ANCOLD Guidelines for Design of Dams and Appurtenant Structures for Earthquake

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1 INTRODUCTION

The purpose of these Guidelines is to provide Owners, Regulators, Consultants, and others involved in the management of existing and new dams and appurtenant structures guidance on the selection of seismic ground motions resulting from earthquakes, and analysis and assessment procedures. The Guideline covers water supply dams, tailings dams and retarding basins (levees are not covered in this Guideline).

The Guidelines are not:

- A textbook on earthquakes and design for seismic ground motions.
- A design code or Australian Standard. Hence the mandatory "shall" or "ensure" has been avoided in the text.

These Guidelines have been written for the guidance of experienced practitioners. There is a need for the users of the Guideline to apply their own experience and judgement in the application of this Guideline. Therefore, it is strongly recommended that personnel who undertake, or at least closely supervise, the assessment procedures outlined within this Guideline are highly experienced in field of the assessment being undertaken.

There have been significant advances in the understanding of earthquakes in Australia since ANCOLD (1998) "*Guidelines for Design of Dams for Earthquake*" was published. These include:

- A recognition that there are active faults in some areas, and other neotectonic faults which have been active in the current regional stress regime and may contribute to the seismic hazard.
- Refinement of earthquake source models for modelling seismic hazard.
- Refinement of ground motion models, and development of models for Australian conditions.
- Refinement in the modelling of uncertainty in the seismic hazard.

The Guidelines provide detailed descriptions of these and guidance on the requirements of seismic hazard assessments.

There have also been advances in the methods available for analysis and design including:

Embankment dams:

- Assessment of the potential for liquefaction of dams and their foundations, and post-earthquake liquefied strengths.
- Numerical modelling of deformations during seismic ground motions and post-earthquake.

Concrete Dams

- Numerical modelling of the dams for seismic ground motions including modelling of deformations and displacements.
- Improved understanding of the importance of identifying and modelling kinematically viable mechanisms for sliding in the foundation.

Appurtenant Structures

- Numerical modelling of the structures for seismic ground motions including modelling of deformations and displacements.

Importantly, there is a greater understanding and expertise within the Australian dam industry than in 1998.

Given the extensive literature relating to seismic hazard assessment, and seismic analysis and design, these Guidelines retain a large body of information to assist users. These are in the Commentary. Users should read the Commentary along with the Guideline Sections so they understand the details and background to the Guideline.

Since the issue of ANCOLD (1998) a number of other ANCOLD Guidelines have also been updated. Where appropriate, references have been made to these other Guidelines rather than repeating sections of them. If inconsistencies occur then this Guideline supersedes older documents.

The Guideline encourages the use of risk based methods for assessing existing and design of new dams. This is because of the uncertainty in the seismic hazard, and the fact that the response of dams and appurtenant structures to seismic loads may change significantly with the seismic ground motion. Therefore, a dam or appurtenant structure that is assessed using a single design ground motion only may not necessarily satisfy ANCOLD (2003, 2017 (pending)) Risk Management Guidelines.

However, it is recognised that many Owners prefer to use a deterministic approach to assessment of design seismic ground motions so both approaches are covered in the Guideline.

For the purposes of defining roles and responsibilities the following terminology has been adopted throughout this Guideline:

- Owner – the entity responsible for the asset, including operation and maintenance and holds a duty of care to the community that the asset risks are as low as reasonably practicable .
- Regulator – the entity responsible for regulation of dams within each state or territory.
- Consultant – the entity responsible for provision of assessment and design services referred within the Guideline. This role may be performed by the Owner, educational or government institution, or by a professional consulting entity suitably qualified to deliver the services.

AS1170 (Structural Design Actions - Part 4: Earthquake actions in Australia) specifically states that dams are not within the scope of the standard and therefore AS1170 should not be used for dams and appurtenant structures.

2 ASSESSMENT OF DESIGN SEISMIC GROUND MOTIONS AND ANALYSIS METHOD

2.1 Earthquakes and Their Characteristics

Users of this Guideline should read the Commentary on earthquakes and their characteristics generally and Australian earthquakes in particular. A good understanding of these is important to using the Guideline.

2.2 Terminology

The following terminology is based on ANCOLD (1998), and ICOLD Bulletin 148 - Selecting Seismic Parameters for Large Dams (ICOLD, 2016a).

Operating Basis Earthquake (OBE) **–** the OBE is that level of ground motion at the dam site for which only minor damage is acceptable. The dam, appurtenant structures and equipment should remain functional and damage from the occurrence of earthquake shaking not exceeding the OBE should be easily repairable.

Safety Evaluation Earthquake (SEE) **–** the SEE is the maximum level of ground motion for which the dam should be designed or analysed. Damage can be accepted but there should be no uncontrolled release of water from the reservoir or tailings from tailings dams.

The term *Safety Evaluation Earthquake* replaces the term *Maximum Design Earthquake* (MDE) in ANCOLD (1998, 2012a, 2013).

The OBE and SEE are expressed in terms relevant to the design of the dam or appurtenant structure. It may be in terms of peak ground acceleration (PGA), peak ground velocity (PGV), or response spectra, which may be accompanied by ground motion time-histories or by proxy information such as moment magnitude Mw representing ground motion duration.

Maximum Credible Earthquake (MCE) **–** the MCE is the largest reasonably conceivable earthquake magnitude that is considered possible along a recognised fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework.

Maximum Credible Earthquake Ground Motion **–** The most severe ground motion affecting a dam site due to an MCE.

Active Faults **–** a fault, reasonably identified and located, known to have produced historical earthquakes or showing evidence of movements in Holocene time (i.e. in the last 11,000 years), large faults which have moved in Latest Pleistocene time (i.e. between 11,000 and 35,000 years ago).

Neotectonic fault - a fault that has hosted displacement under conditions imposed in the current crustal stress regime, and hence may move again in the future.

2.3 General Principles for Selection of Design Ground Motion and Analysis Method

There are two ways of selecting the design seismic ground motion and analysing the structures:

(a) *Deterministic Analysis* – This requires the selection of an OBE and SEE, taking into consideration the Consequence Category of the dam, and the implications of failure of appurtenant structures. The dam and appurtenant structures are analysed for these ground motions. For the SEE it is usual to require a factor of safety to be applied to estimated stresses and deformations within the dam and its foundations to give a low likelihood of the dam failing given the SEE loading.

(b) *Risk Based Analysis*– which assesses the effects on the dam and its foundations of a range of seismic loads up to and greater than the SEE. The design requirement is to satisfy ANCOLD (2003, 2016) tolerable risk criteria. The steps in the process are detailed in Section C2.3.

Preferred method – Based on consideration of all the factors ANCOLD prefers the risk based approach to the deterministic approach for High and Extreme Consequence Category dams. For

Significant and Low Consequence Category dams, deterministic approaches will usually be adequate.

The decision as to whether a risk based or deterministic approach is adopted is for the Owner in consultation with the Regulator.

The Consequence Category of the dam is assessed in accordance with ANCOLD (2012).

2.4 Description of a Probabilistic Seismic Hazard Assessment

A Probabilistic Seismic Hazard Assessment (PSHA) is an evaluation of the ground motion level that will be exceeded at a specified frequency or annual probability. PSHA involves the following steps:

- Identify all earthquake sources capable of generating damaging ground motions at the site, and characterise their location, geometry and sense of slip;
- Characterise the rates at which earthquakes of various magnitudes are expected to occur on each source;
- Characterise the distribution of source-to-site distances associated with potential earthquakes;
- Predict the resulting ground motion level for each earthquake scenario using ground motion prediction equations;
- Combine uncertainties in earthquake size, location and ground motion level using the total probability theorem within a qualified computer program.

Section C2.4 gives a detailed description of the PSHA process.

2.5 Requirements of a Seismic Hazard Assessment

A PSHA is required for both deterministic and risk based approaches.

Where there are active fault(s) in the vicinity of the dam site the deterministic method for Extreme Consequence Category dams also requires assessment of the ground motion at the dam site from the MCE on the active fault(s).

The seismic hazard assessment should be carried out by Seismologists experienced in seismic conditions in Australia, which are different in some regards to what occur elsewhere.

When requesting a seismic hazard assessment the Owner or Consultant should provide site specific details to the Seismologist that may include latitude and longitude for the site, natural frequency of the structures being assessed, specific return periods to be included in the seismic hazard assessment, minimum magnitudes, geological information for the site, Vs30 data, etc.

The seismic hazard assessment for High and Extreme Consequence Category dams should include the following features:

1. *Distributed earthquake source models.* Analyses should use all viable source models. The weighting between these should be determined by the Seismologist in consultation with the Owner and Consultant.
2. *Fault sources*. Active and neotectonic faults which could significantly contribute to the ground motion for the dam should be identified, and be accounted for in the seismic hazard assessment.
3. *Faults in the dam foundation.* Information on any known active or neotectonic faults which have the capability to cause displacement in the foundation of the dam, appurtenant structures, or reservoir rim should be provided.
4. *Ground motion prediction models*. Consideration of multiple alternative ground motion prediction equations (GMPE's) to address epistemic uncertainty. These may vary for cratonic and non-cratonic areas. For non-cratonic areas at least one of the coefficients should be Next Generation Attenuation (NGA), along with one or more of the models developed for Australia. Weightings applied to the models should be determined by the Seismologist in consultation with the Owner and Consultant.

5. *Shear wave velocity of the dam foundation bedrock.* Shear wave velocity of the foundation rock should be determined and included in the ground motion evaluation. The shear wave velocity may vary from valley to abutment sections so the seismic hazard may be different in the valley and the abutments.
6. *Effects of soil overlying bedrock.* If the dam is founded on sediments or deep residual soils and completely weathered rock overlying the bedrock which are expected to have significant nonlinear response, the PSHA ground motions should be specified at the surface of bedrock beneath the sediments or weathered rock at a depth in the profile having a suitable shear wave velocity that is identified jointly with the soil response Consultant, to provide input into a separate study of nonlinear soil response that would not be part of the seismic hazard assessment. The bedrock ground motions may be amplified or de-amplified by the overlying soil depending on the soil depth and properties and the level of the bedrock seismic ground motion.
7. *Response Spectra.* The response spectra used for the hazard assessment should be designed to allow for the response of the dam and appurtenant structures at the dam. Conditional mean spectra may be used to represent the design response spectra for the dam site.
8. *Minimum magnitude earthquake.* The hazard assessment should be carried out for earthquake magnitudes Mw 5 and more. However, under some circumstances, smaller magnitude earthquakes may form the lower limit. With masonry dams, slab and buttress dams, older concrete dams and structural concrete components of dams, Mw 4 earthquake should form the lower limit.
9. *Quantification and reporting epistemic uncertainty.* The epistemic uncertainty in the true value of the hazard due to epistemic uncertainty in the earthquake source (1) and ground motion models (4) should be quantified using the fractiles of the hazard. The report should include median (50^{th} fractile), 85^{th} fractile and 95^{th} fractile percentage spectral response spectra plots.
10. *De-aggregation plots and selection of time-history motions.* The report should include plots showing de-aggregation of the hazard by earthquake magnitude and distance, and based on these contributions provide guidance on how to select time-history motions (accelerograms) suited for the dam and the appurtenant works.
11. *Reservoir-induced seismicity*. For new dams, guidance on whether reservoir-induced seismicity need be considered, and if so, the resultant seismic loading.

For Significant Consequence Category dams, Owners may opt with the advice of their Seismologist and Consultant to not require items (2), (3), and (11); only one GMPE (4), and use estimated shear wave velocities (6); and provide mean estimates without modelling uncertainty (9). However if that study shows that the seismic hazard is critical to the assessment of the risks posed by the dam the more complete assessment of the seismic hazard will be required.

For Low Consequence Category dams and for preliminary studies of higher Consequence Category dams it may be appropriate to conduct an initial assessment based on existing PSHA from nearby dams. Depending on the dam, its characteristics and whether its important potential failure modes are seismic loads related; e.g. liquefaction, a decision can then be made as to whether a site specific PSHA is required.

In some situations for High and Extreme Consequence Category dams a staged approach may be appropriate with a less detailed PSHA as described above for Significant Consequence Category dams used initially, and the more detailed assessment carried out if it is recognised that the seismic hazard is critical. Similarly for Significant Consequence Category dams it may be appropriate to start with an initial assessment based on an existing PSHA from nearby dams, and then conduct to more detailed site specific assessment if it is recognised that the seismic hazard is critical.

It should be noted that PSHAs become dated as new methods are developed and data bases improved. It is unlikely that a PSHA more than about 5 years old will be reliable. The advice of the Seismologist who carried out that study should be sought to advise if a new study is required.

2.6 Selection of Design Seismic Ground Motion - Deterministic Analysis Approach

Operating Basis Earthquake OBE

The selection of the OBE is a matter for the Owner to consider in consultation with the Consultant and other stakeholders. As shown in Table 2.1 the OBE is often accepted as a loading which has a 10% chance of being exceeded in a 50 year period, or an annual probability of exceedance of 1 in 475. However higher or lower annual probability loadings may be adopted depending on the Owners appetite for risk, and the criticality of the dam and appurtenant structures or tailings dam.

Safety Evaluation Earthquake SEE

The recommended Safety Evaluation Earthquake ground motions are shown in Table 2.1.

Table 2.1: Recommended deterministic analysis seismic design ground motions.

Dam Consequence Category	Operating Basis Earthquake OBE (1)	Safety Evaluation Earthquake SEE (2)
Extreme Consequence Category Dams	Commonly 1 in 475 AEP up to 1 in 1,000 AEP	*The greater of*: Ground motion from MCE on known active faults (3) or Probabilistic ground motion Extreme : 1 in 10,000 AEP (4)
High A, B and C Consequence Category Dams	Commonly 1 in 475 AEP up to 1 in 1,000 AEP	Probabilistic ground motion (5) (6) : High A: 1 in 10,000 AEP High B: 1 in 5,000 AEP High C: 1 in 2,000 AEP
Significant Consequence Category Dams	Commonly 1 in 475 AEP	Probabilistic ground motion (5) : 1 in 1,000 AEP
Low Consequence Category Dams	Commonly 1 in 475 AEP	Probabilistic ground motion (5) : 1 in 1,000 AEP

Notes
(1) Owner and other Stakeholders to determine in consultation with the Consultant.
(2) A factor of safety should be applied to estimated stresses and deformations within the dam and its foundations to give a low likelihood of the dam failing given the SEE loading.
(3) Active faults are as defined in Section 2.2. See C2.6 for discussion on active faults.
(4) 85^{th} fractile.
(5) Median, 50^{th} fractile.
(6) The adoption of these SEE for High B and High C Consequence Category dams may prove in some particular cases not to provide an acceptable level of risk in accordance with ANCOLD Risk Management Guidelines. It is therefore recommended that some level of risk assessment be undertaken in these cases before adopting the AEP' stated in the table. If it cannot be demonstrated that an acceptable level of risk would be achieved then an AEP of 1 in 10,000 should be adopted.

2.7 Selection of Design Seismic Ground Motion - Risk Based Analysis Approach

As discussed in Section 2.3, for risk based assessments the seismic ground motions should cover the complete range of feasible loads up to and above the deterministic SEE values.

2.8 Modelling Vertical Ground Motions

The vertical component of ground motions should be included in the time-history accelerograms and used in dynamic analyses for embankment and concrete dams.

For pseudo-static analyses of concrete dams the vertical ground motions should be estimated using the method described in the Commentary.

Vertical ground motions are often not incorporated in simplified deformation analyses for embankment dams. If required the methods described in the Commentary should be used.

2.9 Selection of Response Spectra and Time-History Accelerograms

A response spectrum will be required for each of the earthquake ground motion cases (annual exceedance probabilities (AEP)) to be used in analysing the dam. The response spectra should be prepared by specialist Seismologists. The response spectra are to be site specific, and for the damping factor applicable to the dam. For example, response spectra for concrete dams with linear elastic response will usually be for a 5% damping factor (viscous damping). This linear elastic analysis ignores the hysteresis damping with inelastic behaviour unless a non-linear inelastic analysis is carried out, which is the most accurate to capture the true behaviour of the dam or structures, refer to the Commentary for additional details regarding damping.

For time-history analyses, use at least 3 ground motion records. Generally, accelerograms (acceleration vs time) will be used although some analysis methods prefer velocity/displacement vs time. Use a specialist Seismologist to develop synthetic accelerograms (or velocity/displacement vs time where required) if recorded accelerograms appropriate to the AEP earthquake are not available. Use amplitude scaling or spectral matching to produce accelerograms that will produce a response spectrum that would approximately match the design response spectrum over the range of frequencies equivalent to the range of natural frequencies of interest for the dam.

2.10 Earthquake Aftershocks

Consider whether the dam, or appurtenant structures, which are critical to operation of the dam post-earthquake are likely to be damaged sufficiently by the main earthquake ground motion so as to be more vulnerable to further damage by earthquake aftershocks. If so, seek advice from the Seismologist on the likely magnitude, focal location, and seismic ground motions at the dam site, and analyse the effects on the dam and appurtenant structures. It also needs to be considered how the safety of the dam will be managed in the period through the aftershocks until repairs can be completed.

2.11 Seismic Ground Motions from Earthquakes Induced by the Reservoir

The ground motion from Reservoir Triggered Earthquakes (RTE) should be considered in the seismic hazard assessment for new dams, either risk based, or deterministically. This would be a specific request to the Seismologist for advice on whether RTE should be considered and if so what the loadings would be. Refer to the Commentary for details.

3 ASSESSMENT OF EMBANKMENT DAMS FOR SEISMIC GROUND MOTIONS

3.1 Effect of Earthquakes on Embankment Dams

Earthquakes impose ground motions which result in additional loads on embankment dams over those experienced under static conditions. The earthquake ground motion is of short duration, cyclic and involves motion in the horizontal and vertical directions. Earthquakes can affect embankment dams by causing any of the following:

(a) Settlement and longitudinal and transverse cracking of the embankment, particularly near the crest of the dam.
(b) Liquefaction or loss of shear strength due to increase in pore pressures induced by the earthquake in the embankment and its foundations.
(c) Instability of the upstream and downstream slopes of the dam if the seismic loading leads to strength loss e.g. from liquefaction, within the embankment or foundation sufficient to result in post-earthquake factors of safety less than 1.0.
(d) Reduction of freeboard due to settlement or instability which may, in the worst case, result in overtopping of the dam.
(e) Differential movement between the embankment, abutments and spillway structures leading to transverse cracks.
(f) Transverse cracking in which internal erosion and piping may develop.
(g) Differential movements on active faults passing through the dam foundation.
(h) Overtopping of the dam by seiches induced in the reservoir in the event of large tectonic movement in the reservoir basin.
(i) Overtopping of the dam by waves due to earthquake induced landslides into the reservoir from the valley sides.
(j) Damage to outlet works passing through the embankment leading to leakage and potential piping erosion of the embankment.

The potential for such problems depend on:

- The seismicity of the area in which the dam is sited and the assessed design earthquake.
- Foundation materials and topographic conditions at the dam site.
- The type and detailed construction, and natural period of the dam.
- The water level in the dam at the time of the earthquake.

The amount of site investigation, design, and additional construction measures (over those needed for static conditions) will depend on these factors, the consequences of failure, and whether the dam is existing or new.

There are four main issues to consider:

- Deformations induced by the earthquake (settlement, cracking) and the effects on dam freeboard.
- The potential for liquefaction or strain softening of saturated or nearly saturated sandy and silty soils and gravels with a sand and silt matrix in the foundation, and possibly in the embankment, and how this affects deformations during the earthquake and stability immediately after the earthquake.
- The zoning and design of the dam, particularly the provision of filters, to prevent or control internal erosion of the dam and the foundation, and provision of zones with good drainage capacity (e.g. free draining rockfill).
- For tailings dams using upstream or centreline construction, the potential for liquefaction of loose to medium dense partly saturated tailings where perched water tables or nearly saturated zones may exist above the measured phreatic surface.

3.2 Defensive Design Principles for Embankment Dams

The general philosophy is to apply logical, common-sense measures to the design of the dam, to take account of the cracking, settlement and displacements which may occur as the result of an earthquake. These measures are at least as important (probably more so) as attempting to calculate accurately the deformations during earthquake. The most important measures which can be taken are:

(a) Provide ample freeboard, above normal operating levels, to allow for settlement or slumping or fault movements which displace the crest. For example, one might adopt a narrow spillway with large flood rise and large freeboard instead of a wide spillway with small flood rise and thus usually a lower freeboard, provided the costs were similar.
(b) Use well designed and constructed filters downstream of the earthfill core (and correctly graded rockfill zones downstream of the face for concrete, asphalt and other membrane faced rockfill dams) to control erosion if the core (or face) is cracked in the earthquake. Filters should be taken up to the dam crest level, so they will be effective in the event of large crest settlements, which are likely to be associated with transverse cracking.
(c) Provide ample drainage zones to allow for discharge of flow through possible cracks in the core. For example provide that at least part of the downstream zone is free draining or that extra discharge capacity is provided in the vertical and horizontal drains for an earthfill dam with such drains. In this regard some embankment dam types are inherently more earthquake resistant than others. In general the following would be in order of decreasing resistance:
    - Concrete face rockfill.
    - Central core earth and rockfill.
    - Sloping upstream core earth and rockfill.
    - Earthfill with chimney and horizontal drains.
    - Zoned earth-earth rockfill.
    - Homogeneous earthfill.
    - Upstream construction tailings and other hydraulic fill.
(d) Avoid, densify, drain (to be non-saturated) or remove potentially liquefiable materials in the foundation or in the embankment. Filters, rockfill and other granular materials in the embankment should be well compacted if they are likely to become saturated, so they will be dilatant and not liquefy.
(e) Avoid founding the dam on the strain weakening clay soils and completely weathered rock, or rock with the potential to strain weaken Post- earthquake stability can be an issue if the earthquake causes even relatively minor movements which can take the foundation strength from peak to residual effective stress strength. Clay soils with high clay size fraction, and mudrocks are potentially strain weakening.

