Evaluation of Existing Arch Dam Design Criteria in Lieu of ANCOLD Guidelines

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This paper provides a summary of the current state of practice for arch dam design criteria that have been adopted by some international dam organizations, and where relevant, compares that with the criteria provided in the updated ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams, with the view to provide a basis for consistent and unified design criteria for arch dams in Australia.

The paper draws on the authors' experience with arch dams, including recent experience with a number of arch dam safety reviews in Australia, their past experience with arch dams over 200 m height, as well as their involvement with the development of the mentioned updated ANCOLD Guidelines.

Since the last arch dam was constructed in Australia, a number of international publications have been released on arch dam design practices, providing general information and guidance for the design of new dams and evaluation of the safety and structural integrity of existing arch dams. This paper compares these publications and proposes criteria that are aligned with the ANCOLD gravity dam guidelines.

Keywords: *Arch dams; Design Criteria*

Introduction

The ANCOLD register includes 43 arch dams constructed between 1857 and 1974, ranging from about 15 m to 140 m in height. Although there have been no new arch dams constructed in Australia for many years, the safety of existing arch dams are required to comply with various state dam safety regulatory requirements, which often refer to, or are aligned with, the ANCOLD guidelines.

ANCOLD has developed guidelines to enhance the ability of dam organizations to assure that adequate safety programs and practices are in place. However, only three guidelines cover particular types of dams, whilst the others deal generally with flood capacity and earthquake design, and dam safety and environmental management practices. Except for limited information regarding earthquake design in ANCOLD (1998), and selected information in ANCOLD (2013), none of them deals specifically with arch dams design criteria. In the absence of a single recognized guideline covering the various design aspects of arch dams, there is currently inconsistency in the underlying principles in the review processes of arch dams in Australia.

Since the last arch dam was constructed in Australia, a number of international publications have been released on arch dam design practices, e.g. by the US Bureau of Reclamation (1977, 2006), USACE (1994) and FERC (1999). These publications provide general information and guidance on the design of new arch dams, and for the evaluation of the safety and structural integrity of existing arch dams, including design criteria, material properties, loads and load combinations and evaluation procedures.

This paper provides a summary of the current state of practice for arch dam design criteria that have been adopted by these international dam organizations, and compares that with the criteria provided in the updated ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams (2013), with the view to provide a basis for consistent and unified design criteria for arch dams in Australia.

Key concepts of arch dams

Concrete arch dams are complex three-dimensional shell structures that are thinner but have more redundancies than gravity dams. They carry load both in a vertical plane by cantilever action into the base foundation, as well as horizontally by arch action into the abutments. Arch dams thus resist the water pressure and other loads by self-weight and by transmitting the load by arch action into the valley walls or into concrete gravity thrust blocks. Typically the arch action reduces the bending (cantilever) stresses and adds load carrying capacity. The abutments and foundations must therefore be of sufficient strength to support the arch thrust.

Arch dam structures are curved towards upstream with either single curvature (curved only in plan) or double curvature (curve both in plan and section). An arch-gravity dam or curved-gravity dam has the characteristics of both an arch dam and a gravity dam, but can be thinner than the pure gravity dam.

An arch structure has to be monolithic to achieve successful arch action, i.e. no structural discontinuities, such as open joints or cracks, should exist at the time the water load is applied.

The shape and curvature of an arch dam, its contact with the foundation and the stability of the foundations are the most important design features in providing stability and favourable stress conditions. The desired stresses and stability is achieved most economically by proper shaping and the use of both horizontal and vertical curvature, rather than by adding to the thickness of the dam.

The ideal arch shape depends on the valley shape, with typically a high degree of horizontal curvature in relatively narrow valleys, and poly-centred, parabolic or elliptically shaped arches in wider sites.

Arch dams provide redundant load carrying capacity, i.e. if one part of the structure is overstressed, the load can be transferred to other parts of the structure and transmitted by arch action to the abutments. Where necessary to reduce stresses in the rock, the thickness of the dam near the foundation can be increased by using a fillet, or variable thickness arches, or abutment thrust blocks.

Based on Reclamation (1977) arch dams are generally classified as a thin arch dam if the base thickness to height (b/h) ratio is 0.2 or less, a medium-thick arch dam if the b/h ratio is between 0.2 and 0.3, and a thick arch dam if the b/h ratio is 0.3 or greater.

Typical failure modes

Before considering the design criteria for arch dams, it is useful to review some case histories and understand typical failure modes. Case history on arch dam failures or incidents is limited but available in various publications. Tables 1 and 2 below provide a summary of some known failures and incidents, while ICOLD Bulletin 120 (2001) contains more extensive lists of dams which experience strong seismic ground motion.

Table 1 Incident and failures of arch dams during normal operation and flood events Note 1

Name	Completion	Country	Height	When	What happened
Lake Hodges	1918	USA	41 m	1918	Dam body damaged by cracked piers but did not completely fail.
Gleno	1923	Italy	44 m	1923	Dam failed when nine arches fell due to a poor masonry base.
Manitou	Unknown	USA	15 m	1924	Portion of the dam body failed due to deterioration of the concrete.
Moyie River (Eileen)	1923	USA	16 m	1926	Spillway erosion completely washed out one of the abutments. The abutment was replaced and the dam is still in use.
Lake Lanier	1925	USA	19 m	1926	One of the abutments (cyclopean masonry) washed out as a result of the failure of soft rock in the abutment. The remainder of the dam was unharmed
Vaughn Creek	1926	USA	19 m	1926	The dam failed during first filling as a result of seepage and poor materials in the dam.
Alla Sella Zerbino	Unknown	Italy	12 m	1935	The dam failed as a result of overtopping and sliding on its foundation.
Le Gage	1955	France	46 m	1955	The dam developed extensive cracking on both faces after first filling, which worsened for the next 6 years. After the failure of Malpasset Dam, Le Gage Dam was abandon and a new thicker arch dam was constructed upstream.
Malpasset	1954	France	66 m	1959	The dam failed due to movement of the left abutment, thought to be due to sliding on a rock wedge formed by intersection of a fault with gneissic foliation in the rock.
Idbar	1959	Yugoslavia	38 m	1960	The dam failed during first filling as a result of piping and erosion of the foundation.
Vajont	1959	Italy	276 m	1963	A huge landslide-generated wave overtopped the dam wall by an estimated 100 m. The dam suffered little damage, but the reservoir was a total loss.
Arequipa	Unknown	Peru	Unknown	1965	The dam body failed as a result of fractures caused by a vibrating penstock which passed through the dam.
Matilija	1949	USA	50 m	1965	The dam was judged to be unsafe as a result of deterioration of the concrete due to expansive aggregate and poor foundation conditions. The dam was decommissioned.
Zeuzier	1957	Switzerland	156 m	1978	The dam began to deflect upstream due to riverward movement of the left abutment.
Koelnbrein	1979	Austria	200 m	1981	Cracks and substantial leakage appeared in the lowest foundation gallery when the reservoir was 80% full two years after first filling. Full uplift pressure was observed over the entire base in the central portion of the dam. Major repair was undertaken between 1989 and 1994.
Meihua (Plum)	1981	China	22 m	1981	The experimental dam failed shortly after filling as a result of structural failure due to excessive uplift movement along a peripheral joint. Evidence was observed of sliding both in the arch and downstream direction. The scheme was abandoned after failure.
Leguaseca	1958	Spain	20 m	1987	The dam body failed structurally, apparently due to deterioration due to both aging and the effects of freezing and thawing.
El Fraile	Unknown	Peru	61 m	Unknown	The dam experienced a major slide on one of the abutments during filling. The dam did not collapse. A concrete thrust block abutment was constructed and the dam was saved.
Tolla	1960	France	90 m	Unknown	The dam experienced severe cracking and was buttressed in response. Cracking may have been the result of large temperature stresses.

