength conventional concrete may be appropriate in the heels of RCC dams to increase the tensile capacity of the concrete.

- The avoidance of prominent changes of slope and sharp discontinuities in the foundation profile to reduce the likelihood of high tensile stress zones.
- The avoidance of sharp changes in the upstream or downstream face geometry to reduce the likelihood of high tensile stress zones. High tensile stress zones often result from changing the face geometry to achieve a wider dam crest for a road across the top of a dam. Koyna Dam in India cracked at a change in slope on its downstream face during an M6.5 earthquake in 1967.
- The seismic design of concrete buttress dams. Changes in cross-sections within face slabs and floor slabs are particularly vulnerable to cracking if not detailed for high seismic loads, and slender monoliths have poor seismic resistance to cross valley seismic ground motions. Sefid Rud dam in Iran suffered cracking at the top of its buttress monoliths during an M7.3 earthquake in 1990.

6.8.5.2 Foundation defects and discontinuities

The identification and appropriate treatment of foundation defects is discussed in section 6.4 of these Guidelines. In addition to the measures required for the control of seepage flows, the design challenges for concrete gravity and buttress dams include the identification of all defects and discontinuities that could affect dam stability, the determination of their shear strengths, and the design of any necessary strengthening works to ensure adequate reserves of stability. The assessment of the foundation is one of the most important aspects of the design and safety evaluation of concrete dams as most historical concrete dam failures have resulted from foundation weaknesses (e.g., Sheffield Dam and Morris Shepherd Dam in the USA).

The design of new dams, the safety evaluation of existing dams, and the design of rehabilitation works for existing dams should consider the sliding resistance along any identified joint or shear plane with an orientation that could influence the development of a sliding failure. The Designer should also consider the stability of any combinations of joint or shear planes that form unstable wedges of rock and could result in dam block displacements.

The determination of foundation shear strengths can be difficult and, while the adoption of conservative values from published information may be sufficient for Low PIC dams, foundation shear strengths for Medium and High PIC dams should be based on the results of laboratory tests. Fell at al. (2015) includes recommended practices for the assessment of shear strengths in clean discontinuities, infilled joints and seams showing evidence of previous displacement, thick infilled joints, seams or extremely weathered beds with no evidence of previous displacement, and jointed rock masses with no persistent discontinuities.

6.8.5.3 Shear transfer

Shear keys are resilient features that can provide some load transfer between dam blocks.

For new designs, the stability of straight gravity dam blocks designed by 2-D analysis should not be reliant on load transfer with their neighbouring dam blocks. However, for existing dams constructed with shear keys, it is appropriate to assess and consider the load transfer that can be accomplished during extreme seismic loads or high flood conditions. It is quite likely that load transfer between monoliths and/or the interlocking of monoliths upon initiation of any sliding movements have contributed to the good stability record of concrete gravity dams.

6.8.5.4 Upstream face

The concrete specification for the upstream face of a concrete gravity or buttress dam should encourage long-term durability, crack control and water tightness. Higher strength conventional concrete is commonly used in the upstream face of a mass concrete or buttress dam. For an RCC dam, conventional concrete or grout enriched RCC is often used at the upstream face.

The concrete specification at the upstream heel of a dam should reflect the size and extent of any tensile stresses that develop during unusual or extreme loading conditions. This is particularly important at sites located in areas of high seismic risk. For RCC dams located in areas of high seismic risk, conventional concrete should be used in the upstream heel rather than grout enriched RCC.

Waste water dams or water supply dams in dry regions may require upstream membranes to achieve very low specified seepage requirements.

The durability of the upstream face also needs to be considered if the reservoir is highly acidic. The treatment of water flowing into the reservoir may be an option at a mine site. However, if such a system is unavailable, not applicable, or unreliable, the Designer must demonstrate that the structure meets durability criteria. Sulphate-resisting cements or upstream membranes may provide sufficient resistance. The design life of the solution and the practicality of repair or replacement during the life of the dam should be considered.

6.8.5.5 Drainage facilities

Drainage facilities are frequently installed in concrete dams for the control of uplift pressures at the foundation contact, along a plane of weakness in the dam foundation, and along concrete lift joints. Guidelines for the estimation of uplift pressures are included in the literature (e.g., CDA, 2013; Fell et al., 2015; ANCOLD, 2013). Such guidelines are appropriate for the design of new dams; however, stability assessments for existing dams can be based on measured uplift pressures with appropriate allowances for seepage trends.

Points that should be considered during the design of a drainage system include:

- The location and depth of the foundation drains. Ideally, foundation drains should be located in a gallery, drilled to intersect defects, and extend beneath any potential failure surface. They should also be oriented to intersect foundation defects, installed downstream of any grout curtain, and drilled following the completion of any foundation grouting.
- The spacing and diameter of the foundation drains. Drain spacing will be somewhat dependent on the foundation geology; however, for an efficient drainage system, the spacing should not exceed 3 m. The diameter of the drains should be a minimum of 75 mm to enable drain maintenance.
- The potential for erosion of foundation materials into the drillholes. If any drillhole intersects foundation materials considered likely to erode into the hole, then a suitable filter and screen standpipe should be installed. The filter should be removable for maintenance and replacement.
- The location, spacing and diameter of internal drains. Ideally, they should be located close to the upstream face of the dam and drain into a drainage gallery. Their spacing should be sufficient to ensure they don’t encourage longitudinal cracking and their diameter should be at least 150 mm to minimise the potential for leakage to bypass the drains and to facilitate future cleaning.
- The watertightness of drainage galleries. Drainage galleries should preferably be located within the dam body and not in an area where high tensile stresses could result in the development of a crack between the upstream face and the gallery. Unless suitable mitigating measures can be reliably installed, the watertightness of a gallery located externally at the heel of the dam could be compromised by dam block displacement.
- The reliability of any installed dewatering facilities necessary to pump water from drainage galleries to the tailwater. If the post-earthquake stability of the dam relies on the effective control of uplift pressures, any pump facilities and their associated pipelines should be designed to remain operational following the SEE.
- The future maintenance of the drainage system. Without ready access to the drainage system for inspection and maintenance, its long-term effectiveness cannot be assured, and uplift pressures should be assumed to vary linearly between full headwater pressure at the heel of the dam and full tailwater pressure at the toe of the dam.
- Instrumentation is required to monitor the ongoing effectiveness of the drainage system and confirm design assumptions.

6.8.5.6 Freeboard and crest details

The recommendations in the general section on freeboard for all dams (section 4.5.4) are applicable for concrete gravity and buttress dams. However, concrete gravity and buttress dams can usually accommodate some overtopping without serious damage and, as such, the freeboard provisions can be somewhat less than those detailed in the section on freeboard for embankment dams (section 6.5.5.1).

The quality of the foundation and abutment material in contact with or impacted by overtopping flow is a critical factor in determining the amount of overtopping flow that can be safely discharged without threatening the safety of the dam. From a dam safety perspective, it is important to provide sufficient freeboard to ensure that the safety of the dam, its abutments, and appurtenant structures is not compromised during the IDF. This freeboard may also be important for the continued operation of appurtenant structures, such as spillways, during the IDF. In some cases, additional freeboard provisions may be required by the Owner to meet asset management objectives.

The crest width for the non-overflow section of a concrete gravity or buttress dam is usually set by stability considerations and any access requirements for the maintenance and repair of appurtenant structures (e.g., spillway gates).

6.8.5.7 Horizontal lift joints

Concrete gravity dams and buttress monoliths are generally constructed in horizontal layers. RCC dams may also be constructed using sloping layers, but the resulting lift joints are essentially the same as horizontal lift joints. The bond at horizontal lift joints is critical to:

- Prevent the development of potential sliding failure modes in the dam body.
- To provide adequate tensile resistance between the concrete layers.
- To prevent the development of potential horizontal seepage paths that could result in high uplift pressures within the dam body.

Lift joint preparation is a key factor in achieving adequate bond between concrete layers. Lift joint surfaces should be clean and free of loose material and dirt. Improved bond conditions can be achieved through high pressure water cleaning to ensure the removal of concrete laitance and green cutting of previously placed layers. For RCC dams, the Designer must specify the parameters for the following joint preparation options:

- Fresh RCC directly on a compacted RCC layer. Time limits and temperature parameters before initial set are required and, during this time window, a new layer may be placed over a compacted layer of RCC. After initial set, a cold joint develops which requires treatment with a mortar bedding layer. The time and temperature parameters should be developed and demonstrated during an RCC construction trial.
- Mortar bedding on a cold joint immediately before fresh RCC. The Designer may choose this procedure for all lift joints.

The bond at horizontal lift joints should be confirmed by obtaining cores from a trial RCC embankment and testing them in a laboratory. Bond quality checks, by obtaining cores and testing them in a laboratory, should also be part of the construction quality assurance process.

6.8.5.8 Construction, contraction, expansion and isolation joints

Joints are provided in concrete gravity and buttress dams to minimise cracking and the effects of cracking on relative movement. They include inclined or vertical construction joints for practical concrete construction, contraction joints to regulate the locations of cracks, expansion joints to accommodate volumetric changes in adjacent concrete blocks, and vertical isolation joints to enable movements at specific locations (e.g., directly above a fault in the dam foundation).

All joints require seals to limit joint leakage. The seals must be strong enough to withstand rough treatment during construction and all applied water pressures, flexible enough to accommodate relative movements between adjacent concrete sections, and durable enough to remain effective during the design life of the dam.

A wide range of sealing materials is available. Successful joint systems are heavily reliant on the selection of the most appropriate seal and the design detail for the joint. ICOLD Bulletin 57 (ICOLD, 1986c) provides guidelines for the selection of the most appropriate material and its installation in vertical and horizontal joints.

6.8.5.9 High velocity flow

Surfaces exposed to high velocity flows require considerable attention to detail by the Designer and strong quality assurance requirements during construction. Hydraulic flow characteristics need to be carefully modelled, and negative pressures should be avoided to reduce the risk of cavitation. High quality surface finishes are often required to avoid cavitation type erosion.

Conventional concrete should be the minimum specification for high velocity flow surfaces, including spillway surfaces on RCC dams.

A relatively small amount of bed load can be highly abrasive to concrete surfaces. Silica fume in the concrete mix provides a more durable surface and steel plates are sometimes installed around gates and abrupt transitions to minimise the potential for erosion.

While numerical hydraulic modelling can be used, the limitations of these techniques must be understood. Physical models may be necessary to ensure hydraulically efficient spillway geometry; however, they should only be undertaken by experienced modellers.

6.8.5.10 Concrete mix design

Concrete mix design requires care to ensure the concrete is a reliable construction material and that it delivers the specified strength and durability objectives. In many cases, on-site concrete manufacture will be required. The batching plant for conventional concrete and any plant for the manufacture of RCC must be appropriate for the mix designed and have a production rate that exceeds the maximum required delivery rate at critical times in the construction programme. The Designer should record the results of concrete mix trials, to demonstrate the suitability of the selected mixes, and should receive concrete plant acceptance testing quality control records as evidence that the concrete mixes meet the design requirements.

The control of temperature rise is important in mass concrete and RCC construction. The Designer must specify the mix design, acceptable cement properties, acceptable pozzolanic materials (non-cementitious materials that can replace cement but achieve the specified strength and durability requirements) and acceptable temperature parameters. Methods for determining mass concrete mix proportions are provided in ACI (1989) and methods for determining RCC mix proportions are provided in ACI (1988).

Many concrete gravity dams, including RCC dams, often require cooling and insulating systems for the management of thermal effects during construction. ANCOLD (2013), ACI (1998) and ICOLD Bulletins 107, 126 and 136 (ICOLD, 1997; 2003; and 2009a) include guidelines for assessing the need for temperature control, and outline alternative construction practices for the prevention of uncontrolled cracking.

Consideration should be given to the following during the design and production of concrete mixes for concrete gravity and buttress dams:

- The need for low heat cements.
- The need for low alkali cements to reduce the risk of alkali silica reactions.
- Sulphate resisting cements if there is a risk of exposure to highly acidic water.
- The benefits of pozzolan replacements (Class F (low lime) fly ash is preferred).
- The necessity for silica fume or other additives to meet durability requirements (e.g., for high velocity water surfaces).
- Maximum placing temperatures.
- Minimum placing temperatures.
- Target slumps.
- Quality control test target ranges and acceptance levels.

Production quality control tests should include:

- Consistency tests such as slump tests for mass concrete and Vebe test times for RCC.
- Compressive strength cylinders.
- Tensile tests of cores through lift joints.

6.8.5.11 Post-tensioned anchors

Post-tensioned anchors should not be included as a primary stability means in the design of new concrete dams, especially for normal operating loads. However, anchors are often utilised to raise or improve the stability of existing concrete dams. McInerney et al. (2007) includes a detailed account of present-day practice in North America and provides a benchmark for the design and assessment of stressed anchors for dam projects in New Zealand. Recognising the conclusions reached in the study completed by McInerney et al. (2007), these Guidelines recommend that:

- Without clear evidence that an existing post-tensioned anchor installation is performing as intended, it should be assumed to have no value. Techniques available at the time of preparing these revised Guidelines, utilising indirect methods of non- destructive testing, are unlikely to provide sufficient evidence that anchors are performing as intended.
- New and replacement anchors should be re-stressable post-tensioned anchors incorporating double corrosion protection or an encapsulated tendon. This protection system (Class I protection) encases the prestressing steel inside a plastic encapsulation filled with grout or a corrosion inhibiting compound. An epoxy coated strand tendon grouted into a drillhole that successfully passes a specified water pressure test (refer Post-Tensioning Institute, 2004) also satisfies the requirements for a Class I protection system.
- All post-tensioned anchor installations should be designed in a manner which enables future regular inspection of the anchor heads and load testing of the anchors. Inspection and load testing frequencies, and sample numbers, for anchor installations should reflect the consequences of anchor failure. For a Low PIC dam, it may be appropriate to only inspect and test 10% of the anchors at a frequency of 5 years, while at a High PIC dam it may be appropriate to inspect and test 33% of the cables at a frequency of 5 years.

6.9 Concrete arch dams

6.9.1 Introduction

Arch dams (Figure 6.9) are usually built as independent cantilever blocks separated by vertical contraction joints. The vertical contraction joints are then grouted, at an optimal ambient temperature, so that the structure will act as a monolithic system to distribute loads from the shell to the abutments. Therefore, the construction details and the integrity of the abutments are critical to the safety of the dam.

Arch dams are often classified on the basis of their thickness and geometry (e.g., variable thickness, double curvature, etc.). Gravity arch dams (gravity dams curved in plan) that rely on their curvature to distribute loads into the abutments through arch action require the consideration of both gravity dam and arch dam actions.

Figure 6.9: Concrete Arch Dam – Moawhango Dam (provided by Genesis Energy)

Many early arch dams were designed assuming all horizontal water loads were transferred horizontally to the abutments by arch action and all vertical loads (self-weight and water loads on sloping upstream faces) were carried vertically to the foundations by cantilever action. In some cases, arch thicknesses were determined using the thin cylinder formula while, in other cases, the thicknesses were determined by elastic arch analyses.

Present day arch dam analysis assumes that the horizontal water load is divided between the arches and cantilevers so that the calculated arch and cantilever deflections are equal at all conjugate points in all parts of the dam. The distribution of stresses in an arch dam varies with the horizontal curvature, the shape of the vertical cross-sections, the general dimensions of the structure, and the uniformity of the foundation and abutment profile. For arch dam sites that do not include pronounced irregularities:

- Maximum cantilever stresses usually occur at the base of the highest cantilever. Maximum compressive stresses usually occur in the downstream face at the base of the dam, and tensile stresses often occur in the upstream face at the base of the dam and in the downstream face towards the centre and top of the dam.
- Arch stresses are usually higher towards the centre and top of the dam, and maximum arch stresses usually occur at the crown and abutment sections. At the crown section, high compressive stresses usually occur in the upstream face of the dam and relatively low compressive or tensile stresses in the downstream face. At the abutment sections, stress conditions are usually reversed.

While the design and analysis of arch dams is reasonably straightforward, they do demand higher than normal analytical skills and a sound understanding of the variables that can affect the performance of the dam (e.g., foundation strength and deformation properties, foundation defects and discontinuities, concrete strength and deformation properties, the effects of temperature variations, etc.).

The following subsections discuss potential failure modes for arch dams, loads and loading conditions which must be taken into account during their design, evaluation, and rehabilitation, as well as recommended performance criteria for arch dams. Defensive design details that are important to dam safety are also discussed.

6.9.2 Potential failure modes

Section 6.2.2 provides additional information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams. The general comments on the identification and evaluation of potential failure modes included in section 6.5.2 for embankment dams are also relevant to concrete arch dams.

Fell et al. (2015) includes some statistics on dam failures which highlight that failure rates between 1900 and 1975 for concrete arch dams were 0.7% of dams built. This figure can be compared with the failure rate quoted for embankment dams, over the same time period, which was 1.2% of dams built.

Hansen and Nuss (2011) summarise the seismic performance of seven arch dams experiencing earthquakes with peak ground accelerations greater than 0.3 g. Ghanaat (2004) discusses evidence of failure mode development at Pacoima Dam after the 1971 San Fernando earthquake and the 1994 Northridge earthquake. Observations from the literature indicates that:

- Contraction joints have the lowest tensile capacity and should be expected to open and close during extreme earthquake shaking.
- Seismic ground motions can be amplified significantly at the dam crest. Damage to equipment and buildings from high accelerations at the dam crests highlights the need for careful consideration of amplified ground motions during design.
- Rock mass security in the abutments is vitally important. Ghanaat (2004) noted that rock anchors installed in the abutment at Pacoima Dam after the 1971 earthquake limited the movement of a key rock mass supporting the thrust block during the 1994 earthquake.
- Rock slope failures from abutments and onto access roads can cause significant damage and delayed access to some sites.

Hansen and Nuss (2011) note that a number of features contributed to the lack of complete failures at concrete dams subjected to large earthquake loads. The features included insufficient durations of strong ground motions, the natural frequency of the dam not matching the frequency of the earthquake, load redistribution in the structure, three dimensional effects and increases in the tensile strength of the concrete during dynamic loading. These Guidelines therefore promote the adoption of dam design features that provide increased resilience.

As with concrete gravity dams, known arch dam failures are primarily related to foundation issues. The mechanisms associated with common potential failure modes for arch dams and their foundations are outlined in Table 6.12.

Table 6.12: Initiating mechanism for potential failure modes for arch dams

Initiating mechanism	Common causes
Structural failure along cracked surfaces in the dam	Insufficient shell thickness for applied loads, defects in concrete, deterioration in concrete quality
Structural failure of a cantilever	Excessive opening of vertical contraction joints accompanied by cantilever tensile cracking (e.g., along horizontal lift joints)
Loss of abutment support	Movement of abutment rock wedges formed by discontinuities
Sliding along the concrete/foundation interface, or planes of weakness in the dam foundation	High uplift pressures, insufficient shear strength, inappropriate foundation treatment
Piping of foundation materials	High gradients through foundation, lack of or inappropriate foundation treatment

6.9.3 Loading conditions

Loading conditions that should be considered in the design, evaluation, or rehabilitation of concrete arch dams, as well as general performance criteria, are similar to those outlined in section 6.8.3 for a concrete gravity or buttress dam. Comments on each loading condition and examples of each loading condition for a concrete arch dam are provided below.

- During normal loading conditions, the behaviour of the dam should remain in the linearly elastic range. The normal loading condition should include the consideration of dead loads, hydrostatic loads (headwater and tailwater), loads imposed by silt deposition upstream of the dam and backfill materials adjacent to the dam, and temperature effects (normal, maximum and minimum concrete temperatures).
- During unusual loading conditions, minor non- linear behaviour of the dam is acceptable; however, any necessary repairs should be minor. Analysis should include the consideration of all the loads outlined above for the normal loading condition in combination with the OBE, including hydrodynamic loads, and in combination with an appropriate reduction in the efficiency of underdrains (if present).
- During extreme loading conditions, non-linear behaviour of the dam is acceptable. Analysis should include the consideration of all the loads outlined above for the normal loading condition in combination with the IDF and in combination with the SEE including hydrodynamic loads. The analysis should also address the post-SEE condition, taking into account the possibility of open joints, the possibility of movement in the abutments, the loss of any cohesion at the dam/foundation interfaces, an appropriate reduction in the friction angle at the dam/foundation interfaces, and any increase in the horizontal load that could result from liquefaction of silts deposited immediately upstream of the dam.

6.9.4 Stability and deformation performance criteria

Sliding stability is important for concrete arch dams, not along the foundation contact but at abutment areas where foundation wedges formed by combinations of faults, shears and/or joint sets can affect the overall functionality of the dam by reducing the ability of the abutments to accept the arch thrusts. Compared to concrete gravity and buttress dams, the design and assessment of concrete arch dams is usually more dependent on allowable deflections and concrete stresses.

Arch dams rely on abutments remaining intact to carry the significant thrust forces. Non-linear behaviour may occur in the dam or abutment during extreme earthquakes but the over-riding performance criteria of safely retaining the reservoir applies.

A full design or safety evaluation for an arch dam will require a three-dimensional analysis. Simplified structural theory may be possible for initial assessments of single curvature structures, but a more complex analysis should be completed for final design. Potential abutment wedges need to be identified and analysed for stability using the resultant forces from dam analyses. Guidelines for the design and evaluation of concrete arch dams are included in USACE (1994) and FERC (2016).

Recommended criteria for the design and evaluation of concrete arch dams are provided in Table 6.13. The recommended criteria have been developed from those included in USACE (1994) and FERC (2016).