There are a number of other less important measures:

(f) Use a well-graded filter zone upstream of the core to act as a crack stopper, possibly only in the upper part of the dam. The concept is that, in the event that major cracking of the core occurs in an earthquake, this filter material will wash into the cracks, and prevent the core from eroding further by the crack stopper filtering against the downstream rockfill or filter, thereby limiting flow and preventing enlargement of the crack. If well designed filters are provided downstream, the upstream filter is of secondary importance.
(g) Flare the embankment core at abutment contacts, where cracking can be expected, in order to provide longer seepage paths. Just as (or more) important is to consider the detailing of the contact with concrete walls and the provision of filters downstream of the contacts.
(h) Provide special details if there is likelihood of movement along faults or shears in the foundation.
(i) Site the dam on a rock foundation rather than soil foundation (particularly if it is potentially liquefiable or subject to strain weakening), where the option is available.
(j) Use well graded (densely compacted) sand/gravel/fines or highly plastic clay for the core, rather than clay of low plasticity (if the option is available) (Sherard, 1967). The former is more readily filtered by the downstream zones, and the latter more resistant to erosion than clays of lower plasticity.

When assessing an existing dam, the use of these “defensive design” measures is seldom practical (except in remedial works). However, it is useful to gauge the degree of security the existing dam presents by comparing it with this list. Where the dam fails to meet many or most of these features, particularly (a) to (e), this may be a better guide to the fact that the dam may not be very secure against earthquake than a lot of analysis.

Dams which have well designed and constructed filters, have adequate stability against normal loads, and do not have liquefiable or strain weakening zones or foundations, will be likely to withstand the loading from even very large earthquakes with only minor deformations.

3.3 Seismic Deformation Analysis of Embankment Dams

Embankment dam engineers have recognised for many years that when considering the effects of seismic ground motions, it is deformations, not stability which should be assessed. Most embankments will under large seismic ground motions, yield during part of the loading cycle, resulting in permanent deformations. However that does not mean the dam has “failed” provided the deformations are tolerable.

There are a number of methods available for estimating the deformations which may occur in embankments and their foundations during and post seismic loading. These vary considerably in their degree of sophistication and time to do the analyses. In view of this a staged approach to estimating deformations is recommended as follows:

1. Assess whether the embankment or its foundation are susceptible to liquefaction using the methods described in Section 3.4.
2. For embankments and foundations not susceptible to liquefaction or experiencing significant pore pressure build up or strain weakening:
   2.1. Use one or more of the screening or database methods to estimate the deformation. If the estimates of deformations are much less than what is tolerable; e.g. crest settlements are much less than the available freeboard before the earthquake for the seismic load being considered and the deformation are much less than the width of filter, allowing for the uncertainty of the method, accept that the likelihood or failure by overtopping and loss of filter function is negligible for that seismic ground motion.
   2.2. If the deformations estimated by the screening or database methods are greater than this, use one or more of the simplified methods to estimate deformations. If the estimated deformations are significantly less than the available freeboard and filter width, allowing for uncertainty in the method for the seismic load being considered, accept that the likelihood of failure by overtopping and loss of filter function is negligible for that seismic ground motion.
   2.3. If the deformations estimated by the simplified methods potentially threaten the dam with excessive settlement, opening of transverse cracks that present a significant piping risk or shearing across filters for the maximum seismic ground motion being considered, either use risk based methods to assess whether remedial works or for a new dam, design changes to reduce deformations are required; or use an appropriate advanced numerical method to estimate deformations and then use risk based methods to assess whether remedial works are required.
   Repeat this as required for the full range of seismic loading being considered.
3. For embankments and foundations subject to liquefaction and / or to significant pore pressure build up or strain weakening:
   3.1 Assess post-earthquake factors of safety using the limit equilibrium method. If factors of safety for all reasonable lower bound estimates of the liquefied strength and allowing for pore pressure build up and strain weakening in other zones are greater than 1.1 accept that the likelihood of failure will be small and

probably tolerable subject to the quality and extent of information available. This means that for mean estimates of liquefied strength the post-earthquake factor of safety is likely to be > 1.3 or 1.5.

3.2 If factors of safety are lower than described above, estimate deformations using the simplified deformation analysis method allowing for the range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the very approximate nature of these estimates.

3.3 For High and Extreme Consequence Category dams, regardless of the results of (3.1); and for other dams where factors of safety are lower than described in (3.1), carry out static numerical analyses of the post-earthquake condition for a range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the approximate nature of these estimates.

For cases where the estimated likelihood of failure are intolerable, either design remedial works or for a new dam modify the design so the residual likelihoods of failure are tolerable, or use advanced numerical methods to make more refined estimates of deformations. Allow for the range of liquefied strengths and other properties. Use these to assess the likelihood of failure taking account of the still approximate nature of these estimates.

Also consider interim actions if necessary to reduce the risk until remedial works are undertaken.

The extent to which an embankment is analysed for deformations should be consistent with the Consequence Category and size of the dam. In many cases it may be better to design and construct remedial measures or for new dams modify the design than continuing to do more and more sophisticated analyses.

There may however be situations where the more sophisticated methods are warranted. Even for these the deformations are at best approximate, and controlled by the quality of the data input into the analyses, and the limitations of the methods of analysis.

The more sophisticated methods require expert input to the analysis and selection of properties and should not be carried out other than by experienced persons.

3.4 Assessment of the Effects of Liquefaction in Embankment Dams and Their Foundations

3.4.1 Overall Approach

When assessing the likelihood of failure of an embankment dam by loss of freeboard due to liquefaction of the foundation soil, consider:

(i) The likelihood that the soils are susceptible to liquefaction or cyclic softening.
(ii) The probability, given the earthquake seismic ground motion, that liquefaction occurs.
(iii) Given liquefaction occurs, whether the crest settles sufficiently to lose freeboard, taking account of the reservoir level at the time of the earthquake.
(iv) Given freeboard is lost, whether breach occurs.
(v) Do this for both the upstream and downstream slopes of the embankment and for the varying geotechnical conditions which may apply over the length of the embankment and its foundations.
(vi) If deterministic methods are being used for the assessment, it would be common to assume that the reservoir is at full supply level.

This assessment should allow for the modelling of uncertainties in estimating the seismic hazard, the likelihood of liquefaction, the liquefied strength, and deformations which may result. These are by their nature very approximate. The extent to which this is modelled will depend on the Consequence Category of the dam, and whether the outcomes of the assessment are clear cut or marginal. These are more readily modelled if a risk based approach is being followed.

The following Sections provide guidance on matters relating to liquefaction. The wording is somewhat prescriptive for the sake of brevity, but they are not a standard and well qualified and experienced practitioners may choose to adopt alternative methods if that is appropriate for the conditions they are assessing.

3.4.2 Definitions and the Mechanics of Liquefaction

Liquefaction **–** All phenomena giving rise to a reduction in shearing resistance and stiffness, and development of large strains as a result of increase in pore pressure under cyclic or monotonic (static) loading of contractive soils.

Initial liquefaction **–** is the condition when effective stress is momentarily zero during cyclic loading.

Flow liquefaction **–** is the condition where there is a strain weakening response in undrained loading and the in-situ shear stresses are greater than the steady state undrained shear strength.

Temporary liquefaction **–** is the condition where there is a limited strain weakening response in undrained loading; at larger strain the behaviour is strain hardening.

Cyclic liquefaction **–** is a form of temporary liquefaction, where the cyclic loading causes shear stress reversal and an initial liquefaction (zero effective stress) condition develops temporarily.

Cyclic mobility **–** is a form of temporary liquefaction where the shear stresses are always greater than zero.

Cyclic softening **–** as a term used to describe the reduction of shear strength and stiffness of clays and plastic silts under cyclic loading.

It is recommended that all those involved in liquefaction assessments read the documents referenced in the Commentary to familiarise themselves with the mechanics of liquefaction.

3.4.3 Methods for Identifying Soils which are Susceptible to Liquefaction

(a) Geology and age of the deposit.

Soils which are most susceptible to liquefaction are non-plastic or very low plasticity fills including mine tailings and dredged fills, and alluvial, fluvial, marine and deltaic soils. Residual soils are generally likely to have a low susceptibility,

There is some indication that Pleistocene and older soils may be more resistant to liquefaction than Holocene soils but the evidence for this is not sufficient to be relied upon for dam engineering. This is discussed further in Section 3.4.4 and in the Commentary.

The age of the deposit should be allowed for tailings dams and dredged fills as discussed in Section 3.4.4.

(b) Soil gradation, plasticity, moisture content

Use well established methods such as those described in the Commentary to assess whether a soil is potentially liquefiable. The methods should be used with all required data inputs including in-situ moisture content, Atterberg limits, and fines content.

Where Cone Penetration Test (CPT) data is available use also CPT based methods such as described in the Commentary. CPT or CPTU based methods should not be relied upon alone.

Use at least two and preferably more of the well-established methods to assess the likelihood the soils are potentially liquefiable. A suggested approach for doing this is given in the Commentary.

3.4.4 Methods for Assessing Whether Liquefaction may occur

Use well established methods such as those described in Section C3.4 of the Commentary subject to the qualifications detailed in the Commentary.

If a new consensus method or updates of the methods described herein are published in refereed journals, these methods should be adopted.

Take account of the fact that for some of the referred methods the deterministic methods for estimating Cycle Resistance Ratio (CRR) are for 15% probability of liquefaction, that there are significant uncertainties in the factors used within these methods and as a result, soils which plot within the margins of liquefiable and non-liquefiable soils should be assigned some likelihood of being liquefiable.

These uncertainties are greater for soils below about 15 metres from the surface.

Take account also of the uncertainty in the geotechnical model, and variability of the Standard Penetration Test (SPT) and CPT data, and the uncertainty of the earthquake loading.

Where the K_s and K_a values are critical to the assessment of liquefaction, and / or the liquefiable strata are below 15 metres it may be necessary to seek expert advice and use more advanced methods to assess liquefaction.

Do not use a mix of factors from the different methods because the authors rely on their own approach when developing the data bases upon which the methods are developed.

The effects of ageing of the soil deposit may be taken account of as detailed in the Commentary. This should be done for tailings and dredged fills particularly if they are recently deposited.

3.4.5 Assessing the Strength of Liquefied Soils in the Embankment and Foundation

Use both "critical state" and "normalised strength ratio" methods and apply equal weighting to each method.

Do not allow for any increase in the liquefied residual strength beneath a berm which is to be added to improve post-earthquake stability. If a berm has already been constructed carry out SPT and CPT in the liquefiable soils below the berm and use these data to assess the liquefied strength.

Take account of the fact that there are significant uncertainties in these methods, and use strengths between the best estimate and lower bound strengths.

Use methods published in refereed journals such as those referenced in Section C3.4.5 of the Commentary.

3.4.6 Methods for Assessing the Post-Earthquake Strength of Non-Liquefied Soils in the Embankment and Foundation

Allow for the effects of pore pressure build up during cyclic loading in saturated non liquefied non plastic strata, and for cyclic softening of saturated sensitive plastic strata.

Allow for cracking and potential weakening of compacted fills.

Use methods such as those described in the Commentary.

3.4.7 Site Investigations Requirements and Development of Geotechnical Model of the Foundation

The key requirements are:

(a) Review of available data relating to the embankment foundation.
(b) Develop a preliminary geotechnical model and plan any additional site investigations required.
(c) Carry out site investigations using the methods described in the Commentary.
(d) Refine the geotechnical model using all the available data

Details are given in the Commentary.

3.4.8 Liquefaction Analysis

Follow the procedure detailed in the Commentary.

3.4.9 Assessment of the Likelihood and the Effects of Cracking of Embankment Dams Induced by Seismic Ground Motions

When assessing the likelihood of failure for cracking under earthquake loading leading to internal erosion and piping consider:

(i) The probability, given the earthquake, settlement occurs resulting in transverse cracking.
(ii) Given it occurs, whether it will persist to below the reservoir level at the time of the earthquake or before repairs can be carried out.
(iii) Given it does, whether erosion will initiate along the crack, whether filters will prevent erosion continuing, whether erosion progression and a breach forms.

This is seldom a dominant failure mode because "normal" and "flood" load conditions tend to dominate internal erosion and piping failure modes. It should be noted that dams which are susceptible to cross valley differential settlement may not require large earthquakes to produce transverse cracking.

3.5 Methods for Upgrading Embankment Dams for Seismic Ground Motions

3.5.1 General Approach

The method or methods suitable for upgrading a dam and its foundation for seismic ground motions will be site specific, and where applicable will require a thorough understanding of the liquefaction mechanics, extent, and the consequences for the dam. It will also depend on the objectives, probably measured in residual risk terms.

The remedial measures may consist of one or more of the following:

(a) Do nothing and accept the potential damage and risks.
(b) Adding filters and possibly raising the crest level of the embankment so the consequences of deformations and the resulting risks are reduced to tolerable levels.
(c) Modifying the liquefiable soil in the foundation (and the embankment if applicable), and / or constructing stabilizing berms to limit deformations to tolerable levels.

These works may be required only on the downstream of the embankment, or on both upstream and downstream. Quite commonly only part of the length of the embankment may require remedial works; e.g. if only part is founded upon liquefiable soil. It is not uncommon to use a combination of methods; e.g. to carry out treatment on the downstream but do nothing upstream and tolerate the risks posed by upstream deformations. The ground improvement method used will depend on the nature and depth of the liquefiable soils, the techniques available, and cost.

3.5.2 Embankment Dams not Subject to Liquefaction

The following are the most common remedial measures which may be required:

(a) Provision of filters or upgrade of existing filters in the upper part of the dam. This is usually done to reduce risks of internal erosion and piping to tolerable levels for flood and normal loading, and the upgrade for earthquake loads is achieved at the same time.
(b) For the very few dams where freeboard is insufficient, the embankment may have to be raised, usually along with raising for flood.
(c) Where there are strain weakening soils such as high clay content high plasticity over-consolidated clays in which strains may localise under earthquake deformations, it may be necessary to add a stabilizing berm.

3.5.3 Embankment Dams Subject to Liquefaction

The following are the most common remedial measures which may be required:

(a) Construction of a stabilising berm, most commonly founded upon the potentially liquefiable soil after it has been treated by ground improvement. Alternatively remove the potentially liquefiable soil from the foundation of the berm.
(b) Provision of filters or upgrade of existing filters in the upper part of the dam to reduce the risks of internal erosion and piping to tolerable levels.
(c) Where freeboard is insufficient, the embankment may have to be raised.

Section C3.5.3 in the Commentary includes a summary of ground improvement methods, their limitations and some design details.

3.6 Flood Retarding Basins

The assessment of retarding basins under seismic ground motions should be considered similar to the methods described above for embankment dams. It is to be noted that it is likely that a retarding basin will be empty when subjected to seismic loading conditions and therefore the resulting risks associated with the earthquake are likely to be low. However it is important that the condition and function of the retarding basin post-earthquake be considered with regard to the potential filling of the retarding basin following an earthquake.

4 ASSESSMENT OF CONCRETE DAMS FOR SEISMIC GROUND MOTIONS

4.1 Effect of Earthquakes on Concrete Dams

It is important to recognise the different behaviours that gravity, arch and buttress dams exhibit during and after earthquake shaking.

With gravity dams, cracking can occur which is likely to initiate on the upstream face or at the upstream heel, and propagate along the base of the dam or along lift surfaces. This cracking would then produce increased deformations/movement in the dam, allow increased uplift pressures to develop post-earthquake which could be exacerbated by partial or fully blocked drainage due to the deformations and could damage/rupture passive or active ground anchors due to excessive deformations, thereby reducing post-earthquake stability. Gravity dams are most likely to fail post-earthquake by sliding along the foundation interface, through the foundations or along a weak lift surface. Failure can also be caused by over-stressing at a sudden change in geometry in the upper part of the dam that leads to cracking severe enough to cause toppling/sliding of the upper part of the dam.

Arch dams generally have more complex modes of vibration than gravity dams and some over-stressing is more likely to occur above the base of the dam (as well as along the base of the dam). There is likely to be some redistribution of stresses within the dam to the dam's abutments. Failure is likely to occur due to sliding along the foundation interface, through the foundation or along any weak perimetric joint. Failure could also be due to greater horizontal thrusts and shears being transferred to gravity abutment blocks leading to instability in those abutment blocks. Failure could also occur of the abutment rock mass which supports the arch (hoop) forces from the dam.

Buttress dams may comprise a number of structural elements (especially slab and buttress dams). During the course of earthquake shaking, some of these structural elements may progressively fail. In many circumstances however, this will not necessarily lead to failure of the dam – simply to a redistribution of loads/stresses to other parts of the dam. It is important to appreciate this when analysing the dam. Failure may be either structural failure of the various elements of the dam or global sliding.

4.2 Defensive Design Principles for Concrete Dams

The use of defensive design principles for new concrete dams as well as for the upgrading of existing concrete dams is advised.

Defensive design principles to ensure that these criteria are met, should address the dam structure, the interface between the dam and its foundations, and the dam's foundations. In general terms, the dam or the dam's remedial works should be designed / assessed such that:

(a) Sliding during seismic excitation is ideally prevented, although some deformation may be inevitable under large seismic ground motions. Sliding resistance should be sufficient, potentially with reduced shear resistance along the sliding surface, to meet the post-earthquake stability criteria.
(b) The dam is stable against overturning post-earthquake at all levels of earthquake ground motion up to SEE, allowing for cracking and increased uplift pressure in the foundations caused by deformations during the earthquake. It is to be noted that under post-earthquake conditions that the concrete dam may sit in this condition for some time, this should be taken into consideration in the assessment of the outcomes of the analysis.
(c) Any cracking of the dam or in the foundations will not lead to uncontrolled leakage.
(d) Hydraulic outlet structures are not damaged to the extent that they either allow uncontrolled loss of water which might lead to collapse of the dam or

they cannot be used to lower the storage if necessary.

To achieve the above, the following defensive design principles are recommended:

- Proportion the cross section of a dam with no sudden changes in shape or section stiffness.
- Keep any superstructures on the dam crest to a minimum.
- Avoid sudden changes in the abutment profile which would give rise to stress concentrations, or design to accommodate the differential displacements which are likely to occur at these locations.
- Account for geological features in the foundations which would control the dam's sliding stability. If there are such features, then suitable means of stabilising the dam to account for them should be employed. This is especially true of arch dams, since the interaction of dam thrusts and foundation characteristics is determinative of the whole dam-foundation stability.
- Provide sufficient internal and foundation drainage, and see that drain holes are of large enough diameter that they will remain operative particularly if some sliding is expected under seismic ground motions. It is vital that the drains keep working or that the dam is stable post-earthquake with the drains not operative.
- Undertake adequate investigations of the dam’s foundations and of the lift joints to be able to determine the strength parameters of these features.
- If post-tensioned ground anchors are required for stabilising a dam then ensure that they are un-bonded (except for their anchorage length) - this will allow strains due to any overloading resulting from earthquake loading to be taken over the entire free length of the anchor rather than over a very short length either side of a crack in the dam or foundations – strains taken over the entire free length results in much lesser strains and consequently much lesser stresses due to earthquake loading.
- Detail internal features such as galleries so they do not give rise to stress concentrations which will could lead to excessive cracking.

4.3 General Principles of Analysis for Seismic Ground Motions

See Commentary for some general design principles and refer ANCOLD (2013).

4.4 Seismic Structural Analysis of Concrete Dams

4.4.1 Material Properties

4.4.1.1 Concrete

Refer to the ANCOLD (2013) for information on selecting appropriate strength and stiffness parameters for concrete, when analysing concrete dams. Consideration should be given though to any increase in strength or stiffness due to the short term and high frequency nature of earthquake loading. This is discussed in the Commentary.

In assessing existing concrete dams, it will be necessary to not only investigate the strength and stiffness of the concrete but also the strength of lift surfaces.

4.4.1.2 Foundations

The design of concrete dams requires a thorough knowledge of the foundation conditions.

The design process should include a properly funded geotechnical investigation of the rock foundations.

The investigations should provide a detailed geological model of the dam foundations as described in ANCOLD (2013). The model should be used as the basis for estimating the design strength, compressibility, design uplift pressures for the foundations and kinematically feasible failure mechanisms.

The design team should include specialists with considerable experience in dam foundation investigation and design, engineering geology and rock mechanics. The investigation team should develop the geological model and work with the dam engineering design team to estimate foundation design parameters.

The investigations should be done in stages with the development of the dam design feeding back into subsequent stages of the geotechnical investigations.

A critical issue is whether there are continuous, or near continuous, unfavourably oriented discontinuities in the foundation, such as bedding surfaces, bedding surface shears, joints including stress relief (sheet) joints, faults and shears. The geological investigations should assess the foundation for such features and, if present, the design should make allowance for them.

The shear strength of the foundation should be determined as outlined in Section 5.4 of ANCOLD (2013). The compressibility should be estimated as detailed in the Commentary.