Note 1: Data obtained from FERC (1999) & ICOLD (2001)

Table 2 Incident and failures of arch dams during strong motion shaking Note 1

Name	Completion	Country	Height	Earthquake Event	Magnitude & Acceleration	Effects
Gibraltar	1920, 1990	USA	52 m	Santa Barbara (1925)	6.3 (>0.3g)	No damage resulted. The dam was modified in 1990 with RCC.
Pacoima	1929	USA	113 m	San Fernando (1971)	6.6 (0.6-0.8g)	No damage resulted in the arch, but it opened a joint between the arch and the thrust block.
				Northridge (1994)	6.8 (0.53g, >2.3g at the crest)	It opened the joint (50 mm) between the arch and thrust block.
Ambiesta	1956	Italy	59 m	Gemona-Friuli (1976)	6.5 (0.36g at right abutment)	No damage resulted.
Rapel	1968	Chile	111 m	Santiago 1985	7.8 (0.31g)	Damage resulted to the spillway and intake tower.
				Maule 2010	8.8 (0.302g)	
Techi	1974	Taiwan	185 m	Chi Chi 1999	7.6 (0.5g at the base and 0.86g at the crest)	No damage resulted in the arch, with local cracking of the curb at the crest.
Maina di Sauris	1976	Italy	136		6.5	No damage
Shapai RCC	2003	China	132 m	Wenchuan 2008	8.0 (0.25 to 0.5g)	No damage resulted.

Note 2: Data obtained from Nuss et al (2012)

An arch dam may potentially fail as a result of:

- structural failure within the dam body due to overstressing of the concrete, or during earthquake events due to excessive contraction joint opening combined with cantilever tensile cracking,
- sliding along the dam-foundation interface, or
- movement of the abutment rock wedges formed by rock discontinuities.

The above conditions could be the result of static loading during normal and flood conditions, and additional dynamic loading during earthquake events.

It is evident from Table 1 that concrete arch dams that have performed well under normal operating conditions will likely continue to do so unless something changes. No arch dams are known to have failed statically within the dam body after five years of successful operation having reached its normal operating reservoir level, although four incidents are shown where severe cracking required remedial works.

Changes could result from plugging of drains leading to an increase in foundation uplift pressures, possible gradual creep that reduces the shear strength on potential sliding surfaces, or degradation of the concrete from alkali-aggregate reaction, freeze-thaw deterioration, or sulphate attack.

Under earthquake loading concrete arch dams will respond according to the level and frequency of the shaking, and the reservoir level at the time of shaking. Unlike gravity dams, the most critical case for earthquake loading of an arch dam might not be the reservoir full scenario. A more severe overstress condition could result with the reservoir empty or partially filled.

Structural failure within the dam body

As shown in Table 1, there are no cases of failure within the arch dam bodies due to load overstressing, but there are cases of damage that did not result in dam failure. Four cases involving dam body failure resulted from poor construction materials or concrete material deterioration.

Charlwood and Solymay (1995) reported 32 cases of alkali aggregate reaction (AAR) in concrete arch dams. Most of these dams were constructed prior to or while the understanding of the AAR processes were still emerging. A large number of arch dams subjected to AAR have continued to function adequately, but in some cases strengthening measures were required (Stewart Mountain, Churchill and Gmued Dams), or partial replacement (Matilija Dam), or complete replacement (Drum Afterbay Dam).

As shown in Table 2 there are no known arch dam failures as a result of earthquake shaking. There is thus no direct empirical evidence to indicate how an arch dam would structurally fail under this type of loading. Payne (2002) conducted shaking table model studies to gain some insight as to how an arch dam might fail under earthquake shaking. In these tests:

- failure initiated by horizontal (cantilever) cracking across the lower central portion of the dam,
- followed by diagonal cracking parallel to the abutments,
- then cracking propagated through the model forming isolated blocks within the dam, and
- eventually, the isolated blocks rotated and swung downstream releasing the reservoir.

Vertical contraction joints possess very little or no tensile resistance and might repeatedly open and close during intense earthquake shaking. As postulated by Ghanaat (2004), the contraction joint opening releases tensile arch stresses but increases tensile cantilever stresses. The increased cantilever stresses may exceed tensile strength of the concrete or lift joints, causing horizontal cracks. The resulting partially-free blocks bounded by the opened contraction joints and cracked lift joints may become unstable and cause failure of the dam (see Figure 1).

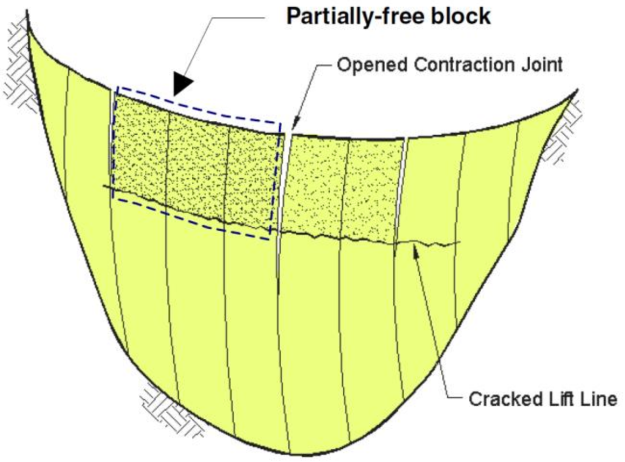

Figure 1 Rotation of blocks caused by cracking and opening of contraction joints (Ghanaat, 2004)

Dam-foundation interface failure

The dam-foundation interface includes the concrete-rock contact, the concrete immediately above up to about the first lift joint, and the foundation rock typically 1 to 2 m immediately below the contact.

There are three types of potential sliding instability cases at the dam-foundation interface, i.e. sliding along the contact between the dam concrete and foundation rock, within the concrete along lift joints, and along planes immediately below the contact.

Sliding instability for the first two cases are less likely because of the wedging produced by arch action and embedment of the structure into the rock. However, arch dams with relatively flat abutment slopes, or arch dams with abutment thrust blocks supported by rock foundations with inadequate shear strength, could be susceptible to sliding along the foundation contact or along planes immediately below the contact, in either or both the arch and downstream directions.

Severe earthquake shaking could break the bond between the dam and foundation, or cause movement along planes below the contact, especially if the foundation was not excavated to radial lines and the excavation surfaces dip downstream on sections cut radial to the dam axis. Resulting sliding and rotation at the base could lead to loss of arch action and subsequently to instability.