Table 6.13: Recommended performance criteria for concrete arch dams

Loading condition	Performance criteria		
	Maximum compressive stress	Maximum tensile stress	Minimum Factor of Safety against sliding
Normal	$0.25f'_c$	f'_t	2.0
Unusual - Flood	$0.4f'_c$	f'_t	1.3
Unusual - Earthquake	$0.4f'_{cd}$	f'_{td}	1.3
Extreme - Flood	$0.67f'_c$	f'_t	1.1
Extreme – Earthquake	$0.67f'_{cd}$	f'_{td}	1.1[3]
Post-earthquake	$0.25f'_c$	f'_t	2.0
Construction (before grouting)	For all cantilevers the resultant should be located within the base and the maximum tensile stress should be f't		

Notes on the use of this table:

1. f'_c is the 28-day compressive strength of the concrete and should be >28MPa, and f'_t is the tensile strength of the concrete. For Low PIC dams f'_t can be assumed to be 10% of f'_c. For Medium and High PIC dams f'_t should be determined from the results of splitting tensile tests.
2. f'_{cd} and f'_{td} are the dynamic compressive and tensile strengths based on the results of laboratory tests for the appropriate rate of loading as determined from the dynamic analysis.
3. For Medium and High PIC dams subjected to extreme seismic loading, time history response analyses should be completed to determine abutment and foundation stability. The Factor of Safety against sliding will vary with time and may be less than 1.0 for one or more cycles if the resulting cumulative displacement is very small and tolerable.

6.9.5 Design details

There are a number of design details for arch dams that can affect dam safety. They include:

- The geometry of the dam.
- The treatment of foundation defects.
- Providing suitable drainage facilities for the control of uplift pressures.
- Providing sufficient freeboard and appropriate crest details.
- The details of the contraction joints between the individual cantilever sections.
- The concrete mix design.

6.9.5.1 Dam geometry

Arch dams are usually constructed in discrete cantilever blocks featuring vertical contraction joints that are grouted on completion. A thermal assessment addressing cooling requirements and construction considerations should be carried out to determine the optimal spacing of these contraction joints.

The cantilever blocks need to be designed for construction conditions until the contraction joints are grouted. The cantilever blocks also need to be checked for excessive tensile stresses if analysis shows contraction joint openings during earthquake loading conditions.

Load distribution into the abutments is most important for arch dams and consideration must be given to the integrity of the abutments, the orientations of the arch thrusts into the abutments, and the stability and deformation of the abutments in response to the arch loads. In addition, it is important to define arch thrust loads on any wedges or blocks formed by discontinuities in the foundation or abutments that could displace under the combination of gravity loads, reservoir water pressure loads in the discontinuities, and arch thrusts.

6.9.5.2 Foundation defects

The identification and appropriate treatment of foundation defects is discussed in section 6.4 of these Guidelines. In addition to the measures required for the control of seepage flows, the design challenges for arch dams include the identification of all defects and discontinuities that could affect dam stability, the determination of their shear strengths, and the design of any necessary strengthening works to ensure adequate reserves of stability.

The design should consider the sliding resistance along any identified joint or shear plane with an orientation that could encourage the development of a sliding failure. The Designer should also consider the stability of any combinations of joint or shear planes that form unstable wedges of rock and could result in the loss of support for the dam. The foundation immediately downstream of the dam is the critical zone for an arch dam. It is essential to ensure that this area is not damaged during the construction process.

The determination of foundation shear strengths can be difficult, and the guidelines included in section 6.8.5 for concrete gravity and buttress dams are also applicable to concrete arch dams.

6.9.5.3 Drainage facilities

While uplift is not usually important in thin arch dams, it can be significant in thick arch dams and in the stability of any blocks or wedges that are formed by discontinuities in the foundation and abutments. The guidelines included in section 6.8.5 for drainage facilities in concrete gravity and buttress dams are appropriate for arch dams; however, additional drainage facilities in the abutments are a common requirement for arch dams.

High pressures can occur in the foundations and abutments of arch dams and drainage can be necessary for the control of foundation and abutment stability. In such cases, drainage is often provided by angled drain holes along the foundation contact and adits, driven into the abutments at appropriate locations which recognise the stresses within the rock mass, with curtains of drilled holes connecting the adits.

6.9.5.4 Freeboard and crest details

The recommendations in the general section on freeboard for all dams (section 4.5.4) are applicable for concrete arch dams. However, concrete arch dams can usually accommodate some overtopping without serious damage and, as such, the freeboard provisions can be somewhat less than those detailed in the section on freeboard for embankment dams (section 6.5.5.1).

The quality of the rock at the impact point for the overtopping flow, downstream of the dam, is the key consideration in establishing the amount of overtopping flow that can be safely discharged without undermining the arch dam foundation. From a dam safety perspective, it is important that the foundation material downstream of the dam is accessible for inspection and evaluation. If there are concerns regarding potential erosion that could undermine the dam foundation, it is essential to provide adequate freeboard to ensure the safety of the dam and its abutments during the IDF.

Many arch dams incorporate free overflow spillways along their crests and consideration must be given to the loads that spillway operation may place on the dam, the dissipation of energy and the control of erosion immediately downstream of the dam, and the accessibility to both abutments for ongoing surveillance, monitoring, and maintenance.

6.9.5.5 Contraction joints

Radial contraction joints are usually provided between the cantilever blocks at approximately 15 m centres, although an appropriate spacing is often determined from temperature studies.

The joints usually incorporate shear keys to provide shearing resistance between the cantilever blocks, waterstops to prevent seepage flows from migrating through the joints, grout stops to confine grouting within specified areas of the joints, and grout pipes for grouting of the joints. Grouting operations should be closely monitored to ensure they don't result in harmful overstress in the dam structure.

6.9.5.6 Concrete mix design

Concrete mix design requirements are similar to those for concrete gravity and buttress dams, and the guidelines provided in section 6.8.5 are appropriate.

6.10 Tailings dams

6.10.1 Introduction

All tailings dams should be designed in accordance with sound dam engineering practice. As such, most of the design recommendations included in these Guidelines are applicable to the design of tailings dams.

While many of the design recommendations included in these Guidelines apply to tailings dams, tailings dams differ from other dams in a number of ways. One of the primary differences is that the construction (deposition) phase of a tailings dam continues throughout its operational life. As such, many of the original design parameters can change during the operational life of a tailings dam. Technical support from design consultants and peer reviewers is typically required on a more frequent basis than for conventional water storage dams. Another distinction is that tailings dams typically remain in a rehabilitated state in perpetuity.

The following subsections provide a brief overview of issues that are particular to the design of tailings dams and, because the design process normally continues throughout their operational lives, the overview includes a discussion on construction methods, operational issues and closure systems. Any recommendations that replace those included in other sections of the Guidelines are clearly emphasised.

In 2020 the International Council on Mining and Metals produced the Global International Standard on Tailings Management (GISTM) (ICMM, 2020). The GISTM outlines seven topics and 15 principles on tailings management. To meet these principles, 77 requirements are outlined. Roles and responsibilities to meet the requirements are outlined in the standard. Compliance with the standard is not required under the legislation in New Zealand. GISTM provides a useful standard specifically on tailings management. Many of the principles outlined in GISTM will be fulfilled by adhering to the requirements of the New Zealand Dam Safety Guidelines as well as the resource management and building consent processes in New Zealand.

While GISTM outlines the principles and requirements for tailings management, technical guidance is found other international guidance produced by ICOLD, ANCOLD, and CDA, among others.

6.10.2 Corporate and management support

Because the construction (deposition) phase for a tailings storage facility can extend over many years and because the tailings must remain safely stored until their toxicity has reduced to environmentally acceptable levels, corporate and management support is critical to the ongoing safety of tailings dams. There is normally no direct financial return from the construction and operation of a tailings dam and the natural temptations are to limit capital expenditure, reduce operating costs, and minimise financial contributions to the closure of a tailings storage facility. Dam safety and environmental standards for tailings dams can only be assured through the ongoing support of corporate and managerial personnel to tailings disposal operations.

6.10.3 Characterisation and behaviour of tailings

Different industrial operations can result in the production of tailings with widely different characteristics. For example, the tailings from a mining operation usually comprise finely ground rock and water that can be used, at least in part, to construct the tailings dams for their retention. Alternatively, tailings produced by most industrial and chemical operations are usually fine grained, fluid or semi-fluid, and are unsuitable for the construction of tailings dams. As such, the retention of tailings from most industrial and chemical operations is achieved by the construction of conventional water storage dams.

The grain size of mine tailings depends upon the characteristics of the ore. The mill processes used to concentrate and extract the metal values and can vary from fine sands to clay sized materials. As such, mine tailings are often transported to the storage facility as slurries, with concentrations varying between 25% and 70% by weight of solids to liquids. The solids content of the tailings stream from the process plant is increased with thickeners. Increased solids content requires increased energy and cost to pump. High solids content slurries typically require positive displacement pumps whereas lower solids contents can be pumped with centrifugal pumps. Thickeners remove water at the process plant rather than at the TSF, which can be of benefit in dry climates where there may be pressure on the water balance, and where higher tailings densities are required. New technologies have led to the alternative of filtered tailings where the water content is reduced to the extent that tailings form a wet material that is able to be mechanically loaded on to a conveyor or trucks, transported, placed and compacted in the tailings storage facility.

Filtered tailings facilities require additional filtration infrastructure, transportation and placement plant, as well as thickeners at an increased capital and operational cost. For high processing rates and wet sites, a conventional slurry tailings storage facility may still be required to take tailings stream as a slurry when weather conditions prevent the transportation and placement of the filtered tailings material. Every site is different, and options require assessment. It is important that the selected option is practical, otherwise it puts pressure on the operation's success.

Typical physical characteristics for various types of tailings are listed in Table 6.14.

Table 6.14: Typical physical characteristics of tailings (source: Fell et al., 2015)

Type of tailings	General characteristics[4]
Ultra-fine tailings, aluminium red mud	Clay and silt, high plasticity, very low density[3] and permeability
Washery tailings, coal, bauxite, some iron and nickel ores	Clay and silt, medium to high plasticity, medium to low density and permeability
Oxidised[1] mineral tailings, gold, copper, lead, zinc, etc.	Silt and clay, some sand, low to medium plasticity, medium density and permeability
Hard rock[2] mineral tailings, gold, copper, lead, zinc, etc.	Silt and some sand, non-plastic, high density, medium to high permeability

Notes:

1. Ore is completely or highly weathered, or altered rock
2. Ore is slightly weathered to fresh rock
3. Assuming no desiccation
4. The potential pollution hazards associated with the storage of tailings slurries vary with the industrial operation. Mining processes that merely grind up an inert ore without the addition of toxic chemicals usually result in no pollution hazard. However, an industrial operation that includes the use of toxic chemicals can result in short-term or long-term pollution hazards. Guidelines for the effective control of pollution hazards are included in the references and further information listed at the end of this module.

6.10.4 Tailings storage methods and deposition

The construction materials and construction methods for tailings dams vary widely to accommodate the particular needs of the selected site, the available materials for tailings dam construction, and the financial and operating policies for the industrial operation. The construction methods usually adopted include:

- The construction of a conventional embankment dam where the structure is required to store water which will be replaced, in part, by tailings during the deposition period, or where the impoundment includes a natural inflow and water storage is required for its control. The embankment can either be constructed to its full height at the commencement of the project, or in stages to spread its construction cost over the planned period of deposition.
- The construction of a tailings dam by the upstream method, where increased tailings storage is provided by constructing additional embankments on the tailings immediately upstream of the starter dam. The additional embankments can be constructed from tailings which have been processed through cyclones and are discharged on the upstream side of the delivery pipeline and, as a result, the crest of the embankment moves upstream as the dam height increases. The additional embankments can also be constructed from waste rock where mining is by open pit.
- The construction of a tailings dam by the downstream method, where increased tailings storage is provided by constructing additional embankments against the downstream shoulder of the starter dam. The starter dam is typically constructed from waste rock or borrow material. The additional embankments can be constructed from the coarse fraction of the tailings which have been processed through cyclones and are discharged downstream to form the embankment, with the fine fraction being discharged into the impoundment. As a result, the crest of the embankment moves downstream as the dam height increases. The additional embankments can also be constructed from waste rock.
- The construction of a tailings dam by the centreline method, where increased tailings storage is provided by constructing additional embankments against the downstream shoulder of the starter dam and on the tailings immediately upstream of the crest of the starter dam. The raises can be constructed from the coarse fraction of the tailings which have been processed through cyclones and are discharged downstream and, when the delivery pipeline and cyclone are raised, discharged upstream. The crest of the final embankment is built directly above the crest of the starter dam. The additional embankments can also be constructed from waste rock.

The three tailings dam construction methods are shown diagrammatically in Figure 6.10. The upstream construction method only shows the use of tailings for construction of the additional embankments.

The particular construction method adopted generally reflects the characteristics of the mine operation (e.g., type of tailings, tailings production rate, site foundation conditions, local seismicity, etc.). However, one important factor that should be considered in the selection of the construction method is the importance of seepage control to the stability of the downstream slope of the embankment. Downstream construction and, to a lesser extent, centreline construction provide more opportunities for the effective control of seepage flows than upstream construction. Downstream construction is also recommended in high seismic environments. Upstream construction should only be used in seismic environments where the tailings can be demonstrated by design and monitoring to be drained, and the rate of rise will not result in the generation of excess pore pressures. Such a design requires rigorous analysis.

The disposal arrangement at the tailings storage facility is largely dictated by the embankment construction method. Where a conventional embankment dam and water storage is utilised, the disposal system may be a single point pipeline discharge into the impoundment. Where tailings are used for the construction of the embankments, the disposal system usually incorporates a delivery pipeline and a bank of cyclones for the controlled discharge and use of the tailings. In all cases, the adoption of a more controlled deposition system that takes into account the slopes formed by the deposited tailings above and below the water level in the storage facility, and the different densities of the deposited materials achieved by different deposition methods, can provide significant economic benefits.

For example, the consideration of deposition angles can increase the volume of material stored for a given embankment height, and beach deposition can result in higher material densities.

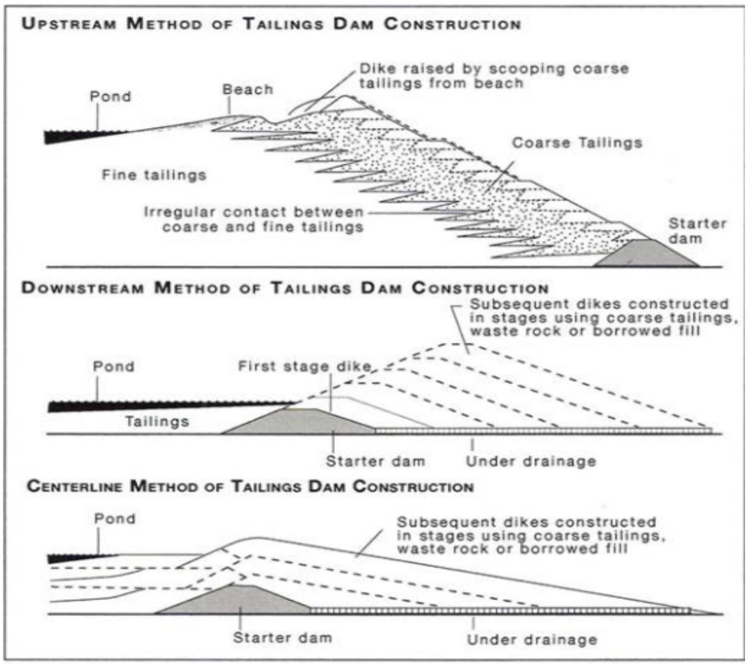

Figure 6.10: Tailings dam construction methods

In most impoundments, the solids settle out of suspension on discharge and the resulting stored material comprises settled solids of variable consistency and a supernatant fluid, usually water, which can be supplemented by runoff and/or direct rainfall. The supernatant fluid may be returned to the processing plant for reuse, stored in the impoundment for future use or for removal by evaporation, or, if sufficiently innocuous, discharged into the downstream catchment.

The main requirement for successful removal of the supernatant is the provision of an outlet facility which can be adjusted as the level of the impoundment increases. The outlet facility (or decanting system) either incorporates an extendible intake and a conduit to convey the discharge away from the embankment; or uses pump barges which reduce the risks associated with raising the level of an outlet facility and to provide more flexibility in the location of the outlet facility within the impoundment.

Filtered tailings facilities are sometimes called dry stacks. However, this terminology can be misleading, as the tailings typically emerge from the filter as a wet soil. During consolidation within the facility, a phreatic surface may develop, leading to saturation of the tailings and the potential for excess pore pressures to develop. Where saturated, tailings are susceptible to liquefaction and specific assessment is required in their design. For large filtered tailings facilities, the consequence of failure is a potential mud flow, depending on how wet the tailings are. Mitigating measures such as seepage collection drains, co-mingling of materials, and a starter embankment may be required. In wet climates, the potential for saturated tailings is higher.

6.10.5 Design overview

The procedures for designing tailings dams can differ significantly from those for conventional water storage dams. Additionally, these procedures are often less well-known and understood compared to those associated with conventional water storage dams. A detailed discussion on the design of tailings dams is beyond the scope of these Guidelines. Designers are instead referred to the relevant ICOLD bulletins, the ANCOLD Guidelines on Tailings Dams (2012, 2019b) and other relevant references listed at the end of this module.

One advantage inherent in the design of tailings dams is that most are built slowly and a design-as-you-go 'observational' approach, using information obtained from the monitoring of dam performance, can often be adopted.

The following material highlights important issues that are particular to the design, construction and operation, evaluation, and closure of tailings dams.

6.10.6 Potential failure modes

Section 6.2.2 provides additional information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams. The guidance on identifying and evaluating potential failure modes for embankment dams, as outlined in section 6.5.2, is also generally relevant to tailings dams.

ICOLD Bulletin 121 (ICOLD, 2001) provides an overview of reported failures for tailings dams and concludes that the main causes of tailings dam failures during operation have been slope instability, overtopping, and earthquakes.

The identification of potential failure modes for a tailings dam should be based on consideration of the construction method, the materials used in its construction and the site-specific characteristics for the dam. However, the information included in ICOLD Bulletin 121 (ICOLD, 2001) indicates that the mechanisms responsible for more common potential failure modes are as outlined in Table 6.15.

Table 6.15: Initiating mechanism for potential failure modes for tailings dams

Initiating mechanism	Common causes
Instability of downstream shoulder	Saturation of downstream shoulder, perched phreatic surfaces, insufficient shear strength, liquefaction of embankment or foundation materials.
Overtopping	Insufficient freeboard to accommodate storms and flood events, inappropriate management of the water balance during operation.
Internal erosion	Inadequate control of seepage, bad filter and drain design, poor design and construction control or earthquake deformations resulting in cracking and leakage paths.

Historical information also indicates that tailings dams constructed by the upstream method are more prone to failure than those constructed by the downstream or centreline methods. While this may merely reflect that the upstream method is the oldest and most commonly used method of tailings dam construction, the downstream and centreline methods do provide more opportunities for the effective control of seepage.

6.10.7 Loading conditions

Loading conditions that should be considered in the design of a tailings dam are similar to those outlined in section 6.5.3 for embankment dams. However, additional external and internal loads should be considered in the design of tailings dams. These include:

- The elevation of the pool water level. The design of conventional water storage dams always includes the consideration of water loads; however, they can be overlooked in the design of tailings dams. Where the pool is located close to the crest of the dam, the direct load of the water should be included in the stability analyses. Where the pool is located away from the crest of the dam, long periods of high water level resulting from continuous rainfall or incorrect operation can result in significant increases in the level of the phreatic surface.
- The rate of rise in the level of the impoundment and its influence on pore pressures. If the rate of rise is greater than the rate of pore pressure dissipation, liquefaction of loose tailings can occur. This is commonly termed 'static liquefaction'. Several observed cases of liquefaction have been initiated by trigger events such as vibrations from passing equipment and an increase in the degree of saturation following heavy rainfall.
- The mass of the tailings and their consolidation can impose a 'downdrag' force on the upstream face of the dam and may result in increased pore pressures. Such forces can be significant for the design of membrane faced dams and decanting facilities.

6.10.8 Stability and deformation performance criteria

As for embankment dams, potential stability failures for tailings dams under different loading conditions are usually assessed in terms of minimum factors of safety. Recommended minimum factors of safety for limit equilibrium stability studies for static loading conditions are listed in Table 6.16.

Table 6.16: Recommended minimum factors of safety for slope stability – static assessment

Loading condition	Slope	Minimum Factor of Safety[1,2]
End of construction of starter dam and construction slopes during operation which do not affect the safe impoundment of contents.	Upstream and downstream, and interim slope profiles without potential for progressive failure.	1.3
Normal during operation (steady state seepage, normal pool level).	Upstream and downstream slopes impounding contents.	1.5
Long-term post- closure (steady state seepage).	Upstream and downstream slopes impounding contents.	1.5
Post-earthquake (residual strength conditions).	Upstream and downstream slopes.	Refer to ICOLD Bulletin 194 (ICOLD, 2022a)[3]

Notes:

1. The Factor of Safety is the factor required to reduce operational shear strength parameters in order to bring a potential sliding mass into a state of limiting equilibrium, using generally accepted methods of analysis.
2. Higher factors of safety may be necessary if there are high levels of uncertainty in the inputs to the stability analysis.
3. The minimum post-earthquake Factor of Safety using limit equilibrium methods depends on the understanding and uncertainty of residual strength parameters, dynamic response, and deformation or damage of the slope post-earthquake. Expert input is required where complex material behaviour or large deformations are possible. Refer to ICOLD Bulletin 194 (ICOLD, 2022a) for discussion on FOS selection.

For seismic loading, the same criteria summarised for embankment dams in Table 6.4 apply.