4.4.2 Loads

4.4.2.1 Static

Static loads on concrete dams shall be in accordance with ANCOLD (2013).

4.4.2.2 Uplift

The pre-earthquake uplift pressure distribution estimated in accordance with ANCOLD (2013) should be assumed to apply during the earthquake.

The post-earthquake uplift pressure distribution should be determined considering the cracking developed during earthquake shaking. Consideration may be given to the effectiveness of internal/foundation drains following the earthquake. If cracking propagates past the line of drains, the capacity of the drains should be assessed to check adequacy for handling potentially larger discharges due to water travelling along the crack.

4.4.2.3 Silt

Consider any potential liquefaction of silt when estimating silt loads in a post-earthquake analysis.

4.4.2.4 Seismic

4.4.2.4.1 General

In preliminary, simplified studies where only a horizontal response spectrum is supplied, the vertical response spectrum may be generated by scaling the horizontal spectral acceleration using the multipliers given in the Commentary. For more detailed analyses us the methods to apply the ground motions described in Section C2.9.

4.4.2.4.2 Inertia

Consider the mode shape of the dam and the associated natural frequency, to determine the acceleration of the dam at various levels within the height of the dam. Ensure that sufficient modes of vibration contribute to the determination of inertia loads.

4.4.2.4.3 Hydrodynamic

Gravity and Buttress Dams:

Pressure distributions which assume the dam is rigid and the water incompressible may be used for preliminary studies, especially for low height dams. Other methods as described in the Commentary should be applied to more complex dam geometry (e.g. fully or partly sloping upstream face) and for high dams.

Arch Dams:

Review the fundamental frequency of the dam with an empty storage compared to the fundamental resonant frequency of the storage. Where the ratio of the two natural frequencies is near unity, it is necessary to consider the flexibility of the dam and the compressibility of the water in the storage. In this case is will be necessary to consider the storage in any finite element model of the dam (e.g. model the storage (all or part) using fluid elements) in order to determine hydrodynamic pressures on the upstream face of the dam. See the Commentary for further information on calculating hydrodynamic pressures for arch dams.

4.4.2.5 Dynamic Earth Pressures

Dynamic earth pressures from silt, fill or abutments should be calculated based on geotechnical principles as discussed in the

Commentary for Section 6.6.4. Pseudo-static or pseudo-dynamic procedures will be sufficient in most cases.

4.4.2.6 Load combinations

Refer to ANCOLD (2013) for applicable load combinations.

Consideration should be given to the behaviour of a concrete dam during an earthquake when the dam is empty or near empty. This is likely to be important in gravity dams which have been strengthened using post tensioned anchors (which are likely to be located near the dam's upstream face). Cracking propagating from the downstream toe of the dam could prejudice the stability of the dam during subsequent flood loading of the dam.

As noted in ANCOLD (2013), consideration also needs to be given to the possible loss of stabilising force from damaged post-tensioned cables by shearing due to sliding displacement or tension overload.

For arch dams especially, temperature effects can have a significant effect on the stress distribution within the dam. However, in most arch dams built with vertical contraction joints, the contraction joints tended to be grouted during winter. Therefore winter represents the temperature stress neutral condition. In summer when the dam deflects upstream due to concrete expansion, the earthquake stresses induced in the dam are likely to be less than in winter. However, each dam should be considered on its merits and at least a qualitative assessment made of how the dam is likely to perform during earthquake loading in combination with different temperature conditions.

4.4.3 Methods Available and When to Use Them

Analysis will be either in the frequency domain where an earthquake response spectrum is used, or in the time domain where time-histories of velocity, acceleration or displacement are used.

For analyses in the frequency domain, obtain a response spectrum for the site (in the case of concrete dams, attenuation or amplification of accelerations through overburden is not likely to be an issue as it may be for embankment dams founded on soil) and for the relevant damping factor and design earthquake peak ground acceleration.

For analyses in the time domain, obtain a minimum of 3 sets (preferably 5 sets) of time-histories (two orthogonal horizontal time-histories and one vertical time-history in each set).

For analyses in the time domain either direct integration or modal superposition methods may be used to determine stresses in the dam. The former is the more accurate but requires considerably greater computer resources (computer memory and run time). The latter will be more conservative as it uses a statistical approach (e.g. square root, sum of the squares) to combine the maximum stresses from the various modes. Consequently, it ignores the sign of the maximum stress for a particular mode.

Undertake structural seismic analysis in a hierarchical manner. That is, analysis should start using simplified linear elastic methods and then progress as required, to more sophisticated methods as given in Table 4-1. The Commentary gives more details on applicability of the methods.

The extent to which a concrete dam is analysed should be consistent with the Consequence Category and size of the dam. In some cases it may be better to design and construct remedial measures than continuing to do more and more sophisticated analyses (e.g. non-linear analysis). There may however be situations where the more sophisticated methods are warranted. The more sophisticated analysis methods should not be carried out other than by experienced persons.

Table 4-1 Hierarchy of Seismic Analysis Methods for Concrete Dams

i.	Simplified, linear elastic methods using a site specific response spectrum on a 2D model of the dam for pseudo-static cantilever type stability analysis – generally only applicable to a concrete gravity dam.
ii.	Simplified, linear elastic methods using a site specific response spectrum on a 2D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam or for the upstream/downstream direction of a concrete buttress dam.
iii.	Linear elastic methods using a site specific response spectrum on a 2D or 3D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam (2D satisfactory for flat sloped abutments or buttress dams considering only upstream/downstream direction); 3D required for steep abutments), and for arch or buttress dams (considering full 3D action).
iv.	Simplified, linear elastic methods using site specific time-history records of ground acceleration or velocity on a 2D or 3D finite element model (FEM) of the dam and foundations – applicable to a concrete gravity dam (2D satisfactory for flat sloped abutments or buttress dams considering only upstream/downstream direction); 3D required for steep abutments, and for arch or buttress dams (considering full 3D action).
v.	Non-linear methods using site specific time-history records of ground acceleration or velocity on a 2D finite element model (FEM) of the dam and foundations so that cracking can be simulated – applicable to a concrete gravity dam or for the upstream/downstream direction of a concrete buttress dam.
vi.	Non-linear methods using site specific time-history records of ground acceleration or velocity on a 3D finite element model (FEM) of the dam and foundations so that cracking can be simulated – applicable to a concrete gravity, an arch dam or for a concrete buttress dam.

The level of complexity of the analysis may be linked to the dam's Consequence Category as indicated in Table 4-2.

Table 4-2 Analysis Method Appropriate for Consequence Category

Analysis Methods	**Consequence Category**		
	Low	**Significant**	**High and Extreme**
2D linear elastic, simplified response spectrum, cantilever analysis - gravity dam only ((i) in Table 4-1)	Yes	Yes	Yes (1)
2D linear elastic FEA, site specific response spectrum-gravity dam & buttress dam in u/s-d/s direction ((ii) in Table 4-1)		Yes	Yes
2D or 3D linear elastic FEA, site specific response spectrum- all concrete dams ((iii) in Table 4-1)			Yes
2D or 3D linear elastic FEA, site specific accelerograms- all concrete dams ((iv) in Table 4-1)			Yes
2D non-linear FEA, site specific accelerograms - gravity dam & buttress dam in u/s-d/s direction ((v) in Table 4-1)			Yes
3D non-linear FEA, site specific accelerograms – all concrete dams ((vi) in Table 4-1)			Yes

Notes

(1) Consideration needs to be given by the dam Owner in consultation with the Consultant as to whether a 2D analysis is considered sufficient for a High and Extreme Consequence Category dam. In most cases a higher level of analysis would be required.

4.4.4 Analysis in the Frequency Domain

See Commentary for details.

4.4.5 Analysis in the Time Domain

See Commentary for details.

4.5 Approach to Analyses and Acceptance Criteria

4.5.1 During the Earthquake

4.5.1.1 Gravity Dams:

The following are the steps in the analyses and acceptance criteria. The extent of analyses will depend on the Consequence Category of the dam, and the purpose of the analysis; e.g. concept versus detailed design. For a detailed design or assessment of a High or Extreme Consequence Category dam linear elastic time-history or non-linear -history time analyses should be carried out. It is likely that a response spectrum analysis will give overly conservative results. This will be important if the response spectrum analysis gives an extent of cracking during earthquake shaking that would lead to inadequate post-earthquake stability.

1. Carry out pseudo-static analysis using 5% damping factor. If factors of safety are ≥ minimum required by ANCOLD (2013) then behaviour during earthquake loading is satisfactory.
2. If factors of safety are < required by ANCOLD (2013) carry out linear elastic response spectrum analysis or linear elastic time-history analysis. If stresses using 5% damping factor indicates stresses ≤ dynamic strengths, then behaviour during earthquake loading is satisfactory.
3. If (2) indicates stresses > dynamic strengths, then carry out analysis with response spectrum modified for 10% damping factor. If stresses > dynamic strengths then undertake linear elastic time-history analysis to estimate Demand Capacity Ratio (DCR) and cumulative time for non-linear behaviour. Assess according to relevant USACE manual (e.g. USACE EM 1110-2-6053).
4. If (3) indicates that the linear elastic analyses undertaken indicate unacceptable DCR, carry out a non-linear time-history analysis to estimate the extent of sliding/rocking that the dam undergoes. Assess this against the estimated maximum sliding/rocking that the dam can undergo. This assessment should consider excessive permanent leakage that may occur through the dam or its foundations, and the effect on internal drains and post tensioned anchors.

4.5.1.2 Arch Dams:

Carry out a 3D linear elastic finite element analysis (FEA) of the dam and its foundations using response spectra, in the first instance. Go to more sophisticated 3D linear elastic FEA using time-histories and 3D non-linear FEA using time-histories as might be dictated by the results of the less sophisticated analyses and the dam's (and foundation's) geometry/properties. See Commentary for details.

4.5.1.3 Buttress Dams:

The requirements are that for both upstream/downstream and cross valley ground motions:

- The structure has satisfactory factors of safety against sliding and overturning as required for concrete gravity dams or that permanent deformations are tolerable.
- The face slabs are not dislodged and satisfy normal structural reinforced concrete requirements;
- The buttresses do not fail in buckling when assessed using normal reinforced concrete design principles;
- Sufficient struts remain intact such that the strength of the buttresses is not compromised.

4.5.2 Post-Earthquake

4.5.2.1 Gravity Dams:

Regardless of the method of analysis used to analyse the dam during earthquake shaking, carry out a post-earthquake stability analysis. This analysis should consider damage to the dam occurring during the ground motion (e.g. disruption of drains, failure of post tensioned anchors) and changes in uplift pressure distribution due to cracking caused during earthquake ground motions. Reduced or residual shear strength parameters due to strain weakening should be used where appropriate within the foundations and at lift joints. The minimum post-earthquake factor of safety for sliding should satisfy ANCOLD (2013) Table 6.1 and 6.2 criteria.

4.5.2.2 Arch Dams

Consider extent of cracking at base of dam caused by earthquake shaking in determining redistribution of stresses in the dam and in determining post-earthquake sliding resistance.

Consider extent of cracking in both the arch and any gravity abutment sections in considering the change of loading on the abutment sections.

Use residual shear strength parameters within the dam and the foundation where appropriate according to the requirements of ANCOLD (2013).

4.5.2.3 Buttress Dams

Carry out post-earthquake analysis following the principles for gravity dams as discussed in Section 4.5.2.1.

The minimum factor of safety for sliding and overturning should satisfy the requirements of ANCOLD (2013) for gravity dams.

4.5.3 General

See Commentary.

5 ASSESSMENT OF TAILINGS DAMS FOR SEISMIC GROUND MOTIONS

5.1 Some General Principles

This Section of the Guideline supplements the ANCOLD Guideline on Tailings Dams (ANCOLD, 2012a). Those involved with tailings dams should refer to that Guideline.

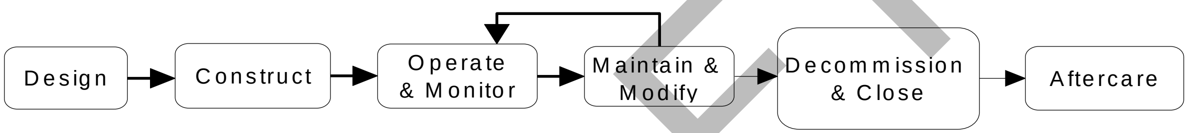
Where there are conflicting data in the two Guidelines this Guideline represents ANCOLD's position.

Where there are additional State regulations for tailings dams these regulations must be complied with.

Tailings dams and their potential impacts must be effectively managed throughout their life, from design through operation to closure and post closure. The post closure period may be very long; e.g. ANCOLD (2012a) suggest 1000 years.

The flow diagram below illustrates a cyclical return step during operation indicating possible modifications to ensure that the dam achieves its ultimate goal - sustainable closure with minimal post closure risks and associated costs.

These considerations can impact on the seismic design requirements of tailings dams as described below.



Tailings dams may be designed and constructed by a number of methods (ANCOLD, 2012a):

- *Downstream*, usually incorporating a starter dam.
- *Centreline*, where part of the embankment is built over the tailings beach.
- *Upstream*, where most of the embankment is built over the tailings beach.

A combination of these methods may be used; e.g. downstream construction for the majority of the dam, with small raises using upstream or centreline construction.

If downstream construction methods are used there are few if any differences between such dams and conventional water retaining dams from the seismic design viewpoint.

However, if centreline or upstream construction is used there are a number of the methods described in this Guideline for conventional dams which are not applicable. These are discussed below.

There is also a greater susceptibility to liquefaction and strain weakening leading to large deformations and potentially slope instability which needs to be taken into account.

5.2 The Effects of Earthquakes on Tailings Dams

For downstream construction these are as for conventional embankment dams as detailed in Section 3.1. If the water storage is kept remote from the dam by good water management practices the implications of cracking of the dam potentially leading to internal erosion and piping may be less than for a conventional dam.

For tailings dams constructed by centreline and upstream construction methods there will be a greater emphasis on liquefaction and strain weakening, and potential for large deformations, internal erosion and piping in cracks resulting from deformations and slope instability.

5.3 Defensive Design Principles for Tailings Dams

These are as for conventional embankment dams as detailed in Section 3.2. For tailings dams constructed by centreline and upstream construction methods there will be a greater emphasis on the potential for liquefaction and strain weakening and slope instability.

Particular care and conservatism is required if upstream construction methods are proposed for High and Extreme Consequence Category tailings dams if they are constructed of tailings materials which are potentially liquefiable. This is due to the difficulty of maintaining good construction practices throughout the life of the dam, and the uncertainty of predicting the liquefied strength of the tailings and post-earthquake deformations. If upstream construction methods are used it should only be by involving Consultants expert in these matters, using conservative assumptions and requiring confirmation of assumed parameters during construction.

Upstream construction may reasonably be used for small raises of tailings dams constructed by downstream methods where it can be demonstrated with a high degree of confidence that the potential effects of liquefaction and / or strain weakening of the tailings can be accommodated with a large margin of safety.

Where upstream construction is used, this must recognise that such dams are not suitable for water storage other than the minimum amount required for achieving decant water quality. Water storage limitations should be set, and stringently adhered to during operation and for the life of the dam, taking into account the lack of filters for this type of dam and the potential deformation, which should ensure no contact of stored water to the perimeter embankment.

Where centreline undertaken, specific defensive design principles include:

(a) Use conservative design methods including estimates of the extent of liquefaction and residual strength parameters for stability analysis;

(b) Include buttresses constructed of high strength, non-liquefiable materials as part of the outer shell design;

(c) Avoid water storage on the dam surface;

(d) Where feasible use “integrated waste management” principals to construct conservative embankments from mine waste rock;

(e) Provide internal drainage systems to lower phreatic surface and reduce the level of saturation of tailings;

(f) Maximise the density and reduce saturation by well-planned and managed tailings discharge aimed at minimising thickness of fresh tailings discharge layers and maximising evaporation and drying shrinkage;

(g) Maximise the density and reduce saturation by mechanical working of the tailings using purpose built equipment such as Amphirolls (Archimedes Screw Tractors):– note that this method has been able to successfully compact tailings to non-liquefiable conditions equivalent to “dry-stacking”;

(h) Use dry-stacking methodology including filter-press technology to dewater tailings and potentially allow placement and compaction of tailings to non-liquefiable density or saturation levels.

5.4 Design Seismic Ground Motions and Analysis Method

Design seismic ground motions for tailings dams during operation are as for conventional dams and are given in Table 3.2. The Consequence Category should be determined as detailed in ANCOLD (2012), Table 1. It should be noted that for tailings dams the impact on the natural environment may be a controlling factor.

The design seismic loads for tailings dams post closure should be the SEE. The SEE should be re-evaluated at the time of closure to allow for any development in understanding of seismic hazard and for any changes in the consequences of failure, and that allowance should be made for potential increases in the PAR and hence consequences post closure.

5.5 Seismic Deformation Analysis of Tailings Dams not Subject to Liquefaction

These are as for conventional embankment dams as detailed in Section 3.3.

For tailings dams constructed by centreline and upstream construction methods there will be a greater emphasis on liquefaction and strain weakening. None of the “screening”, “database” or “simplified” methods are applicable to tailings dams where liquefaction or other significant strain weakening may occur. Deformations may be estimated by the methods described for static and advanced numerical deformation analyses as described in Sections C3.3.4.3 and C3.3.4.4 of the Commentary.

5.6 Assessment of the Effects of Liquefaction in Tailings Dams and Their Foundations

These are as for conventional embankment dams as detailed in Section 3.4.

Allow for the effects of age of the tailings deposit as described in Section 3.4.3 and Section C3.4.3 in the Commentary.

5.7 Methods for Upgrading Tailings Dams for Seismic Loads

These are as for conventional embankment dams as detailed in Section 3.5.

6 ASSESSMENT OF APPURTENANT STRUCTURES FOR SEISMIC GROUND MOTIONS

6.1 The Effects of Earthquakes on Appurtenant Structures

A number of subsidiary structures associated with a dam are essential for the dams operation. For these structures it is important that their functional and structural integrity is retained in the event of a notable earthquake. This is particularly the case where the appurtenant structure may be required to release water from the reservoir in a controlled manner to lower the storage following an earthquake. It is therefore necessary that not only the intake/outlet structures and their gates and valves remain serviceable but also that there is proper access to these structures. Bridges and roads may need to remain in a sound state after an earthquake depending upon their importance.

The most important factors in considering the earthquake resistant design of an appurtenant structure are:

- Whether or not failure of such a structure could lead to loss of control of the reservoir following an earthquake,
- Where the structure is required for post-earthquake operation to lower or maintain the reservoir level so that the structure maintains its functionality or allow repairs.

Generally, appurtenant structures should be such that:

- They maintain their normal operating condition after an OBE
- They are not damaged to an extent where they could allow sudden or uncontrolled loss of water from the storage for ground motions up to the SEE.
- Following ground motions up to the SEE, appurtenant structures should operable to an extent that the dam is able to pass floods while repairs are carried out.

The level of assessment and design ground motions of critical appurtenant structures is subject to the risk assessment and operational requirements of the component and should be determined by the Owner and Consultant. It may be required for appurtenant structures that are critical to the operation and safety of the dam following an earthquake, e.g. low level outlet works that have the ability to draw down the reservoir, that these structures maintain function post-earthquake loading conditions with a high degree of confidence. To achieve this may require them to be designed for the SEE.

When designing and assessing appurtenant structures it is important to note that these are hydraulic structures and the assessment needs to be conducted accordingly. Australian Standards such as AS1170 (Structural Design Actions) are typically not appropriate for these structures.

When assessing appurtenant structures consideration also needs to be given to other external factors that could influence the operation of the structure such as rock fall impacts, restricted access for operation, loss of power, etc.

This following sections provide the information for the various types of appurtenant structures including:

- Defensive design principles;
- Performance criteria for the Operating Basis Earthquake (OBE) and the Safety Evaluation Earthquake (SEE); and
- Recommended analysis procedures.

6.2 Intake Towers

6.2.1 General

Dam intake/outlet facilities typically comprise of the intake/outlet structure, intake/outlet tunnel, exit structure, access bridge (at times) and operating equipment including pipework, valves, generators, electrical control panels, etc.

Many intake/outlet towers, termed intake towers from herein, are either free standing on an enlarged base or foundation mat placed on the reservoir bottom, or are deeply founded through bedrock or soil, away from the dam. Others are embedded within earthfill dams, or structurally tied to the upstream face of concrete dams. Towers built within embankment dams may interact dynamically with the surrounding materials. Reservoir water surrounds most towers, sometimes up to a significant height. Hence they are subjected to hydrodynamic interaction effects. Some contain inside water, which also affects seismic response.

6.2.2 Defensive Design Principles

The following defensive design principles are suggested for new structures and upgrades to existing structures:

- Site the intake tower on a foundation where there are no geological features that could lead to uneven displacement or deformations during the earthquake event.
- Where possible, the intake tower should be socketed into rock to assist with preventing the tower from sliding.
- Where possible, install grouted dowels in the rock foundation as a redundancy against uplift and rocking.
- Avoid so far as is practicable sudden changes in profile that could give rise to stress concentrations.
- Provide horizontal reinforcement designed to prevent vertical reinforcement from buckling and to confine concrete when it is in compression. The horizontal reinforcement should be placed on the outside of the vertical reinforcement.
- In the design of towers, practices should be adopted that ensure ductile behaviour while suppressing brittle failure modes including: meet minimum reinforcing steel requirements such that the nominal strength moment is equal to 1.2 times that cracking moment; provide adequate confinement at splice locations and plastic hinge regions: provide anti-bucking hoops/ties in plastic regions; provide adequate splice and anchor lengths; avoid locating splices in inelastic regions; and provide direct and continuous loads paths.
- For access bridges to intake towers provide appropriate support mechanisms designed to prevent damage to both the bridge and the intake tower due to deflections occurring during the seismic event.