Foundation failure

Actual arch dam failures have resulted from foundation deficiencies, which included sliding of large blocks bounded by geologic discontinuities within the foundation and abutments, or along planes of weakness (three failure cases and three incidents shown in Table 1).

Although no arch dam foundations are known to have failed because of earthquake shaking, they have not been subjected to unprecedented seismic design loads.

The most critical mode of foundation instability involves sliding on discontinuities (joints, faults, shears, bedding planes, foliation, clay seams, shale beds etc.) within the foundation. Sliding in the foundation typically occurs along a single failure plane (plane sliding) or along the line of intersection of two of these planes (wedge sliding). To be kinematically capable of failure, the direction of sliding surfaces must intersect or "daylight" a free surface downstream from the dam. While it might be capable of bridging a small unstable foundation block at the bottom, large, unstable wedges of rock in the abutments could endanger the safety of the arch dam.

For thin arch dams, sufficient movement may be generated in the foundation during the shaking to cause rupture of the dam body. Even if movement initiates but does not cause dam failure, water forces acting on the block planes may still increase as a result of the movement. Stability analyses simulating post-earthquake conditions are thus required to assess the likelihood of post-earthquake instability.

Overtopping failure

During large floods the arch wall could be subject to overtopping and erosion of the abutments. Table 1 includes one such case where overtopping erosion at the contact zone resulted in sliding failure.

Although no arch dams are known to have failed statically due to overstressing within the dam body, a possibly more serious condition occurs when there is an abutment foundation block upon which the dam rests, that could erode due to overtopping flows, or become unstable under increased loading due to the flood conditions.

The loss of part of the abutment and foundation near or at the toe of the arch wall could enable plane or wedge sliding by exposing (daylighting) sliding planes, by removing passive resisting rock, or by changing the deformations of the dam and redistribution of stress state at the region and applying more forces to the wedge.

It is important to perform abutment stability analyses under flood loading considering the increase in dam thrust on the foundation blocks and the increased hydrostatic forces on the block bounding planes.

Existing design references

A number of internationally recognised design guidelines and manuals applicable to arch dams have been published since 1953, as listed below. This list is not exhaustive and several other guidelines and manuals could be applied to arch dams, e.g. regarding site investigations, spillways, outlet works and dam safety management practices.

United States Bureau of Reclamation

- Guide for Preliminary Design of Arch Dams (1977)
- Design of Arch Dams (1977)
- Design Criteria for Concrete Arch and Gravity Dams (1977)
- Guidelines on Foundation and Geotechnical Studies for Existing Concrete Dams (1999)
- State-of-Practice for the Nonlinear Analysis of Concrete Dams at the Bureau of Reclamation (2006)

United States Army Corps of Engineers

- Arch Dam Design, EM 1110-2-2201 (1994)

- Response Spectra and Seismic Analysis for Concrete Hydraulic Structures, EM 1110-2-6050 (1999)
- Time-History Dynamic Analysis of Concrete Hydraulic Structures, EM 1110-2-6051 (2003)
- Stability Analysis of Concrete Structures, EM 1110-2-2000, (2005)
- Earthquake Design and Evaluation of Concrete Hydraulic Structures, EM 1110-2-6053 (2007)

Reclamation and USACE have also prepared the following joint publication:

- Best Practices in Dam and Levee Safety Risk Assessment, Chapter 21 – Risk Analysis for Concrete Arch Dams, (2010)

United States Federal Energy Regulation Commission

- Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 11 – Arch Dams (1999)

ANCOLD

ANCOLD has no publications specifically for arch dams; however ANCOLD (1998) contains guidance related to earthquake design of arch dams, while ANCOLD (2013) contains information that could be applied to materials and load conditions for concrete dams in general.

The remainder of this paper draws to a large extent on the information provided in the above references.

Material parameters

Reclamation, USACE, FERC and ANCOLD all have similar approaches in defining material properties and they often refer to the same past studies and reports for typical values.

The material parameters for both the dam wall and foundations are project and site specific. Due to limited space this paper cannot discuss this topic in sufficient detail and the reader is therefore referred to the extensive coverage of this topic in the design references listed in the previous section. However, it is emphasised that a thorough knowledge must first be gained on a dam's original design and its performance history and records, to provide a basis for evaluation and any further studies and investigations that might be required.

The dam and foundation material parameters should be determined on the basis of field and laboratory investigations. In assigning strength and stiffness parameters for the foundations, it is essential to firstly derive a proper geological model for the foundations. This should be undertaken by a geologist with assistance where appropriate, by a rock mechanics expert. In most cases, the required parameters will be determined by rock defects rather than by the rock mass.

Where the field or laboratory determination of certain material parameters is neither cost effective nor conclusive, the parameters can be estimated by existing correlation relations or using the same parameters in similar projects. In these cases, their effects on the dam response should be evaluated by parameter sensitivity analyses. The USA Electric Power Research Institute investigated the factors that influence uplift pressure distributions in concrete dams and foundations, to establish ranges of shear and tensile strengths and cohesion values typical of concrete (parent concrete, bonded and unbonded joints), and concrete to rock interfaces (EPRI, 1992). In the absence of site specific testing, this document provides valuable information.

The determination of the condition of the lift joints and the overall strength of the dam based on limited available testing remains a challenge for dam engineers. Over the last 50 years Reclamation has performed strength and frictional tests on numerous concrete dams of different ages, with construction dates ranging from 1905 to 1993. Dolen (2011) processed the data of these tests and reported on the strength and frictional properties of parent concrete and lift joints, grouping the results by dam ages. In the absence of site specific information this data provides valuable information to understanding the joint strength in relation to construction practices over the years.

Dolen presented ratios of bonded to unbonded lift joints for dams of different ages. A practical approach to account for the portion of lift joints that are not bonded in the global lift strength properties, as proposed by Dolen, consists of reducing the average test values based on the estimated fraction of bonded lift joints.

Loads

General

Reclamation, USACE, FERC and ANCOLD all have the same design loads and generally the same definitions, although some of the references define the load types in more detail.

Arch dams are designed for the same loads as gravity dams with the exception of the temperature load which has a significant influence in arch dam design.

The loads for which arch dams must be designed can be categorized as static or dynamic loads. Static loads are sustained loads that do not change, or change very slowly compared to the natural periods of vibration of the structure, e.g. dead load, hydraulic load, loading from silt and backfill materials, dynamic forces from flowing water changing direction, uplift, forces from ice expansion or impact, and stresses caused by temperature changes.

Dynamic loads are transitory in nature and typically seconds in duration, e.g. earthquake-induced forces, blast-induced forces, fluttering nappe forces, or forces caused by the impact of ice, debris, or boats. Because of the speed at which they act, the inertial and damping characteristics of the dam as well as its stiffness affect the dam's behaviour.

Dead load

Dead load includes the weight of both the concrete and appurtenant structures (gates, bridges, and outlet works). The dead load is normally imposed on cantilever monoliths prior to the grouting of the contraction joints (no arch action) and should be taken into account when analysing an arch dam, which is different to applying the self weight of a gravity dam. The weight of appurtenances is typically negligible compared to the dam itself;

however, massive outlet works and overflow ogee weir spillways may have noticeable effects on the static and dynamic stresses.