The above recommended factors of safety are similar to those recommended for embankment dams; however, for tailings dams, careful consideration needs to be given to the following:

- The stability of the tailings dam, during all stages of construction, to its maximum height. This is particularly important for a clay foundation where the increase in strength resulting from consolidation under the increasing weight of tailings may be insufficient to maintain stability as the height of the dam increases.
- The form of construction. For upstream construction the failure surface is likely to include a large mass of tailings materials. For downstream and centreline construction, with good drainage, the failure surface is often located within the constructed embankment. However, where failure mechanisms include a plane of weakness in the foundation, the failure surface can project up through the full tailings profile.
- The spatial and temporal (time) variability in the density of the tailings materials and hence their shear strength.
- The variability in the permeability of the coarse and fine tailings and its effect on the dissipation of pore pressures.
- The potential for chemical reactions to modify the physical properties of the tailings and foundation materials.
- The potential for liquefaction of the tailings materials and the consequent reduction in their shear strength (particularly for upstream construction). This includes the potential for static and seismically induced liquefaction.
- The potential for high saturation levels above the phreatic surface.
- The potential for phreatic surface rise after the main earthquake and before the aftershock.

As stated in section 6.5.4 for embankment dams, the use of simplified stability and deformation methods is appropriate for most applications. Numerical methods should be utilised for High PIC dams where simplified stability and deformation studies may not adequately consider the complex behaviour affecting performance of the tailings facility.

6.10.9 Design details

From a dam safety perspective, there are a number of design details for tailings dams that are additional to those outlined in section 6.5.5 for embankment dams. These warrant careful attention during the design process. Details are addressed in ICOLD bulletins and are discussed below.

6.10.9.1 The tailings storage site

The site for a conventional water storage dam is largely dictated by available water sources, topographical conditions and foundation conditions. This is not always the case for tailings dams which, for ease of tailings disposal, are usually sited within close proximity to the mining or industrial operation. In addition, a tailings dam site must provide security for the long-term storage of the tailings following closure.

6.10.9.2 The deposition method

The selection of an appropriate deposition method is critical to the design process, particularly for dams constructed from tailings. Many factors need to be taken into consideration in selecting the deposition technique including the topography of the site, the physical and chemical characteristics of the tailings materials, the supply rate and slurry concentration of the tailings, climate conditions, surface water and processing inflows, and the rate of rise in the level of the tailings impoundment. Where filtered tailings are used in the operation, poor weather conditions are an important consideration in the feasibility of the method. The selected deposition or disposal method will largely govern the form of the tailings dam, including its geometry and any internal drainage facilities necessary for the control of seepage flows.

6.10.9.3 The water balance

Inadequate management of the water balance is one of the primary causes of tailings dam failure, and it is most important that the available water storage and the installed capacities of the decanting and spillway structures can safely accommodate all inflows to the tailings storage facility. Normal inflows include the tailings transport water, direct rainfall into the impoundment, runoff from upstream catchments, and seepage discharges captured and returned to the storage facility. Abnormal inflows, which must be allowed for in the design, can include water or effluents delivered from extraneous sources and extreme storm events not allowed for in the design. Often tailings dams are designed for no or very unlikely discharge with excess water re-cycled for use in the process plant or pumped for water treatment prior to release. The water balance may rely on diversion drains to divert runoff from upslope.

A water balance model should be established using realistic assumptions and should be regularly reviewed and updated during operation to account for actual operating experience and changes in basic assumptions (e.g., quantity of tailings produced, volume of water pumped from tailings dam, catchment area).

6.10.9.4 Freeboard

Refer also to the recommendations in the general section on freeboard for all dams (section 4.5.4) and the sections on freeboard for specific dam types, particularly section 6.5.5.1 for embankment dams. ICOLD Bulletin 194 (ICOLD, 2022a) provides further guidance. In some cases, resource consents for tailings dams may also set freeboard requirements.

In comparison to a conventional water storage dam, the crest of a tailings dam is usually under continual construction. It is most important that the freeboard between the supernatant pool and the dam crest can always safely accommodate the design flood event. In cases where the dam is being formed using the tailings being deposited, the freeboard may only be provided by the slope and length of the exposed tailings beach.

6.10.9.5 Seepage

Seepage interception facilities and collection drains must be filter compatible with the tailings and foundation materials. Geosynthetic fabrics can become blocked and can tear and should not be used in lieu of sand/gravel filters for the control of internal erosion in the body of or beneath a tailings dam. They should only be used for interception facilities and collection drains where they can be readily exposed, repaired or replaced.

6.10.9.6 Beach length

For upstream and centreline construction, the stability of the embankment is largely governed by the position of the phreatic surface. To maintain the phreatic surface at the design distance from the downstream face, and therefore maintain an adequate level of embankment stability, a minimum beach length should be specified. This is to keep the normal operational pond at distance from the crest and allow drainage to be effective adjacent to the tailings dam wall. It may be acceptable for the pond to come closer to the crest temporarily during a flood. Assessment of this scenario may require a transient analysis of the pond level, seepage conditions in the tailings, and stability.

6.10.9.7 Decanting and spillway facilities

The sizing and operation of decanting and spillway facilities are critical to the safety of a tailings storage facility. Under sizing or inappropriate operation of the decanting and spillway facilities can result in a rise in the pool level with consequential effects on freeboard and dam stability.

6.10.9.8 Designing for closure

The planning and provision for closure should be incorporated within the design from the start of the project to ensure the long-term environmental safety of a tailings storage facility. Planning for closure from the start can allow material movements to be scheduled to minimise closure liabilities (bonds) and final closure costs.

6.10.10 Construction and operation

The recommendations included in Module 4 (Construction and Commissioning) and Module 5 (Dam Safety Management) relating to construction personnel, construction contracts, construction planning, quality management, construction records, and dam safety assurance are relevant to the construction and operation of tailings dams. However, there are significant differences in the construction and operation of tailings dams which can affect dam safety. These include:

- Often the construction of a tailings dam is undertaken by a mining company as a component of a mining operation. In such a case it is natural that the primary focus will be on production and the efficiency of the mining operation, and that the construction of the tailings dam will be of secondary importance. From a dam safety perspective, it is most important that the construction of a tailings dam is appropriately resourced and managed to ensure the design intent is achieved. Dam safety can only be assured through the appointment of a construction team with an appropriate level of experience in tailings dam construction, the implementation of appropriate quality control procedures, and the ongoing support of the design consultant and corporate and management personnel throughout the tailings disposal operation.
- As outlined earlier, the construction of a tailings dam continues throughout the operational life of the tailings storage facility and ongoing design support is essential to ensure the design intent is met during all stages of construction. Regular monitoring of the dam's performance (e.g., piezometric pressures, seepage flows) and testing of the deposited tailings materials (e.g., water content, density, permeability, shear strength) will be necessary to confirm design assumptions and support the 'design-as-you-go' (observational) approach.
- It is not uncommon for the rate of mining - and consequently the quantities of tailings - and processing and operational procedures to change during the life of a tailings dam. It is important that the implications of such changes on the design are considered and appropriate amendments made where necessary. Regular reviews are recommended.
- The construction and operation process will vary according to the physical and chemical properties of the tailings materials and the rate at which they are delivered to the storage facility. Unlike a conventional water storage dam, the construction of a tailings dam must reflect the requirements of the mining process, and the operation of the dam will commence shortly after the onset of its construction.
- The operation process should be supported by a formal detailed manual (or manuals) that outline requirements for operation, maintenance, and surveillance. Unlike a conventional water storage dam, where such a manual is often prepared towards the end of dam construction, an operation, maintenance, and surveillance manual for a tailings dam should be prepared ahead of dam construction. The manual should be similar to that outlined in Module 5 (Dam Safety Management) but it should also include activities related to the operation, maintenance, and surveillance of the tailings disposal system, such as:

  - Operation and maintenance of the tailings delivery system.
  - Ensuring that the deposition process achieves adequate particle size segregation on the beaches.
  - Maintenance of the beach length, slope and freeboard.
  - Water balance operating procedures.
  - Operation and maintenance of the decanting facility.
  - Regular testing of the deposited tailings.
- An Emergency Action Plan should be in place throughout the operational life of the tailings storage facility. Guidelines for the preparation of an Emergency Action Plan are included in Module 6 (Emergency Preparedness).

6.10.11 Closure

A tailings impoundment will generally remain in existence long after the associated mine or processing plant has ceased operation and it is important that the impoundment, tailings dam and associated structures remain safe in the long-term. It is therefore important that the closure of a tailings storage facility is given early consideration during the design process. ICOLD Bulletin 153 (ICOLD, 2013) provides advice on design for closure. The international trend is towards a design life of in excess of 1,000 years.

It may not be practical to design all elements for such long design lives in closures. A shorter design life may be acceptable where repair or replacement of specific elements are practical. Where elements cannot be repaired or replaced the facility closure design shall consider the eventual failure of such elements. Where elements require repair or replacement, it is typical to leave financial instruments or have agreements in place at the end of the project so that the site can continue to be operated, maintained, and continue surveillance.

Design loading for closure may need to be higher than operation. GISTM provides guidance on the design levels for closure for flood and earthquake. For lower consequence structures, the design levels are higher than operation due to the longer design life. For higher consequence structures they are typically the same as the design loading is already at the maximum.

While environmental controls are important to ensure the long-term environmental safety of a tailings storage facility, their design and management are beyond the scope of these Guidelines. From a dam engineering perspective, important issues that should be addressed in the design of a tailings dam closure include:

- The long-term stability of the tailings dam. This should allow for the potential of large earthquakes after closure of the facility. The closure design of TSF should be based on the highest SEE design loads. The protection of the outer embankment slopes against the effects of erosion.
- The ongoing effectiveness of the seepage control facilities to effectively manage the residual moisture in the tailings and any springs identified beneath the impoundment.
- The long-term ability of the facility to safely manage and discharge catchment rainfall and runoff. Long- term safety may necessitate the management and discharge of larger rainfall and runoff events than those that were adopted for the operation of the tailings storage facility. This can arise because often runoff from upstream will be diverted past the tailings dam during operation but will run through the tailings dam after closure.
- Ongoing surveillance and monitoring, and reviews of the surveillance and monitoring results, to identify any adverse trends which could affect the safety of the tailings facility.

6.11 Flood detention dams

6.11.1 Introduction

Flood detention dams are often owned by local authorities and are intended to modify the flood hazard profile for developed areas downstream of the dam. They can range in size from small stormwater ponds in urban settings (often with only significant inflows from stormwater pipe reticulation) to large dams on urban or rural streams.

Flood detention dams are designed to capture storm runoff from storm events up to a certain design threshold, storing floodwaters temporarily in the upstream holding reservoir and then gradually releasing those floodwaters over time. These dams reduce downstream flood flows to mitigate flooding impacts, particularly in urban areas.

Flood detention dams normally do not impound any water under dry weather conditions and convey flows in the natural watercourse by means of a low-level culvert through the base of the dam. However, in some cases, they may store water for amenity or environmental reasons. A wetland constructed within the holding reservoir impounded by a flood detention dam may also perform a stormwater quality function.

Flood detention dams often incorporate a low-level culvert structure through the base of the dam functioning as primary outlet structure and an auxiliary overflow spillway as illustrated by the example shown in Figure 6.11.

Figure 6.11: Hopua Te Nihotetea flood detention dam on Raumanga Stream, Whangarei, operated by Northland Regional Council (photo provided by Damwatch Engineering)

6.11.2 Design floods

Flood detention dams reduce downstream flooding for flood events up to the Service Level Inflow Flood, by attenuating flood flows through a low-level service spillway (typically a conduit through the impounding embankment). The Service Level Inflow Flood differs from the Inflow Design Flood (IDF) conventionally used for dam design and for which minimum values are specified in Table 4.1 of this module. The IDF is discussed further below.

Typically, the Service Level Inflow Flood will be determined by the dam Owner and may be guided by local planning regulations. The Service Level Inflow Flood is not related to dam safety but is set by the dam Owner to provide a specified level of flood protection for downstream assets such as public infrastructure and residential buildings. The Service Level Inflow Flood is typically the 1 in 50, 1 in 100 or 1 in 200 AEP flood although it may not necessarily have an AEP associated with it. Historically the Service Level Inflow Flood selected for a flood detention dam may not have included allowances for future climate change. For new flood detention dams, it is recommended that consideration be given to including allowances for future climate change with selection of the Service Level Inflow Flood. If the Service Level Flood is exceeded in a storm event, the level of downstream flood protection may be compromised.

The level of flood protection provided downstream of the dam may change over time as a result of various factors including but not limited to changes to hydrology, geomorphology, land use, and societal expectations. Dam Owners should be aware that the value of the infrastructure and residential development protected by the dam may also change over time.

The Inflow Design Flood (IDF) relates to dam safety. The selection of the IDF for a flood detention dam is the same as for other dams. During extreme flood events in excess of the Service Level Inflow Flood, flood flows will discharge through the auxiliary spillway, and downstream flooding will increase as a result of the step change in the discharge. In flood events larger than the Service Level Inflow Flood, the percentage reduction in peak flood flow downstream of the detention dam reduces, and in extreme floods (e.g. significantly greater than the service level inflow flood), flooding experienced downstream may be similar to that which would be experienced without the dam being present. The catchment characteristics downstream of the dam, such as tributary inflow hydrographs, will play a role in the magnitude of this effect.

The peak reservoir extent should be identified to prevent inappropriate future development within the reservoir extent. Such identification may be required within local planning rules.

6.11.3 Hydrology

Hydrology assessments for flood detention dams should be undertaken to a level required by the PIC (as for any other dam). Many local and regional authorities have prescriptive hydrology methods which may be required for the Service Level Inflow Flood. However, these methods may not necessarily be transferable or appropriate for determination of the IDF.

Prescribed council methods may require the use of:

- a nested rainfall storm over a set duration,
- a particular runoff loss method, or
- non-calibrated site-specific runoff parameters.

Use of a nested rainfall storm is not appropriate for any assessment of the effectiveness of a flood detention dam as the storage characteristics of the impounded reservoir significantly alter the downstream flood hazard profile. Use of a nested rainfall storm may, therefore, give a misleading picture of the downstream flood hazard. Instead, a series of rainfall storms covering a range of durations should be considered so that the critical storm duration can be established. This will provide a more accurate picture of the downstream flood hazard.

6.11.4 Potential failure modes

Most flood detention dams are constructed as earth-fill embankment dams. The relevant potential failure modes are therefore the same as those described in section 6.5.2 for embankment dams. However, as flood detention dams normally do not impound water in dry weather conditions, any failure of a flood detention dam would more likely occur under flood conditions.

For any flood detention dam of a different construction type to an embankment dam, the relevant potential failure modes would be those described in either section 6.6, 6.7, 6.8 or 6.9.

6.11.5 Loading conditions

Loading conditions for the design and rehabilitation of flood detention dams will be the same as those described in section 6.5.3 for embankment dams unless the flood detention dam is of a different construction type.

For those flood detention dams which do not impound water under dry weather conditions, the normal hydrostatic loading condition on the upstream face will not apply.

For High PIC flood detention dams that normally impound little or no water, the designer may consider SEE loadings at Low PIC standards on the basis that it is not credible to have a coincident SEE and extreme flood. However, consideration needs to be given to the ability of such dams to safely impound a flood event in a post SEE damaged state should a flood event occur before remedial works can be completed. An approach to temporary risk mitigation following earthquake damage to a flood detention dam that may be appropriate in some cases could be to consider modifying the dam to prevent it storing a significant volume of water in a subsequent flood event.

6.11.6 Stability and deformation performance criteria

As for embankment and other types of dams, potential stability failures under different loading conditions should be assessed in terms of minimum factors of safety (refer to section 6.5.4 for embankment dams).

6.11.7 Design details

Design details for flood detention dams are as described in the earlier sub-sections for the different dam types (for example section 6.5.5 for embankment dams).

The recommendations on freeboard for all dams (section 4.5.4) and freeboard for embankment dams (section 6.5.5.1) are also applicable for flood detention dams.

The operational management of hazards associated with dams in general is discussed in Module 7 of these Guidelines.

However, there are several operational health and safety factors which need specific consideration in the design of a low-level culvert structure for a flood detention dam:

- The prevention of unauthorised public access to a low-level culvert under normal flow conditions.
- Facilitation of routine safety inspections of a low-level culvert with respect to confined space entry.
- Provision of easy access for the maintenance of a low-level culvert including post-event clearance of debris blockages.
- Access for clearance of debris under flood conditions.

If a dam has both a primary (or service) and an auxiliary spillway, the primary spill outlet should typically be designed to pass at least a 1in100 AEP flood without the auxiliary spillway operating.

6.12 Appurtenant structures and Dam Safety Critical Systems (DSCS)

6.12.1 Introduction

The Building Act 2004 defines an appurtenant structure, in relation to a dam, as "a structure that is integral to the safe functioning of the dam as a structure for retaining water or other fluid". This is interpreted to be primarily concerned with safe containment of a reservoir. As such, appurtenant structures are those structures, other than the main dam itself, at the dam site, or located around the perimeter of a common reservoir, that are designed, and are required for the safe containment and control of the common reservoir, under all loading conditions.

A structure is an appurtenant structure if;

- **It is necessary to retention of the reservoir over the full range that can be impounded by the main dam** (i.e., a dam structure such as wing dam, saddle dam, natural feature modified to act as a dam).
- **It has a reservoir operational or spill discharge function** (i.e., hydraulic structure – spill outlet – spillway, sluiceway, low-level outlet, or inlet or intake structures).
- **It has a reservoir drawdown function, i.e., partial or full dewatering of the reservoi**r (i.e., hydraulic structure – spill outlet – spillway, sluice or low-level outlet).
- **It is a conduit connected to the reservoir that cannot be effectively isolated[1] from the reservoir** (i.e., closed conduit – penstock, tunnel or pipework, or open conduit – canal or channel, or surge chamber).

In identifying appurtenant structures, it is helpful to think of the reservoir as a system.
Appurtenant structures can be considered in the following categories:

- **Appurtenant dam structures** which have only a reservoir retention function.
- **Appurtenant hydraulic structures** which have a discharge control function or a conveyance function in addition to a retention function.

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1. Effective isolation requires that the flow control device can close fully under the maximum feasible operating conditions, including extreme conditions, i.e. rupture of the downstream conduit.

Appurtenant dam structures are necessary in addition to the main dam for safe retention of the reservoir over the full range that can be impounded by the main dam and can include structures such as: wing dams; saddle dams, or natural features modified to act as a dam. These structures should be designed as dam structures using the criteria and considerations provided in sections 6.2 to 6.11 of this module. It is important to recognise that the potential impact of failure of these structures may be different to the main dam and should be specifically identified for design and dam safety management purposes.

Appurtenant hydraulic structures are necessary to convey the reservoir contents, or to control flow into or out of the reservoir. Conveyance structures can include closed conduits (pipelines, penstocks, and tunnels), and open conduits (canals, channels and transition structures). Flow control structures can include spill outlets, inlet or intake structures which are free discharge or controlled gate or valve systems.

Structures with a conveyance function which **can** be effectively isolated from the reservoir by a flow control structure need not be considered as appurtenant hydraulic structures, as their failure would not lead to an uncontrolled release of reservoir.

Structures with a flow control function that is not dam safety critical to the main dam or reservoir need not be considered as appurtenant hydraulic structures, as their failure would not impair the safety of the main dam or reservoir.

Appurtenant hydraulic structures may incorporate systems for the monitoring, controlled discharge or release of the reservoir contents for operational or dam safety purposes. These systems typically involve mechanical, electrical, control and communication equipment, as well as redundant power supplies. Where these systems have a function to protect the dam against the development of a potential failure mode they are classified as Dam Safety Critical Systems (DSCS) – refer to section 6.12.9 and 6.12.10.

6.12.2 Appurtenant dam and hydraulic structures

For **appurtenant dam structures** which have only a retention function: the considerations of potential failure modes, loading conditions and performance criteria are those for the relevant dam type as described in sections 6.2 to 6.11 of this module, bearing in mind that where the appurtenant dam structure provides a dam safety function to the main dam, the loading conditions and performance criteria will be those applicable to the PIC of the main dam.

For **appurtenant hydraulic structures** which have a conveyance or discharge control function in addition to a retention function (such as spill outlets and or inlet structures), the following subsections provide guidance and information on aspects of appurtenant hydraulic structures.

Note that the dam structure elements of appurtenant hydraulic structures (i.e., concrete dam elements of a spill outlet or flow inlet structure, or embankment sections of a spill or inlet structure) should also make use of the information in this module for the relevant dam or material type.

6.12.3 Potential failure modes

Section 6.2.2 provides general information on the requirements for the application of FMEA processes to assess potential failure modes in the design and operation of dams.

Inadequate design and/or inappropriate operation of appurtenant hydraulic structures, and DSCSs are significant factors in dam failures internationally, particularly for embankment dams (ASDSO, 2024).

Some of the mechanisms responsible for common potential failure modes for dams, which are related to the design and operation of appurtenant hydraulic structures, are outlined in Table 6.17. The general comments on the identification and evaluation of potential failure modes included in sections on specific dam types, (i.e., sections 6.2 to 6.11 of this module) may also be relevant to appurtenant hydraulic structures.