6.2.3 Performance Requirements

OBE: Static and dynamic loads to induce maximum concrete and steel reinforcement stresses within the elastic region (i.e. limited amount of reinforcement yielding) for strength design purposes and the tower and its base is to remain stable.

SEE: Significant amount of reinforcement can yield and damage to the tower may occur. The tower and its base and should not be damaged to an extent that it leads to an uncontrolled release of water from the reservoir. Intake towers required for the emergency release of water following an earthquake event should remain functional.

6.2.4 Analysis Procedures

The analysis methods for intake towers are described here in general terms only. It is recommended that the reader refer to ICOLD (2002), USACE (2003b), USACE (2007) and the Commentary for additional detailed information on the analysis procedures. The more sophisticated methods require expert input to the analysis and should not be carried out other than by experienced persons.

The general issues and potential modes of failure that need to be examined in the seismic response of intake towers include the following:

- Flexural displacement demands exceeding the flexural displacement capacity;
- Shear demands exceeding shear (diagonal tension) capacity;
- Shear demands exceeding the sliding shear capacity; and
- Moment demands exceeding the overturning capacity (rocking).

There are a number of methods available for the assessment of intake towers, these are provided below ranging in increasing level of complexity. It is to be noted that care is to be taken when selecting models with increasing levels of complexity and is to be based on the judgement of experienced personnel. This is particularly the case with non-linear analysis where the assessment can often be complex and time consuming. In this instance it needs to be considered whether remedial measures would be more beneficial than further analysis.

Table 6-1: Hierarchy of Seismic Analysis Methods for Intake Towers

Strength Assessment	
i.	Response Spectrum: The response spectrum analysis is adequate for towers whose responses to earthquakes are within the linear elastic range. If it is determined that the tower remains within its elastic range under the OBE and meets the ductility requirements for the SEE, then the tower is considered acceptable.
ii.	Linear Time-History Analysis: Applicable if tower exceeds requirements of the Response Spectrum Analysis. If the demand capacity ratios, cumulative duration of bending moment excursions and extent of reinforcement yielding meet those described in the Commentary then the tower is considered acceptable.
iii.	Non Linear Analysis: The nonlinear analysis of towers can be complicated and time consuming. In this instance it needs to be considered whether remedial measures would be more beneficial than further analysis.
Stability Assessment - Sliding	
i.	Response Spectrum: The OBE is considered an unusual condition and a factor of safety of 1.3 is required. The SEE is considered an extreme condition and a factor of safety of 1.1 is required.
ii.	Linear Time-History Analysis: The tower should meet the requirements described in Section 6.2.3 for the OBE and SEE. Within this analysis, the stability is maintained and sliding does not occur if the factor of safety is greater than 1.
iii.	Non Linear Analysis: In the nonlinear time-history analysis an assessment can be made of the total permanent sliding displacement of the tower. It then needs to be assessed whether the permanent displacements meet the criteria for the SEE described in Section 6.2.3.
Stability Assessment – Rotational	
i.	Tipping Potential Evaluation: The tower may start to tip and start rocking during an earthquake. The assessment of the tipping potential for the tower, either a rigid or flexible tower, is provided in USACE (2007). If it is determined that no tipping occurs, ie. the structure does not break contact with the ground, then the tower is considered stable. If tipping occurs then the rocking block analysis should be conducted.
ii.	Rocking Block Analysis: USACE (2007) and the Commentary provide the procedures for assessment of the tower rocking. The tower should meet the requirements described in Section 6.2.3 for the OBE and SEE.

Note. For sliding in the foundation the principles of selection of the strength of the foundation and factors of safety should be as detailed in ANCOLD (2013).

6.3 Spillways

6.3.1 General

Typically, spillways are constructed of mass or reinforced concrete. Seismic loads often control the design of such structures. Spillway component structures can be grouped into three general classes, including inlet structures (inlet and/or crest structures including gates), chutes (conveyance structures such as floor slab with walls connecting the inlet structures to the terminal structure), and the terminal structure (hydraulic-jump, stilling basin, flip bucket, impact structure, etc.). Each of these spillway components is covered in the following section.

6.3.2 Defensive Design Principles

For defensive design principles for mass and structurally reinforced components of spillways, refer to Section 4.2.

6.3.3 Performance Requirements

OBE: Spillways should maintain their normal operating condition after an OBE.

SEE: Spillways, including gates that retain permanent storage at the time of an SEE should not fail to an extent where water is released in an uncontrolled manner. Following ground motions up to the SEE spillways should be operable to an extent that the dam is able to pass floods while repairs are carried out.

6.3.4 Analysis Procedures

General

Any large mass spillway structures that retain the reservoir should be designed in accordance with Section 4 – Concrete Dams. For all other spillway structures, Table 6-2, taken from ICOLD (2002) with some additions, provides some guidance on the recommended analysis methods for the components of spillway structure.

Table 6-2: Analysis method of spillway component structures

Spillway Structures	Components	Usual Approaches	Recommended models
Inlet and crest structures	Morning glory drop inlet structures	Response spectrum Linear time-history	3D
	Overflow structures: Straight ogee crests, Labyrinth, Fuse gates	Pseudo-static Using elastic foundation	2D plane strain of plain stress or 3D
	Siphon structures	Response spectrum	2D or 3D
	Fuse plug structures: zoned embankment	Deformation analysis Newmark method or Liquefaction potential (refer Section 3)	2D
Chutes	Conveyance structures: Floor slab and connecting walls	Pseudo-static Using elastic foundation	2D plane-strain or plane-stress
Terminal Structures	Hydraulic jump Stilling basin Flip bucket Impact structures	Pseudo-static Using elastic foundation	2D

Additional inertia loading from operational equipment, piers, etc. needs to be considered in the assessment of spillway structures. For spillway piers, the effect of the bridges and operating equipment needs to be considered in the assessment of the piers.

6.4 Spillway Gates

6.4.1 General

Spillways often incorporate gate systems that retain permanent storage and / or fulfil dam safety functions. These gates can, at times, be used to allow the controlled release of water in a potential dam safety emergency, including post earthquake.

Spillway gate types typically include crest mounted radial gates, orifice radial gates, vertical lift wheel gates and flap gates.

6.4.2 Defensive Design Principles

The following defensive design measures are recommended:

- Provide sufficient flexibility and details in the gate system to accommodate expected differential movements during an earthquake event.
- Evaluate the reliability of the operating system, including appropriate access, for operation post earthquake, refer to Sections 6.8 and 6.9 for additional details.
- The gate drive systems should not fail or distort; such as drive shafts and hydraulic cylinders.

6.4.3 Performance Requirements

OBE: Spillways gates and their operating gear should maintain their normal operating capability after an OBE.

SEE: Spillways gates that retain permanent storage at the time of an SEE should not fail to an extent where water is released in an uncontrolled manner. The SEE should not cause gates to distort or gate piers to permanently displace to an extent where the gates become jammed. The hoist system or a backup arrangement should remain functional.

6.4.4 Analysis Procedures

If spillway gates are located on the top of a concrete dam, the spillway gates will need to be analysed as an integral part of the dam. Amplification of the ground accelerations could be significant (in some cases there may be a two – to fivefold increase in magnitude).

Seismic loads transmitted from spillway gates to trunnion pins and trunnion blocks should also be accounted for when designing or assessing a gated crest structure. These loads can be significant due to their concentrated nature.

Hydrodynamic loads are typically assessed as described in Section 4.4.2.4.3, and applied as added masses to a model. When using this approach, the following items need to be considered as they can impact on the hydrodynamic loads applied to the gates:

- The depth of the water against the gate;
- The depth of water against the full spillway structure;
- The fundamental frequency of the gate compared to the fundamental resonant frequency of the storage; and
- The position of the gate relative to the upstream face of the spillway.

With the development of numerical modelling, the use of fluid or acoustic elements within a three dimensional model is now possible and can be adopted, particularly for structures with a complex geometry. For further details on seismic induced loads on spillway gates refer to USBR (2011).

The complete gate system should be considered, from incoming power supply, electrical components, backup supplies, hoist design and gate design, emergency bulkheads and cranes, operator access.

The effect of dynamic amplification on critical components (eg hydraulic cylinders) may need to be considered.

6.5 Outlet Works - Water Conduits, Gates and Valves

6.5.1 General

Water conduits such as pipelines, penstocks, tunnels and low-level outlets may be required for reliable, controlled, rapid emptying of the reservoir. Water conduit design should be such that it does not lead to failure or compromise the functioning of the dam and its foundation. In addition, in the case of water supply reservoirs in populated areas, the safety and operability of the outlet pipelines, gates and valves become significant factors affecting the maintenance of drinking water supplies as well as water to fight-fires and assisting with post-earthquake recovery functions.

6.5.2 Defensive Design Principles

The following defensive design measures are recommended:

- Provide sufficient flexibility in the outlet conduit system to accommodate expected differential movements during an earthquake event.
- Weak zones, faults and active fault crossings should be avoided where possible for outlet conduits. If such areas cannot be avoided, then design details should be utilised which can accommodate displacement and differential movements.
- Tunnel plugs can be incorporated as part of the conduit design to prevent uncontrolled releases of water should damage or conduit failure occur.
- Where possible, pipelines or penstocks, including supports and anchor blocks, should be founded on suitable rock, soil, or stabilised soil which is capable of minimising differential movement and settlement due to seismic ground motion.
- Pipelines or penstocks should not be founded on low-density, non-plastic soils that are subject to liquefaction or high levels of strain that could cause damage even during a moderate earthquake.
- Consider the differential loadings that can occur on conduits which pass through different zones in an embankment.
- Consider rock falls that could occur as a result of an earthquake and potentially damage the pipeline or penstock.
- For pressurised systems consideration needs to be given to the hydrodynamic forces that can be generated during an earthquake, refer to the Commentary.
- Ensure the gate actuation systems can withstand vibration. Hydraulic cylinders are vulnerable to vibration. Rope supported gates may be susceptible to vibration under certain conditions and position.

6.5.3 Performance Requirements

Outlet Conduits

OBE: Static and dynamic loads to induce concrete and steel reinforcement stresses which satisfy AS3600 Concrete Structures.

SEE: Conduit not to collapse or rupture. Collapse could lead to an undermining and subsequent failure of the embankment. Rupture could cause piping or destabilise an embankment by a marked increase in pore pressure.

Pipelines and Penstocks

OBE: Static and dynamic loads to induce steel stresses which satisfy AS4100 Steel Structures.

SEE: Pipelines and penstocks required for emergency releases or post-earthquake recovery not to collapse or rupture.

Gates and Valves

OBE: All gates and valves to maintain their normal operating capabilities.

SEE: Emergency closure and regulating gates and valves (especially low level release valves) to maintain operating capability – the storage may need to be quickly lowered if parts of the dam are damaged and need remedial works or relief of hydrostatic loads.

6.5.4 Analysis Procedures

The complete outlet system should be considered, including power supply, electrical components, backup supplies, valve and gate design and operation, emergency bulkheads and cranes, operator access.

Outlet Conduits

Use methods detailed in the Commentary. A 2D analysis is usually sufficient for the seismic assessment of an outlet conduit.

Pipelines and Penstocks

Consideration needs to be given to the potential of rocking of pipelines and penstocks within outlet conduits and the potential amplification effects that can occur.

6.6 Retaining Walls

6.6.1 General

Retaining walls are often critical components to a spillway or dam structure. They include gravity, semi gravity and non-gravity walls with seismic loads often controlling the design of such structures. During an earthquake, a retaining wall can be subjected to dynamic soil pressures caused by motions of the ground and the wall that need to be accounted for in the design and assessment of the wall. For retaining walls that retain embankments, consideration needs to be given to the performance of the backfill material as this has the potential to lead to piping or large deformations, particular consideration needs to be given to backfill materials that are potentially liquefiable.

6.6.2 Defensive Design Principles

The following defensive design measures are recommended:

- Avoid sudden changes in the retaining wall profile which would give rise to stress concentrations;
- Site where there are no geological features in the foundations which would decrease the retaining walls sliding stability - if there are, then suitable means of stabilising these features or accounting for them in the design should be employed.
- Provide sufficient drainage provided behind the retaining wall. It is vital that the drains keep working or that the retaining wall is stable post-earthquake with the drains not operative;
- If post-tensioned ground anchors are required for stabilising a retaining wall design so that they are un-bonded (except for their anchorage length), refer to Section 4 for additional details on anchors;
- For new structures and upgrades design and specify the backfill to the retaining wall so that the backfill materials are not liquefiable as this can lead to significant deformations of the backfill material, increasing the potential for opening a gap between the wall and backfill resulting in piping and increasing the loads on the retaining wall.
- For retaining walls that retain embankments, the wall be sloped on the embankment side to maintain positive contact with the embankment following deformations that may occur due to the earthquake.

6.6.3 Performance Requirements

OBE: Retaining walls should maintain their normal operating condition after an OBE. Static and dynamic loads to induce maximum concrete and steel reinforcement stresses which are within the elastic region.

SEE: Walls which retain part of an embankment dam shall not collapse or deform to an extent that could lead to embankment failure due to piping or breach due to significant deformations.

6.6.4 Analysis Procedures

Gravity retaining wall structures which if they fail could result in breach of the dam should be designed in accordance with Section 4 – Concrete Dams. Cantilever retaining walls which if they fail could result in breach of the dam should be designed to the same principles but using design methods appropriate for such walls as described in the Commentary.

The dynamic loads applied from the backfill material need to be included. Depending on the magnitude of wall movements the backfill material is said to be in yielding, non-yielding, or intermediate state. Further information on the assessment of the backfill loads is provided in USACE (2007) and in the Commentary.

6.7 Parapet Walls

6.7.1 General

Parapet walls are often used to increase the height of embankment dams and are generally constructed using either cast-in-situ or precast concrete components. These walls are above the full supply level of the dam and do not retain permanent storage.

6.7.2 Defensive Design Principles

- Provide sufficient flexibility in parapet wall joint details to allow for deformations due to the earthquake event.
- Consider the potential for internal erosion and piping underneath the walls and if required install suitable filter zones.

6.7.3 Performance Requirements

OBE: Parapet walls should maintain their normal operating condition after an OBE.

SEE: Parapet walls can be expected to deform and be damaged in a SEE. The Owner and Consultant should consider the potential for damage, how this may be minimised and details provided so that the dam will satisfy tolerable risk criteria resulting from floods following the earthquake and before the parapet wall can be repaired.

6.7.4 Analysis Procedures

Refer to the Section 6.6: Retaining Walls for analysis procedures. Consideration of the performance and deformation of the parapet wall needs to be given as part of the earthquake assessment of the underlying dam structure. Consider the amplification effects that can occur at the top of the dam.

6.8 Mechanical and Electrical Equipment

6.8.1 General

For gates and outlet works to remain functional following an earthquake event any mechanical and electrical equipment required for the operation of the gates and valves also need to remain operational. These items in existing dams are often not designed for seismic ground motions and can sometimes be fragile and may be subjected to accelerations many times greater than the peak ground acceleration due to amplification of the ground motion. Depending on its resonant frequency, equipment may be susceptible to dynamic amplification resulting in high stresses. Some 'off the shelf' equipment may not be suitable for seismic loading and vibration.

6.8.2 Defensive Design Principles

The following defensive design measures are recommended:

- Anchor mechanical and electrical components to maintain their stability during earthquake events and their operability following the event. This is typically relatively inexpensive to incorporate into new and existing structures.
- The reliability of the power supply should be evaluated. Consideration should be given to alternative power supplies for critical components. Consider the possibility of landslides and structural movement on cable runs and hydraulic pipework. Control buildings should be designed to withstand earthquake loading. Consider fuel tanks, switchboards, transformers, cable trays and support of starter batteries.
- Critical crane equipment should be appropriately anchored to avoid misalignment. Especially applicable to cranes supporting emergency bulkheads or required for emergency remedial works.
- Hoist systems (shafts, hydraulic cylinders and pipework etc.) should be appropriately designed.

6.8.3 Performance Requirements

All mechanical and electrical equipment and components should meet the design requirements of the equipment that they operate. All equipment should be appropriately anchored to prevent sliding, overturning or impact loading. All power facilities, including electrical conduits and buildings that might house engine generator sets, should be designed to functionally survive the design seismic ground motion.

6.8.4 Analysis Procedures

A pseudo-static method of analysis is usually sufficient for the assessment of mechanical and electrical components. This needs to take into account that the accelerations may be greater than the bedrock peak ground acceleration, and the potential for dynamic amplification of the vibration in some equipment.

6.9 Access Roads and Bridges

6.9.1 General

In order to allow access to the dam for inspection or operation of outlet works following an earthquake event, the bridges and roads may need to remain in a sound state depending upon the importance of the dam and its appurtenant structures.

There is a limit to what a Dam Owner has control over in this instance as typically roads and bridges to access dams are not owned or maintained by the Dam Owner, this needs to be considered in assessing the requirements for operation of appurtenant structures following an earthquake.

6.9.2 Defensive Design Principles

The following defensive design measures are recommended:

- For bridges over spillways that are combined with or attached to the supporting structure for operating mechanisms that open and close gates a number of defensive design principles can be incorporate to prevent the following from occurring:
  - Distortion of the piers that could cause binding of gates or removal of support for the bridge from its bearings.
  - Failure of the girders, supports, handrails, etc. that could interfere with the gate operating machinery or interfere with opening of the gates.
  - Failure of the beam seats that could interfere with the gate operation
  - Differential movement that could cause hydraulic lines or other power supply functions to fail.
  - Distortion that could interfere with communication/control functions.
- For access bridges to intake towers the appropriate support mechanisms should be chosen to prevent or limit damage to both the bridge and the intake tower due to deflections occurring during the seismic event. This includes provision that sufficient longitudinal and lateral movement is available in the bridge supports.
- For critical structures an assessment may need to be conducted on the potential for rock falls or landslides and the impact they may have on either the structure itself or on access to the structure. It may be impractical to prevent rock falls, even in events more frequent than the OBE, and rock fall protection structures may be required.

6.9.3 Performance Requirements

OBE: Access roads and bridges to the dam and its appurtenant works that are owned and maintained by the Dam Owner should so far as is practicable remain passable after an OBE event. Therefore, likely rock fall or landslip areas along the access roads may need to be checked for potential instability. The impact of this on access roads and bridges that are not owned or maintained by the dam owner becoming impassable needs to be considered in assessing the required operation of appurtenant structures following an earthquake.

SEE: Access roads to the dam and its appurtenant works may become impassable after an SEE event. However, for access roads that are owned and maintained by the Dam Owner they should so far as is practicable be easily cleared or made accessible if there are no alternative access routes available. Access bridges that are owned and maintained by the Dam Owner should remain capable of carrying the design loads if there are no alternative access routes available. The impact of this on access roads and bridges that are not owned or maintained by the Dam Owner becoming impassable needs to be considered in assessing the required operation of appurtenant structures following an earthquake.

For bridges over spillways these may be combined with or attached to the supporting structure for operating mechanisms that open and close gates. For purposes of dam safety, such bridges should not impede the ability to open and close spillway gates following the occurrence of the SEE.

6.9.4 Analysis Procedures (Bridges)

Bridges should be assessed using an appropriate dynamic analysis of the pier and bridge system and should be in accordance with AS5100 – Bridge Structures. For intake tower access bridges, the analysis may be part of an overall dynamic analysis for an intake tower. The pier and bridge system should be examined for seismic ground motions in the direction of the bridge access and perpendicular to the bridge access. Loads resulting from seismic ground motions may be combined on a square root of the sum of the squares (SRSS) basis.

6.10 Reservoir Rim Instability

If there are existing or potential landslides on the reservoir rim, assess the likelihood that the slides will activate under seismic loads, and if so, whether the slide debris will reach the reservoir and at what velocity and volume. Then assess the potential for waves to be generated by the landslide debris, and if so, the effects on the dam and appurtenant structures.

C1 INTRODUCTION

No commentary.

C2 ASSESSMENT OF DESIGN SEISMIC GROUND MOTIONS AND ANALYSIS METHOD

C2.1 Earthquakes and Their Characteristics

C2.1.1 Seismology for Earthquake Hazard Studies

C2.1.1.1 Earthquake Mechanisms and Terminology

An earthquake is the sudden slip on a fault that is produced when stress within the earth builds up over a long period of time until it eventually exceeds the strength of the rock, which then fails and a break along a fault is produced. It may take tens, hundreds or thousands of years for the stress to build up in a particular area, and it is then released in seconds. Part of the energy is transmitted away as seismic waves and part as heat.

The fault displacement in a particular earthquake may vary from a few millimetres up to a few metres. Once ruptured, the fault is a weakness, which is more likely to further rupture in future earthquakes, so a large total displacement may build up from many earthquakes over a long period of time. This may eventually measure kilometres for thrust faults produced by compression, or hundreds of kilometres for horizontal strike-slip faults such as the San Andreas fault in California.

As shown in Figure C2.1 the point on the fault surface where a displacement commences is called the hypocentre or focus, and the earthquake epicentre is the point on the ground surface vertically above the hypocentre.