Temperature load

Temperature loading is the most important difference between gravity and arch dams analysis. The temperature load in arch dams results from the differences between the closure temperature and concrete temperatures in the dam during its operation.

The closure temperature is the concrete temperature at the time of grouting of the contraction joints. It can also be considered as the stress-free temperature, i.e. there will not be any thermal stresses in the dam as long as the temperature of the dam remains at the closure temperature. However, once the average concrete temperature through the thickness exceeds the closure temperature, the resulting positive temperature loading will cause compressive stresses in the arches, which in turn will result in deflection into the reservoir. The opposite happens when in the winter the concrete temperature drops below the closure temperature and the arches experience tension which causes downstream deflection.

There are two ways which are generally adopted for defining the internal concrete temperature causing loading and thermal stress: conduct steady state temperature calculations based upon water and air face temperatures (such as USACE (1994) or Stucky and Derron (1957) for nonlinear distribution of temperature through the dam thickness), or assume a cyclic variation of ambient temperature with respect to time (seasonal variations) and conduct a harmonic temperature analysis, with due regard being given to the time dependant temperature condition of the reservoir water.

In the case of old arch dams where construction methodology, vertical joint treatment and construction timing are unknown, a practical approach is to adopt the average ambient temperature as the closure temperature. However, it would be advisable to undertake a sensitivity analysis using the average summer and average winter temperatures as the closure temperature.

Further details about temperature loading are contained in USACE (1994) and Stucky and Derron (1957).

Hydrostatic hydraulic loads

Reservoir and tailwater loads

Water loads include hydrostatic pressures on the dam faces resulting from the reservoir and tailwater during the normal and flood conditions. Unlike a gravity dam for which higher reservoir levels would result in more critical cases, an arch dam may experience higher tensile stresses on the downstream face under low reservoir levels.

As tailwater acts in the opposite direction than the headwater, it reduces the deformations caused by the headwater and thus reduces both tensile and compressive stresses below the tailwater levels. This effect is negligible when the tailwater depth is less than 20% of the dam height. Below this level it is generally considered conservative to then ignore tailwater loads in the stress analysis for simplicity (FERC, 1999). However, if tailwater affects uplift pressure on a failure plane on which sliding stability is being analysed, tailwater uplift should still be considered.

Uplift and pore water pressures

Uplift or pore water pressures develop when water enters the spaces and cracks within the body of an arch dam, as well as in the foundation joints, cracks, and seams.

During static loading conditions the effect of pore water pressure is to reduce normal compressive stresses within the concrete and to increase the corresponding normal tensile stresses should they exist. Considering the cumbersome process to include pore pressures in finite element models, combined with the relatively minor change in stress, pore pressures and their effects within arch dams have often been ignored in the absence of any cracks. McKay & Lopez (2013) proposed a practical methodology for including uplift and pore pressures.

According to FERC (1999) uplift does not need to be considered in the stress analysis for thin arch dams. Uplift should always be considered in the sliding stability analysis and be applied as external loads on both faces of tensile cracks at the dam-foundation interface. In the absence of field data and seepage analysis, uplift can be represented as described in ANCOLD (2013).

Silt load

The need to apply silt pressure in arch dam analysis depends on the sediment depth. According to FERC (1999) for U-shaped and broad base arch dams, sediment depth of less than 0.25 of the dam height produces negligible deformations and stresses, and thus their effects may be ignored. For V-shaped dams the effects of silt pressure may be ignored if the depth of sediment is less than 0.33 of the dam height.

Ice load

Although uncommon in Australia, ice can produce significant loads against the face of an arch dam and must be considered where reservoir freezing can be expected. Static ice loads is produced by the ice in contact with the dam when the reservoir is completely frozen, and dynamic ice loads by sheets of ice colliding with the dam.

Hydraulic loading of spillways

Forces produced by discharge through a spillway located on an arch wall are usually insignificant and typically ignored. USACE (1992) provides methods for determining spillway pressures if hydrodynamic forces could affect the dam.

Arch dams with overflow spillways can also be subject to forces produced by a fluttering nappe, which is caused by resonance between air trapped in the cavity between the nappe and the downstream face of the dam, as well as by spillway gates that transfer dynamic loading to the top of the dam. Such vibrations could be of importance to the safety of tall and thin arch dams. The phenomenon, and methods to prevent such vibrations, is thoroughly described in ICOLD (1996).

Hydrodynamic hydraulic loads

As the dynamic interaction that occurs between the reservoir and the dam during an earthquake can have a significant effect on the earthquake response of the dam, it must be considered in the dynamic analysis. Because the inertia force of a structure is a function of acceleration and mass, hydrodynamic interaction has a larger influence on thinner, less massive dams. There are three formulations for modelling hydrodynamic interaction:

- use of lumped mass (e.g. determined using the Generalised Westergaard theory of added mass to model incompressible fluid);
- incompressible fluid finite element equivalent added mass; and
- compressible fluid added mass, added damping and added forces with and without reservoir absorption.

Westergaard's theory of added mass, as developed in 1931, is reasonably appropriate only when assuming incompressible reservoir acting on a rigid straight gravity dam perpendicular to a wide valley, and with a vertical upstream face. For curved surfaces like arch dams, a Generalized Westergaard Method accounts for dam curvature and dam flexibility (Kuo, 1982). This method assumes that the hydrodynamic pressure at any point on the upstream face is proportional to the total acceleration acting normal to the dam at that point. The application of the Generalised Westergaard method is described in more detail in FERC (1999) and Reclamation (2006).

Incompressible fluid formulations ignore the compressibility of water and assume the reservoir floor and the upstream extent of the reservoir are rigid and ignore accelerations at these locations. Thus, the pressures induced on the dam from accelerations applied to the reservoir bottom are ignored.

Modern finite element analysis (FEA) software incorporates compressible water effects, which model the interaction between dam and reservoir and the proper transmission of pressure waves in the upstream-downstream direction. The use of compressible water effects is described in USACE (1999 & 2003) and Reclamation (2006).

Fluid elements are used to model the reservoir-dam and reservoir-foundation interaction more accurately. A three-dimensional mesh of fluid elements is developed to represent the reservoir. The use of fluid elements is described in Reclamation (2006).

Earthquake load

Arch dams are expected to respond linearly under the Operational Basis Earthquake (OBE), assuming continuous monolithic action along the entire length of the dam. If damage did occur, it should be possible to repair it while the dam remains operational.

The SEE (Safety Evaluation Earthquake, to replace Maximum Design Earthquake in the update of the ANCOLD (1998) guidelines) is the highest adopted magnitude the dam is required to withstand. The dam is allowed to respond nonlinearly and suffer significant damage, but without a catastrophic failure. The SEE to be used in the analysis of arch dams is defined in ANCOLD (1998), which is currently under review.

For preliminary linear response spectra analysis, site-specific response spectra of earthquake ground motions should be developed by experienced seismologists. The spectra should be developed for 5% damping, and relationships or factors provided to obtain response spectra for higher damping ratios (as high as 10%) if required for the analysis. These relationships or factors may be based on a documented site-specific study; alternatively, the relationships presented by Newmark and Hall (1982) may be used.