Table 6.17: Initiating mechanisms for potential failure modes relevant to appurtenant hydraulic structures

Initiating mechanism	Common causes
Overtopping	Inability to control reservoir level, leading to overtopping of main dam or appurtenant structure, due to: • Insufficient spill outlet capacity in an inflow event, due to failure to correctly assess all potentially feasible inflow events (i.e., from catchment inflow flood, failure of upstream structures or features, changes in assessment methodology, catchment characteristics or probable maximum precipitation). • Insufficient spill outlet capacity in an inflow event, due to unavailability of all or elements of spill outlet structures, gates, valves or their control and operating systems (may occur due to maintenance, repair, or refurbishment outages). • Inappropriate operation of spill outlet facilities • Inability to operate spill outlet facilities (spillway blockage, gate jamming through pier deformation, equipment malfunction, control systems failure, power supply failure, lack of access for manual operation, no operator for manual operation). • Inappropriate management of water balance in reservoir, due to inadequate or inappropriate 'rules' for operation of spill outlets during an unusual or extreme inflow event, or failure to apply rules appropriately. • Inappropriate management of water balance in a canal or pumped storage system. • Inappropriate management of water balance in a tailings storage facility. • Lack of, or inadequate, Emergency Backup systems to implement spill outlet operation in the event of loss of operational control. • Failure of Emergency Backup systems to effectively implement spill outlet Dam Safety protection function, on demand or in the event of loss of operational control. • Loss of freeboard at Appurtenant Structures due to gradual or instantaneous slumping or collapse of all or some elements of structures.
Erosion of Spill Outlet structure	Inability to manage the energy from flow discharge leading to failure of the spill outlet structure, i.e. • Failure of spill outlet conduit (chute, penstock, pipe or tunnel) during a spill operation, leading to backward erosion to the reservoir, due to inadequate design, construction or poor condition. • Failure of spill outlet energy dissipation (stilling basin, plunge pool, disperser valve) during a spill operation, leading to backward erosion to the reservoir, due to inadequate design, construction or poor condition.
Erosion of embankment materials	Inadequate detailing of embankment/structure interfaces (e.g., spillway, intake and conduit interfaces), inadequate filter protection systems adjacent to appurtenant structures, rupture of pipeline through embankment.
Structural failure of gate or valve system	Structural collapse of gate or valve housing structural elements, gate or valve structural elements, gate or valve operation, power or control elements. Overstressing of gate arms, gate bearing seizure, failure of gate lifting ropes or rams. Damage to valve cone and/or collar, valve positioning control.

6.12.4 Loading conditions

For **appurtenant dam structures** which have no function other than reservoir retention, the loading conditions to consider are those for the relevant dam type as discussed in sections 6.5 to 6.11 of this module.

For the design of new, or safety evaluation and/or rehabilitation of existing **appurtenant hydraulic structures** and DSCS, the loads and load combinations which will be applicable relate to the dam safety function(s) they perform.

The loads and load combinations selected should align to the potential impact classification of the main dam, unless the appurtenant hydraulic structure does not provide any dam safety protection function to the associated main dam. In such cases, the design loads and load combinations selected need only be those appropriate to the consequences of its own failure.

The process for selection of loads and load combinations for a specific appurtenant hydraulic structure or DSCS, is similar to that outlined in section 6.8.3 for a concrete dam and should also be guided by the performance criteria for appurtenant hydraulic structures and DSCS discussed in section 6.11.4, section 6.11.5, and section 6.11.10.

It is frequently necessary to analyse the performance of the appurtenant hydraulic structure, the DSCS, or a component of the appurtenant hydraulic structure or DSCS, under various loading configurations and operating conditions to allow for the configurations that the system may experience during normal, unusual and extreme load cases and scenarios.

Seismic loads require specific consideration. Where appurtenant hydraulic structures and DSCS are located in prominent positions on dams or abutments, or above the crest of the structure, they are likely to be exposed to amplified ground motions during earthquakes. Amplification of ground motions is of particular concern when the fundamental frequency of the appurtenant hydraulic structure or DSCS aligns closely with the frequencies of the ground motions.

Various reviews of earthquake related damage to dams (e.g. Fell et al., 2015; Wieland, 2012; Hansen and Nuss, 2011) note earthquake damage to hydraulic structures and DSCS.

6.12.5 Performance criteria – general

The basic performance criteria for appurtenant hydraulic structures are similar to those outlined in section 6.8.4 for concrete dams and the recommendations relating to sliding factors of safety, the position of the force resultant, and normal compressive stresses are applicable. These are summarized in Tables 6.18 and 6.19.

The performance criteria for the appurtenant hydraulic structures and DSCS that fulfil a safety function are more demanding than the performance criteria for the related dam, as the appurtenant hydraulic structures and DSCS are required to be functional during, and/or following, unusual and extreme load events and scenarios, such as flood or earthquake. Guidance on the earthquake design of appurtenant concrete structures is provided in USACE engineering manual Earthquake Design and Evaluation of Concrete Hydraulic Structures (USACE, 2007).

In many cases, structures will need to be capable of fulfilling their dam safety critical function for an extended period (weeks, months or possibly years) following a damaging load event, to allow the safety of the dam and reservoir to be managed while the dam is made safe, or remedial action is planned and implemented. Design and safety evaluation should consider this requirement.

The plant and equipment used in DSCS should be extremely robust and reliable, with adequate provision for redundancy.

Table 6.18: Recommended minimum sliding and flotation Factors of Safety for appurtenant hydraulic structures

Loading condition	Minimum sliding Factor of Safety [1,2]		
	Friction and cohesion present		Friction only present
	Not well defined	Well defined	Well defined
Normal	3.00	2.00	1.50
Unusual	2.00	1.50	1.30
Extreme - Flood	1.50	1.30	1.10
Extreme – Earthquake	(note 4)		
Post-earthquake		1.2	
Loading condition	**Minimum flotation Factor of Safety [c)]**		
Normal	1.3	1.3	1.3
Unusual	1.2	1.2	1.2
Extreme - Flood	1.1	1.1	1.1
Extreme – Earthquake	(note 3)		
Post-earthquake		1.2	

Notes:

1. Refer Table 6.9 for general notes.
2. Based on the following references:
   a) Stability Analysis of Concrete Structures (USACE, 2005).
   b) Guidelines for Design of Dams and Appurtenant Structures for Earthquake (ANCOLD, 2019a).
   c) Guidelines on Design Criteria for Concrete Gravity Dams (ANCOLD, 2013).
3. The earthquake load case is used to determine the post-earthquake condition of the structure (refer Table 6.9) and a minimum Factor of Safety is not given. If sliding/flotation stability assessments indicate displacement, then the Designer needs to consider the amount of displacement of the appurtenant structure and its impact on post-earthquake dam safety and operations.

Table 6.19: Recommended position of the force resultant for appurtenant structures

Loading condition	Position of the force resultant [1, 2,3]
Normal	Middle 1/3
Unusual	Middle 1/2
Extreme - Flood	Within the base
Extreme – Earthquake	(note 4)

Notes:

1. Refer Table 6.10 for general notes.
2. Guidelines on Design Criteria for Concrete Gravity Dams (ANCOLD, 2013).
3. Stability Analysis of Concrete Structures (USACE, 2005).
4. Rocking may occur under extreme earthquake loads and some permanent displacement could result. The Designer needs to determine whether this occurs and, if it does, evaluate its impact on post-earthquake dam safety and operations.

6.12.6 Performance criteria - seismic

For seismic load, additional load condition and performance criteria include:

- Appurtenant hydraulic structures and DSCS elements must be able to perform their dam safety hazard mitigation function effectively following any seismic event, including aftershocks, up to and including the SEE event load case.
- Design these structures to withstand seismic loads, including amplified loads (due to physical location or configuration) as well as structural movement (gross or differential).
- Consider all components of seismic shaking (upstream/downstream, cross valley, vertical, temporal) and as well as the potential for ground tilting or displacement.
- The support structures must withstand the opening and/or closing loads of the system in its post-SEE condition, even if elements are damaged (e.g., for a gate system consider damaged or deformed gates, or piers, damaged or deformed winch platforms, damage to power supplies, control, and communications equipment). The structural elements that support, or house all of the elements of a DSCS, and the individual components that make up the system, must be designed to ensure the system is able to withstand the SEE event load case, and perform their dam safety functions post-SEE.
- Where equipment has an opening or closing function then the support structure must be able to sustain the opening and/or closing loads of the system in its post-SEE condition (e.g., consider damaged or deformed gates, or piers, damaged or deformed winch platforms, damage to power supplies, control and communications equipment).
- Provide safe access to inspect, operate, maintain and make temporary repairs of appurtenant hydraulic structures and DSCSs for all design or safety evaluation load case and scenario. (this can be partly accommodated by planned emergency response actions for the dam – refer to Module 6 Emergency Preparedness).
- Provide safe access to carry out surveillance and monitoring of civil elements of appurtenant hydraulic structures (e.g., spillways, stilling basins and their underdrainage systems, and tailrace areas), for all seismic load scenarios.
- Provide safe access to carry out condition monitoring of all necessary DSCS elements (e.g., gate and valve systems, power, control and communications equipment, etc.) following seismic events up to and including the design or SEE.
- Some damage to appurtenant hydraulic structures and DSCS equipment as the result of the SEE is acceptable; however,
  - Damage which could potentially lead to failure in a single event is not acceptable.
  - Damage that could potentially lead to failure in an unusual flood event following an extreme seismic event, and before the structures or DSCS can be repaired, is not acceptable.

6.12.7 Performance criteria - flood

For flood loads, additional load conditions and performance criteria include the following.

- Appurtenant hydraulic structures and all elements of the relevant DSCS are required to function effectively for inflow events up to, and including, the design or safety evaluation flood.
- Appurtenant hydraulic structures and all elements of the DSCS are required to withstand flood safety evaluation event loads, including high reservoir levels and spill discharges, as well as those loads and conditions that can be anticipated prior to, during, and following an extreme inflow event (e.g., wind, waves, rain, spray and extremes of temperature).
- Provide adequate freeboard and protection to all vulnerable elements of the appurtenant hydraulic structures and DSCS to mitigate threats due to high reservoir levels, wave action, etc..

- Reservoir level monitoring systems used for dam safety critical functions (e.g., emergency backup monitoring and control of reservoir levels and spill discharge) should be completely independent of those used for normal reservoir operations. This requires that they be separately located, and configured such that reservoir effects such as drawdown, waves and other flow hydraulic loads do not adversely affect their function.
- Provide safe access to inspect, operate, maintain and make temporary repairs to DSCS for all flood scenarios. (This may be partly accommodated by planned emergency response actions for the dam – refer to Module 6 Emergency Preparedness).
- Provide safe access to carry out surveillance and monitoring of civil elements of appurtenant hydraulic structures (e.g., spillways, stilling basins and their underdrainage systems, and tailrace areas), during the flood events up to and including the design or safety evaluation event.
- Provide safe access to carry out condition monitoring of all necessary elements: DSCS (i.e., gate and valve systems, power, control and communications equipment), etc. during the flood events up to and including the design or safety evaluation event.
- Some damage to appurtenant hydraulic structures and DSCS equipment as the result of the Safety Evaluation flood event is acceptable, however:
  - Damage which could potentially lead to failure in a single event is not acceptable.
  - Damage that could potentially lead to failure in an unusual flood event following an extreme flood, and before the structures or DSCS can be repaired, is not acceptable.
- Following an unusual or extreme inflow event, it can be expected that the reservoir will be full or close to full and the likelihood of spill from subsequent inflow events will be very high. Design and/or safety evaluation should consider how successive inflow events can be managed safety, particularly as it is possible that due to extreme inflow events the normal operational outlets from the reservoir may be damaged, or not usable (e.g., hydro dam - generation not possible from a hydro plant, or water supply dam - water distribution systems damaged).
- The duration of the period of sustained high reservoir levels, and/or further spills, before the appurtenant hydraulic structures can be fully inspected or repaired may be weeks, months or potentially years. The ability of damaged appurtenant hydraulic structures and DSCS to fulfil their safety functions in these post event scenarios should be an explicit design and safety evaluation consideration.

Other structures and systems that are not critical to the safe retention of the reservoir, or do not have a dam safety critical function in a potential failure mode are usually designed in accordance with relevant industry standards, and/or the applicable building regulation requirements.

6.12.8 Appurtenant hydraulic structures - spill outlets and discharge inlets

6.12.8.1 Spill outlet - introduction

The primary route for the contents of a reservoir to be released will be by some form of operational outlet, e.g., penstocks for a hydro dam, distribution pipes or channels for a water supply dam. However, spill outlets are a critical appurtenant hydraulic structure for all dams. Spill outlets are required to safely discharge flood water or surplus fluid that cannot be used for the reservoir's operational purpose or stored within its operational storage range.

The importance of adequate and safe spill outlet capacity cannot be overstated. Overtopping is identified as the cause of 26% of dam failures, with spill outlet failures - stemming from inadequate capacity, poor design, or maloperation - accounting for the majority of these failures. Embankment dams are highly likely to fail if overtopped, and concrete dams are vulnerable to failure from overtopping if not specifically designed for that mode of operation.

In the design of a new dam or spill outlet, it should be borne in mind that future modifications to increase the discharge capacity of an existing spill outlet may be much larger than the initial cost of providing the same increase in discharge capacity at the time of building the spill outlet initially. This is of particular importance in view of the uncertainty that climate change brings to the estimation of extreme inflows.

The spill outlet facilities required for each dam will be specifically designed to accommodate the unique characteristics and operational requirements of that dam. These may be a single outlet configured to allow the full range of required functions and discharges, or may involve two or more outlet facilities with the functions and required discharge capacities shared amongst them.

Individual spill outlets may act as main (service) spill outlets, when they provide a continuous or frequent release of water, or as auxiliary or emergency spill outlets, where they supplement the capacity of the main spill outlet and will only operate infrequently, or under unusual or extreme conditions.

6.12.8.2 Spill outlet - definition and elements

Spill outlets are defined as follows (after ICOLD, 2012):

- Service spill outlet: Primary outlet, used for discharging smaller more common floods and operational discharges.
- Auxiliary spill outlet: Secondary outlet, which complements the capacity of the service spillway to discharge the IDF.
- Emergency spillway: Supplementary outlet which enters into operation for floods greater than the IDF, or if the capacity of the service and auxiliary spillways is affected by an unusual event such as gate malfunction, power failure or partial/total blockage.

The total capacity and capabilities of the spill outlets provided must be adequate for all spill discharge scenarios. Ample spill outlet capacity for all load cases and scenarios is critical to dam safety, and the spill outlet(s) at a dam must have the geometric, hydraulic and structural capacities to safely contain all of the forces and pressures that spill discharge will impose. The spill outlet surfaces must be able to resist the erosive forces from scour, damage from cavitation in high velocity locations, abrasion from any material entrained in the flow. Spill outlets must be able to dissipate the energy from the water released in such a way that excessive erosion does not occur anywhere within the spill outlet conduits or where the outlet releases its discharge to the receiving water course. In many cases flow control is also required to manage the rate of discharge from the reservoir.

It should be noted that it is not usual to include the capacity of operational discharge outlets from a reservoir in the spill outlet capacity for passage of unusual and extreme flows. This is because, in most cases, it cannot be certain that the operational process (e.g., water supply, hydro generation, irrigation supply) will be able to operate in such an event, and so that discharge cannot be relied on.

Spill outlets typically consist of four main elements, and the carefully considered configuration and design of each of these elements is critical to the safe function of the whole spill outlet as a DSCS.

The four elements are:

- **Approach channel** – which conveys water from the reservoir to the control structure and is intended to ensure adequate hydraulic conditions of flow to the flow control structure.
- **Inlet** – provides the hydraulic control (establishes the discharge capacity) for the spill outlet. The two types of hydraulic inlet are
  - uncontrolled (free-flow spill outlet), and
  - controlled (gated, or staged spill outlet).
- **Conduit** – the conveyance structure provided to safely convey the flow from the flow control structure and discharge it downstream of the dam. This can take the form of an open conduit (channel or chute), or a closed conduit (pipe, tunnel or culvert) where flow may be free surface or pressurised. Open channels and chutes are most typically sized to fully contain the design flow, while providing sufficient freeboard to accommodate the effects of flow bulking due to air entrainment, shock waves, surface roughness, spray and splash, and any modelling uncertainties.
- **Energy dissipation/terminal structure** – This structure either dissipates most of the kinetic energy associated with the spill flow and conveys the water from the conveyance feature to the downstream river or stream. or conveys high energy flow downstream where the kinetic energy is dissipated within the natural river or stream channel. The requirement for an energy dissipation structure is based on the risk posed by erosion of the downstream channel and the threat this poses to the dam.

6.12.8.3 Spill outlet - location and type

The location and type of spill outlet(s) provided at a dam are governed by economic and safety considerations depending on the type of dam. These are in turn dominated by the topographical and geological conditions on the reservoir banks and at the receiving watercourse. For a spill outlet upgrading the location and type of the additional spill outlet may also be influenced by the location and condition of the existing spill outlets, the interfaces of the new structure with the existing dam (including potential differential settlement) and the conditions for flood diversion during construction. For Medium and High PIC dams, the provision of a low-level spill outlet facility that enables the reservoir to be significantly lowered in a potential dam safety emergency should be considered, particularly for embankment dams located close to active faults.

Commonly used terms for spill outlet types in New Zealand are spill outlet, sluice outlet and low-level outlet, although other terms are also used, and these terms are used differently in other countries. These terms tend to relate the location of the outlet in relation to the reservoir surface, and the functions they provide to the operation or management of the reservoir.

Spillway outlets are typically located close to the reservoir surface and provide the primary spill outlet. Sluice outlets are typically located lower in a reservoir and provide the ability to flush sediment past the dam and lower the reservoir below the normal operating range. Low-level outlets tend to be construction bypass structures which are also designed to provide similar functions to a sluice outlet during the operating life of the reservoir system.

The typical functions and uses associated with each spill outlet type are shown in Table 6.20.

Table 6.20: Spill outlet types and functions

Spill outlet function	Spillway outlet	Sluice outlet	Low-level outlet
Operational control of reservoir	√		
Release operational flows	√		
Environmental release	√	√	√
Sediment release		√	√
Pass usual flood flows	√		
Pass unusual flood flows	√	√	
Pass extreme flood flows	√	√	√
Partial drawdown of reservoir	√	√	√
Maximum drawdown of reservoir		√	√
Back up for spillway		√	√
Inflow diversion for construction			√

USBR (2022) provides extensive information on spill outlets and guidance on their selection and design. Section 3.5.2.3 and Table 3.5.2.3-1 of USBR (2022) summarise key factors that guide selection of spill outlet types that are most suited for a given new dam or refurbishment.

Note that spill outlets should be described to reflect the full range of functions they are required to provide in the operation and safety of the reservoir system and should be designed to meet the criteria relevant to these functions.

6.12.8.4 Spill outlet - flow control types

Uncontrolled or free-overflow outlets come in many forms. Their primary advantages are design and construction simplicity, freedom from operating mechanisms, and no requirement for operations personnel. However, they also require a combination of long crest length, and/or high flood surcharge (affecting the minimum reservoir freeboard required above the normal maximum operating storage level) in order to pass unusual to extreme flows, and they cannot allow reservoir drawdown, or passage of sediments. They are generally more suited to water supply dams where close control over reservoir level is less critical, locations with lower extreme flood magnitudes remote sites where access for operations and maintenance are difficult, and regions of high seismicity, where designing gate systems to meet post SEE serviceability requirements is difficult.

Controlled or gated spill outlets also come in many forms. Their advantages are reduced flood surcharge requirement, greater control over reservoir operation, ability to allow reservoir drawdown in an emergency, generally lower overall cost for reservoirs with large spill flows. In addition, the spill outlet crest can be set below the normal maximum operating level, allowing a high proportion of the design discharge to be available when gates are opened. They can be installed in orifice outlets below the normal maximum control level to enable sluice and low-level outlets which can allow reservoir drawdown and passage of sediments. However, they are less able to accommodate increases in capacity without significant dam crest raising in the case of changes to the hydrological inputs. Furthermore, DSCSs require robust and reliable plant and components to assure adequate dam safety, and generally require robust automation systems, and/or availability of skilled operations and maintenance personnel. They also require a means to isolate the gates from the reservoir to allow some maintenance and testing operations.

Uncontrolled spill outlet types include: open channel, drop inlet (morning glory or intake tower), weir (ogee crest, straight drop, labyrinth, piano key, side channel), and bathtub. Controlled or stage outlet types include: gated (drop inlet, freefall, ogee crest, orifice, side channel), fuse plug, fuse gate, and siphon.

Further guidance is provided in USBR (2022):

- Section 3.5.2 (USBR, 2022) provides detailed descriptions of each flow control type, their characteristics and applications.
- Section 3.5.2 (USBR, 2022) provides detailed descriptions of each flow control type. Section 3.5.2.3 and Table 3.5.2.3-1 of the section provides a summary of the spillway types, and their characteristics and applications.

Examples of some of these uncontrolled and controlled spill outlet types from New Zealand dams are shown in Figures 6.11 to 6.21.