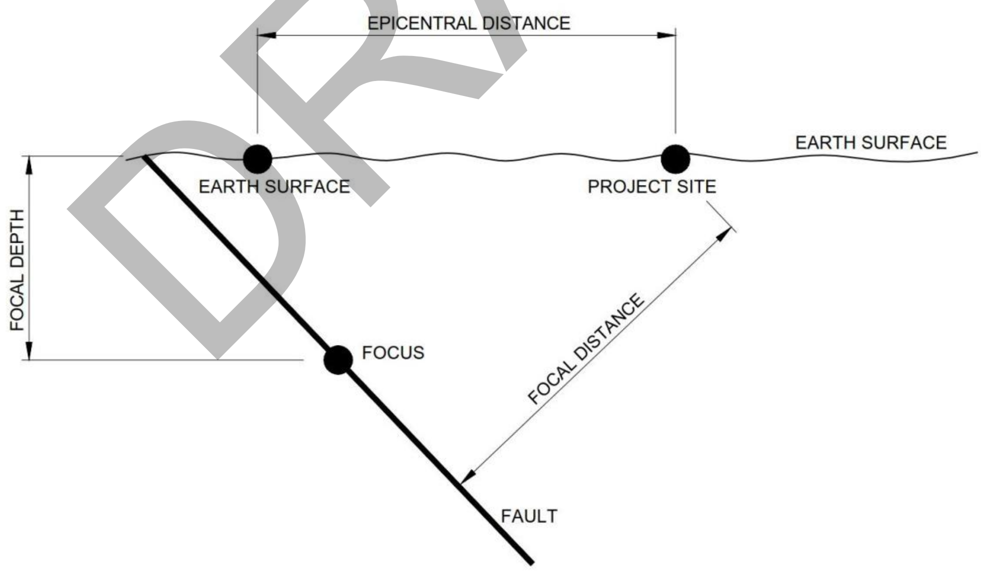


Figure C2.1 Definition of earthquake terms.

The fault slip initially propagates away from the hypocentre in a circular manner, but its eventual rupture extent is variable, sometimes propagating in both directions along strike and sometimes only in one direction. Maximum energy release is not necessarily near the hypocentre, and is often not even close to the hypocentre.

The hypocentral distance from an earthquake to a point is the three-dimensional slant distance from the hypocentre to the point, while the epicentral distance is the horizontal distance from the epicentre to the point.

Tectonic stress within the earth is caused by deformation that results from plate movement. The stress can be resolved into three orthogonal principal stresses, the maximum, intermediate and minimum principal stresses, usually denoted by σ_1, σ_2, and σ_3.

For crustal earthquakes, one of these principal stresses is usually near vertical, so the other two will be near horizontal. The stress directions determine the type and orientation of the faulting.

The vertical principal stress at any depth usually has a value comparable with the 'lithostatic pressure', or ρgh, where ρ is the density of the rocks above the point. The two near horizontal principal stresses can be higher or lower than this value, or one can be higher and the other lower, depending on which of σ_1, σ_2, or σ_3 is vertical.

If maximum principal stress σ_1 is vertical, or 'normal' to the Earth's surface we get a normal fault, resulting in a horizontal extension. If the minimum principal stress σ_2 is vertical we get a reverse fault, resulting in horizontal compression. If the intermediate principal stress σ_3 is vertical a strike-slip fault is produced, in which two blocks move horizontally relative to each other.

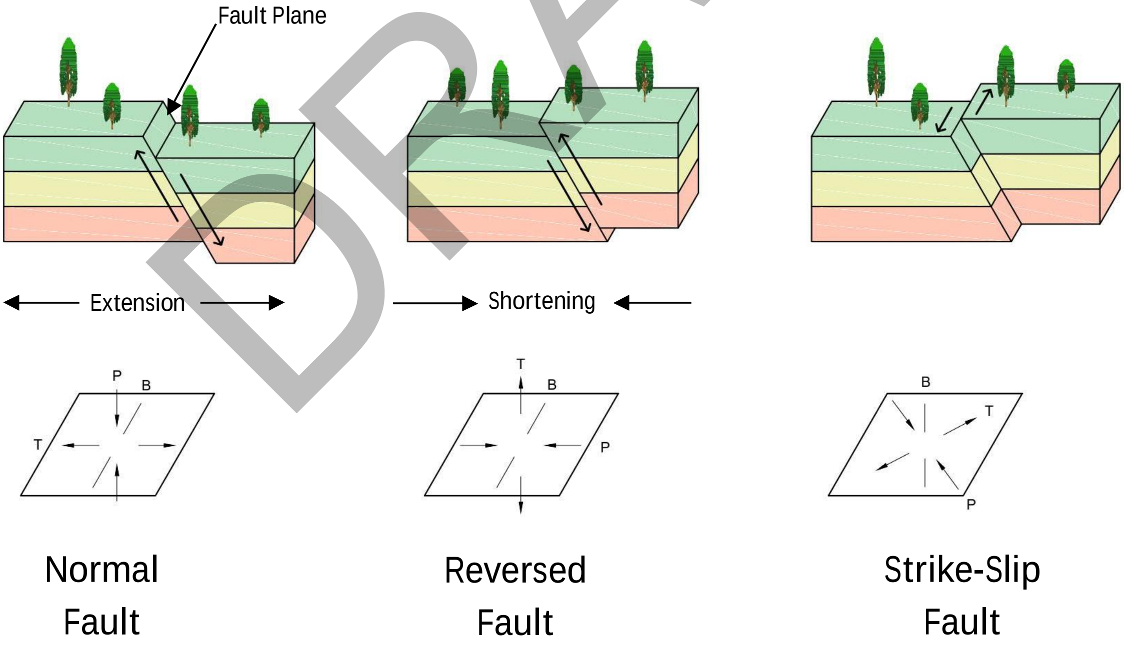


Figure C2.2: Principal stress orientation and fault types (P: Maximum, B: Intermediate, T: Minimum)

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A fault will only be active if its type and orientation is consistent with the principal stress orientation, to within about 20 degrees. Ancient faults that are not oriented appropriately are very unlikely to fail, but those that are so oriented may be reactivated. Stress orientations may be inferred from earthquake mechanisms, or obtained from the World Stress Map.

In Australia, all measurements to date show the maximum principal stress is near horizontal, with a high level of stress compared with the lithostatic pressure. The minimum principal stress is usually vertical so most faults are reverse. In some locations the intermediate principal stress is near vertical, so active faults at these places are likely to be strike-slip.

Using traditional mining terms for normal and reverse faults, the block that lies over the dipping fault is called the hanging-wall block, and the one under the fault is the foot-wall block.

For strike-slip faulting, standing on one block and facing the other, then observing which way the other is moving (right or left), defines right- or left-lateral strike-slip faulting.

C2.1.1.2 The Fault Rupture Process

The earthquake process is more complex than the simple displacement between two blocks shown in Figure C2.2.

A fault is a weakness that has experienced many ruptures in the past, each one providing an incremental shift between the blocks on either side. As tectonic deformation takes place at a very slow but fairly constant rate, the relative motion between blocks is concentrated along the fault, with bending that results in shear strain, and the accumulation of shear strain energy in the blocks on both sides of the fault as shown in Figure C2.3.

Since the Earth's surface is a free surface, this will result in folding and a vertical offset. This deformation happens very slowly over a very long period of time. The folding has low amplitude relative to the horizontal extent, metres vertical over kilometres or tens of kilometres horizontal distance. Reverse faults give folding in the hanging-wall block, similar to the folds produced by subduction in the over-riding plate, but without outer-rise folding on the subducting plate. Small earthquakes occur throughout the folded region during deformation.

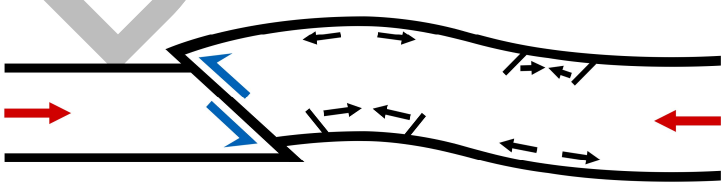


Figure C2.3: Deformation associated with reverse faulting.

The main active reverse fault dips under the uplifted block, so epicentres of foreshocks and aftershocks that are on this fault extend for tens of kilometres on the up-thrown block side of the fault surface rupture, with few earthquakes in the footwall block. Some foreshocks and aftershocks may also occur above the main fault in the up-thrown block.

Fault Rupture and generated seismic waves

When the stress level exceeds the strength of the fault at a point on the fault, it ruptures. The stored strain energy allows the rupture to propagate along the fault, despite considerable loss of energy due to friction and some due to radiation of seismic waves. Propagation continues until there is insufficient strain energy to continue the rupture, kinetic energy drops to zero, and the rupture displacement stops.

When the rupture occurs the release of shear strain energy leads to the production of a shear wave (S wave) that travels through rock at speeds of about two-thirds the speed of the P wave (compressional wave), whose motion is in the longitudinal and thus strongest on the vertical component on the ground surface. The S wave is a transverse wave with motion perpendicular to the direction of propagation, and is strongest in the horizontal plane on the ground surface. The amplitudes of the P and S waves decrease with distance due to geometric spreading and absorption of wave energy.

The stored compressional and shear strain energy are the tectonic energy that will be released in the earthquake.

The strain energy accumulates slowly over a long period, until it exceeds the strength at a point on the fault plane, which then ruptures. The rupture front spreads over the fault plane at about 3 km/s, with one block moving at about 1 m/s relative to the other, decreasing with time due to friction and loss of available strain energy.

Seismic waves leave the fault rupture at a wave velocity depending on local rocks, but typically 4 to 6 km/s for P waves, with ground motion velocities starting at about the relative block motion velocity (1 m/s).

Surface Rupture

Almost all earthquakes, especially larger earthquakes, occur on existing faults. This is because faults are weaker than surrounding unbroken rock, and are much more likely to fail again when stress rebuilds. Earthquakes nucleate within the brittle zone of the shallow crust, which is typically about 10 to 20 km thick. There is a brittle-to-ductile transition at the base of this seismogenic zone, typically at a depth of 10 to 20 km, caused by the increase in temperature with depth in the earth. Because rocks near the surface are relatively weak in non-cratonic regions of Australia (as defined in Clark et al., 2011; 2012; Figure C2.20), there is another brittle-to-ductile transition at shallow depths, typically a few km, so that few earthquakes originate at shallow depths (in the top one or two kilometres). It is common for surface rocks to be folded in response to faulting at depth, giving a monocline and scarp at the surface, but without a surface fault. In cratonic regions of Australia, which include most of the western part of Australia, rock near the surface is quite strong and shallow earthquakes are more common that in non-cratonic regions.

Surface rupture occurs when a fault break reaches the ground surface. It may produce a vertical or horizontal offset, or both, with a displacement of millimetres to a few metres, and a length from metres to tens of kilometres. Surface rupture has occurred frequently in the past century in cratonic regions of Australia, but is expected to be less common in non-cratonic regions of Australia.

A site will have surface rupture potential if an existing fault is found which has been active during the current stress regime. Clark et al. (2011) describe neotectonic features have undergone displacement under the current stress regime in Australia, and hence may have the potential for displacement in the future. The age of the current stress regime in Australia is estimated to lie in the range of 5 to 10 million years (Sandiford et al., 2004).

C2.1.1.3. Earthquake Magnitude

Earthquakes vary enormously in size. Richter (1935) defined a magnitude scale to indicate the size of an earthquake as

$$M_L = \log_{10} A$$

where

M_L is the Richter local magnitude scale

A = maximum seismic wave amplitude (in thousandths of a millimetre) recorded from a standard seismograph at a distance 100 km from the earthquake epicentre

or

$$M_L = \log_{10} A - F(D) + k$$

where

F(D) = distance correction; k = scaling constant.
This allows calculation of M_L from seismographs which have recorded the earthquake at different distances from the earthquake.

Other magnitude scales have been defined, including the Moment Magnitude (M_W) which assigns a magnitude to the earthquake in accordance with its seismic moment M_O, which is directly related to the energy released by the earthquake:

$$M_W = (\log_{10} M_O / 1.5) - 10.7$$

where

M_O is the seismic moment in dyn-cm.

Moment magnitude is the scale most commonly used for engineering applications.

These magnitude scales give similar values that can range from 0.0 to over 9.0. For each unit of magnitude there is a tenfold increase in ground displacement, and a thirtyfold increase in seismic energy release.

Another measure of earthquake size is the fault area, or the area of the fault surface, which is ruptured. The fault area ruptured in an earthquake depends on the magnitude and stress drop in the earthquake. Typically, a magnitude 4.0 earthquake ruptures a fault area of about 1 square kilometre, magnitude 5.0 about 10 square kilometres, and magnitude 6.0 about 100 square kilometres (perhaps 10 x 10 kilometres).

There are approximate relationships between the magnitude of an earthquake and the rupture area of a fault and other parameters, such as Gibson (1994) as shown in Table C2.1 and Leonard (2010).

Table C2.1: Approximate measures of shallow earthquake ruptures. (Modified from Gibson, 1994).

Magnitude	Rupture Area (km2)	Typical Length x Width (km x km)	Fault Slip ~Length/20,000 (metres)	Rupture Duration ~Length/3 (seconds)	Energy Released (MJ)	Global Average Number per year
2.0	0.01	0.1 x 0.1	0.005	0.03	60	2,000,000
3.0	0.1	0.3 x 0.3	0.015	0.1	2,000	200,000
4.0	1	1 x 1	0.05	0.3	60,000	20,000
5.0	10	3 x 3	0.15	1	2 million	2,000
6.0	100	10 x 10	0.50	3	60 million	200
7.0	1000	30 x 30	1.5	10	2,000 million	20
8.0	10,000	200 x 50	5.0	60	60,000 million	1.0

C2.1.1.4 Maximum Credible Earthquake Magnitude

In view of the impracticability of identifying all the faults on which major earthquakes may occur, it is necessary to use probabilistic methods to estimate expected ground motion versus Annual Exceedance Probability (AEP). For this, it is necessary first to assess the magnitude of the maximum credible earthquake, Mmax.

If the earthquake catalogue only covers a short period compared with the required return period, then the activity from a large surrounding area may be considered when estimating Mmax, perhaps as large as the whole of Australia or even including other intraplate areas over the earth. To give some appreciation of the likely maximum credible magnitude, Seismologists have considered the credible maximum lengths and widths of faults that may rupture to cause the earthquake. Because of the limited seismogenic width of the shallow crust, large earthquakes have long rupture lengths, as indicated in Table C2.1.

Very few intraplate earthquakes are larger than magnitude 7.5. A series of very large intraplate earthquakes in the New Madrid area of Missouri, USA, in 1811 to 1812, had original published magnitude values exceeding 8.0 (Johnston and Shedlock, 1992). Recent authors believe that the New Madrid earthquakes were considerably smaller than this (Evernden, 1975, M6.9; Gomberg, 1992 M7.3; Hough et al, 2000).

There is general consensus among Australian Seismologists that the maximum credible magnitude of an earthquake in Australia is about magnitude 7.5. This is based on consideration of the limited depth range at which crustal earthquakes can occur, and the sensible maximum length of fault that will rupture. It was originally suggested that a magnitude 7.5 earthquake could occur anywhere in the country, but it is possible that this value may be reduced if there is evidence that active faulting, especially surface faulting, has not occurred in particular locations.

Given the shallow seismogenic depths in Eastern Australia, the value of Mmax 7.5 is possibly conservatively high. However for return periods to 1000 years, (annual probability of 0.001) decreasing Mmax to 7.2 or 7.3 would give negligible effect on probabilistic ground motion estimates. For longer return period (lower annual probability) motion, the lower value could still be justified, especially if there is little seismological and especially geological evidence for the existence of large nearby faults.

C2.1.2. Earthquake Recurrence

C2.1.2.1. Introduction

Earthquake source regions are either known faults, or source zones where faults are unknown or earthquakes are distributed over many small faults. A zone can be any volume in the earth, but is usually defined as a polygonal prism with vertical sides and horizontal top and bottom. For determination of earthquake recurrence, the main characteristic of each zone is that it is reasonable to expect that earthquake activity is uniform throughout the zone.

For known faults we need to know the type of fault, location and dip direction. The best measure of the activity on the fault is the average slip rate, usually given in millimetres/year for active regions, or metres/million years in stable regions.

If the earthquake catalogue included all events within the entire zone, at all times during the observation period, then a recurrence plot of rate of activity against magnitude could be determined by simply counting the number of events with each magnitude and dividing by the number of years of observation.

Unfortunately earthquakes and seismograph coverage are not so simple. Earthquakes cluster in time and space with long periods of quiescence. Seismograph coverage varies with time, usually giving complete coverage to lower magnitudes as seismograph density increases, but sometimes deteriorating as individual seismographs are removed, or seriously deteriorating as seismograph networks are removed.

The coverage of a zone depends on the location of seismographs relative to the zone. If seismographs are aligned in one direction relative to the zone, either inside or outside the zone, then coverage may vary widely across the zone, with complete coverage being limited to larger events.

C2.1.2.2. The Gutenberg-Richter Relationship

In seismology, the Gutenberg–Richter law expresses the relationship between the magnitude and the total number of earthquakes in any given region and time period of at least that magnitude.

$$\log_{10} N = a - bM$$

Or

$$N = 10^{a-bM}$$

where:

- N is the number of events having a magnitude $\geq M$
- a and b are constants

The parameter b (commonly referred to as the "b-value") is commonly close to 1.0 in seismically active regions. This means that for given a frequency of magnitude 4.0 events there will be 10 times as many magnitude 3.0 earthquakes and 100 times as many magnitude 2.0 earthquakes.

The a-value indicates the total seismicity rate or activity of the region.

C2.1.2.3. Random and Non-Random Processes

The earthquake recurrence rate is usually estimated by assuming that the earthquake activity rate observed over the recent past will continue unchanged.

Earthquake hazard studies usually assume that earthquake recurrence is a random process. That is, the probability of an earthquake does not change with space or time, and all earthquakes are independent of each other.

One of the most obvious examples of non-random activity in time is the foreshock-main shock-aftershock sequence. In hazard studies, if it is assumed that the main shock will be the most damaging event in the sequence, foreshocks and aftershocks can be removed (de-clustered) and the recurrence of main shocks is determined, normally leading to an increase in estimated hazard because the "b-value" is then lower.

Non-random activity in time is clear in an earthquake cycle represented by a sequence including a period of quiescence, possible precursory events for large earthquakes, foreshocks, main shock, aftershocks, possible adjustment events and another quiescence period.

Non-random activity in space is largely related to faults. Large earthquakes only occur on large faults and small earthquakes most often occur on small faults. This means that the relative number of small to large earthquakes, the Gutenberg-Richter b-value, varies depending on local geology, not only in its value but also in the magnitude range of applicability. Variations in b-value are particularly obvious with earthquake depth with high values at shallow depths and low values for deep earthquakes. There is increasing evidence of earthquake activity being affected by recent past activity in neighbouring regions, both on a single large fault (or plate boundary) and on other nearby faults.

One of the most common ways to force the seismicity of an area to fit a single Gutenberg-Richter distribution is to use large zones that include many earthquakes from a wide range of sources. This leads to "large-zone hazard dilution", where the estimated hazard in more active regions is significantly reduced, while that in larger areas of low activity is increased slightly.

Future earthquake hazard studies will continue to rely on past earthquakes, but will give greater consideration to geological and geodetic data and processes, including stress, strain energy, and the dynamics of deformation in a complex geological environment.

C2.1.2.4. Time-Magnitude Plots

The earthquake recurrence or seismicity (seismic activity) of an area must take the range of earthquake sizes into account. There are many more small earthquakes than large. As discussed above, in most places around the earth the b-value is 1.0. The b value may be 1.3 or higher if there are many small earthquakes, or 0.7 or lower if there are relatively few small earthquakes.

The recurrence rate is complicated by the wide range of earthquake magnitudes that are reported, or recorded and located. Most historical earthquakes larger than about magnitude 4.0 or 5.0 are reported in newspapers because of the intensity of ground motion that was felt, or the damage that was caused. National seismograph networks can currently record all earthquakes exceeding about magnitude 2.0 to 3.0 depending on the seismograph density and distribution. A dense local network of seismographs about a site can record earthquakes exceeding about magnitude 0.0 to 1.0.

The Gutenberg-Richter relationship is used to quantify the earthquake recurrence for a region. The activity (a) is given by the intercept on the vertical axis, the relative number of small to large magnitudes by the gradient (or b-value) and a maximum magnitude is applied.

The b-value is not a constant, but varies with space and time. If a number of different processes are included, each with their own b-value and other parameters, the apparent seismicity of the region may still have a well-defined b-value, but at some intermediate value depending on the region and time interval considered (the average of a set of straight lines gives another straight line, provided that linearity occurs for all earthquakes over the magnitude range considered).

If the seismicity is dominated by a swarm of shallow earthquakes then the observed b value will be too high to allow extrapolation to higher magnitudes, resulting in hazard estimates too low, or if it is dominated by a few moderate to larger earthquakes (or the catalogue is incomplete), the observed b-value will be too low to allow reliable extrapolation, resulting in hazard estimates that are too high.