For more detailed linear and non-linear time-history analysis, acceleration time histories of ground motions should be developed consistent with the latest guidelines as for example contained in FERC (1999) and USACE (2003). Acceleration time histories should be developed for three components of motion (two horizontal and one vertical). Time histories may be either (a) recorded or simulated-recorded time histories or (b) response spectrum matched time histories. For recorded or simulated-recorded time histories, a minimum of three sets of recordings should be used.

Recorded earthquake ground motions at Pacoima Dam during the 1994 Northridge earthquake indicated that the seismic input for arch dams might vary along the dam foundation interface. At present time scarcity of data prevents a realistic definition of such non-uniform free-field motions for arch dams, even though procedures for handling non-uniform input have been developed. In view of these difficulties, the use of standard uniform seismic input is currently still acceptable.

Loading Combinations

Arch dams are designed for two groups of loading combinations. The first group combines all the static loads and the second group takes into account the effects of earthquake. In addition, depending on the probability of occurrence of the cases in each group, they are categorised as Usual, Unusual, and Extreme loading combinations.

Table 3 on the next two pages presents a summary of the static and dynamic loading combinations used by Reclamation, USACE, FERC and ANCOLD, including the loading combination categories. Although having a similar approach, they differ with regard to categorising certain loading combinations. The designer should however assess each load case to ensure that it is applicable to the project and that it is properly classified under one of the three categories.

Table 3 Summary of loading combinations [Note 1]

USACE (1994)	Reclamation (1977 & 2006)	FERC (1999)	ANCOLD (1998) [Note 2]	ANCOLD (2013) [Note 2]	Proposed for arch dams
Static Usual					
$D + T_w + H_x + U$ $D + T_s + H_x + U$ $D + H_n + T_x + U$	Reclamation (2006): $D + H_n + T_w + U$ $D + H_n + T_s + U$ *Reclamation (1977):* $D + T_w + H_x + S + I + U$ $D + T_s + H_x + S + U$ $D + H_n + T_x + S + I + U$ $D + H_l + T_x + S + I + U$	$D + T_w + H_x + S + I + U$ $D + T_s + H_x + S + U$	$D + T_w + H_n + S/B + I + U$ $D + T_s + H_n + S/B + U$ PQ: $D + H_n + S/B + U + I$	$D + H_n + S/B + U + I$ $D + H_{50} + S/B + U$	$D + T_w + H_x + S/B + U + I$ $D + T_s + H_x + S/B + U$ (If H_x is uncertain, check for H_l and H_f.) $D + H_{50} + T_x + S/B + U$ $D + H_n + T_x + S/B + U + I$
Static Unusual					
$D + H_s + T_x + U$ [Note 3] $D + H_l + T_x + U$ [Note 4] $D + H_e + T_x + U$ [Note 4]	Reclamation (2006): $D + H_l + T_s + U$ [Note 4] $D + H_f + T_w + U$ *Reclamation (1977):* $D + H_f + T_x + U$	$D + H_l + T_x + S + U$ [Note 4] $D + H_e + T_x + S + U$ [Note 4] $D + T_s + H_f + S + U$ $D + T_w + H_f + S + I + U$ $D + T_x + H_f + S + I + U$	$D + T_w + H_f + S/B + U$ $D + T_s + H_f + S/B + U$	$D + H_{500}\text{-}H_{2000} + S/B + U$ PQ: $D + H_{50} + S/B + U$ $D + H_{100} + S/B + U_{bd}$ $D + H_{100}$ (1 gate closed) $+ S/B + U$ $D + H_{100} + S/B + U$ + Wind seiche and wave action	$D + H_{500}\text{-}H_{2000} + T_x + S/B + U$ PQ: $D + H_{50} + T_x + S/B + U$ $D + H_{100} + T_x + S/B + U_{bd}$ $D + H_{100}$ (1 gate closed) $+ T_x + S/B + U$ $D + H_{100} + T_x + S/B + U$ + Wind seiche and wave action $D + H_l / H_e + T_x + S/B + U$ [Note 4]
Static Extreme					
$D + H_f + T_x + U$				$D + H_f + S/B + U + I$ $D + H_{100}$(>1 gate closed) $+ S/B + U$ $D + H_{lw} + S/B + U$	$D + H_f + T_x + S/B + U + I$ $D + H_f + T_w + S/B + U + I$ $D + H_f + T_s + S/B + U + I$ $D + H_{100}$(>1 gate closed) $+ T_x + S/B + U$ $D + H_{lw} + T_w + S/B + U$
Dynamic Unusual					
OBE $+ D + H_n + T_x + U$ OBE $+ D + H_e + T_x + U$			OBE $+ D + T_w + H_n + S/B + I + U$ OBE $+ D + T_s + H_n + S/B + U$ OBE $+ D + T_w + H_e + S/B + I + U$ OBE $+ D + T_s + H_e + S/B + U$	OBE $+ D + H_c + S/B + U$	OBE $+ D + H_n + T_w + S/B + U$ OBE $+ D + H_n + T_s + S/B + U$ OBE $+ D + H_e / H_l + T_w + S/B + U$ OBE $+ D + H_e / H_l + T_s + S/B + U$

Table 3 continued

USACE (1994)	**Reclamation (1977 & 2006)**	**FERC (1999)**	**ANCOLD (1998)** [Note 2]	**ANCOLD (2013)** [Note 2]	**Proposed for arch dams**
			Dynamic Extreme		
SEE + D + H_n + T_x + U	Reclamation (2006): SEE + D + H_n + T_c + U SEE + D + H_n + T_w + U SEE + D + H_n + T_s + U *Reclamation (1977):* *SEE + D + T_w + H_x + S + I + U* *SEE + D + T_s + H_x + S + U* *SEE + D + H_n + T_x + S + I + U* *SEE + D + H_l + T_x + S + I + U*	SEE + D + T_w + H_x + S + I + U SEE + D + T_s + H_x + S + U	SEE + D + H_c + S/B + U	SEE + D + H_n+ S/B + U	SEE + D + H_n+ T_w + S/B + U SEE + D + H_n+ T_s + S/B + U SEE + D + H_e / H_l + T_w + S/B + U SEE + D + H_e / H_l + T_s + S/B + U

H_n: Usual (normal) reservoir level with associated tailwater level
H_x: Reservoir level at the time with associated tailwater level
H_l: Lowest operating reservoir level with associated tailwater level
H_e: Empty reservoir level with no tailwater level
H_c: Critical reservoir level with associated tailwater level
H_f: Maximum flood reservoir level with associated tailwater level
H_{lw}: Landslide generated wave reservoir level with normal tailwater level

H_{50}: 1in 50 AEP reservoir level with associated tailwater level
H_{100}: 1in 100 AEP reservoir level with associated tailwater level
H_{500}: 1in 500 AEP reservoir level with associated tailwater level
H_{2000}: 1in 2000 AEP reservoir level with associated tailwater level

T_x: Temperature at the time
T_s: Summer temperature
T_w: Winter temperature

D: Dead load including appurtenances
U: Uplift
U_{bd}: Uplift with blocked drains
S: Silt (if applicable)
B: Backfill against dam (if applicable)
I: Ice (if applicable)
PQ: Post earthquake

Notes:

1. This table contains a collation of loading combinations proposed by the respective publications. Not all the cases would apply to a specific dam and the design should use judgement in selecting the combinations and categorising them.
2. The combinations for ANCOLD (1998) and ANCOLD (2013) were developed for gravity dams and are included here with the view to achieve proposed combinations for arch dams that are reasonably consistent with these ANCOLD guidelines.
3. This case applies only to dams that are normally at a low level or empty, such as flood retention dams.
4. For flood retention dams these cases should be considered under Usual Static.
5. Uplift should be determined by the reservoir level and tailwater level taking into account any foundation drains.
6. Silt, backfill against the dam and ice should be included only if applicable.