Figure 6.11: Controlled - low-level outlet – Clyde Dam (provided by Contact Energy)

Figure 6.12: Uncontrolled - spillway morning glory spill outlet – Lower Huia Dam (provided by Watercare Services)

Figure 6.13: Uncontrolled - spillway - free overflow ogee crest weir – Waitaki Dam (provided by Meridian Energy)

Figure 6.14: Uncontrolled - spillway - side spill labyrinth weir – Ohau C Canal (provided by Meridian Energy)

Figure 6.15: Uncontrolled – spillway - lateral or side channel spill outlet – Upper Mangatawhiri Dam (provided by Watercare Services)

Figure 6.16: Controlled - spillway - radial gate – Benmore Dam & Te Anau Lake Control (provided by Meridian Energy)

Figure 6.17: Controlled - spill gates - flap gate at Paerau Weir (provided by Manawa Energy)

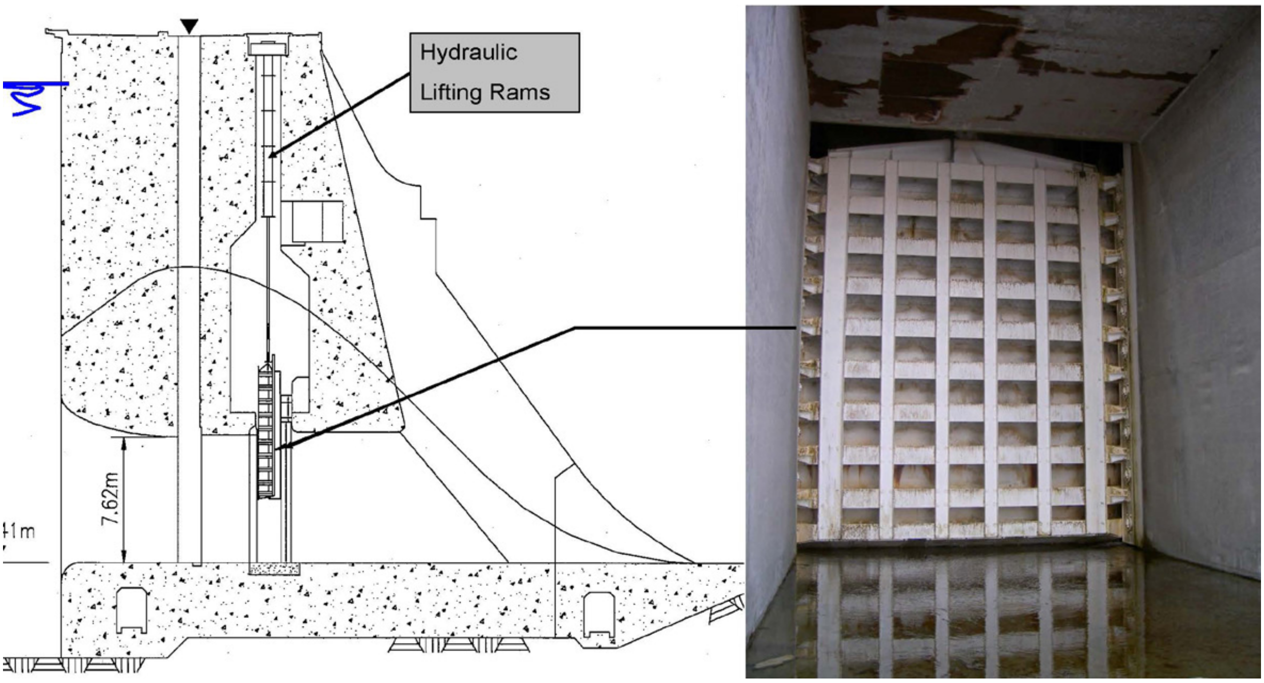

Figure 6.18: Controlled - sluice gates - orifice vertical lift sluice gate Benmore Dam (provided by Meridian Energy)

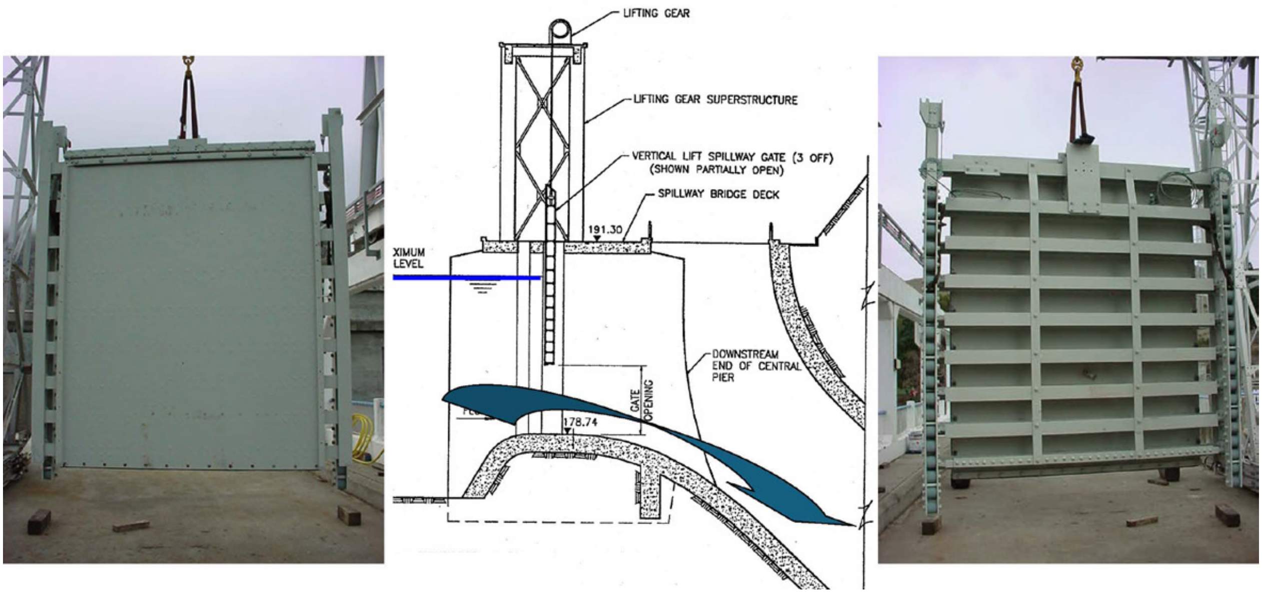

Figure 6.19: Controlled- spill gates - vertical lift wheel gate – Maraetai dam – Provided by Mercury Energy

Figure 6.20: Controlled - inflatable rubber dam (not in NZ)

Figure 6.21: Controlled - spillway - auxiliary spill outlet with fusible weir Opuha Dam (provided by Opuha Water)

6.12.8.5 Spill outlets - design considerations - general

USBR (2022) provides a checklist that summarises the USBR approach to analysis and design of both new spill outlets and modifications of existing spill outlets. This includes “Checklist – Spill Outlet Design Considerations” (section 3.3.2, and 3.3.2.1 - checklist) which itemises technical activities. A corresponding table in the USBR (2022) publication shows the design steps.

The UK Environment Agency, DEFRA, also provides current guidance on the main design principles, methods and current best practice for the design of new spill outlets and evaluation and upgrading of existing spill outlets (DEFRA, 2022a, 2022b, 2022c).

All DSCS spill outlets and discharge inlet structures should be designed to loading and performance criteria consistent with the PIC of the main dam. as such, in addition to fulfilling their operating and DSCS design functions for normal, unusual and extreme loading conditions, they should also be capable of safely withstanding the loading conditions that occur during such events and be operational during and/or following such events.

A key requirement is that the spill outlets should be designed such that damage that could trigger a potential dam failure mode in a single event is not feasible, and that the inspection and adequate repair of damage from a large single event is practical within a reasonable period, given the likelihood of a subsequent unusual spill discharge.

It is important to consider the likely operational scenario for the reservoir and spill outlet following a damaging load event (earthquake, flood or other) as well as the duration for which the spill outlet(s) may have to operate before inspection and repair can be conducted. This consideration should account for potential damage to the normal operational outlets from the reservoir, or damage to the main dam or its appurtenant structures.

Design and emergency preparedness provisions should also consider that an extreme inflow event is possible at any time during the life of a dam and can occur with relatively little warning (i.e., days of warning at best). In an extreme inflow event, the dam and its spill outlets will be tested to their design limits. Once the event has started, it cannot be stopped, and there is very little the dam Owner can do to address any issues that arise during the passage of the event. Thus, the dam and its spill outlets must withstand the design or safety evaluation event without failure and without damage that could lead to the initiation of a potential failure mode by following the 'flood rules' for the dam.

The design of a dam and its appurtenant structures should include the development of documented flood rules that will define how inflow events will be safely managed for the reservoir using combinations of reservoir flood storage and spill outlet capacity. These flood rules should take account of the downstream effects of the flood discharges and be agreed with the Owner, consenting authorities and - where appropriate - the downstream communities. These flood rules will need to be reviewed and may need to be updated during the dam's lifespan as understanding of inflow hazard develops. Flood rules should be adequately documented, along with review and updating processes.

Where the dam is in a cascade scheme, all the dams in the same catchment are likely to experience the extreme event at the same time and must all withstand the event without failure, or the possibility of significant intervention. Therefore, it is of critical importance that the spill outlets and all the associated DSCS for a dam are designed and constructed to be robust, reliable and to be able to be maintained to be available to pass an extreme flow event at any time during the lifetime of the dam.

General considerations for selecting type, size, and number of spill outlets for a new or existing dam include project requirements (frequency and duration of operation, along with flood control requirements); dam type (concrete, embankment, composite); site conditions (topography, geology, and climate); hydrologic and seismic loading requirements; and requirements for diversion during construction.

The design of reservoir spill outlets is a complex multidisciplinary task involving detailed hydrology, hydraulic, geotechnical, structural, and sometimes mechanical and electrical calculations and analyses. Often, the design must interface with operational, maintenance, environmental, social, or other aspects. These aspects are particularly important where upgrading spill outlet capacity at existing reservoirs. For such projects, the ongoing operational requirements, flood conditions to be accommodated during the construction phase, and the forms of temporary works and their potential impact on the integrity of the existing structures may be governing factors for the spill outlet design.

As covered in section 3 of this module, lead designers for spill outlets should ideally have experience in the civil, structural, hydrological, hydraulic, and geotechnical aspects of spill outlet design, as well as familiarity with the specific types of structures and materials used. This expertise allows designers to balance often competing - and at times opposing - design needs in the interests of the project's safety and economy.

Optimisation of the design should be achieved through comparative analysis of several viable options developed based on the spill outlet location, type, and size. The design may also be affected by the dam type (for example, concrete or embankment) or could affect the latter in the case of new reservoirs. Analyses should consider the cost, benefits, and risks associated with each option and make a recommendation on the preferred option.

The optimum location of each spill outlet will be site specific, but there are some overarching considerations to keep in mind when locating spill outlets.

- Wherever it is practicable, spill outlets and their discharges should be sited well clear of an embankment dam, and clear of a concrete dam if its foundation is potentially erodible. This is to minimise the risk of damage to the dam in extreme events and/or failure of a spill outlet.
- Where the total design discharge for a reservoir is to be provided by more than one spill outlet, the spill outlets must be designed and configured so that, in combination, they can safely discharge the design flood.
- Auxiliary spill outlets and discharge channels should be located and/or protected to minimise the potential for erosion to threaten the safety of a dam, to result in excessively high downstream discharges, and to result in excessive downstream environmental damage.

Preferred locations for a spill outlet include:

- Dam abutments, adjacent to or near the ends of the dam (especially embankment dams). This includes both surface and subsurface (tunnel) spill outlets for embankment, concrete, and composite dams. The location of the spill outlet will depend on topography, geology, and economics.
- Reservoir rim (located away from the dam). This includes both surface and subsurface (tunnel) spill outlets for embankment, concrete, and composite dams. The location of the spill outlet will depend on topography, geology, and economics. It should be noted that, although a spill outlet can be located on or through the reservoir rim, care should be taken to evaluate the exit channel and downstream area. In some situations, locating a spill outlet on or through the reservoir rim would allow discharges to enter a different drainage area than that associated with the main river. During spill outlet operation, this could adversely impact downstream areas that were not subject to flooding prior to the dam or spill outlet being constructed. The downstream consequences, including both property damage and potential loss of life, must be fully evaluated before determining the location of a spill outlet that could discharge flows into a different drainage area or a tributary that joins the main waterway downstream of the dam.

A spill outlet should not be located over or through an existing or new embankment dam, or the embankment portion of a composite dam, unless there are very unusual circumstances. An unusual circumstance might involve an existing embankment dam where the reservoir surcharge and discharge capacities of existing hydraulic structures are inadequate to safely pass flood events and the dam abutments and/or the reservoir rim do not offer feasible locations for a spill outlet. In this case, an auxiliary or emergency spill outlet in the form of overtopping protection of the embankment dam could be considered. Further information on overtopping protection is detailed in USBR (2014).

A spill outlet can be located on or through (integral with) an existing or new concrete dam. Also, the concrete portion of a composite dam may be able to accommodate the spill outlet. Locating the spill outlet on or through the concrete dam or the concrete portion of a composite dam would be acceptable provided it does not cause unacceptable stress concentrations within the dam.

There may be both economic and technical reasons to have the spill outlet integral with the dam, which could provide spill outlet releases with the most direct path between the upstream reservoir and the downstream river or stream.

Although spill outlets have been placed in or on new conventional mass concrete dams with minimal disruption to construction operations, care must be taken if placing spill outlets in or on a new roller compacted concrete (RCC) dam to avoid significant impacts to construction operations. To minimise impacts consideration should be given to either isolating the spill outlet from RCC operations and/or constructing a stepped chute spill outlet which utilises the downstream dam (stepped) face.

Spill outlet foundations require specific consideration. A spill outlet can be located on rock or soil foundations but, if available, it is highly recommended that a spill outlet be located on a rock foundation. Significantly more robust design and construction considerations will be needed for a soil foundation, as well as greater maintenance and monitoring during operation, particularly of the under drainage system.

For all spill outlets, the likelihood and consequences of spill outlet blockage should be carefully evaluated. This is particularly important for orifice spill outlets with small inlet structures (e.g., sluice and low-level outlets, tunnel spill outlets, morning glory spill outlets) and service spill outlets for flood detention dams. Usually, where there is a significant likelihood of spill outlet blockage, the preferred option is to provide an auxiliary spill outlet and discharge channel.

Where flow conditions in approach channels are unsymmetrical, spill outlet facilities differ from conventional spill outlet designs, or energy dissipation facilities incorporate flip buckets or nappes discharging into scour holes, hydraulic modelling should be completed to confirm design assumptions and finalise design details (e.g., head/discharge characteristics, chute wall heights, energy dissipation details, flow conditions downstream of energy dissipation facilities).

In many cases, computational hydraulic modelling will be sufficient (e.g., head/ discharge characteristics, chute wall heights); however, physical hydraulic modelling may be necessary for the detailed design of spill outlets with unsymmetrical inflow conditions, unconventional energy dissipation facilities and flip bucket shapes, and the determination of stable energy dissipation scour hole dimensions. However, the capabilities of computational hydraulic modelling are improving over time and may be viable for these applications.

Bridges located above spill outlet gate structures should be suitably restrained to ensure they do not obstruct flow in extreme events, are operational following the SEE, and to ensure they do not impede gate operation following the SEE. Spill outlet bridge performance is especially important if spill outlet operating equipment required for post-SEE operation is fixed to the bridge. In addition, any gantry cranes which service appurtenant structures should be designed to be parked in locations where they cannot prevent or obstruct spill operation if toppled in a high wind or seismic event. Alternatively, they should incorporate anchoring systems to prevent the toppling of the gantry and crane under extreme wind or seismic loads.

Care is required in the design of spill outlet features to ensure spill outlet discharges are not adversely affected by physical impediments (e.g., the bottoms of gates in their fully open positions, winch platforms, gate trunnion bearings, stored stoplogs, boat and weed booms) and high tailwater conditions.

Adequate provision should be made to allow ongoing inspection, test and maintenance of all elements of each spill outlet, such that it can be relied upon for safe operation, particularly unusual and extreme flood at all times throughout its operating life.

Provision should also be made to allow for condition monitoring inspection and potentially temporary repairs during spill events, noting that unusual and extreme spill events can require prolonged spill outlet operation. This is required as, for an extreme flood event, all of the spill outlet facilities will typically be required to operate at, or close to, their full capacity. It should be noted that even for unusual flood events, the spill outlets will be subject to loads and demands beyond those they can be tested for during commissioning and in operational flow testing.

Where local conditions can adversely affect the reliability of gate operation, consideration should be given to the adoption of an ungated spill outlet. Local conditions that can affect the reliability of gate operation include a remote site, difficult site access, a lack of skills for gate operation and maintenance, debris accumulation, reservoir slope instability, and short peaking times for flood events. If an ungated spill outlet has piers and a bridge, these must be positioned to provide sufficient flow passage and minimise the risk of debris accumulation.

6.12.8.6 Spill outlet - design considerations - flow duration and intensity

It is important to consider the effects of spill flow duration and intensity on the safety of a dam.

Where the hydrology and scheme configuration for a particular dam indicates that its spill outlets can be expected to be required to discharge large flows for sustained periods, severe damage can occur and may affect the safety of a dam. In such cases, it is prudent to adopt conservative design details for all elements of the spill outlets.

Conversely, where the hydrology and scheme design are such that spill outlets are expected to discharge high flows only for short durations and that damage in such events will not affect the safety of a dam, design details can be less conservative, with provision for any necessary repairs or protective works to be undertaken following damaging flood events.

6.12.8.7 Spill outlet - design considerations - low-level outlet structures

For Medium and High PIC dams, the necessity for a low-level outlet facility that enables the reservoir to be lowered in an emergency to mitigate potential failure modes, or operationally to allow inspection or maintenance activities, should be carefully considered. This consideration is particularly important for embankment dams situated near active faults.

If a low-level outlet is not installed for reservoir dewatering, then the ability of other facilities (e.g., low set spill or sluice outlet gates, penstock intake structures and water supply outlets) to provide sufficient dewatering in a potential dam safety emergency should be considered. The hazard and risk posed by the remaining pool after reservoir lowering (i.e., the likelihood and consequences of a dam failure following the reservoir lowering) need to be understood. An understanding of the capability to lower and control the reservoir level under post-SEE conditions is essential for Medium and High PIC dams. USBR and DEFRA provide guidance on recommended drawdown rates for reservoirs; however, ideally the requirements should be informed by assessment of the drawdown rates necessary to mitigate the development of potential failure modes for the specific dam and appurtenant structures.

Low-level outlet facilities often incorporate intake/outlet towers which can be free-standing on an enlarged base or foundation mat, or structurally tied to the upstream face of concrete gravity dams. Examples include the shafts of morning glory spill outlets and water intake towers. The failure of an intake/outlet tower during or following a large earthquake could result in the uncontrolled release of a reservoir and, as such, all intake/outlet towers should be designed to accommodate SEE loadings.

ICOLD Bulletin 123 (ICOLD, 2002b) provides guidelines for the seismic analysis and design of intake/outlet towers. The partial embedment of intake/outlet towers in new embankment dams needs careful consideration due to the potential for embankment displacements associated with seismic load cases.

6.12.8.8 Spill outlet - design considerations - approach channel

The levels and configuration of the inlet structure should be designed to meet the following main requirements for unusual and extreme flood load cases:

- Provide sufficient flood freeboard to the dam crest or to the critical water-tight element of the main or appurtenant structures (i.e., concrete dam crest or embankment dam core).
- Allow safe passage, or retention and removal, of floating debris.
- Prevent erosion of or of upstream of the inlet, including by reducing approach velocity.
- Achieve acceptable flow transitions into the flow control or transmission conduit.
- Avoid significant vibrations due to un-cushioned fall of water from height or hydraulic jumps in the vicinity of the embankment.
- Allow control of the reservoir outflow and downstream flooding
- Control the frequency of operation of the auxiliary or emergency spill outlet, to effectively manage their risk of failure and maintenance cost.
- Prevent significant noise generation by reducing the energy dissipation at the inlet as much as practically possible, where it is located close to an inhabited area.

In addition, for projects involving upgrading the spill outlet capacity of existing reservoirs, there may be a requirement to prevent any increase of downstream flood risk in low-order flood events.

6.12.8.9 Spill outlets - design considerations - inlet - use type flow control

Auxiliary spill outlets with fusing mechanisms to initiate discharge require special care in their design. The sizing and sequencing of the fuse elements require very careful consideration. In addition, the incremental consequences of fuse elements breaching to the downstream area need to be evaluated to determine the size of the fuse elements that will provide an appropriate balance between dam safety in a flood event and downstream consequences. It should be noted that fuse type outlets are not permitted in some jurisdictions due to the incremental hazard to downstream population from faulty operation of the fuse devices.

Fusing mechanisms must be reliable throughout the lifetime of the dam, vegetation should be managed, and vehicles should not be allowed to travel across fuse plug embankments. Pilot channels or similar features that encourage initiation of fuse breach as designed should be installed in fuse plug embankments to enhance their reliability and be maintained during their life.

An example of an auxiliary spill outlet with a fuse weir mechanism is shown in Figure 6.21.

6.12.8.10 Spill outlet - energy dissipation - design considerations

The extent of energy dissipation to be provided downstream of the conveyance structure generally depends on the erodibility of the receiving watercourse and the risk of undermining the spill outlet structure. This could result in head-cutting of the reservoir embankment or pose a risk to adjacent structures or the environment.

Normally, the safest, but most expensive, method of preventing excessive erosion downstream of the conveyance structure is to provide full energy dissipation of the high velocity supercritical flow within a concrete-lined energy-dissipation structure. In this way, flow will enter the receiving watercourse in subcritical mode with a well-established, close to uniform, velocity profile and will not pose a risk of erosion. However, this would require a very long structure which is usually not considered economical.

Therefore, it is generally considered acceptable that a portion of the high turbulence generated during the process of energy dissipation persists downstream of the energy dissipator. This is subject to flow being subcritical with suitable erosion protection provided to prevent undue erosion of the downstream channel, and a suitable concrete or piled cut-off at the end of the energy dissipator, to prevent undermining of this structure. Such erosion protection should be sized to resist the maximum predicted bottom velocity and would normally take the form of rip-rap rock.

Where the energy dissipation structure is founded on competent rock, it may be possible to demonstrate that the structure will not be undermined by erosion during the extreme load flood event. In this case, the extent of erosion protection provided could be limited to the risk associated with the infrequent maintenance or reinstatement that the Owner would be willing to accept in exchange for the initial saving achieved.]

To achieve subcritical flow conditions downstream of hydraulic jump stilling basins, sufficient water depth and basin length must be provided. The tailwater water depth required for submergence of the hydraulic jump should exceed the subcritical conjugate depth of the incoming supercritical flow by 10 to 15%. It could be achieved via an end sill such that the water depth remains independent of any changes occurring within the receiving watercourse during major storm events.

It is important that the end sill is carefully designed so as not to cause undue erosion downstream of the stilling basin, while creating a 'reverse hydraulic roller' to ensure that downstream scoured bed material is driven upstream against the downstream end of the basin, effectively protecting the downstream end from being undermined (Mason, 2004). A suitable drainage arrangement should also be provided to allow the stilling basin to be regularly inspected.