This assumption of temporally random earthquake occurrence clearly does not apply to earthquake clusters consisting of possible foreshocks, a main shock, and aftershocks. For many years, standard practice has been to consider the recurrence of clusters rather than individual events, with the cluster represented by the main shock, which is the event in the cluster that will give strongest motion, and usually (but not always) cause the most damage. A de-clustered earthquake history is used, and foreshocks and aftershocks are simply not considered in the earthquake recurrence calculations.

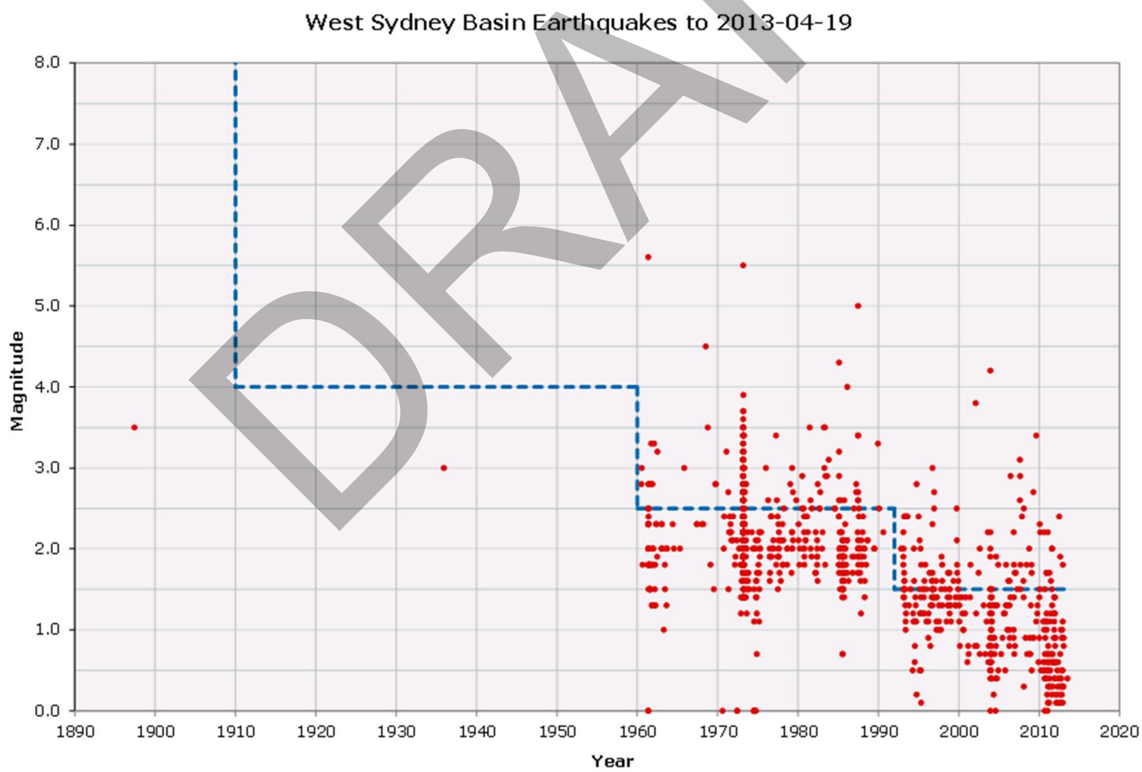


Figure C2.4: Earthquake history for the West Sydney Basin zone.

As an example the seismological history of the West Sydney Basin zone is shown in C2.4, with the dashed line showing the estimated variation in magnitude threshold for complete coverage of larger magnitudes, generally decreasing as the seismograph network density improves over time.

The difference in rates of activity for magnitudes 1.5 to 2.2 between 1970 and 1988, and between 1992 to the present is probably because the original catalogue was not fully "de-blasted", so contains many quarry and mine blasts as well as earthquakes. The vertical lines of events show clusters, usually a main shock with foreshocks and aftershocks.

The next stage in quantifying the activity within the zone is to convert the historical data into cumulative recurrence data. This is shown in Figures C2.5 and C2.6.

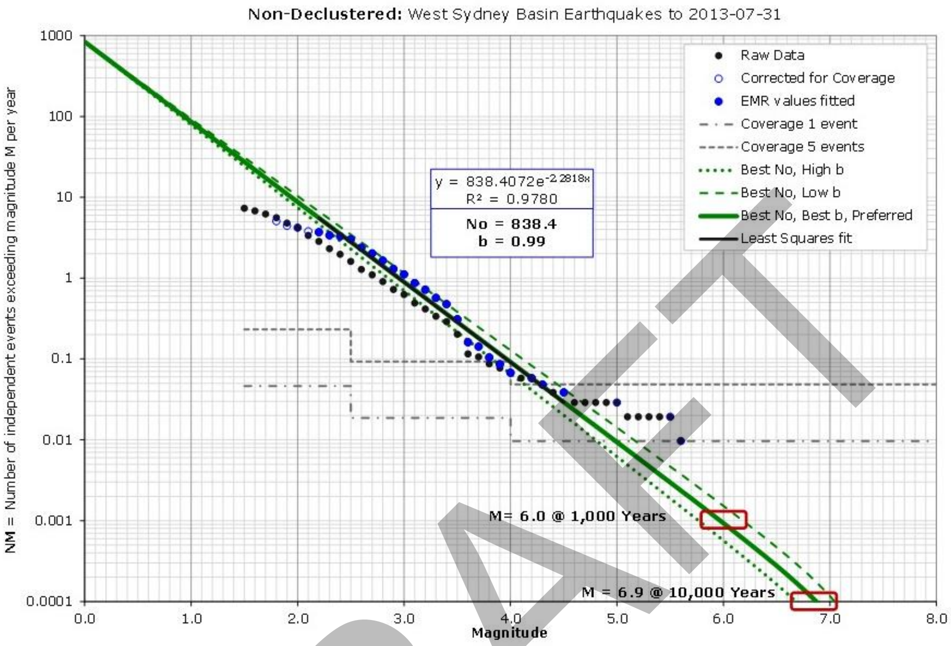


Figure C2.5: Earthquake magnitude recurrence, not de-clustered.

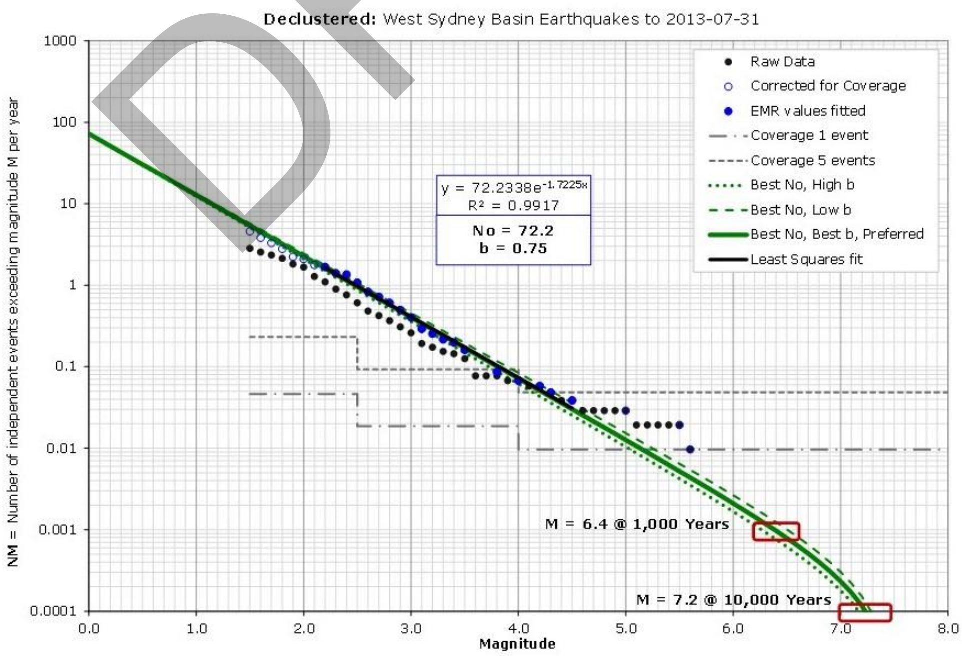


Figure C2.6: Earthquake magnitude recurrence, de-clustered.

Note that in this example that the earthquake with a 1/1000-year average recurrence interval is magnitude 6.0 for the original data and 6.4 for the de-clustered data. The corresponding values for 10,000-year average recurrence interval are magnitude 6.9 and magnitude 7.2 respectively.

C2.1.2.5. Geologically Observed Slip Rates vs. Extrapolated Observed Seismicity

Larger active faults appear to have geologically observed slip rates that require greater earthquake activity rates than the extrapolated seismicity estimates (Figures C2.6 and C2.7,). Earthquake recurrence estimates made by extrapolating small earthquake recurrence will under-estimate the hazard in these regions.

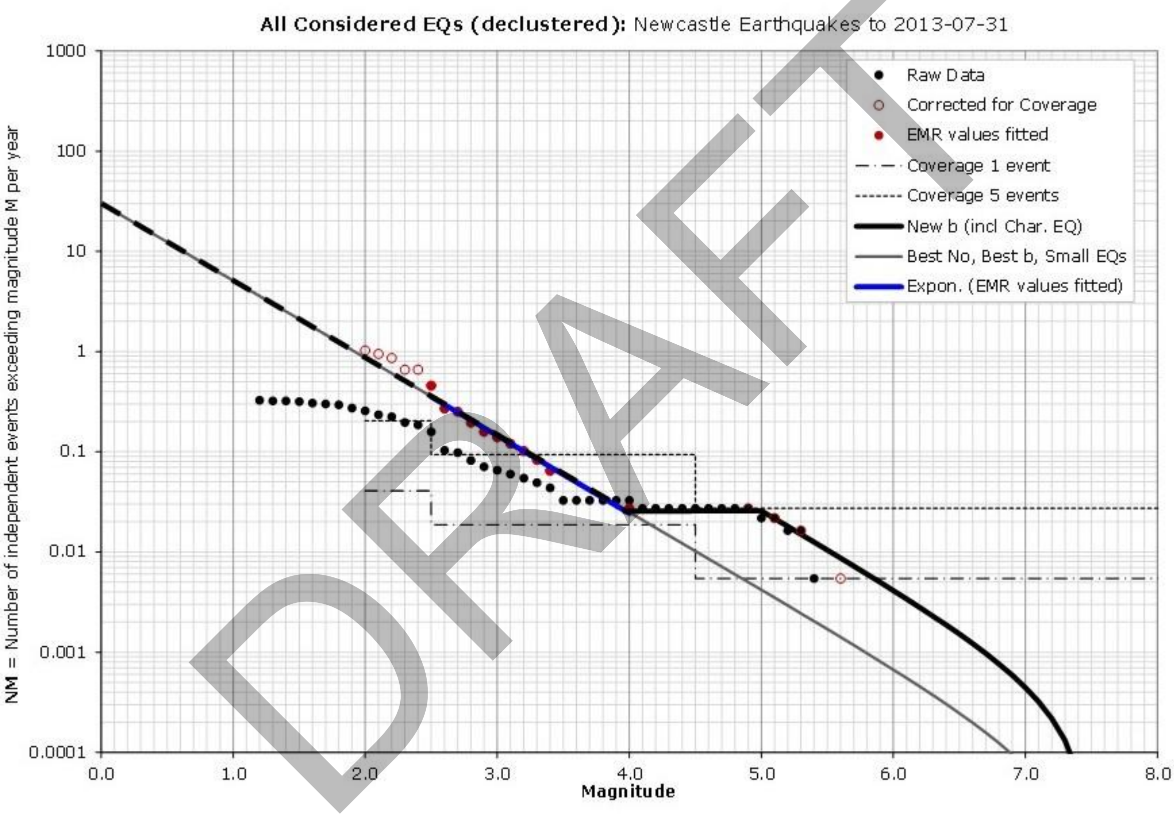


Figure C2.7: Earthquake magnitude recurrence for the Newcastle zone.

In areas with no obvious large active faults, such as large areas of undeformed flat-lying sediments, the observed recurrence rates are lower than the extrapolated estimates (or zero), with fewer (or not so large) earthquakes. This suggests that these smaller earthquakes occur in regions with small faults, and lower maximum magnitude. Earthquake recurrence estimates made by extrapolating small earthquake recurrence will over-estimate the hazard in these regions.

C2.1.3. Seismic Ground Motion

C2.1.3.1. Ground Motion Measures

Earthquake ground vibration is recorded by a seismograph or a seismogram. Most modern seismographs record three components of motion: east-west, north-south and vertical.

The rupture time for a small earthquake is a fraction of a second, for earthquakes of magnitude 5.0 it is about a second, and for large earthquakes may be up to tens of seconds. However the radiated seismic waves travel at different velocities, and are reflected and refracted over many travel paths, so the total duration of vibrations at a site persist longer than the rupture time, and show an exponential decay.

As discussed above, several types of seismic wave are radiated from an earthquake. Body waves travel in three dimensions through the earth, while surface waves travel over the two-dimensional surface like ripples on a pond. There are two types of body wave (P waves and S waves), and two types of surface waves (Rayleigh and Love waves).

Primary or P waves are ordinary sound waves travelling through the earth. They are compressional waves, with particle motion parallel to the direction of propagation.

Secondary or S waves are shear waves, with particle motion perpendicular to the direction of propagation. The amplitude of S waves from an earthquake is usually larger than that of the P waves.

P waves travel through rock faster than S waves, so they always arrive at a seismograph before the S wave.

The frequency content of seismic ground motion covers a wide range of frequencies up to a few tens of hertz (cycles per second). Most engineering studies consider motion between about 0.2 and 25 Hz.

The amplitude, duration and frequency content of seismic ground motion at a site depend on many factors, including the magnitude of the earthquake, the distance from the earthquake to the site, and local site conditions.

The larger the earthquake magnitude, the greater the amplitude, the longer the duration of motion, and the greater the proportion of seismic energy at lower frequencies. A small earthquake has low amplitude (unless it is very close), short duration, and has mainly high frequencies.

The smaller the distance from an earthquake to the site, the higher the amplitude. The duration is not strongly affected by distance. High frequencies are attenuated by absorption within the ground more quickly than low frequencies, so at greater distances the proportion of seismic vibration energy at high frequencies will decrease.

C2.1.3.2. Earthquake Intensity

Earthquake Intensity is a measure of the effect of the seismic waves at the surface, and is normally given on the Modified Mercalli Intensity scale; a copy of which is attached in Appendix A. This is an arbitrary scale defined by the effects observed (whether sleeping people were woken, trees shaken, etc.) and on the damage caused. Normally the maximum intensity occurs near the epicentre of the earthquake, and intensity then decreases with distance. However this may be affected by the orientation of the rupture, or local ground conditions such as topography or surface sediments.

C2.1.3.3. Attenuation and Amplification of Ground Motion

Seismic ground motion generally attenuates with increasing distance from the source due to radiation and hysteretic damping. High frequency motion is attenuated more quickly with distance than lower frequency motion.

In the old, hard rocks in cratonic regions such as the shield areas in Western Australia and in eastern Canada, earthquakes of magnitude 5.0 can be felt at much greater distances than in Eastern Australia, over 400 km compared with less than 150 km, and cratonic ground motion models must reflect this. Unfortunately, there is very little strong motion data available from such areas.

In cratonic areas there are often very hard crystalline rocks to near the surface and earthquake depths tend to be quite shallow. This can lead to generation of strong surface waves by relatively small earthquakes with long period motion (about 2.0 seconds in the Yilgarn of Western Australia). Ground motion models for this region (Liang et al., 2008; Somerville et al., 2009) incorporate such unusual characteristics.

In selecting attenuation relationships or ground motion prediction equations care is needed, and attention paid to the mechanism of the source earthquake, e.g. whether shallow intraplate or deep plate boundary earthquakes.

At low amplitude levels, the ground motion is amplified by the near-surface geology because it is generally more flexible than unweathered bedrock. The amplification increases as the shear wave velocity of the surface geology decreases due to impedance amplification. The shallow shear wave velocity is quantified, for example, by Vs30, the time-averaged shear wave velocity in the upper 30 metres of the ground. However, at high amplitude levels in soft soils, weak surface materials absorb seismic energy rather than transmit it unchanged, thus tending to reduce amplitudes at the surface. The amount of attenuation depends on the properties of the materials, and especially their thickness.

In horizontally stratified sediments, the near surface layers will vibrate preferentially at their own natural frequencies, depending on their thickness and elastic properties. The earthquake motion at the natural frequencies of the near-surface layers is amplified, while motion at other frequencies may be little affected or even attenuated. The amplification effect can be especially pronounced for deep soft sediments that have very low damping, such as those underlying Mexico City.

Dams (like all other structures) each have their own natural frequency depending on their mass and stiffness, usually in the range from about 0.5 to 5 Hz for embankment dams and 2 hertz to 20 hertz for concrete gravity dams.

C2.1.3.4. Site Response – Near-Surface Amplification and Attenuation

Ground motion at a site on the Earth's surface may be significantly affected by near-surface geology and topography. There are many phenomena that affect near-surface motion, including:

- Impedance amplification
- Resonance of surface sediments
- Additional sedimentary basin effects
- Variation of frequency-dependent attenuation, Q(f), in surface sediments
- Scattering in complex geology
- Reflections in stratified surface geology
- Groundwater
- Wave conversions at interfaces
- Focusing by complex structures
- Topographic effects

Combinations of impedance amplification and resonance can give amplifications of bedrock ground motion that can exceed x 4 to x 8 at resonant frequencies. Resonant motion gives greater amplification to horizontal motion than to vertical motion. Site response can vary over very short distances, such as tens of metres.

Site response can be measured by comparing surface motion with bedrock motion.

Measurement of ambient vibrations at a point on the surface can clearly indicate the natural frequencies of the sedimentary column simply because the spectrum shows a high peak. In other cases using single point measurements, the ratio of horizontal spectral motion over vertical spectral motion will give some idea of both natural frequencies and the degree of amplification, because horizontal motion is amplified much more than vertical motion.

Measuring ambient vibrations with a small array can reveal more information about the sedimentary column, such as the Spectral Autocorrelation (SPAC) method which gives a velocity dispersion curve from which the variation of shear wave velocity with depth can be estimated. These methods usually consider depth ranges from metres to hundreds of metres.

Site amplification has little effect on hard rock sites, but can dominate the hazard on sites with deep soft sediments.

Free-Surface Amplification

The amplitude of seismic waves increases as they approach the Earth's (free) surface, at which they are reflected. This amplification can be considered in terms of constructive interference between the up going and down going waves, and can give surface motion up to about double that at depth for a uniform half space. However, as noted above, the near surface material usually has lower shear wave velocities than bedrock, so the surface motion is usually more than twice that at depth. The nature and degree of free surface amplification varies with topography, even in fresh, strong rock. Changes in soil thickness above an irregular bedrock surface can give complex surface amplification that varies with wave duration.

Resonance

Resonance in the surface sediments causes amplification at particular frequencies, especially at the natural frequency of the sediments. This depends on the thickness and elastic properties of the sediments. Seismic motion recorded on hard rock tends to be broadband, while that recorded on soft sediments is usually dominated by the resonant frequency.

Attenuation

In surface sediments, high frequency vibrations are attenuated much more with distance than low frequencies. If sediments are very thick, much of the high frequency motion will be lost and peak surface accelerations will be relatively low, even if resonance has amplified motion at the low resonant frequency.

Local site conditions on a relatively small scale, particularly soft surface sediments but also topography, can significantly affect surface motion from an earthquake. These effects can vary rapidly with location, showing significant changes over distances within tens or hundreds of metres.

Estimating Site Response in Practice

Site response, or site amplification, depends on many phenomena, including variation in impedance depending on the rock properties, variations in attenuation especially through soft rocks, resonance in surface layers especially in soft rock, resonance of sedimentary basins, conversion of wave types between P, S and surface waves at the surface or at boundaries beneath the surface, and focusing of waves by irregular surfaces.

The treatment of each of these can be from simplistic to complex. The critical phenomena will vary from site to site. Some of the phenomena are period (or frequency) dependent and require spectral variations (e.g. resonance, attenuation), and others vary little with frequency (e.g. impedance amplification).