Table 4 Summary of factors of safety

Parameter	USACE (1994)	Reclamation (1977 & 2006)	FERC (1999)	ANCOLD (1998) Note 1	ANCOLD (2013) Note 1	Range	Proposed for arch dams
Static Usual							
Compression	4.0	3.0-not more than 10.3 MPa (4.0) Note 2	2.0	4.0	3.3	2.0-4.0	**3.3**
Tension	1.0	Not more than 1.03 MPa	1.0	1.0	1.0	1.0	**1.0**
Sliding	2.0	3.0	1.5 (2.0) Note 3	1.5 – 2.0	1.5 / 2.0 / 3.0 Note 4	1.3-3.0	**1.5 / 2.0 / 3.0 Note 4**
Static Unusual							
Compression	2.5	2.0 not more than 15.5 MPa (2.7) Note 2	1.5	2.7	2.0	1.5-2.7	**2.0**
Tension	1.0	Not more than 1.55 MPa	1.0	1.0	1.0	1.0	**1.0**
Sliding	1.3	2.0	1.5	1.3 – 1.5	1.3 / 1.5 / 2.0 Note 4	1.3-2.0	**1.3 / 1.5 / 2.0 Note 4**
Static Extreme (maximum flood condition only)							
Compression	1.5	Note 5	Note 5	Note 5	1.3	1.3-1.5	**2.0**
Tension	1.0	Note 5	Note 5	Note 5	1.0	1.0	**1.0**
Sliding	1.1	Note 5	Note 5	Note 5	1.1 / 1.3 / 1.5 Note 4	1.1-1.5	**1.3 / 1.5 / 2.0 Note 4**
Dynamic Unusual (cases including OBE)							
Compression	2.5	2.0 not more than 15.5 MPa (2.7) Note 2	-	2.7	2.0	2.0-2.7	**2.0**
Tension	1.0	Not more than 1.55 MPa	-	1.0	1.0	1.0	**1.0**
Sliding	1.3	2.0	-	1.3 – 1.5	1.3 / 1.5 / 2.0 Note 4	1.3-2.0	**1.3 / 1.5 / 2.0 Note 4**
Dynamic Extreme (cases including SEE)							
Compression	1.5	1.0 (1.3) Note 2	1.1	1.3	1.3	1.1-3.0	**1.3**
Tension	1.0	1.0	1.0	1.0	1.0	1.0	**1.0**
Sliding	1.1	1.0	1.1	1.2 – 1.4	1.1 / 1.3 / 1.5 Note 4	1.0-1.5	**1.1 / 1.3 / 1.5 Note 4**

Notes:

1. These guidelines were developed for gravity dams.
2. Values in brackets are for the foundations.
3. Value in brackets is for internal shear.
4. Factors are for "Residual strength c' and φ' well-defined" / "Peak strength c' and φ' well-defined" / "Peak strength c' and φ' not well-defined" (refer ANCOLD (2013) for further explanation of these conditions).
5. These references included maximum flood in the static unusual category.
6. The factor of safety for allowable tensile strength should be applied to the concrete under consideration, i.e. either the parent or lift joint strength.

Acceptance criteria

The structural integrity of an arch dam is maintained and it is considered safe if overstressing, sliding and other possible modes of failure will not occur.

Reclamation, USACE, FERC and ANCOLD differ slightly in the values assigned to determine the allowable stresses and the factors of safety against sliding.

Table 4 presents the factors of safety used by each of the organisations for the allowable stresses and stability. This table also includes the factors suggest by the authors of this paper when assessing arch dams in accordance with ANCOLD guidelines, as further discussed below.

Allowable stress

The ultimate load-resisting capacity of an arch dam is determined by the compressive strength of the concrete, unless foundation or another mode of failure occurred first. The tensile strength of the concrete is however an important consideration, particularly in estimating seismic safety of concrete arch dams.

As discussed by Gillan et al (2011) the general stress-strain behaviour of concrete can be characterized in four stages:

- In the first stage the stress-strain curve is considered to be linear elastic, i.e. there would be no permanent deformation. According to ACI (1996) mass concrete material behaves linearly up to approximately 35 % of the ultimate strength.
- In the second stage the stress-strain curve consists of some inelastic behaviour (cracking). The load would result in minor permanent deformations and strains in the material. According to ACI (1996) the growth of internal microcracks commences in the concrete at

loads equal to approximately 35 to 50 % of the ultimate strength.

- The third stage consists of large inelastic strains, so that there is a noticeable change in deformation. In this stage there is stable crack growth in the concrete, meaning that cracks will form but not initiate failure.
- The fourth stage is the fracture stage where deformations are great enough to produce unstable crack grow and eventual failure of the concrete.

The above mentioned concrete behaviour was considered when comparing the factors of safety given in the existing references and suggesting factors to be adopted for arch dams. For load combinations in the Static Usual category it is assumed that the concrete behaviour is limited to the linear elastic first stage and a factor of safety for compressive strength of 3 would be appropriate, with 3.3 adopted in ANCOLD (2013). Compared to the factors used provided in the references by USACE, Reclamation and FERC, the same factors as used in ANCOLD (2013) is suggested for arch dams.

For the less frequent load combinations in the Static Unusual, Static Extreme and Dynamic Unusual categories it is assumed that the concrete behaviour is limited to the second stage, which can result in microcracking and minor permanent deformations and strains in the concrete that would not affect the operation of the dam. Therefore, a factor of safety for compressive strength of 2 to 3 would be appropriate, with 2.0 adopted in ANCOLD (2013). Compared to the factors provided in the references by USACE, Reclamation and FERC, the same factors as used in ANCOLD (2013) is suggested for arch dams, except for the Static Extreme category, as discussed in the second paragraph below.

For the rare load combinations in the Dynamic Extreme category (SEE) it was assumed that the concrete behaviour is limited to the third stage. This assumes that the concrete may crack and experience permanent deformations and damage due to the load, but not enough to cause failure. A factor of safety for compressive strength of 1.1 would be appropriate, with 1.3 adopted in ANCOLD (2013). Compared to the factors provided in the references by USACE, Reclamation and FERC, the same factors as used in ANCOLD (2013) is suggested for arch dams.