In order to prevent hydraulic jump sweep out, the stilling basin should be long enough to accommodate the hydraulic jump roller length and the backwater curve upstream of the end sill. For preliminary design purposes, a total stilling basin length 25% greater than the predicted hydraulic jump length could be used.

Where the stilling basin is constructed on rock, and the tailwater depth is generated by the receiving watercourse, there may be potential for the stilling basin length to be shortened to about 60% to 80% of the length of the hydraulic jump. However, this would require a robust assessment of the potential impacts and risks to the downstream environment, infrastructure or public.

Where the tailwater depth is controlled by an end sill, reducing the length of the stilling basin below the length of the hydraulic jump could cause a severe sweepout and damages further downstream which may be difficult to predict.:

Careful consideration should also be given to tailwater conditions for the energy dissipator, which affect the design and performance of energy dissipation facilities, accounting for the potential for long-term changes in the tailwater conditions that could result from modified downstream river conditions.

6.12.8.11 Guidance on gate and valve systems used in spill outlet and discharge inlet systems

Appurtenant hydraulic structures often incorporate gate and/or valve systems. These gate and valve systems perform operational and/or dam safety functions during normal, unusual, and extreme loading conditions and/or following unusual and extreme loading conditions. They are the most common form of DSCS found in dam systems. Other types of DSCS are listed in section 6.12.9 and 6.12.10.

The plant and equipment making up the gate or valve system varies according to the layout and features adopted for the specific a dam. Plant and equipment often includes:

1. Data acquisition systems – metrological info, rainfall, reservoir inflows, reservoir storage level monitoring and data transmission system.
2. Data processing systems – analysis and assessment of reservoir inflows and storage, alert processes if spill likely.
3. Communication systems between the reservoir and reservoir control and/or operations staff, and between data processing and spill outlet control systems.
4. Spill outlet control systems including, gate controls, Programmable Logic Controllers (PLCs).
5. Power supply systems to all data acquisition, processing, communications and control systems, including backups and fuel supplies.
6. Flow control devices position control (opening and closing) equipment, e.g., winch and rope hoist systems, hydraulic ram systems, motorised valve actuators.
7. Flow control devices – gates or valves and ancillary equipment – bearings, seats, seals, guides, wheels, etc.
8. Support structures, to flow control equipment, piers, deck or bridge for position control, power and control equipment.

Guidance on Items 1 to 6 can be found in sections 6.12.9 and 6.12.11. This section provides guidance on the selection and design of item 7 - flow control devices.

Guidance relating to the operation, inspection, maintenance and testing of DSCS gates and/or valves are included in Module 5 (Dam Safety Management).

6.12.8.12 Gate & valve type suitability for design applications

Table 6.21 has been developed from Sehgal (2000). This table outlines selection criteria for spill outlet gates and their control and operating equipment.

Table 6.21: Suitability of gate types to spill outlet design requirements

Design req'	Radial gate	Radial gate	Vertical gate	Vertical gate	Flap gate	Inflatable rubber
	Crest Mounted	Submerged	Crest	Submerged	Crest	Crest
Passage of Flood	Yes	Yes	Yes	Yes	Yes	Yes
Storage above spill outlet crest	Yes, Gate height limited by height of piers	Yes, Gate height not critical to storage	Yes, Gate height limited by height of piers	Yes, Gate height limited by height of piers	Yes, Height limited~5m Due to high lifting loads	Yes, Height limited ~5m Due to strength of material
Passage of Floating debris	No Except when gate fully open. Underflow gate	No	No Except when gate fully open. Underflow gate	No	Yes Overflow gate	Yes Overflow gate
Sediment sluicing	Yes, but ineffective for high crest heights	Yes Limited area of effect large rocks or debris may prevent full closure	Yes, but ineffective for high crest heights	Yes, Not preferred. Sediment can jam wheels. Turbulent flow can damage wheels. Large rocks or debris may prevent full closure	Limited Except when gate fully open	No, except when gate fully open

USBR (2022) also provides detailed descriptions of each flow control type (refer section 3.5.2 of USBR, 2022).

Design of the gate structure should take account of all feasible gate operating and opening configurations.

Examples of feasible loading conditions include, but are not limited to, the following:

- Normal loading conditions – All fully gates/valves open, all gates/valves fully closed, adjacent gates/valves fully open and fully closed. Various configurations of unequal opening and partial openings also need to be considered.
- Unusual loading conditions – All maintenance bulkheads in place, adjacent maintenance bulkheads in place and not in place, stilling basin dewatered for inspection or the completion of remedial works, and normal gate/valve configurations with the OBE.
- Extreme loading conditions – All gates/valves open for flows up to the IDF, adjacent gates/valves open and closed for flows up to the design flood or Safety Evaluation flood, configurations of unequal opening and partial opening for flow sup to the IDF, discharges through flow conduits and energy dissipation or terminal structures up to the IDF and rapid reductions in discharge during recession of the IDF. Normal gate/valve configurations for seismic events up to the SEE, annual flows through the spill outlet for a period following an SEE event, and dewatering flows following an SEE event.

These operating conditions occur in spill outlet, canal inlet and penstock intake structures, and bottom outlet gate/valve structures.

The design of spill outlet should also consider the potential need to close the flow control gates during an extreme inflow event in order to inspect and/or carry out temporary repairs to the spill outlet conduit or energy dissipation structures. The gates should provide sufficient freeboard such that access to the spill outlet during an event of (say) 70% of the IDF will be feasible for inspection and temporary repair. The design flow rates should be determined considering the potential failure modes for the specific spill outlet.

In addition to normal, unusual, and extreme seismic and flood loading conditions, all designs should include consideration of other unusual loading conditions including equipment malfunction (e.g., hoist rope failure, seized trunnion or roller bearing, jammed gate), gate over-pour, floating debris and sunken log impacts and effects, and flow induced vibrations. ICOLD Bulletin 102 (ICOLD, 1996a) includes detailed discussion on flow induced vibrations and guidelines to limit flow induced vibrations.

- All gates should be designed and detailed to limit deformation that could result in gate jamming or loss of sealing capacity, under normal, unusual and extreme load conditions. Excessive leakage could limit access for unjamming operations, or for access to inspect and repaid the flow conduit and energy dissipation structures.
- All gate and valve designs should consider the durability of materials and critical components (e.g., bearings, bushes, trunnions, winches, lifting points, pins, rams etc.) to assure that they are appropriate to maintaining the serviceability of the gate or valve during it design life.
- Designs should minimise the threat of corrosion that may affect availability and reliability or reduce service life of the flow control system.
- All structural arrangements should facilitate ready access for the operation, inspection, maintenance, repair or replacement of gates, valves and their components.
- Safety in Design methodologies should be employed to assure that there is safe access to operate, insect test and maintain the systems under all conditions, including emergency conditions and during exceptional circumstances (e.g., storm, high winds, failure of external power supplies, extreme winter or summer conditions, etc.).
- Design should include provision of facilities to allow the flow control equipment to be isolated (e.g., stoplogs, bulkheads, guard valves, etc.) to allow the safe and inspection, maintenance and repair of DSCS gates and valves for normal, unusual and extreme load conditions, as well as emergency conditions and during exceptional circumstances.
- Similar isolation facilities should be provided on the downstream side where access is not readily available due to elevated tailwater conditions.
- These isolation facilities are not primary flow control devices and will generally only be installed and operated under no-flow conditions. However, while generally not required to perform a dam safety critical function, consideration should be given to designing bulkheads such that they can be closed against flow, to allow flow through a jammed or damaged gate to be stopped without the need for the reservoir to be drawn down to the gate sill level.
- Isolation facilities should be designed such that they can be installed and removed effectively and quickly under all operating conditions. Consideration should be given to the crest level of these isolation facilities, including wave freeboard allowance and to minimise the risk of them being overtopped in the event of a rapid inflow to the reservoir while in service.

An example of an upstream bulkhead is shown in Figure 6.22.

Figure 6.22: Upstream bulkhead

Flow control gates and their ancillary equipment (including dam safety critical equipment) typically have design and service lives that are significantly less than the design life of the dam or appurtenant structure. Accordingly, all such equipment should be designed and detailed in a manner that enables practical inspection, maintenance, testing and repair.

Consideration should be given to how the equipment, including embedded components, can be replaced when it is no longer capable of reliably fulfilling its design functions. This is a particular consideration for main elements such as gates, gate trunnion, gate trunnion anchor systems, gate lifting systems etc. Ideally these main elements should be able to be replaced without having to excessively constrain reservoir operations, or in a way that increases the risk of failure of the dam during replacement. The replacement requirements and methodologies should be documented and agreed with the Owner in the design process.

Design of dividing walls and piers in spill outlet gate structures should consider the cross-valley component of the SEE earthquake loadings, as well as any amplification effects, and to ensure that post event deformations of the structures would not result in jamming of the flow control device (gate or valve). Examples of gate structures with piers are shown in Figures 6.16 and 6.17.

6.12.8.13 Conduits and conveyances - design considerations

There are a number of design details for appurtenant hydraulic structures that can affect dam safety and warrant careful consideration during their detailed design.

Design details associated with conduits through and beneath embankment dams, and interfaces between concrete structures and embankment dams are discussed in section 6.5.5. Additional design details associated with conduits, ungated and orifice spill outlets, and low-level outlet facilities are discussed in ICOLD bulletins, as well as USBR and UK Environment Agency guideline documents.

The size of the conveyance structure is largely dictated by the hydraulic capacity required to pass the predicted reservoir outflow and, for open channels, by the additional need to provide a suitable freeboard to accommodate air bulking, shock waves, surface roughness, spray and splash and any modelling uncertainties in order to prevent overtopping. However, in the most typical case of an open channel spill outlet structure, there could be different width/height combinations that would allow the passage of this flow. The selection of the optimal parameters should be based on comparative analysis of a range of spill outlet chute width/height configurations giving due consideration to all relevant design aspects and implications resulting from variations in chute configuration.

For example, reducing the spill outlet chute width could have several beneficial effects, including:

- reduced number of slab joints,
- potential improvement of the dissipation of the maximum uplift pressures acting along the centreline on the base slab,
- easier access for cleaning and inspection of under-drains, where present, and
- improved resistance to uplift and increased structural strength/reduced bending moments.

Consideration should also be given to the possible adverse effects of reducing the spill outlet width, including:

- Increase in the reservoir maximum water level, where the chute width may affect the overflow weir length or its submergence.
- Increased structural depth and water depth resulting in increased bearing pressure, settlement, and loading on the side walls.
- Potential increase in the foundation depth of the collecting channel where flow is subcritical. This may be due to the need to prevent the subcritical flow controlling the reservoir level. An increase in the foundation depth might increase uplift pressure, where it is present, and may also require a larger drainage capacity and more costly access and maintenance arrangements. It would also increase the construction cost.
- Increase in the unit discharge and corresponding hydrodynamic forces acting on the spill outlet structure which are then transferred to its foundation.
- Increase in the length and depth of the energy dissipator.
- Potential generation of cross waves and resulting dynamic forces where the conveyance structure becomes narrower than the inlet structure.
- Increased risk of cavitation where such risk is present.
- Increased risk of blockage by floating debris where the width reduction becomes significant enough to cause blockage.
- Increase in the freeboard required to accommodate air bulking, surface roughness, wave action, spray, splash.

An increase in the spill outlet width would naturally result in the opposite effects.

Conveyance structures must provide sufficient freeboard to account for:

- Non-uniform and asymmetric flow conditions.
- Shock waves, roll waves and flow regime transitions.
- Increases in surface roughness due to abrasion or scour.
- Flow bulking due to air entrainment.
- Spray and splash.
- Modelling uncertainty, particularly when using simplified models.
- Construction accuracy tolerances, particularly of joints.

Where these phenomena cannot be accurately determined and assessed, an additional safety margin should be allowed for the worst-case scenario.

Simplified empirical formulae for minimum freeboard should not be used in conditions that differ from those for which they were developed (e.g., non-uniform flow conditions, converging or diverging chutes, bends in horizontal alignment, increased air entrainment due to strong shockwaves, roll waves or regime transitions, etc.).

Design of conveyance structures with complex geometries in Medium or High PIC Dams should consider the use of three-dimension physical hydraulic models or numerical CFD models to more accurately estimate flow patterns and behaviours. However, modelling of air entrainment remains a complex issue and should be conservatively assessed during design.

Limited overtopping of chute sidewalls by shock waves may be acceptable if it can be demonstrated that the ground adjacent to the chute can tolerate the effects of overtopping in the worst-case design condition, without inducing failure of the conveyance structure or dam. This requires careful analysis of the magnitude and cumulative duration of overtopping to estimate the extent of potential damages and their impact on the conveyance structure and dam, as well as careful design and detailing. The potential impact of overtopping on the drainage system's ability to perform its function must also be considered.

NZS 3114:1987 Specification for concrete surface finishes (Standards New Zealand, 1987) is commonly used to specify the surface finishes and tolerances for abrupt or gradual variations from the design geometry for spill conveyance structures. Experience has found that the NZS 3114 surface finishes and tolerances are very difficult to achieve in such structures. International practice for spillway specifications has advanced and manages cavitation risk through specification of more achievable tolerances and/or the inclusion of aeration devices. Achieving adequate construction quality through consistent concrete supply, good compaction, smooth slip-form progression, surface consistency and well-formed joints - thereby avoiding excessive re-work, grinding or patching - has been found to be more critical to good long-term conveyance performance than stringent geometrical surface tolerances. Use of cavitation indices can be considered as a basis for determining appropriate concrete surface tolerances in conveyance structures.

6.12.8.14 Conduit underdrainage - design considerations

Underdrainage for spill outlets should be provided where there are no other more cost-effective or reliable alternatives for mitigating the risk of flotation, rather than being used solely as a redundancy measure. Where drainage is provided, it will also mitigate the risks of frost heave, scour of foundation material, and reduction of the shear strength of the foundation material due to partial saturation and dynamic loading.

The drainage system should be designed to ensure safe spill outlet performance under all design and operational scenarios. In particular, it should be designed to allow effective monitoring of performance during spill, as well as allowing for inspection, cleaning and maintenance. The Independent Forensic Team report into the Oroville Spillway incident provides background on the consequences of poor design and management of underdrainage, and guidance on best practice (IFT, 2018).

All drainage systems in contact with soil material should be provided with suitably designed graded sand filters to prevent erosion of the underlying soil material and also preventing erosion of the filter material into the drainage system. Filters should also have sufficient permeability to pass the required drainage flows without risk of clogging.

Due to their vulnerability to clogging, geotextiles or other woven or nonwoven fabrics should not be used instead of soil filters in any underdrainage systems or in back-of-wall drainage systems which are critical to the safety of the spill outlet.

Where spill outlets are to be installed over earth embankments, a reliable drainage solution with a high degree of redundancy is required. This could be achieved by providing a continuous drainage layer consisting of a coarse granular material and a suitable filter layer. Both layers should be sized and compacted to provide ample drainage capacity and prevent liquefaction and frost heave. Measures should be taken to prevent concrete or blinding concrete from entering the drainage layer, such as using suitable barriers, using low slump class blinding concrete, or other measures.

Piped drainage systems should provide ample redundancy to allow for cleaning and maintaining of the system without loss of drainage capacity. This is particularly important where longitudinal collector drains are provided under wide chutes.

All pipes should be designed with sufficiently large diameters, long radius bends, and steep gradients to allow the drainage system design flow rate to be conveyed under free-surface flow conditions with a velocity preventing sedimentation. They should not be positioned close to the spill outlet structure or laid at too shallow a depth, to prevent risk of damage by the spill outlet vibrations or freezing.

Where the insulation requirements for protecting the under-drainage pipework from freezing cannot be met by the spill outlet concrete slab thickness and the thickness of the drainage layer above the drainage pipe alone, suitable additional insulation should be provided.

No drainage outlets should discharge within the spill outlet channel or energy dissipation structure where high water levels and pressure fluctuations could be present, as they could be transmitted beneath the upstream slabs or behind the spill outlet walls. This could increase the uplift pressure acting on parts of the spill outlet above the pressure they have been designed for.

The spill outlet underdrainage system should be separated from the spill outlet wall backfill and hillside drains so that it does not become overwhelmed during a severe rainfall event in line with the spill outlet design event.

6.12.8.15 Conduits through or beneath dams – design considerations

Where conduits are installed through or beneath dams they should ideally be designed to ensure non-pressurised flow conditions. They should also incorporate upstream bulkhead facilities to isolate the conduit from the upstream reservoir and be of a sufficient size to enable inspection and the completion of any necessary repairs.

Where pressurised conduits are installed through or beneath embankment dams - preferably only in Low PIC dams less than 10 m in height - they should incorporate suitable facilities for the detection and monitoring of any conduit leakage, and an upstream valve to enable maintenance or replacement of the outlet control valve. They should also be designed to withstand the internal pressures associated with rapid closure of the outlet control valve and be pressure tested to 150% of the maximum operating pressure prior to backfilling.

Where pressurised conduits are installed through concrete dams, they should be steel lined and incorporate an upstream bulkhead facility to isolate the conduit from the upstream reservoir. The steel lining should be designed to withstand the maximum negative and positive internal pressures associated with rapid start up and closure of the downstream control facility (i.e., valve or turbine).

Ideally, any necessary changes in the directions of spill outlet chutes should be included where subcritical flow conditions occur. If changes in alignment are necessary where supercritical flow conditions occur, they should be large radius bends and should be designed to ensure the resulting shockwaves are contained within the chute.

Particular attention should be given to the adverse effects of high velocity flow, turbulence and abrasion. High velocity flow and turbulence effects include cavitation, the initiation of high uplift pressures beneath slabs, and slab vibration. Abrasion effects, which are normally limited to low-level orifice spill outlets, can result in sufficient concrete loss to necessitate the completion of concrete repairs. ICOLD Bulletins 58 and 81 (ICOLD, 1987 and ICOLD 1992b) provide guidelines for the effective control of cavitation damage, and the design of chutes/tunnels and energy dissipation facilities.

The hydraulic design of a low-level outlet structure or culvert through the base of a dam requires consideration of a range of factors including:

- The horizontal and vertical alignment of the outlet structure relative to the natural watercourse.
- The form of the structure entrance and the corresponding head loss characteristics.
- The potential for outlet entrance blockage by flood-transported woody debris and other detritus, and for blockage or ingress by flood-transported sediment material.
- Air entraining vortex formation at a partially submerged entrance under flood conditions.
- The flow regime through the main body of the outlet structure under flood conditions including the effects of tailwater level influence and the potential for pressurisation of the culvert refer to further discussion below).
- Whether there is a need for ventilation of the outlet structure to supplement the supply of air from the upstream and downstream ends, and to mitigate the effects of potential flow pressurisation including the occurrence of pressure surges.
- The need for energy dissipation at the outlet structure exit under flood conditions and the form of any energy dissipation structure.
- The performance of any energy dissipation structure and the scour risk under flood conditions at the outlet structure exit.
- The facilitation of fish passage under normal flow conditions (NIWA, 2018b).

General guidance on hydraulic considerations for the design of outlet works including low-level culverts is provided in Chapter 4 of the USBR Design Standard No. 14 (USBR, 2022).

Guidance on the potential for low-level outlet structure blockage by floating debris and non-floating debris is provided in Book 6 of Australian Rainfall and Runoff (Weeks and Rigby, 2019). This includes guidance on:

- Assessment of blockage potential.
- Assessment of likely blockage levels.
- Hydraulic analysis of blocked structures.
- Management of blockage for new structures.
- Retrofitting of existing structures for managing debris blockage.
- Debris control structures for mitigating the potential for debris blockage.

Culverts through earth and rockfill type embankment dams should be designed so they do not operate as pressurised pipes during the IDF and smaller floods. If pressurisation cannot be avoided, consideration must be given to additional defensive measures to prevent the possibility of pressurised leakage from the culvert into the dam embankment.

6.12.9 Identifying Dam Safety Critical Systems (DSCSs)

Appurtenant hydraulic structures frequently incorporate Dam Safety Critical Systems (DSCSs). These are integrated plant systems designed to fulfil one or more dam safety critical functions essential for the safe control of reservoir contents and discharges.

These systems can be categorised as follows:

- **Passive Systems**: Do not involve mechanical, electrical, or automated equipment (e.g., over topping crest weirs).
- **Active Systems**: Involve mechanical, electrical, or automated equipment

When designing a new dam, managing the safety of an existing dam, or rehabilitating an existing dam, it is necessary to:

- Clearly identify the DSCSs.
- Ensure each DSCS is designed for relevant load cases and scenarios; and meets relevant dam safety performance criteria.

Note that these systems may also be located in the main dam structure, and the same considerations apply.

Not all plant systems or elements in dams or appurtenant hydraulic structures are DSCSs, as some serve only operational or ancillary functions.

In order to identify which systems and elements are considered ‘dam safety critical’, the following methodology is recommended (noting that other robust and industry recognised methodologies may also be used):

Methodology for identifying dam safety critical systems and elements:

1. **Identify the potential failure modes**: These are the ways in which the structures that retain the reservoir could potentially fail to function. For each potential failure mode, identify the key ‘trigger’ or initiating events. For example, a potential failure mode could be the flood overtopping the dam core, with the key trigger being a high reservoir level.
2. **Establish the dam safety critical functions**: These are the dam safety critical functions that are required to prevent each potential failure mode from progressing to failure if a trigger or initiating event occurs. For example, lake level high detection is triggered when the lake level exceeds a certain threshold requiring the release of surplus inflow.
3. **Identify the Dam Safety Critical Systems**: DSCSs are integrated plant systems that are required to perform a dam safety critical function. Categorising these systems is at the discretion of the dam Owner. However, some common examples include spill gate systems and penstock rupture detection systems.

4. **Identify the dam safety critical elements**: A dam safety critical element is a component of a DSCS that is essential to the dam safety critical function. The process involves discerning between critical and non-critical system elements.