For many earthquake hazard studies it is normal practice to calculate bedrock motion for surface outcrop or at the surface of bedrock below sediments. Unless the site is near to an active fault, there is little variation in estimated bedrock motion over distances of kilometres, and the same bedrock motion can be applied at various locations about the sites. That does not hold if the rock foundation conditions are greatly different between the river bed and the abutments. The bedrock motion can then be used to estimate surface motion or motion of the dam, listed in order of increasing complexity, accuracy and cost:

- When using Australian Loading code AS1170.4 (2007), the bedrock ground motion values given would be multiplied using the AS1170.4 site sub-soil class factors, which vary from 0.8 for hard rock to 3.5 for very soft soils. This does not consider frequency dependent resonance. AS1170.4 has limited application for dams because it only provides loads for the 10% chance of exceedance in 50 years, or 1 in 475 annual probability.
- For the next level of design, for horizontally stratified surface layers, a local value of Vs30 allowing for the Vs of the sediments overlying bedrock can be used to recalculate the results to give an average estimate of site response that considers impedance amplification, but does not consider frequency dependent phenomena such as resonance. Vs30 is the time averaged shear wave velocity in the top 30 metres of rock and sediments, calculated from the inverse of the mean slowness in the top 30 metres (using seconds per metre rather than metres per second), or 30 divided by the total vertical shear wave velocity (Vs) travel time through these 30 metres. This gives a greater weighting to low velocities rather than to high velocities. Use of Vs30 to estimate soil amplification at a specific site assumes that the average shear wave velocity, shear modulus reduction, and damping increase profiles represented in the strong motion database used to develop the ground motion prediction equations are a reasonably accurate representation of the specific profiles at the site. If this is not the case, then a site-specific analysis of the soil amplification (described next) should be done.
- If site specific response must be considered, and if the site has near horizontal stratification, the results provided can be used to select design earthquakes for use with a SHAKE type program to estimate surface motion and cyclic stresses in the foundation overburden and embankment. The site is represented by a one-dimensional model giving the shear wave velocities of each layer down to bedrock (not just the surface 30 metres), plus densities, and the depths of the layer interfaces. If conditions are appropriate and the shear wave velocity variation is known or can be estimated, this is relatively easy to do.
- If frequency dependent site response is considered, and if the site has topographic variations, or does not have near horizontal sub-surface stratification, the results provided can be used to select design earthquakes for use with a finite-element or finite-difference computer program to estimate surface motion. The site is represented by a two- or three-dimensional model giving the distribution of sub-surface materials. In some cases this may be a combined site-structure model, incorporating site-structure interaction. Two-dimensional models require much data and complex computation, and three-dimensional models require a great deal more. This is seldom done in practice as one dimensional models are usually adequate. More than one 1D model may be used if conditions vary across a dam site.
  - A time-consuming and generally impractical method in anything other than highly seismically active areas is to determine a site transfer function empirically by installing seismographs at the site and nearby on bedrock (either in a borehole or on a rock outcrop within a distance of some kilometres). Comparison of spectra from regional and distant

earthquakes will give frequency dependent site response for short and long period motion respectively.

In the case of foundations on rock with a fairly typical Vs30 of 760 m/s, there will be relatively little resonance or other frequency dependent site response. Inclusion of the Vs30 term will account for some impedance amplification.

C2.1.4. Earthquake Hazard in Australia

C2.1.4.1. Australian Earthquakes

The Australian continent is within a tectonic plate shared with most of India and some of New Zealand, and all of its earthquakes are intraplate. The plate boundaries to the north and east are among the most active on the Earth. Possibly as a result of this, Australia is one of the most active intraplate areas on the Earth. Despite this, the hazard is quite low when compared with active plate boundary areas.

Most people in Australia can expect to feel an earthquake about every five to ten years, although many of these will not be recognised as an earthquake. Most Australian earthquakes that are reported are smaller than magnitude 3.0 and are heard with a noise like a distant quarry blast or thunder (due to coupling of P-waves in the ground into sound waves in the air), often followed by a slight vibration.

Only a small proportion of the earthquakes that are felt, perhaps about 5%, will cause any damage in their epicentral area. If they occur in an inhabited area, most earthquakes of magnitude 4.0 to 5.0 will cause some minor damage, while magnitudes greater than 5.0 may cause significant damage.

By contrast, in an active plate boundary area like New Britain or Bougainville in Papua New Guinea, earthquakes are felt very often, on average every week or two. These are normally felt rather than heard, with any sounds being the reaction of a building to the vibrations rather than the earthquake itself. A very small proportion of these felt Papua New Guinea earthquakes, perhaps about 0.1%, will cause damage in their epicentral area, and earthquakes smaller than magnitude 6.0 rarely cause damage.

There are a number of factors that influence the estimation of earthquake hazard in Australia. One is the short duration of documented history, with a little over 200 years of data about Sydney, and considerably less for most of the continent.

Another factor is the large area of the country relative to the size of the population. Seismographs are distributed relatively sparsely, limiting the accuracy of earthquake locations and magnitudes.

As was the case over most of the Earth, seismograph coverage of local earthquakes in Australia was only established in about 1960, following the International Geophysical Year. While there should be some link between population and seismograph density with more instrumentation in the populated southeast, coverage is highly variable and still far from complete. The Australian National Seismograph Network, operated by Geoscience Australia, aims to locate all earthquakes in Australia larger than magnitude ML 3.0. Local seismograph networks have non-uniform coverage, often with good coverage of large dams and poor coverage of major cities.

Figure C2.8 shows the location of Australian earthquakes with magnitude exceeding ML 4.0 in the period 1850 to 2014. Note that many earthquakes prior to 1960 were not located so are not shown.

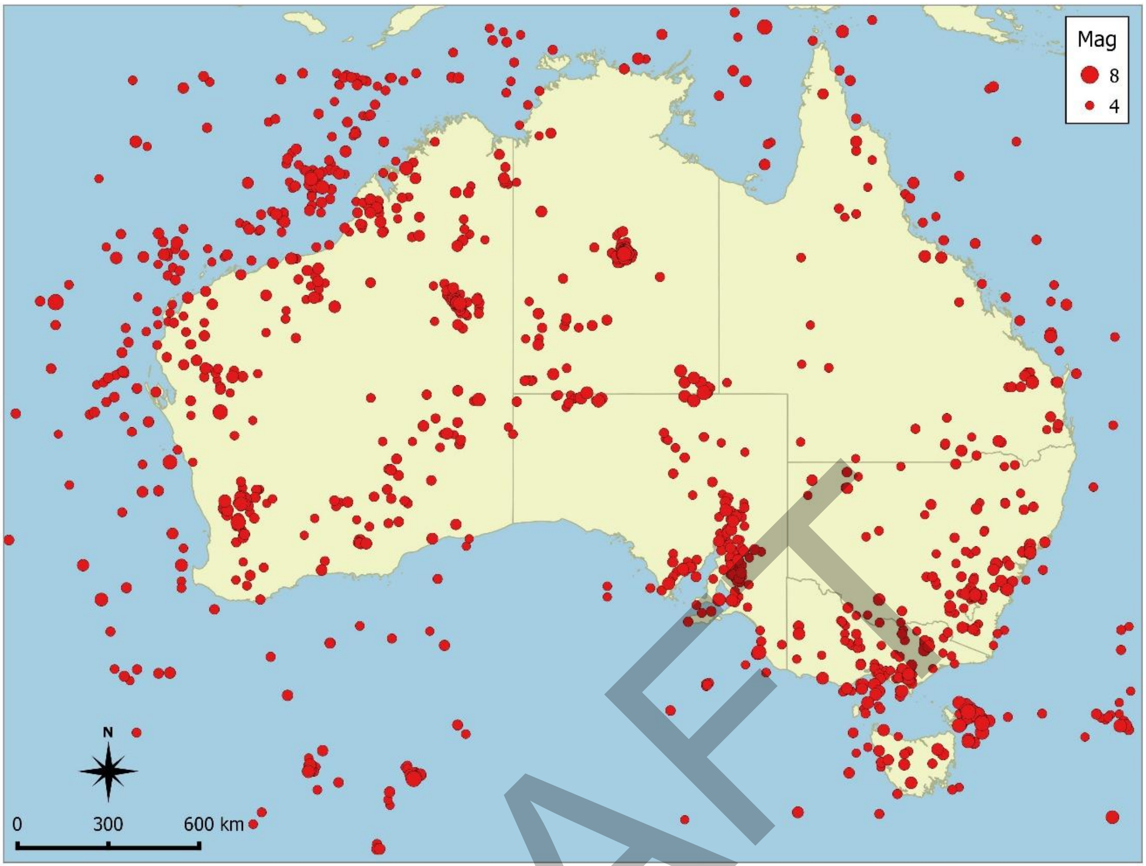


Figure C2.8: Australian earthquakes exceeding magnitude 4.0 from 1850 to 2016

Table C2.2 lists some of the largest earthquakes which have occurred in Australia. It can be seen that there are several in the range M6.5 to 7.0.

Table C2.2: Significant Australian earthquakes

Date	Place	Mag	Imax	Damage, Approximate A$(2016).
1892-01-26	Flinders Island, Tas	MP 6.9	7+	Offshore epicentre, little damage. Felt through Tasmania and East Victoria.
1897-05-10	Beachport, SA	MP 6.7	8	Damage in Kingston, Robe, Beachport, major liquefaction; felt from Ceduna to Melbourne.
1902-09-19	Warooka, SA	MP 6.0	8	At Warooka chimneys fell, walls partially demolished, few buildings without damage.
1903-07-14	Warrnambool, Vic	MP 5.3	7	Extensive minor damage and liquefaction at Warrnambool. Followed similar event in April.
1918-06-06	Bundaberg, Qld	MP 6.0	6	Epicentre offshore, minor damage in Rockhampton.
1941-04-29	Meeberrie, WA	MS 7.2	8	Isolated area. Damage to remote farm houses including cracked walls, burst tanks.
1954-02-28	Adelaide, SA	ML 5.4	8	Widespread minor damage, A$200m. Epicentral was rural but is now urban.
1961-05-21	Robertson, NSW	ML 5.6	7	Damage in the Moss Vale, Robertson and Bowral area, A$10m.
1968-10-14	Meckering, WA	MS 6.8	9	Most buildings in Meckering destroyed, $90m. 32 km surface rupture.
1969-10-14	Boolarra, Vic	ML 5.6	6	Cracked walls and fallen chimneys in the epicentral area.
1973-03-09	Burragorang, NSW	ML 5.0	6	Minor damage in Picton, Bowral and Wollongong, A$7m.
1979-06-02	Cadoux, WA	MS 6.2	9	Many buildings at Cadoux destroyed, $9m. Only one injury. 15 km surface rupture.
1986-03-30	Marryat Creek, SA	MS 5.9	8	Epicentre in remote area. Cracked walls in nearest homestead, 13 km surface rupture.
1988-01-22	Tennant Creek, NT	MS 6.8	9	Epicentre in remote area. Damage A$4m, mainly to damaged gas pipeline, 35 km surface rupture.
1989-05-28	Uluru, NT	mb 5.7	6	Minor damage at Uluru National Park (Ayers Rock). Epicentre west of Mt Olga.
1989-12-27	Newcastle, NSW	ML 5.6	8	Thirteen fatalities, $4500m plus damage. Widespread minor to moderate damage.

C2.1.4.2. Mechanism of Australian Earthquakes

Almost all Australian earthquakes have mechanisms with the maximum principal stress near horizontal. Most have the minimum principal stress near vertical, producing reverse or thrust faults. There may be some strike-slip movement, when the failure is on a fault that is oriented at an angle other than 90° to the principal stress direction.

Compression giving reverse and thrust faults produces surface uplift, so these earthquakes are most likely to occur in areas where mountains are developing (such as the eastern Highlands or Flinders Ranges), and less likely under sedimentary basins (such as the Murray-Darling Basin or the Great Artesian Basin).

There are a number of active faults upon which several episodes of movement have been proven in the last 100,000 years. These include the Edgar fault; Tasmania, Hyden fault, Western Australia; Wilkatana and Roopena, South Australia; and Cadell, Victoria/New South Wales (Clark et al, 2012).

Large Australian earthquakes (e.g. Tennant Creek, 1988), have occurred on faults had not been previously been recognised and mapped. Crone et al. (1997) describe evidence that there had been no movement on the fault at Tennant Creek for 200,000 years.

Based on the available information, it would appear that differences between earthquake ground motion s in non-cratonic regions of Australia and those from reverse fault earthquakes in USA, China, etc., data from which are included in some empirical ground motion prediction models are based, are not sufficiently great as to invalidate the use of those models in Australia. This is an aspect that will need to be further assessed as more ground motion data from Australian earthquakes is gathered.

More detailed design methods are based upon seismic response analyses to actual ground motions rather than empirical methods so this is not an issue provided there is a careful selection of ground motion histories to represent the earthquake conditions in Australia.

C2.1.4.3. Earthquake Depths

Usually, earthquake depths can be precisely determined from local networks only if the distance to the nearest seismograph is not greater than about twice the earthquake depth. The depths of most Australian earthquakes are poorly constrained, because of their relatively shallow depths and the low density of seismographs.

If it is not possible to constrain an earthquake depth using seismograph data or other observations (e.g. a magnitude 1.0 earthquake that is felt must be within a couple of kilometres of the surface), an arbitrary “normal” depth may be used. Typical values used in Australia include 0, 5, 10 or 33 kilometres. These may or may not be realistic. Some observatories may select a depth from a range of standard values, depending on the character of the recorded waveforms (e.g. a small magnitude earthquake with large surface waves must be shallow).

Eastern Australian earthquakes are usually at depths between 1 and 20 km, while those in Central Australia may be a little deeper, and those in Western Australia may be a little shallower. Australia’s deepest known earthquake occurred at 39 km depth offshore from Arnhem Land in 1992 (McCue and Michael-Leiba, 1993).

In Eastern Australia, earthquakes at depths of less than 5 km are regarded as shallow, and greater than 15 km as deep. The Newcastle earthquake of December 1989 was at a depth of about 12 km.

All of these Australian earthquakes are very shallow compared with those in plate boundary areas, where subduction earthquakes can occur as deep as 700 km, and depths of less than 70 km are regarded as shallow, and greater than 300 km as deep.

It seems that small earthquakes tend to occur more often at shallow depths, while moderate to large earthquakes rupture at greater depths (Allen et.al, 2004). Many of the larger Australian earthquakes have occurred in the cratonic regions and have rupture through to the surface due to the high strength of the surface rocks, giving a surface fault rupture.

Shallow earthquakes often have many aftershocks, some with magnitudes approaching that of the main shock. Deep earthquakes usually have few aftershocks, and these are usually no larger than one magnitude unit below the main shock magnitude.

Because of their shallow depth, small Australian earthquakes are often heard or felt. Magnitude 1.0 events can be felt to a distance of about 1 km, and magnitude 2.0 to about four kilometres. These are slant distances, so only very shallow events of these magnitudes will be felt.

Because the short travel distances from shallow earthquakes do not give much attenuation of high frequency vibrations, and if a relatively high stress drop gives a high proportion of high frequency vibration from the earthquake source, these events have enough energy in the audio range to allow them to be heard. For many such small earthquakes the sound heard is more significant to an observer than the vibrations felt.

For similar reasons, moderate magnitude earthquakes in Australia produce motion with strong peak ground accelerations, and can cause significant damage. The Newcastle earthquake was only of magnitude ML 5.6, but caused extensive damage. However much of this occurred in areas where the significant depth of alluvium overlying bedrock amplified the ground motions.

In summary, Australian earthquakes are:

Intraplate and continental, so they are infrequent

- Most people feel earthquakes just a few times in their lifetime
- However, Australia is one of the most active intraplate areas

Distributed over many small to moderate faults

- Fault lengths rarely exceed 100 km,
- Low maximum magnitude, perhaps Mw 7.5
- Hazard is quite widely distributed

Shallow, from surface to 20 km

- Small events often felt and heard within a very limited area
- Moderate magnitudes can cause damage
- Above Mw 6 usually gives surface rupture, especially in cratonic regions

Dominant horizontal compression

- Reverse faults predominate, strong and relatively inactive, so stress levels are high
- Earthquakes may have high stress drops, giving high frequency, high acceleration, short duration motion

C2.1.4.4. Australian Earthquake Hazard Maps

As presented in ANCOLD (1998) earthquake hazard maps for Australia have changed considerably over the years as more earthquakes are recorded in an increasing seismograph recording network, and as earthquakes such as Tennant Creek have occurred in what was thought to be a low seismic hazard area.

In practice these hazard maps are of limited use for dams because they only consider the 10% chance of exceedance in 50 years, or 1 in 475 / annum hazard which is too frequent for design ground motions for most dams.

As a result site specific hazard studies are generally required for dams.

C2.1.5. Reservoir Triggered Earthquake

Reservoirs may induce seismicity by two mechanisms. Either the weight of the water may change the stress field under the reservoir, or increased ground water pore pressure may decrease the stress required to cause an earthquake. In either case, reservoir triggered earthquakes (RTE) will only occur if relatively high stresses already exist in the area. If a recent large earthquake has relieved the stress, perhaps in the last few hundred years for low seismicity areas like Australia, then RTEs are unlikely to occur.

RTE events initially occur at shallow depth under or immediately alongside a reservoir. As years pass after first filling, and groundwater pore pressure increases permeate to greater depths and distances, the events may occur further from the reservoir. This occurs at a rate of something like one kilometre per year. RTE's are experienced under new reservoirs, usually starting within a few months or years of commencement of filling, and usually not lasting for more than about twenty years. Once the stress field and the pore pressure field under a reservoir have stabilised, then the probability of future earthquakes reverts to values similar to those estimated if the reservoir had not been produced. Most of the earthquake energy does not come from the reservoir, but from the normal tectonic processes. The reservoir simply acts as a trigger.

If there is a major fault near the reservoir, RTE events can exceed magnitude 6.0 (e.g. Xinfengjiang, China, 1962, M 6.1; Koyna, India, 1967 M 6.3). Such events will only occur if the fault is already under high stress. Several Australian reservoirs may have triggered earthquakes exceeding magnitude 5.0 (e.g. Eucumbene, 1959, M 5.0; Warragamba, 1973, M 5.0; Thomson, 1996, M 5.2), and others may have triggered smaller earthquakes.

It is more common for a reservoir to trigger a large number of small shallow earthquakes, especially if the underlying rock consists of jointed crystalline rock like granite (e.g. Talbingo 1973 to 1975; Thomson 1986 to 1995). These events possibly occur on joints rather than established faults, so are limited in size, and only give magnitudes up to 3 or 4. Their shallow depth means that even events smaller than magnitude 1.0 may be felt or heard.

RTEs have been observed for over one hundred reservoirs throughout the world, and small shallow induced events have probably occurred under many others. A relatively high proportion of reservoirs with RTE seismograph networks do record such activity. A high proportion of RTE examples occur in intraplate areas, with above average rates in China, Australia, Africa and India.

It is not easy to predict whether a particular new reservoir will experience RTE because the stress and strength at earthquake depths cannot normally be measured. For the same reason, prediction of normal tectonic earthquakes has been unsuccessful in most parts of the world.

It seems that RTE with many small events is more likely to occur in intraplate areas with near-surface crystalline rocks like granite, rather than sedimentary rocks. A larger magnitude RTE event can only occur if there is an existing fault of sufficient dimension that is late in its earthquake cycle, with the stress already approaching the strength of the fault.

ICOLD (1983, 1989, 2012), conclude that there is documented evidence to prove that impounding of a reservoir sometimes results in an increase of earthquake activity at or near the reservoir. ICOLD (1983) conclude that:

- Earthquakes of magnitude 5 to 6.5 were induced in 11 of 64 recorded events.
- The greatest seismic events have been associated with very large reservoirs (but there is insufficient data to show any definite correlation between reservoir size and depth and seismic activity).
- In view of the above, a study of possible induced seismic activity should be made at least in cases where the reservoir exceeds $5x10^8 m^3$ in volume, or 100 m in depth.
- The load of the reservoir is not the significant factor; rather it is the increased pore water pressure in

faults, leading to a reduction in shear strength over already stressed faults.

ICOLD (2012) indicate that there are so far only six generally accepted cases of reservoir triggered seismicity where the magnitude of the event exceeded 5.7. The largest recorded magnitude event that is believed to be due to a reservoir-triggered event is 6.3.

C2.2Terminology

Active fault. The definition adopted for an active fault is adapted from ICOLD (2016a).

ICOLD (2016a) include "or very long faults which have moved repeatedly in Quaternary time (1.8 million years). For this guideline this is not included as such very infrequent activity would be too conservative for use in the deterministic analysis approach. These faults are included in PHSA as neotectonic faults.

The United States Nuclear Regulatory Commission (USNRC) define a capable fault as a fault which has exhibited one or more of the following characteristics:

(a) Movement at or near the ground surface at least once within the past 35,000 years or movement of a recurring nature within the past 500,000 years.
(b) Macro-seismicity instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
(c) A structural relationship to an active fault according to characteristics (a) or (b) that movement on one could be reasonably expected to be accompanied by movement on the other.

This is broadly consistent with the ICOLD (2016) definition. Active faults are included in the Probabilistic Seismic Hazard Assessment (PSHA) and in assessing the MCE for Standards based design.

Neotectonic fault. The definition is taken from Clark et al (2012). Such faults are assumed to be potentially active in assessing the PSHA. They represent faults which have ruptured in the last 5 to 10 million years.

C2.3 General Principles for Selection of Design Ground Motion and Analysis Method

There are two ways to select the design ground motion. These are related to the analysis method. They are:

(a) Deterministic Analysis. Design ground motions at the dam site are defined in probabilistic terms; and where there are active faults in the vicinity of the dam, also by the ground motion resulting from the MCE on the faults.
(b) Risk based Analysis. A risk based approach requires assessment of the seismic ground motions at the dam site up to and beyond the SEE for use in risk analyses.

The steps involved in the risk analysis process are:

(i) Determine the AEP of earthquake ground motion (P_E) over the range of earthquake events which may affect the dam. Table C2.3 gives an example for AEP vs ground acceleration.
(ii) Determine the conditional probability (P_{BC}) that for each of the ground motion ranges (e.g. 0.125 g to 0.175 g in Table C2.3) the dam will fail resulting in uncontrolled release of the reservoir. In assessing this conditional probability all modes of failure should be considered and the probabilities combined, making allowance for interdependence and mutual exclusivity or otherwise (e.g. for embankment dams, slope instability, piping, liquefaction/instability, and for concrete gravity dams, overturning, and sliding).
(iii)Assess the probability of failure for each range of ground motion by

multiplying the AEP with P_{BC} i.e. $P_B = P_E \times P_{BC}$ – see Table C2.3.

(iv) Sum the probabilities to give the overall annual probability of failure due to earthquake.