ANCOLD (2013) adopted only one set of criteria for the Extreme category, which includes both the maximum design flood and the maximum safety evaluation earthquake, although they relate to the static and dynamic strength parameters respectively. This implies that for the flood load condition concrete behaviour is allowed beyond the second stage, i.e. in the third stage which includes cracking and permanent deformation. Whereas the SEE loading is a cyclic loading, the extreme flood condition is a sustained loading. The authors believe that the factors in ANCOLD (2013) for the flood loading condition (Static Extreme) may be too low and the allowable behaviour of the concrete should be limited to the second stage for which the factor of safety for compressive strength of 2 to 3 would be appropriate.

In Table 4 for all the cases the factor of safety for allowable tensile stress is unity. The intent of any design should be to minimize or limit tensile stresses to localized areas by reshaping and / or redesigning the dam. A dam designed with high tensile stresses in too many areas, even though within the allowable limits, might exceed the compressive limits under one or more loading combinations. When the tensile strength of the concrete is exceeded and cracking occurs, the uncracked portion of the cantilever would tend to carry more compression while also increasing the balance of the loads carried by the arches. If the cracking becomes widespread, too much of the load would have to be carried by the arches. The uncracked portion of the cantilevers could exceed the compressive strength of the concrete and cause crushing failure of the concrete. Subsequent joint opening and cracking and load redistributions might eventually exhaust the capacity of the concrete, or might form surfaces along which partial sliding could occur. Therefore, since compression is the dominant mode of failure of an arch dam, and since the other concrete properties are a function of the compressive strength, a more conservative approach is taken in establishing the allowable compressive stresses compared to the allowable tensile stresses.

Pursuant to the above, one of the objectives in arch dam design is minimizing the magnitude and the locations of tension in the dam. Tensile stresses are however inherent to most arch dams and therefore require further consideration.

Arch dams typically exhibit tensile stresses at the downstream face along the foundation under the low reservoir – high temperature conditions, which includes the construction period. This condition is regarded as a significant problem as long as the stability of the cantilevers is not in question. Even if some cracking has occurred, the additional hydrostatic load and the resulting downstream deflection will cause the cracks to close. Tension at the upstream face of the dam however requires more careful consideration, due the possibility of a seepage path through the dam if cracks were to develop and extend through the thickness of the dam. Cracked cantilevers do not necessarily imply a dam failure, as loads carried by the cantilevers before cracking will be transferred to the arches and adjacent cantilevers. A nonlinear analysis is required to ensure that the compressive stresses of the remaining uncracked section of the cantilever and the other arches and cantilevers remain within the allowable concrete stresses.

As already mentioned the current engineering guidelines recommend a factor of safety of 1.0 to determine the allowable tensile stresses. The authors believe that this factor should be used with caution and only be used when extensive testing has been undertaken so that sufficient statistical data is available to estimate the existing tensile strength with confidence.

In the absence of tensile strength data of the concrete, Chopra (1994) suggested that the tensile-compressive strength relationships determined by Raphael (1984) could be used if sufficient compressive strength data is available.

When assessing existing dams with insufficient or no construction records, the number of tensile strength tests are usually limited and insufficient to make reliable statistical correlations. This could lead to overestimating the existing tensile strength of the concrete when applied to the entire dam. It is therefore deemed appropriate to apply a strength reduction factor and assume a lower and more conservative tensile strength for the entire dam.

The ACI Building Code (ACI, 2008) states that one of the reasons for strength reduction factors is to account for potential understrength members due to variations in material strengths and dimensions. In plain (mass) concrete, since both flexural tension strength and shear strength depend on the tensile strength characteristics of the concrete, with no reserve strength or ductility possible due to the absence of reinforcement, equal strength reduction factors for both bending and shear are considered appropriate. Consequently, ACI suggests a strength reduction factor of 0.6 for bending and shear, based on reliability analyses and statistical studies of concrete properties, as well as calibration against past practice. A practical approach is therefore to apply the strength reduction factor of 0.6, as suggested by ACI for tensile strength in plain concrete, to determine the existing tensile strength of the concrete when only limited testing has been undertaken. The factors of safety for tension as given in Table 4, are then applied to the reduced tensile strength in order to estimate the allowable tensile strength.

Stability of cantilevers during construction

As arch dams are constructed in monoliths, because of the vertical curvature, the monoliths may be unstable against overturning prior to the grouting of the monolith joints and the filling of the reservoir. When assessing the construction phase dam empty case, the stability of the cantilevers must be assessed to assure that each cantilever is stable at different stages of construction.

Dynamic loading

Establishing the acceptability of performance of arch dams under dynamic load cases is a complicated process which cannot be summarized in a table and is discussed in the next section under “Analysis methods and evaluation”. The factors of safety given in Table 4 are only the first step in determining the safety criteria and should not be regarded as absolute limits. These factors should be applied to the test results for the appropriate rate of loading as indicated by the dynamic analysis, or as determined for the static analysis and modified according to the approach discussed by Raphael (1984).

Analysis methods and evaluation

Concrete arch dams are complex three-dimensional shell structures that rely on both cantilever action (on vertical planes) and arch action (on horizontal planes) for transferring the loads to the foundation, which is deemed to include the entire length of concrete rock interface. Hence, two dimensional analyses cannot provide realistic results and the three dimensional geometry of the dam, foundation and reservoir (if required) has to be considered in the analysis to provide a more realistic idealisation of the structural system.

Method of analysis

Three-dimensional finite element analysis is preferred for the static and dynamic analysis of arch dams. The trial load method is considered outdated, but may still be used for preliminary static stress analysis only if the dam has a simple geometry and uniform material parameters can be assumed for the concrete and for the foundation rock in a low hazard area. Other mathematical formulations and approaches can also be employed, but the accuracy of such methods should be verified by comparison with the finite element analyses

With the development of the finite element method, the advances in dynamic analysis procedures and the availability of high capacity computers, the older traditional methods of analysis have been abandoned. The use of the finite element analysis technique, with its versatility and capability to deal with structural, geotechnical and fluid aspects, allowing a more realistic analysis of virtually any type of dam, has become the standard practice for dam design and analysis.

Different elements have been used in the past to model arch dams, including shell elements and solid elements. Shell elements (with five or six degrees of freedom) are consistent with the behaviour of the dams; however they are unable to accurately model stress distribution through the dam thickness. Therefore, three dimensional brick/solid elements (with three degrees of freedom) are now routinely used for the analysis.

Evaluation for static loading

Idealization of the dam and an appropriate portion of the foundation rock as an assemblage of finite elements is the first step in a typical finite element static analysis. Each individual static load case as explained in the previous sections is applied to the model separately and the results should be obtained and reviewed to facilitate examination of the consistency of the results. The load cases are then combined based on proposed load combinations and finally the system is solved to determine displacements at nodal points of the assembled structure, stresses computed at various locations (such as the Gauss integration points) and arch thrusts and shears exerted on the dam abutments and / or thrust blocks.

At each nodal point, three displacement components corresponding to a global system of axes are computed. The magnitudes and deflected shapes of the resulting displacements provide important data that can be used to visually evaluate the overall behaviour of an existing dam (or acceptability of a new design), although the magnitudes of the resulting deformations are not directly used in safety evaluation of arch dams. The deflection patterns should vary smoothly from point to point and are used to evaluate the adequacy of the design/analysis by visual means.