Examples of DSCSs and the dam safety critical functions that they are required to perform include:

- Gate system required to act as a barrier (closed), allowing no uncontrolled release of the reservoir.
- Gate system required to act as a flow control and operate on demand to safely reduce extreme high-levels, due to excess reservoir inflows, loss of operational discharge, or other unusual events.
- Gate system required to act as a flow control and operate on demand to allow drawdown of the reservoir in a dam safety emergency.
- Penstock rupture protection system, required to detect uncontrolled leakage from a penstock system, and initiate closure of inlet control gate(s), to isolate penstock(s) from reservoir.
- Reservoir extreme high-level protection system, required to detect extreme high reservoir levels (or uncontrolled rising reservoir level) and initiate opening control gate(s) to control reservoir levels.
- Reservoir extreme low-level protection system, required to detect extreme low reservoir levels (or uncontrolled falling reservoir level) initiate closure of inlet control gate(s) from upstream reservoir.

In addition, DSCSs include the equivalent valve systems for each of the above functions.

Other examples of DSCSs are:

- Dam foundation uplift pressure relief system, required to maintain control of uplift pressures acting on dam.
- Toppling block and flashboard weir systems, that activate under unusual or extreme reservoir water levels to prevent overtopping of dam structures.
- Gravity discharge dam uplift drainage dewatering system, required to discharge normal to extreme uplift drainage inflow from dam body, to prevent excessive uplift pressures (if gallery above tailwater level).
- Pumped discharge dam uplift drainage dewatering system, required to discharge normal to extreme uplift drainage inflows from dam body to prevent excessive uplift pressures (if gallery below tailwater level). The DSCS will include, sump level detection devices, sump pumps, pump control systems, power supplies, pump discharge pipework (and/or other conduits).
- Foundation seepage monitoring system, required to detect development of seepage through dam (in the specific case, for a post fault rupture load case scenario).
- Bulkhead doors or valves that isolate sections of galleries within the body of a dam from each other, to prevent high inflows or inundation, or high pressures in one section of a gallery from affecting others.

Potential failure mode assessment for a specific dam or scheme may identify other DSCSs. NZSOLD requests that designers and Owners of such systems advise NZSOLD of these systems so that they can be considered in further updates of these Guidelines.

Guidance on requirements for DSCSs is given in section 6.12.10, and on reliability assessment for DSCSs in section 6.12.11.

6.12.10 Requirements for Dam Safety Critical Systems (DSCSs) and equipment

The following is drawn from ICOLD (2021) Operation of Hydraulic Structures of Dams, Bulletin 178.

The design of appurtenant hydraulic structures is governed by numerous factors: dam type, topographic and hydrological potential of the site and existing technology, and each scheme will be unique because of those factors. This means that it is not possible to set precise rules for the design and configurations for DSCSs.

However, there is a clear trend towards the increasing use of automation, telemetering and remote control in all operations connected with the hydraulic appurtenant structures and DSCSs for dams.

In designing new, or assessing existing, DSCSs and the equipment included in them (e.g., gates, power supplies, operating systems, control systems, etc.), reliability assessment would ideally be used as the means to assess demand and availability requirements for the DSCS to assist designers and/or safety reviewers to determine the configuration required to effectively mitigate all identified potential failure modes for all applicable loads cases and scenarios. Section 6.12.11 provides guidance on DSCS reliability assessments.

However, it is recognised that the information to robustly assess the reliability of much of the mechanical, electrical and control plant and equipment used in such systems is not well understood for dam applications. Further, there are no widely accepted tolerable risk criteria for such systems yet established in New Zealand.

The effort required to carry out such assessments - in order for them to be considered as robust- is significant, as it relies on the input of numerous specialists (including reliability engineers), and generally requires independent peer review. Such assessments may be considered impractical, or not financially justified for many new or existing dams, so the following recommendations are provided to guide designers and dam Owners.

6.12.10.1 Terminology, definitions, and background:

- Basic Process Control System (BPCS) is a functional safety term used to describe the operational control system, and its associated equipment, that does not perform any Safety Instrumented Function (SIF) as defined by IEC61511 (IEC, 2016a, 2016b, 2016c). In a dams context, this is commonly referred to as Programmable Logic Controller (PLC).
- Safety Instrumented System (SIS) is an autonomous set of controls in hazardous applications, designed to detect and mitigate hazards by enforcing a safe state upon detecting unsafe conditions, in strict compliance with the IEC61511 standard (IEC, 2016a, 2016b, 2016c).
- Layers of Protection (LOP) is also referred to as ‘defence in depth’ and is commonly depicted using the ‘Swiss cheese model’. It describes the use of multiple independent barriers that are independent from the triggering event and from other layers of protection. Bowtie diagrams have been used in recent years to illustrate, and aid in the understanding of these layers. However, it is important to note that barriers identified in a bowtie diagram may not be independent.
- Failure modes in the context of control equipment can be classified as systematic failures, common cause failures or random hardware failures. Dam Owners should take reasonable care to reduce these failure modes to acceptably low levels.
- Systematic failures are characterised as faults that consistently and predictably occur under certain conditions. These failures often originate from human errors at various stages throughout a system’s lifecycle. This could include a lack of understanding of the fundamental requirements during the design phase, improper maintenance or operation of hardware, or mistakes in software or application programming. Systematic failures are minimised by having robust lifecycle management procedures to ensure that each stage of the system’s lifecycle is undertaken with an appropriate level of diligence and care.
- Common Cause failures, refer to the simultaneous failure of multiple system components due to a single shared cause, often leading to a significant reduction in system reliability or safety, e.g., fire, seismic event, humidity, flooding. Typical measures to mitigate Common Cause Failures include diversity, segregation, diagnostic capability, and reduced proof testing intervals.
- Random Hardware failures, describe failures in hardware that occur at unpredictable times but at predictable rates. Typical measures to mitigate random hardware failures include redundancy, hardware fault tolerance (MooN), diagnostics and reliability analysis based on failure rates.

6.12.10.2 General guiding principles

A fundamental guiding principle is to ensure the resilience of DSCSs and dam safety critical functions by eliminating single points of failure. This is achieved through the application of sound engineering principles and the implementation of multiple, independent protective measures to mitigate potential failure modes. Other relevant guiding principles are redundancy, diversity, segregation, defence in depth, fault tolerance, and fail to a safe condition.

The DSCS and plant must be appropriate to:

- the potential failure mode being mitigated - e.g., local manual operation of plant, or use of portable power supplies - should not be relied upon where access to site in the load scenario, or within the timeframe for development of the potential failure mode are not realistically achievable, and
- the load scenario related to the potential failure mode - e.g., the DSCS requirements - may be different for flood load cases than for seismic and post seismic cases.

For every potential failure mode that is identified, it is beneficial to consider and evaluate the Layers of Protection (LOP) and dam safety critical functions assigned to each layer. Each LOP protection layer can be evaluated using robust methods that would typically include demonstrating independence from the triggering event and from other layers of protection. Evaluating independent LOP provides insight to the overall reduction in risk and if there are sufficient barriers to mitigate the risk, much as Bowties are currently used. If risk has not been reduced So Far As Reasonably Practicable (SFARP), it may be necessary to consider introducing additional layers of protection such as a Safety Instrumented System (SIS) or strengthening existing barriers.

IEC61511 (IEC, 2016a, 2016b, 2016c) Functional safety - Safety instrumented systems for the process industry sector provides comprehensive guidance and methodologies that are relevant and applicable to dams applications. There are known limitations that make implementation of this standard impractical in many cases. However, that should not dissuade dam Owners from utilising the standard as a valuable reference and guide, to bridge gaps that are not adequately covered by other standards, particularly in respect to enhancing the safety and reliability of control systems.

The UK Health and Safety Executive (HSE) published a study in 1995 that is commonly cited throughout modern functional safety literature. That report identified that nearly 60% of major incidents were attributed to incorrect specification and design. In that context, it is critically important that the design or evaluation of DCS Functions are undertaken with an appropriate level of skill and care (HSE, 2003).

Control and networking equipment must be appropriately safeguarded from advanced cyber security threats and malicious actors within the control system environment. In that context, consideration needs to be given to appropriate segregation between IT and OT networks, communications protocols, networking firewalls, physical security and access to equipment, and monitoring of networks.

It is crucial to acknowledge the potential for human error. Despite the best training and intentions, operator errors can occur, leading to system failures. Therefore, any design of a DSCS must incorporate this reasonable likelihood. It is not sufficient to rely solely on human response for safety measures unless it can be shown that human intervention can be relied upon to operate the DSCS in all scenarios that can be associated with the PFMs it is required to mitigate. Where human intervention cannot be relied upon, the design should include robust safeguards that can function independently of human intervention. These safeguards could be automated systems or 'fail-safe' mechanisms that activate in the event of operator error.

For a Medium PIC dam, it is considered best practice to satisfy the following minimum criteria:

- At least two independent sources of electrical supply to support local or remote gate operations.
- At least two permanently installed diverse energy sources to support local operations, e.g., permanently installed diesel electric generator, diesel hydraulic or other.
- There should be no single points of failure that result in the loss of more than one DSCS.
- There should be no single points of failure that result in the loss of a dam safety critical function unless it can be demonstrated that:
  - there are adequate backup DSCSs that can fulfil the dam safety critical function, or
  - an effective human response by trained and competent personnel can be assured under all foreseeable circumstances within a period adequate to prevent the relevant potential failure mode from eventuating.
- Instrumentation dedicated to a specific DSCS function should not be repurposed for another DSCS or dam safety critical function. This practice would only be appropriate when strict adherence to an applicable standard such as IEC61511 (IEC, 2016a, 2016b, 2016c) is maintained.
- Local manual controls are required for DSCSs if remote or automatic controls are unavailable, including any critical indications required to operate the equipment manually.

- A DSCS and dam safety critical function should be designed and installed to enable periodic testing from source to the final element.
- Control and networking equipment including sensing equipment is to be physically secure from the public and/or malicious actors.
- Appropriate safeguards from cyber security threats that align with recommended practices in this area.

For a High PIC dam, it is considered best practice to include the following criteria in addition to those for a Medium PIC dam:

- At least three independent sources of supply to enhance the security of supply for the remote and automated response of the DSC Systems. (Many critical installations have three independent supplies, refer Lewin, 2008).
- An overtopping protection system that is independent of the operation control system is required unless it can be demonstrated and independently verified that it is not required (for example where there is sufficient freeboard to allow a human response for all foreseeable scenarios).
- While somewhat subjective, the DSCSs and dam safety critical functions of High PIC dams should be more rigorously safeguarded from systematic, common cause, and random hardware failures.
- Segregation of DSCSs and use of diverse routes for power, control and comms links.

Good practice in design of DSCSs includes:

- Providing separate storage for reserve fuel supplies, multiple backup power and control units, diverse routing of power and communications cables or pipework for back-up units, locating portable back-up units in a location separate to the main power supply systems, providing back-up input transducers such as storage level indicators; and having multiple alternative communication systems (e.g., satellite phone, line of sight and/or trunked radio, cell phone).
- For most DSCSs, hand powered mechanisms are not considered a suitable back up actuation system. However, for specific cases, such as valves and small gates, which require only low operating forces, they may be assessed as suitable, provided that access to operate the manual system is assured for the relevant potential failure modes, and manual actuation can be completed quickly enough to mitigate the failure threat.
- Alternative actuation systems should be considered, e.g., if the primary actuation is an electric motor, then independent hydraulic or pneumatic actuation should be considered for back-ups.
- Ensuring DSCSs have the backup energy available to perform the dam safety critical function a number of times, over an extended period.
- Ensuring the backup system has the capacity to perform its function sufficiently quickly to mitigate all considered potential failure modes – e.g., raising spill gates quickly enough to prevent dam over topping, or trigger penstock intake gate closure quickly enough to mitigate risk to people at the powerhouse.
- Considering the likelihood that the number of competent operators and maintainers available in an emergency event is likely to be limited, and the time to implement any required repairs or operations will be longer than in normal conditions.
- The plant and equipment used in DSCSs should be extremely robust and reliable, with adequate provision for redundancy. So that, with appropriate ongoing inspection, test and maintenance, it can be relied upon to perform its design function on demand at any time during its operational life.
- The plant and equipment used in DSCSs should be configured such that it can be replaced and/or upgraded as required throughout the whole-of-life of the dam, without compromising the normal operations of the dam, or increasing the risk of dam failure while being replaced or upgraded.
- Safe access to inspect, operate, maintain and or repairs to appurtenant hydraulic structures and DSCSs, or all foreseeable load cases.
- Safe access to carry out surveillance and monitoring of civil elements of appurtenant hydraulic structures (e.g., spillways, stilling basins and their underdrainage systems, and tailrace areas).
- Safe access to carry out condition monitoring of all elements DSCSs (e.g., gate and valve systems, power, control and communications equipment), including during flood events.
- Adequate lighting should be provided for safe access and operation of the facilities at night.

- DSCSs and plant should, as far as practical, be located where water from leakage or rupture of water- carrying conduits cannot threaten their integrity. If this is not practical, appropriate protection systems should be provided to ensure the operation of the DSCS is not adversely affected.
- Facilities and structures housing relevant DSCSs and plant need to be safely accessible and functional for the post-earthquake scenario. In many instances this will require that the serviceability load case for these structures is the SEE for the dam.
- All associated power supply cables, hydraulic piping, and control and communication cables should be designed and detailed in a manner which ensures their availability following an extreme event (e.g., adequately supported to withstand the SEE, allow of differential or gross movement of structures and elements in the earthquake event). This may require physical clearance, and looped connections across contraction joints.
- Where the function of a DSCS will, or may, rely on human intervention to provide adequate risk mitigation, the nature of the human intervention should be documented as an emergency preparedness procedure (refer to Module 6 -Emergency Preparedness) and all of the physical resources that will be necessary to implement the intervention identified and maintained to be available at all times (e.g., suitable vehicle, access equipment, PPE, tools and spares, materials, manuals and documentation, etc.) and this human intervention practiced regularly as part of emergency response procedures.
- All DSCSs and emergency operating devices should have protected access for all emergency conditions.
- There is a need to minimise the chance of failure of the complete DSCS, due to a single event. Such events could include sabotage, fire, earthquake, fuel contamination, lightning strikes, extreme weather events, extreme temperatures, rockfall, communication failure or local control malfunction.
- Potential 'single points of failure' include, loss of a common switch board, control Programmable Logic Controller (PLC) or electrical transformer; a fire damaging all equipment housed in a single structure or location; type fault failures (equipment or coding) if identical devices are used for main and back up equipment.

6.12.11 Reliability assessment of Dam Safety Critical Systems (DSCSs) and equipment

Reliability and risk assessment are regularly used to identify critical components and weaknesses in systems.

Reliability assessment of new and existing DSCSs is recommended in their design and in safety evaluations. Reliability of each system should be considered in the context of the overall risk profile of the dam, including both upstream and downstream effects of failure of the DSCS.

There are broadly three categories of reliability assessment:

- Qualitative only, where the DSCS is assessed and potential failure mechanisms and reliability Issues identified, but no analysis undertake to estimate the probability of operational failure. This assessment is commonly completed and addresses potential failure modes. It sometimes includes an assessment of consequences. A type of formalised assessment typically used is Failure Modes and Effects Analysis (FMEA), or if consequence is included, Failure Modes Effects and Criticality Analysis (FMECA) (refer ANCOLD, 2022).
- Quantitative based, where the DSCS is analysed (e.g., using fault tree analysis), reliability issues identified, and reliability expressed as a value representing the probability of the DSCS failing to operate or failing to mitigate the potential failure mode (FMECA - refer ANCOLD, 2022). Quantitative reliability is theoretically the estimated number of successful operations out of a total number of operations but, in practice, is our confidence that a component will operate as planned when on demand in a potentially threatening event.
- Semi-quantitative, where qualitative assessments use readily available quantitative data to help to calibrate and ground the judgement of operational failure likelihood.

Experience has shown that judgments of the assessment of the reliability of plant systems (including DSCSs) are unreliable without a systematic, structured analysis of the total system, involving specialists in the assessment methodology, the systems and plant under assessment, and the load cases being considered.

For this reason, semi-quantitative assessment is recommended to ensure that the assessment is grounded against available quantitative data. It is also recommended that the assessments are informed by review of documented incidents and failures of relevant DSCS and plant.

For high hazard and/or high value structures, where there is sufficient quantitative information to allow a quantitative reliability of assessment of a DSCS, this should be considered.

Any assessment should ideally consider the complete system, including hardware, software and 'liveware' (i.e., human factors) (Hobbs and Azavedo, 2000). Human factors to be considered include the effectiveness of identifying the initiation of a potential failure mode, communication and decision-making systems in an event, the availability of appropriately skilled resources to respond to events (e.g., operations, electrical and mechanical maintenance staff, dam safety engineers), the training status of potential responders, and the likelihood of operator error. Another factor receiving increasing attention in reliability assessments are security Issues including vandalism, sabotage, and potential terrorist attacks, both directly and through cyber networks.

Assessments should consider complete load case and event scenarios. For an inflow event, consider from the initial receipt of inflow data (rainfall, stream gauging, meteorology etc.), through the processing of that data and the associated decision processes, to communication of operational decisions to site, the ability of operators or responders to reach the site under flood or storm conditions and implement required actions, power supplies, equipment, operation and control, right through the potential event, i.e., allowing for maintaining response for a period of many hours or days, including at night.

Studies have shown that human factors are critical factors in gate reliability, often dominating gate failure results (refer to section 6.12.10 for guidance on appropriate design).

Studies have also shown that the probability of a DSCS operating successfully improves if time is available to undertake repairs in the event of a failure to operate, or to correct a wrong decision. Hence, for any study it is important to establish the time available between the initiation of a potential failure mode and it developing to a point where dam failure becomes likely.

In many cases, this period will either be unable to be determined with confidence, or will be insufficient for reliance to be placed on human intervention (e.g., in an flood inflow event human intervention may not be possible or reliable as access to the dam site and plant may not be available for an extended period, a jammed gate, or failed hoisting system, may not be able to be returned to service following a damaging earthquake as the access and/or resources and equipment required to restore the gate to operation may not be available for an extended period). In such instances, DSCSs will need to be designed to be fully automated and sufficiently reliable that the safety of the dam does not rely on human intervention in that event.

A critical factor in reliability assessment, particularly qualitative studies, is obtaining relevant data on equipment incidents and failures.

Information for the reliability and actual failure modes of DSCSs and plant in dam applications is very limited, and dam Owners and Operators are encouraged to record, and share data on component and operational failures of DSCSs and plant with other dam Owners, NZSOLD, and the dam engineering community.

7. Performance monitoring

7.1 Introduction

Dam safety does not only depend on proper design and construction, but also on acceptable long-term performance.

Consistent, reliable visual inspection of dams is one of the most important tools for monitoring long-term performance. However, where particular design assumptions need to be validated, potential failure modes have been identified that require monitoring, or the consequences of failure are high, extensive instrumentation may be necessary for the monitoring of dam performance. The need for instrumentation to check the validity of design assumptions and monitor the performance of a dam or appurtenant structure, and the ability to monitor dam performance (e.g., it can be difficult to monitor seepage at dam toes which are founded on river gravels or inundated by tailwater conditions), should always be considered in the design of a new dam or the rehabilitation of an existing dam. In some cases, where the design assumptions are conservative and the consequences of failure are minor, the expense associated with the installation, and ongoing monitoring by instrumentation may be unwarranted.

In addition to the above, instrumentation for existing dams and their appurtenant hydraulic structures is usually less extensive than that for new dams and their appurtenant hydraulic structures. This is because the performance of existing structures will generally have been established under normal loading conditions and some unusual loading conditions, and the retrofitting of instrumentation can be expensive.

Where instrumentation is installed, it should:

- Not be used as a replacement for regular visual inspections but as an aid to augment the ongoing assessment of dam performance.
- Be appropriate to the dam type and enable the monitoring of identified potential failure modes.
- Be simple, reliable, robust and - where warranted - sensitive with sensors that are easy and safe to install, calibrate, maintain and operate.
- Be calibrated on a defined, regular basis.
- Be installed in a manner which does not adversely affect the integrity of the dam (e.g., the drilling of boreholes and the grouting of piezometers within a dam core can leave potential weaknesses in the core if incorrect techniques are used – refer section 5.2). Include sufficient instrumentation for the measurement of parameters critical to dam safety and incorporate some redundancy to allow for instrument failures and the cross-checking of results.
- Include, where appropriate, an automated monitoring system for the more frequent monitoring of key dam safety parameters such as reservoir water level, piezometric levels, and seepage flows and turbidity levels. ICOLD Bulletin 118 (ICOLD, 2000) includes a discussion on automated monitoring systems and their application to dam safety.
- Where automated monitoring systems are included, manual monitoring backup systems should be available to provide calibration and enable the monitoring of performance in the event of system failure.
- Include the establishment of warning and alarm levels for the items being monitored, beyond which action is taken to review and ensure the continued integrity of the dam.

ICOLD Bulletins 104, 118, 129, 138, 141 and 158 (ICOLD, 1996b; 2000; 2005b; 2009b; 2011b; 2018b) discuss instrumentation objectives and various instrumentation techniques. Instrumentation systems for the monitoring of dam performance are fully detailed in the literature (e.g., FERC, 2016 Fell at al., 2015; and technical data sheets produced by instrument manufacturers).

The measurement of reservoir or pond levels, as well as earthquake ground motions where applicable, is common for embankment dams, concrete dams, and tailings dams. The following subsections provide an overview of typical instrumentation systems that can be installed to verify design assumptions and assist in monitoring the long-term performance of embankment dams, concrete dams, and tailings dams. Guidelines for the monitoring of long-term performance and the interpretation and reporting of monitoring results are included in Module 5 (Dam Safety Management).