Since maximum stresses in an arch dam usually occur at the faces of the structure, normal stresses resolved into arch, cantilever and principal stresses at the upstream and downstream faces of the dam are the primary stresses used for the evaluation of the analysis results. However, shear stresses induced in the body of the dam by bending and twisting moments should also be examined to assure that they are within the allowable limits. The evaluation of the safety of an existing dam involves comparing the maximum computed stresses with the allowable compressive and tensile strengths of the concrete. The largest compressive and tensile stresses should be less than the corresponding allowable strengths of the concrete considering the factors of safety established in Table 4 for each particular loading combination.

Whenever the overall stresses in the structure are below the allowable values as specified in the previous sections, the design is considered to be adequate or the existing dam is safe from stress distribution through the dam wall point of view. A well-designed arch dam will develop only compressive stresses under the static loads and these are generally much smaller than the allowable compressive strength of the concrete.

The stress results produced by the linear finite element analysis usually indicate some areas of tensile stress in the dam. Whilst tensile strength of the parent concrete could be high, it should be kept in mind that a typical arch dam is made of concrete blocks divided by lift joints and vertical contraction joints, with lower tensile and shear strengths than the parent concrete (or even a pre-existing crack with no tensile strength). Therefore it is not appropriate to evaluate the indicated tensile stresses of a finite element model in terms of allowable tensile stress for the parent concrete alone.

If the compressive or tensile stresses in some locations of the dam predicted by linear elastic analysis exceed the allowable strengths, they may not reliably predicted the extent of the cracking (damage) or the true behaviour of the dam. For these cases, nonlinear response of concrete dams can be employed to provide a more accurate response of the dam and the damaged region of the dam. However, the predictions of the extent of the damage obtained from these analyses are quite sensitive to the assumed nonlinear properties of concrete or joints.

Evaluation for seismic loading

There are no codes and regulations which are universally applicable to the earthquake resistant design of concrete arch dams. The performance criteria, therefore, should be discussed on a case-by-case basis.

The earthquake response of an arch dam is assessed through a staged approach. Usually, the linear response-spectrum mode-superposition method is used as first approach, but if maximum stresses exceed the allowable values, a time-history analysis is required to assess the severity of joint opening and tensile cracking. The basic results of a response-spectrum analysis consist of the maximum nodal displacements and element stresses. Because the response to the three earthquake components (two horizontal plus vertical) are developed independently, the maximum dynamic responses due to the earthquake components are further combined by the SRSS method to include the effects of all three components. It is notable that the resulting dynamic responses obtained in this manner have no sign and may be interpreted as being positive (e.g. tensile stress) or negative (e.g. compressive stress). In the response-spectrum method, total stresses are estimated by linear combination of the dynamic stresses with static stresses due to the usual loading combination and compared with the allowable values. This method requires a qualitative judgement of how stresses will be redistributed during joint opening and cracking. This evaluation is done in lieu of more prolonged time-history analysis. This approach is not sufficient for some situations and a more detailed analysis using time-history techniques may be required.

The evaluation criterion for time-history analysis is more involved than simple stress checks. As described by Ghanaat (2004), a systematic interpretation and evaluation of these results in terms of the stress demand-capacity ratios, cumulative overstress duration, spatial extent of overstressed regions, and other considerations form the basis for an approximate and qualitative estimate of damage (see Figure 2). This evaluation is applied to the damage control range of strains. If the estimated level of damage falls below the acceptance threshold for a particular dam type, the damage is considered to be low to moderate and the linear time-history analysis would suffice. Otherwise the damage is considered to be severe, requiring a non-linear time-history analysis to determine whether or not it would lead to failure of the dam.

Horizontal lift joints and vertical contraction joints should be assumed to crack when subjected to tensile stresses exceeding their tensile strengths. The dam may be considered safe for the SEE if, after the effects of crack and joint opening have been accounted for, it can be shown that the concrete is not over-compressed and free cantilevers do not topple.

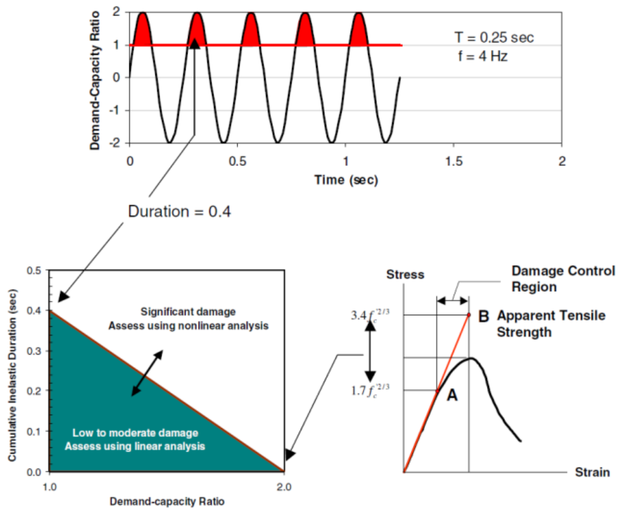

Figure 2 Illustration of seismic performance and damage criteria (Ghanaat, 2004)

Post-earthquake safety

A post-earthquake safety evaluation is required to assure the safety of the dam if a damaging SEE should occur, or the predicted performance of the dam due to a postulated SEE should indicate substantial damage. This evaluation should consider the effects of static loads as well as severe aftershock earthquakes that invariably occur after any major quake.

Sliding stability

To assure safety against sliding along identified feasible failure planes in the dam, at the dam-foundation interface, or in the foundation, the shear friction factor of safety should higher than those given in Table 4 for normal, unusual and extreme loading. These safety factors assume that stability has been evaluated with respect to conservative shear strength parameters. For major dam structures subjected to severe seismic loading, time-history analyses should be considered for abutment and foundation stability instead of the usual pseudostatic analyses. In time-history analyses, the factor of safety varies with time and may become less than 1.0 for one or more cycles provided that the resulting cumulative sliding displacement is very small and can be tolerated.

Stability analyses of foundation blocks typically involve uncoupled analyses whereby loading from the dam is calculated from finite element analyses and applied in a separate rigid block foundation analysis. When time-history rigid block analyses are performed and the factor of safety drops below 1.0 during the earthquake, the permanent displacement could be estimated using the Newmark type method. Such an estimate would be a conservative worst case displacement since it is assumed that the loads follow the block as it displaces. In reality, loads would change direction and be redistributed by the dam. In certain critical cases, a coupled dam-foundation analysis may be warranted, but this is not often the case.

Acknowledgement

The authors which to acknowledge Mr Brian Cooper, who reviewed this paper also from the perspective as a member of the Working Group that is updating the ANCOLD (1998) guidelines, and commented as follows:

"I am in general agreement with the principles and concepts presented in this paper. They are consistent with what has been written to date, in the revised ANCOLD earthquake guidelines which covers earthquake analysis of all concrete dams. There may be some minor differences in the factors of safety but these will be worked out as the revised guidelines are further developed. The revised guidelines are less cook-book and more guidelines – more along the lines of this paper."

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