7.2 Embankment dams

Instrumentation typically installed for monitoring the behaviour of embankment dams includes:

- Observation wells and piezometers for the measurement of groundwater levels in the abutments, and piezometric pressures in the embankment and its foundation.
- Weirs and other measurement facilities for the monitoring of seepage flows.
- Surface monuments for the monitoring of horizontal movements and settlements.
- Strong motion stations for the measurement of ground accelerations due to earthquakes in large High PIC dams located in proximity to highly active seismic sources.

Observation wells and the types of piezometers installed in a dam and its foundation should be appropriate for the ground conditions in which they are installed and for the required characteristics of piezometric response (e.g., time lag). Advantages and limitations of the available types are outlined in Table 7.1.

Table 7.1: Advantages and limitations of common observation wells and piezometers (modified from FERC, 2016)

Type	Advantages	Limitations	Can be manually read?	Can be readily rehabilitated?
Observation Well	Simple device, inexpensive, easily automated	Applicable only in uniform materials, not reliable for stratified materials, long time-lag in impervious soils	Yes	Yes
Standpipe Piezometer	Simple device, inexpensive, reliable, easily automated, can be subjected to rising or falling head tests to confirm function	Long lag-time in impervious soils, porous tips can clog due to repeated inflow and outflow, not appropriate for artesian conditions, can be damaged by consolidation of soil around standpipe	Yes	No
Single Tube Piezometer	Same as for open standpipe piezometer	Same as for open standpipe piezometer but appropriate for artesian conditions	Yes	No
Twin-Tube Hydraulic Piezometer	Simple device, moderately expensive, reliable, short time-lag	Cannot be installed in a borehole, generally cannot be retrofitted, moderately complex monitoring and maintenance, periodic de-airing required, moderately complex to automate	Yes	No

Pneumatic Piezometer	Moderately simple transducer, moderately expensive, reliable in the medium term, very short time-lag, elevation of readout independent of tip elevation and piezometric level	Moderately complex monitoring and maintenance, dry air and readout device required, sensitive to barometric pressure, performance can deteriorate after many years, moderately expensive readout, complex to automate and cannot be automated over long distances	No. Requires a readout unit	No
Vibrating Wire Piezometer	Moderately complex transducer, simple to monitor, very short time-lag, elevation of readout independent of tip elevation and piezometric level, output signal can be transmitted over long distances, easily automated	Lightning protection recommended, sensitive to temperature and barometric changes, risk of zero drift but some models available with in situ calibration check	No. Requires a readout unit or datalogger	No

Dual piezometers, or individual piezometer installations, isolated in the core and in the foundation beneath the core (upstream and downstream of any impermeable cutoff) provide information which can be used to establish general seepage behaviour through and beneath a dam. Where these instruments connect directly with a seepage path, recorded piezometric pressures will provide an insight into hydraulic gradients and may indicate the occurrence of internal erosion.

Weirs, Parshall flumes and calibrated containers are commonly used for the monitoring of seepage flows. The advantages and limitations of each flow measurement device are as follows:

- Weirs are simple, reliable, inexpensive and require little maintenance. They have the capability to capture sediment being transported by seepage flows and thus allow the identification of developing internal erosion features. However, they require a restriction to the flow channel and a sufficient elevation change to prevent the tailwater from affecting the weir discharge.
- Parshall flumes are simple, reliable and require little maintenance. However, they do not have the capability to capture sediment and they are likely to be more expensive than weirs to install. They should only be used where seepage volume is required, and internal erosion is not a concern.
- Calibrated containers are reliable for low flows, and they are inexpensive. However, they require a free-falling flow, they are inaccurate for large flows, and they are labour intensive.

Surface monuments are usually established on the abutments, along the crest and on the downstream slope of an embankment dam for the monitoring of dam deformations. Care needs to be taken to ensure that the monuments are founded on materials that are not susceptible to shrinkage and swelling. A wide variety of survey techniques can be used for the monitoring of dam deformations:

- Alignment surveys are the simplest and most accurate method for the monitoring of horizontal deformations on straight dams.
- Levelling surveys are the simplest and most accurate method for the monitoring of vertical deformations.
- Triangulation and trilateration surveys, using theodolites and electronic distance measuring instruments, are frequently used for the monitoring of horizontal and vertical deformations where measuring points do not lie along a straight line or where lines of sight are obstructed. While such surveys are highly accurate, they require an experienced survey team with specialist survey equipment and involve relatively complex calculations.

In some cases it may be necessary to install additional instrumentation for the monitoring of foundation settlement and embankment consolidation during and following construction (e.g., internal settlement gauges, borehole inclinometers), deformations and joint openings in concrete face slabs and plinths during construction and commissioning (e.g., joint meters), and seepage water characteristics following the identification of increased seepage flows (e.g., seepage turbidity levels, seepage water temperatures).

7.3 Concrete dams

Instrumentation typically installed for monitoring the behaviour of concrete dams includes:

- Observation wells and piezometers for the measurement of groundwater levels in the abutments, uplift pressures at the dam/foundation contact and, if necessary, uplift pressures beneath potential failure surfaces in the foundation and blocks or wedges in the abutments.
- Weirs for the monitoring of seepage flows from internal and foundation drains.
- Survey points for the monitoring of dam deformations.
- Internal deformation instruments to measure tilt, rotation and horizontal deformation.
- Strong motion stations for the measurement of ground accelerations due to earthquakes in large High PIC dams located in proximity to highly active seismic sources.

Piezometers installed from within drainage galleries or from downstream toes of concrete gravity dams enable the confirmation of uplift assumptions adopted during the design of the dam, the provision of uplift pressures for incorporation in stability studies, and the identification of any reductions in the efficiency of drainage systems. In existing concrete gravity dams where piezometers are not installed, uplift pressures can be measured at selected drains. The monitoring of uplift pressures beneath thin arch and buttress dams that are not founded on slabs is not normally necessary as uplift pressures usually have a minimal effect on dam stability.

Weirs and calibrated containers are commonly used for the monitoring of seepage flows, and their advantages and limitations are as outlined in section 7.2. For concrete gravity dams, it is good practice to divide the seepage monitoring system into separate catchments to enable the location of any seepage increases or decreases. Reductions in seepage flow may indicate reductions in the efficiency of underdrainage systems.

Any loss of structural integrity in concrete gravity dams will usually manifest itself at the vertical contraction joints between adjacent dam blocks. Because of the monolithic behaviour of concrete arch dams, any loss of structural integrity will usually manifest itself as horizontal displacements along the arch. In recognition of the above, deformation systems usually include:

- Survey points along the crests or in the galleries of concrete gravity and buttress dams for the monitoring of upstream/downstream deformations and vertical deformations. Alignment surveys are usually adopted for the monitoring of upstream/downstream deformations, and levelling surveys are usually adopted for the monitoring of vertical deformations.
- Joint meters across vertical contraction joints in concrete gravity dams. Simple scribe marks can be made across all monoliths at the surface or in galleries in concrete gravity or arch dams to detect differential foundation displacements that may develop gradually under static loading or in response to earthquake or high flood loadings.
- Survey points along the crests of concrete arch dams for the monitoring of upstream/downstream deformations and, in some cases, survey points on the downstream faces adjacent to the abutments for the monitoring of chord distances.

As stated above for embankment dams, it may be necessary in some cases to install additional instrumentation for the monitoring of dam performance. Examples include plumblines for the measurement of a dam's response to variations in reservoir level and temperature, inclinometers and extensometers for the measurement of movements within the dam foundation, and instruments for the measurement of concrete and reservoir water temperatures.

7.4 Tailings dams

Instrumentation typically installed for monitoring the behaviour of tailings dams during their construction includes:

- Weirs and other measurement facilities for the monitoring of seepage flows.
- Open standpipe or vibrating wire piezometers for measuring the location of the phreatic surface.
- Open standpipe or vibrating wire piezometers for the monitoring of pore pressures in the tailings, embankment and foundation.
- A survey network, and in some cases inclinometers for the monitoring of horizontal movements and settlements.
- Strong motion stations for the measurement of ground accelerations due to earthquakes in large High PIC tailings storage facilities located in proximity to highly active seismic sources.

Weirs and calibrated containers are commonly used for the monitoring of seepage flows, and their advantages and limitations are as outlined in section 7.2. Seepage flows are normally manually read but can be automated and continuously monitored in a control room.

Manually read open standpipe or vibrating wire piezometers are usually adopted for monitoring the location of the phreatic surface. Standpipes can be installed on the ground before any tailings are placed and additional pipe added as the tailings rise. Because of the anisotropic permeability associated with the method of deposition, the elevation of the phreatic surface indicated by a standpipe based on the ground is usually lower than the actual surface. More accurate positions of the phreatic surface can be obtained from standpipes installed in the tailings with their intake filters placed a short distance below the position of the phreatic surface indicated by the deeper standpipes.

Pore pressures can be measured by open standpipe piezometers if the pressures are not sufficiently high to result in water level rises to the tops of the standpipes. Where pore pressures are high, they are usually monitored from a remote location using vibrating wire piezometers. Vibrating wire piezometers are precise, they can be easily read and automatically recorded, and they are easily installed. In addition, they are not affected by electrical disturbances other than voltage overloading from lightning, against which they should be protected.

Vertical and horizontal deformations of tailings dams constructed by the upstream method are usually monitored along the crest of the starter dam and on the berms of the downstream slope of the tailings dam. Inclinometers may be appropriate in some circumstances. Levelling and alignment surveys are usually adopted for the monitoring of deformation; however, more accurate survey methods (i.e., triangulation and trilateration surveys) should always be available to enable the completion of more detailed surveys in a potential dam safety emergency. For tailings dams constructed by the downstream and centreline methods the surfaces of the dam are only provisional before the embankments are raised; however, survey reference points can be placed on the crest of the starter dam and subsequent stages but the monitoring will only be effective for a short time period. More information is included in ICOLD Bulletin 104 (ICOLD, 1996b).

Monitoring of the installed instrumentation during the latter stages of construction of a tailings dam normally continues throughout closure of the tailings dam; however, the monitoring frequencies usually become progressively less as the instrumentation demonstrates satisfactory long-term performance of the dam. For tailings dams constructed by the downstream and centreline methods, survey reference points are usually established on the crest and downstream face of the completed dam to enable the monitoring of deformation.

8. Design processes

8.1 Feasibility studies and design of new dams

The dam design process is typically undertaken in a series of stages which include:

- A prefeasibility (or concept) study to identify a preferred dam site for development. The work completed is predominantly office based and utilises existing information that is available in the public domain and from a variety of central and local government agencies (e.g., topographical maps, geological maps, hydrological data).
- A feasibility study to establish the technical and economic feasibility of development at a preferred dam site. The work usually includes field activities (e.g., geological mapping, drilling), laboratory testing of potential construction materials, and office studies to develop a preferred general arrangement for the dam and an estimated cost for its development.
- A preliminary design to address any significant issues identified during the feasibility study, develop preliminary design details for the dam, and refine the estimated cost for its development. The resulting documentation is usually sufficient to support an application for water permits and land use consents.
- A detailed design to resolve all outstanding issues, complete the detailed design for the dam, and prepare all necessary documentation (i.e., drawings, technical specifications, contract documents) for its construction. The resulting documentation should be sufficient to support an application for building consents. Design details should be complete and constructible, areas of uncertainty should be identified, hold and witness points identified and listed, and contingent details in place before construction commences. Refer to Module 4 for more details.

As outlined in section 3, each stage of the design process should be undertaken by personnel with appropriate backgrounds of experience. Significant benefits can result from the early engagement of peer review services for the design of Medium and High PIC dams, and the later engagement of experienced contractors to assist in the identification of construction issues and risks, the assessment of construction methods (e.g., diversion arrangements) and the estimation of construction costs.

The design of a dam usually reflects precedent designs, the results of analytical studies, and the experience and personal preferences of the Designer. While alternative dam designs are often possible for a particular dam site, it is most important that the adopted design reflects the characteristics of the project and the dam site. Project and dam site characteristics that can strongly influence the design include:

- The function of the dam and its proposed operating regime. The reservoir for a hydroelectric scheme usually has a small operating range while the reservoir for a water supply scheme usually has a wide operating range. Impoundment upstream of a flood detention dam only occurs during large flood events.
- A requirement for staged construction to reflect a projected growth in water demand.
- The length of the season available for dam construction. Embankment dams typically have shorter construction seasons than concrete dams.
- The availability and quality of local materials for the construction of a dam. Some sites may have large resources of materials suitable for dam construction, while at other sites the resources may be limited.
- The size and shape of the dam site. Narrow dam sites with steep walls may necessitate special design provisions for an embankment dam or, if other site characteristics are appropriate, the development of a concrete dam.
- The local geology and the quality of the dam foundation. A site with a shallow rock foundation may require little foundation treatment in comparison to that necessary for a site with a deep alluvial foundation.
- The climate and likely weather conditions during the construction of the dam. Fine grained soils require dry weather for construction while coarser grained soils and rockfill can usually be placed during wet weather.
- Diversion requirements during construction. If diversion capacities are exceeded by a flood during construction, embankment dams present a far greater hazard to downstream communities and infrastructure than concrete dams.

8.2 Design of rehabilitation works

The design of rehabilitation works to address identified potential failure modes or dam safety deficiencies would typically be undertaken in a series of stages which include:

- A thorough evaluation of the existing risks and the necessity for the proposed rehabilitation works.
- A preliminary design to identify and consider rehabilitation alternatives, develop preliminary design details for the preferred remediation alternative, and complete an estimated cost for its completion. The resulting documentation should be sufficient to support any necessary applications for variations to existing water permits and land use consents.
- A detailed design to provide the necessary risk reduction, complete the detailed design for the works, and prepare all necessary documentation (i.e., drawings, technical specifications, contract documents) for their construction. The resulting documentation should be sufficient to support an application for building consents.

Personnel requirements for the design of rehabilitation works are similar to those outlined in sections 3 and 8.1.

It is most important that the final rehabilitation works design properly addresses the identified dam safety deficiency, and that the solution is compatible with the characteristics of the existing dam and the dam site. Issues that can strongly influence the design of rehabilitation works for any dam include:

- The effects of the rehabilitation works on existing consents for scheme operation. For example, the completion of the rehabilitation works may necessitate a reduction in the consented reservoir level and variations in the consented discharges to the downstream river.
- The ability of the dam to fulfil its intended function during the completion of the rehabilitation works. For example, the lowering of a reservoir level may have a significant impact on the ability of an Owner to meet bulk water supply targets or may significantly reduce the head available for electricity generation.
- The risk to the safety of the dam during the completion of the rehabilitation works. For example, the rehabilitation of toe drainage facilities in an embankment dam could affect the stability of the downstream shoulder, and the rehabilitation of a spillway facility could markedly reduce the ability of the dam to safely manage flood events during the construction period.
- The time and cost required for completion of the rehabilitation works. To minimise the effects of rehabilitation works on normal business operations, a dam Owner may consider a rehabilitation option that results in partial reduction of the risk rather than an option that results in full mitigation of the risk.
- The availability and quality of local materials for the construction of the rehabilitation works. Material shortages at some dam sites could significantly influence the scope and characteristics of the rehabilitation works.

8.3 Design oversight and amendments during construction and commissioning

Dam design is never complete until the dam or dam rehabilitation has been constructed and all facilities have been commissioned. It is essential that the dam Designer provides input through the construction and commissioning processes. It is essential that the Owner understands the need for Designer involvement, including the likelihood of design changes during construction and the need for design support during commissioning, with their associated costs, and has appropriate funding in place to support both activities.

Dam Designers should be familiar with the specific guidance on the roles and responsibilities of the Designer during construction and commissioning provided in Module 4 as follows:

- Role, responsibilities and experience of the Designer for dam construction - Section 2.2.2
- Design and build model of contracting - Section 2.5.3
- Hold and witness points - Section 2.8.6
- Value engineering - Section 2.9
- Design change management - Section 2.10
- Role and responsibilities of the Designer during commissioning - Section 4.2.2

The Designer must remain alert throughout construction for any changes in conditions or properties from those assumed in the design. For example, foundation excavation may expose important foundation features that were not identified during the site investigation, the quantities and qualities of the borrow materials may be more variable than anticipated in the design, groundwater inflows during construction may necessitate modifications to the designed seepage control measures, and design changes may be necessary to suit a Contractor's preferred construction method. A dam design or rehabilitation must be continuously reviewed and re-engineered during construction to ensure the final design is compatible with the conditions encountered during construction. This is especially true for dam rehabilitation projects where the actual as built conditions found during construction may be different than those presented on available drawings for the original construction.

Any changes to the design necessary to address observed site conditions or Contractor preferences must be completed to the same standards as the original design but, most importantly, must be addressed in a thorough manner to ensure that any changes do not inadvertently create a risk elsewhere. Even small design changes must not be considered in isolation as significant reductions in dam safety can result from a sequence of relatively minor seemingly unrelated modifications. Design changes that materially alter a building consent issued for the construction of a dam or rehabilitation works will require the approval of the regulator responsible for administering the building consent process.

In the case of tailings dams, construction is normally undertaken in stages with the dam in operation. Detailed design is also undertaken in stages and should take into account the experience obtained during operation, and changes in tailings production and characteristics as well as changes in operating procedures and water management.

8.4 Design documentation

From a dam safety perspective it is most important that a design is translated into clearly understood construction drawings and specifications, and that an appropriate design report is completed which records all design data, philosophies and assumptions, and defines areas requiring re-evaluation or confirmation during construction. Such documentation is required to accompany a building consent application for a dam (refer Module 1).

Construction drawings and specifications must clearly describe any particular requirements to be achieved in areas critical to dam safety. For example, for an embankment dam specific fill materials and compaction requirements may be required adjacent to conduits and concrete structures to minimise the potential for erosion along concrete/embankment interfaces. Typical earthworks specifications for the construction of subdivisions and road embankments are inadequate for the construction of embankment dams. The same applies to concrete dams. For example, specific foundation treatment may be necessary to minimise the potential for erosion along a shattered zone that trends across a rock foundation. While it is not customary practice, consideration should also be given to including a commentary in the construction documents that highlights areas critical to dam safety, the reasons for their specified treatments, and the possible consequences of failing to meet the specified requirements.

Design reports are typically completed at the end of each stage of the design process, whether they are for the design for a new dam or the design of a rehabilitation project. Those completed at the end of the preliminary design should incorporate sufficient detail to support an application for water permits and land use consents, or any variations to existing water permits and land use consents. Those completed at the end of the detailed design should be sufficient to support an application for building consents. The final design report, which should be completed towards the end of construction, should be appropriate to the PIC of the dam and should clearly document all assumptions, criteria and methods adopted for the design of the dam and its associated hydraulic structures or, as appropriate, the design of the rehabilitation works. In many cases it may be sufficient to amend the design report completed at the end of the detailed design and include an appendix that documents any design changes adopted during construction.

The final design report for the design of a new straightforward Low PIC dam designed by precedent will normally be relatively short in comparison to those completed for Low PIC dams designed by empirical methods and those completed for Medium and High PIC dams. However, all final design reports should include:

- A description of the background to the project, the purpose of the dam, and the characteristics of the dam site.
- An assessment of the PIC for the dam and its associated appurtenant hydraulic structures.
- An assessment of the hazards that can affect the safety of the dam (e.g., floods, earthquake ground motions, reservoir landslides).
- An assessment of the potential failure modes, defensive measures adopted in the design to prevent the potential failure mode from developing, and surveillance measures proposed to monitor for early warning signs of a potential failure mode developing during the operational phase.
- The characteristics of the site geology, the dam foundation, and the materials utilised for dam construction.
- The philosophy adopted for the design of the dam and its associated hydraulic structures, and the criteria and methods adopted for their design.
- The expected performance of the dam and its associated hydraulic structures during normal, unusual and extreme loading conditions.
- The rationale for and a description of any instrumentation installed for the monitoring of dam performance.
- Surveillance and monitoring proposals for ongoing dam safety assurance.
- Relevant appendices (e.g., hydrological data, geological maps, seismic hazard studies, field investigation records, laboratory investigation results).

Similarly, the final design report for the rehabilitation of a straightforward Low PIC dam designed by precedent will normally be relatively short in comparison to those completed for Low PIC dams designed by empirical methods and those completed for Medium and High PIC dams. However, all final design reports should include:

- A description of the purpose and characteristics of the existing dam, the identified dam safety deficiencies, the risks associated with the identified dam safety deficiencies, and the objectives of the rehabilitation works.
- A description of any site-specific considerations (e.g., site geology, available materials) or constraints (e.g., timing, flood management during construction, Owner requirements) that influenced the selection and design of the rehabilitation works.
- The philosophy adopted for the design of the rehabilitation works, and the criteria and methods adopted for their design.
- A description of the conditions encountered during the construction of the rehabilitation works.
- The expected performance of the rehabilitated dam during normal, unusual and extreme loading conditions.
- The rationale for and a description of any instrumentation installed for monitoring the performance of the rehabilitated dam.
- Surveillance and monitoring proposals for ongoing dam safety assurance.
- Relevant appendices (e.g., hydrological data, geological maps, seismic hazard studies, field investigation records, laboratory investigation results).

Copies of all key design records, drawings and documentation should be provided to the dam Owner at the end of the project.

Further information on documentation of construction and commissioning is covered in Module 4 of these Guidelines.

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Other resources

ANCOLD Guidelines: **www.ancold.org.au/product-category/guidelines/**

ASDSO Resource Center: **www.damsafety.org/resource-center**

CDA Guidance Documents: **www.cda.ca/publications/cda-guidance-documents**

ICOLD Bulletins: **www.icold-cigb.org/gb/publications/bulletins.asp** (registration required, login available for NZSOLD members)

USACE Publications: **www.publications.usace.army.mil/**

USSD White Papers: **www.ussdams.org/about/white-papers/**

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