(v) Assess whether the resulting risks are tolerable.

Table C2.3. Example of assessing the probability of failure by earthquake.

Acceleration	Annual Probability (P_E)	Conditional(1) Probability (P_{BC})	P_B(2)
<0.075g	0.874	0.0005	0.0004
0.075g to 0.125g	0.100	0.005	0.0005
0.125g to 0.175g	0.015	0.05	0.0007
0.175g to 0.225g	0.007	0.1	0.0007
0.225g to 0.3g	0.003	0.3	0.0009
>0.3g	0.001	0.5	0.0005
Total	1.000		0.0037

(1) Given the earthquake occurs
(2) $P_B = P_E \times P_{BC}$

Preferred method.

The risk based approach is preferred for assessment of existing dams for the following reasons:

1. In the deterministic approach it is difficult to account for the fact that the conditional probability of failure of the dam given the SEE is not 1, and may be very different for different types of dams and foundation conditions. Hence either these conditional probabilities have to be estimated, or it has to be understood that the different dams have quite different annual probabilities of failure, even though they are assessed for the same SEE.
2. Deterministic methods potentially mask the fact that many dams have some likelihood of failure at ground motions less than the SEE.
3. Some design methods are by their nature probabilistic; e.g. liquefaction assessments, and hence inherently risk based.
4. Risk based methods are commonly used for Portfolio Risk Assessments (PRA) in Australia, and if deterministic methods are used they may result in a dam passing the deterministic method but not satisfying the PRA tolerable risk criteria.

C2.4 Description of a Probabilistic Seismic Hazard Assessment

C2.4.1 Inputs into PSHA.

(a) Earthquake Forecast (EQF)

The PSHA uses an earthquake forecast, which predicts the locations, magnitudes, and frequencies of occurrence of all earthquakes that contribute seismic hazard at the site. The earthquake forecast is based on earthquake source models, which are of two kinds: distributed earthquake sources, and fault sources. To account for epistemic uncertainty in earthquake source models, alternative viable earthquake source models should be used in a logic tree framework, with weights given to each alternative model.

(b) Ground Motion Prediction Equations (GMPE)

Ground motion prediction equations provide estimates of the ground motions at a site having specified site conditions located at a specified distance from an earthquake of specified magnitude and other source properties (such as style of faulting). They specify the ground motion by peak ground acceleration and response spectral acceleration (5% damping).

In the past, site conditions were parameterized in GMPE's as a generic geological category such as rock or soil, but most current GMPE's typically now use the time-averaged shear-wave velocity in the upper 30 metres of the bedrock profile (Vs30) that underlies the site.

In cases where soil overlies bedrock, especially where the soils are expected to have significantly non-linear behavior, non-linear site response analysis should be carried out using ground motions estimated at an "engineering bedrock" level below the soil profile by the PSHA. These analyses are carried out by the dam Consultant.

To account for epistemic uncertainty in earthquake ground motion models, alternative viable earthquake source models should be used in a logic tree framework, with weights given to each alternative model.

C2.4.2 The PSHA Process

(a) Method

PSHA is based on methodology originally proposed by Cornell (1968), outlined schematically in Figure C2.9. If seismicity is considered to follow a random Poisson process, then the probability that a ground motion, such as Spectral Acceleration (SA) exceeds a certain value (s) in a time period t is given by:

$$P(SA > s) = 1 - e^{-f(s)t} \quad \text{Equation 1}$$

where $f(s)$ is the annual mean number of events (also known as "annual probability of exceedance") in which the ground motion parameter of interest exceeds the value 's'.

Some prefer to report this in terms of computing s for a certain probability of occurrence, P, in a time period t. For example a ground motion level having a 10^{-4} / annum probability of exceedance may be expressed as equivalent to a return period of 10,000 years.

The annual probability of exceedance is calculated as follows:

$$f(s) = \sum_{i=1}^{Faults} \left(\iint_{m,r} f(m)(P(SA > s \mid m, r) P(r \mid m)\, dm\, dr \right)_i$$

Equation 2

where:

f(m) = probability density function for events of magnitude m_i (from the EQF).

P(SA>s|m,r) = probability that *SA* exceeds *s* for a given magnitude *m* and distance *r* (from the GMPE).

P(r|m) = probability that the source to site distance is *r*, given a source of magnitude *m* (from the EQF).

The probabilistic analysis is performed using a computer program of the type described further below. The computer program should allow the treatment of two types of uncertainty: epistemic uncertainty and aleatory variability.

(b) Treatment of Random Variability within the PSHA Hazard Integral

Aleatory variability (randomness) results from randomness in natural physical processes that coexist in nature. The size, location, and occurrence time of the next earthquake on a fault, and the site-to-site random variability in the ground motion level at a given distance from a given earthquake (shown in Box 4 of Figure C2.9) are examples of quantities considered aleatory. In current practice, these quantities cannot be predicted, even with the collection of additional data. Integration over aleatory variabilities is carried out within the hazard curve (Equation 2) to yield a single hazard curve.

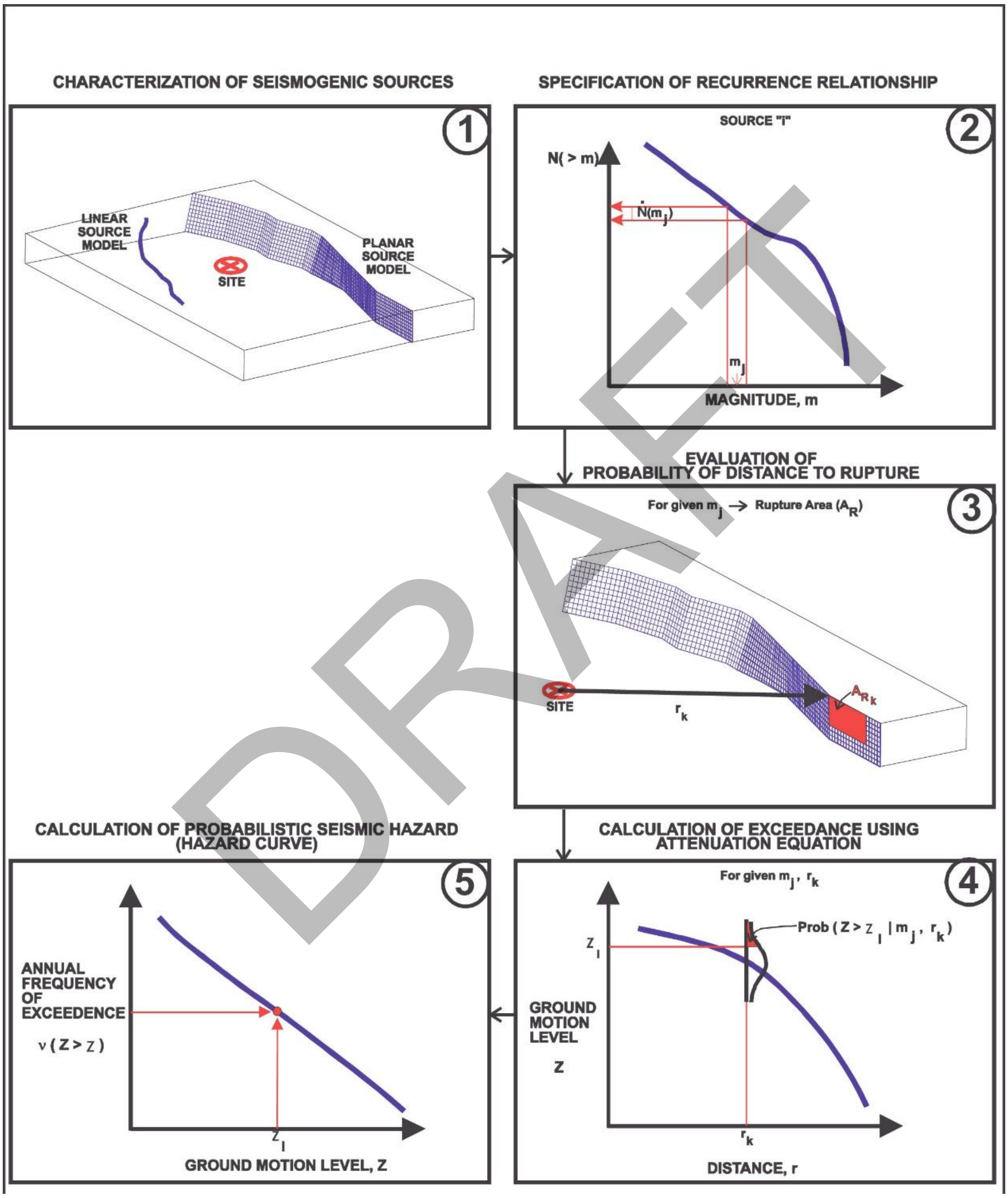


Figure C2.9 Schematic diagram of probabilistic seismic hazard analysis. The attenuation equations shown in Box 4 are also called ground motion prediction equations.(Source: Paul Somerville)

(c) Treatment of Epistemic Uncertainty outside the PSHA Hazard Integral

Epistemic uncertainty results from imperfect knowledge about the process of earthquake generation (e.g. in alternative viable earthquake forecast models) and the assessment of their effects (e.g. in alternative viable ground motion prediction models). Viable alternatives are mutually exclusive, and are treated using logic trees outside the PSHA hazard integral. Epistemic uncertainties are thus expressed by incorporating multiple hypotheses, models, or parameter values. These multiple interpretations are each assigned a weight within the logic tree framework, resulting in a suite of hazard curves and their associated weights. Examples of logic trees for distributed earthquake sources and fault sources are shown in the upper and lower parts of Figure C2.10.

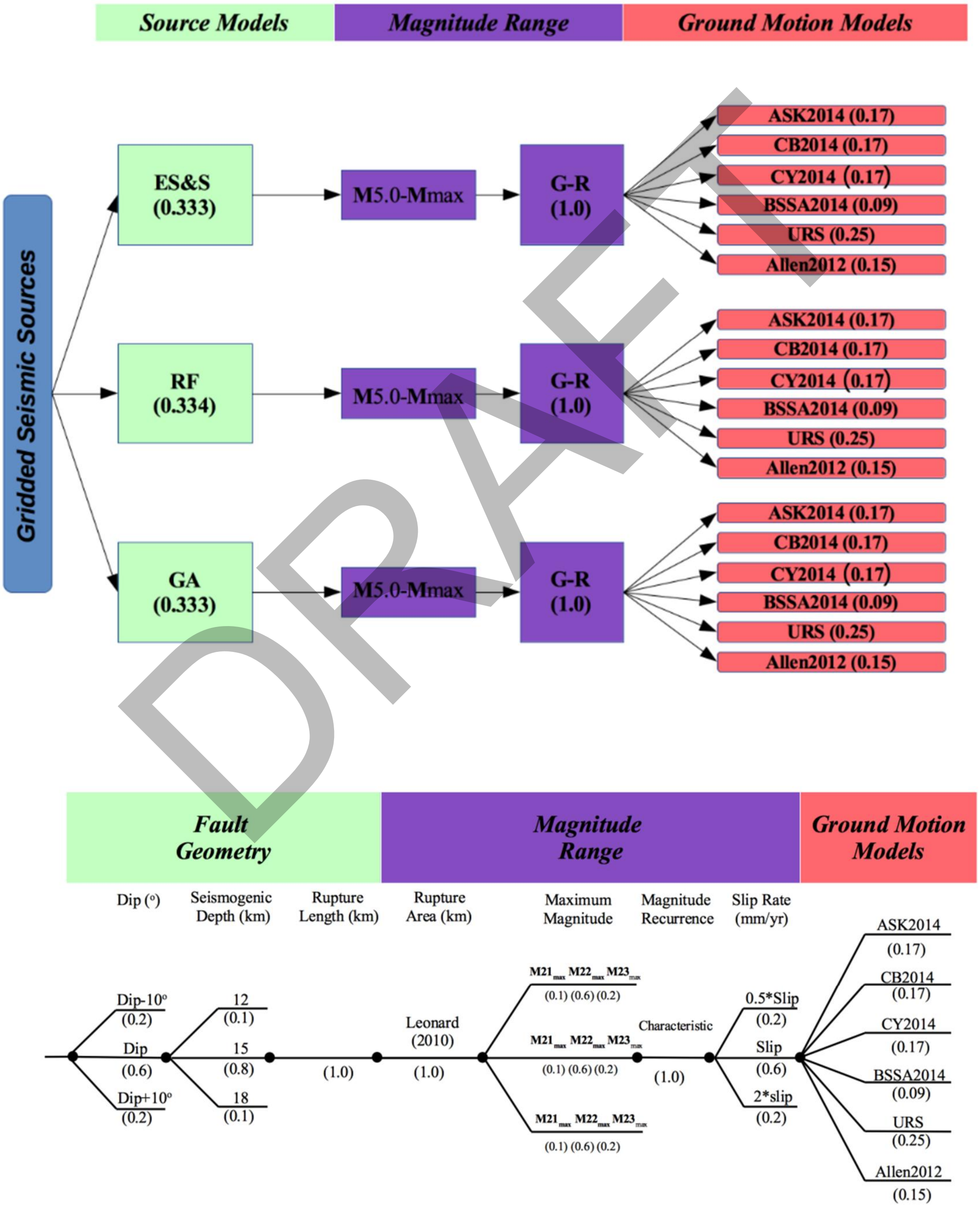


Figure C2.10. Example logic trees for the treatment of epistemic uncertainty in distributed earthquake sources (top) and fault sources (bottom) in PSHA. (Source: Paul Somerville)

C2.4.3 Products of PSHA and their analysis.

(a) Hazard Curves

The PSHA estimates the ground motion level as a function of annual probability of exceedance, or return period. Figure C2.11 shows peak ground acceleration hazard curves for a group of distributed earthquake sources and fault sources, shown by the colored lines, with the black curve showing the combined total hazard.

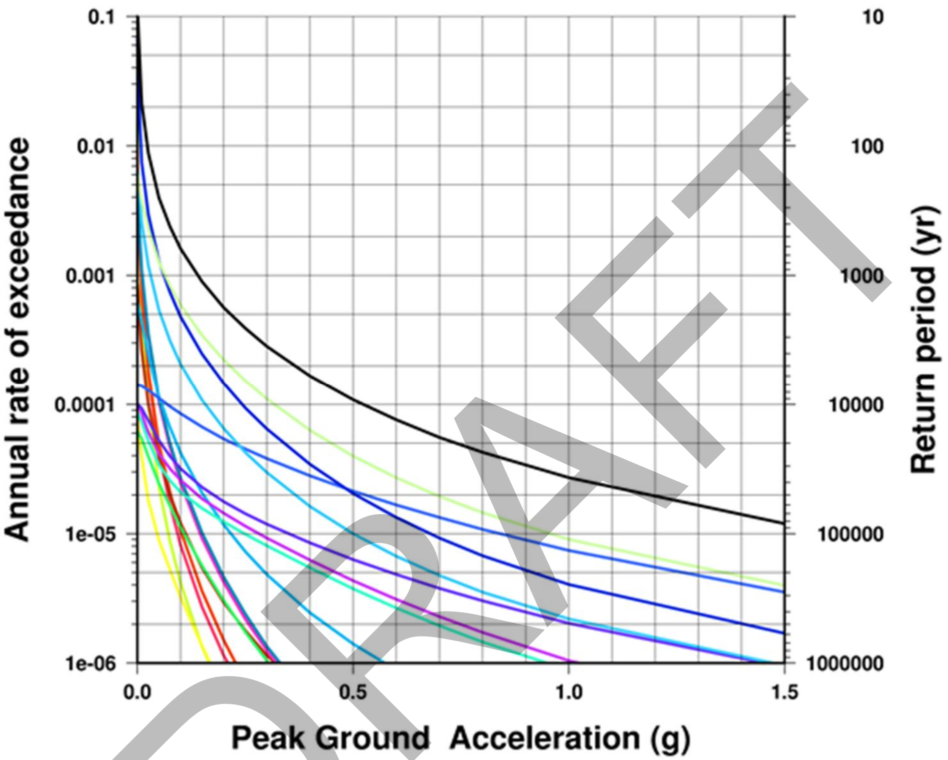


Figure C2.11 Example hazard curve for a group of distributed earthquake sources and fault sources, with the uppermost black curve showing the combined total hazard. (Source: Paul Somerville)

(b) Uniform Hazard Response Spectra (UHS)

Uniform hazard curves for a set of return periods are obtained from the hazard curves for each of a set of ground motion periods, including PGA, which is equivalent to the zero period response spectral acceleration (Figure C2.12).

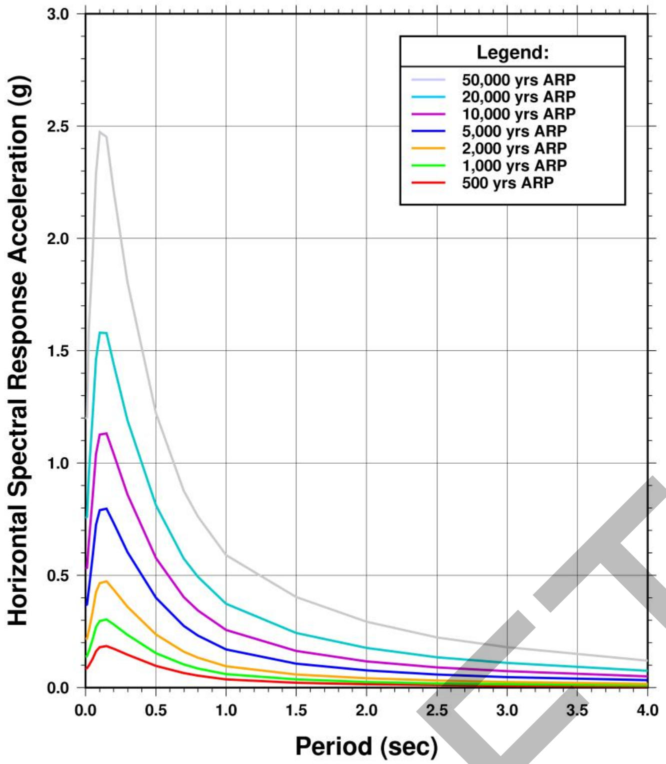


Figure C2.12. Equal hazard response spectra for a range of return periods.

(c) Fractiles of the Uniform Hazard Response Spectrum (UHS)

There is epistemic uncertainty in the true value of the seismic hazard results due to uncertainty in the true state of nature. In the example shown in Figure C2.13, three distributed earthquake source models (AUS5, RF and GA) were used because of uncertainty about the degree to which each is correct. Similarly, six ground motion models were used because of uncertainty about the degree to which each is correct. This kind of uncertainty is modeled using logic trees, with the alternative branches (corresponding to the alternative choices) being given weights. The seismic hazard is calculated for each of these branches, in this case, for each of the eighteen combinations of distributed earthquake source model and ground motion model. As a result, eighteen seismic hazard curves are obtained. The usual practice in seismic hazard analysis is to use the mean of these hazard curves, to ensure that large hazard values are given appropriate weight.

The fractiles of these multiple hazard curves are used to quantify the epistemic uncertainty in the true value of the hazard. Figure C2.13 shows the 95th, 85th, 50th (i.e. median), 15th, and 5th percentiles of the hazard for a given return period, as well as the mean hazard. The mean hazard is generally slightly higher than the median. The fractiles represent the degree of certainty that the true value of the hazard does not exceed the value given by the fractile. For example, there is a 50% chance that the true value of the hazard exceeds the median (50th fractile) value, and a 15% chance that it exceeds the 85th fractile.

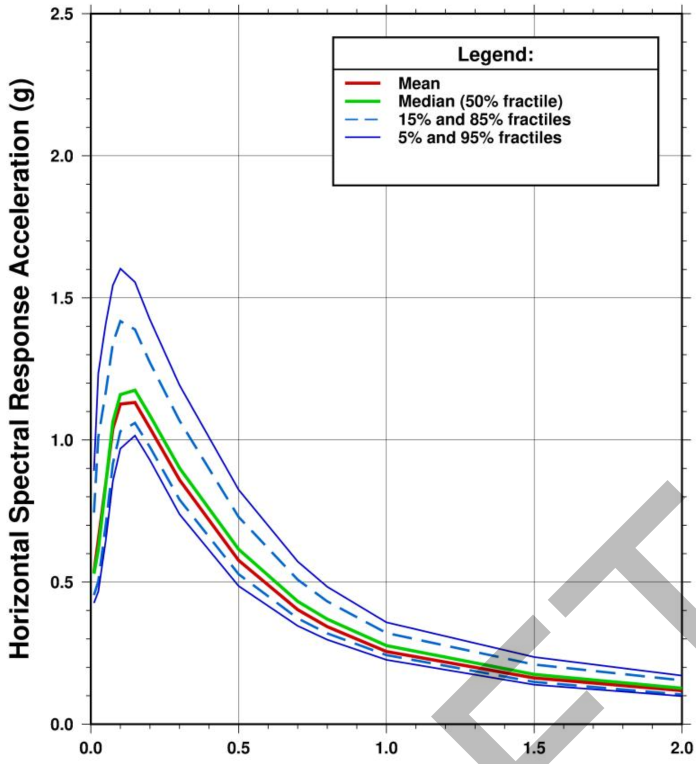


Figure C2.13. Fractiles of the hazard for a return period of 10,000 years (10^{-4}/annum)

(d) De-aggregation of the UHS

The probabilistic UHS obtained through PSHA takes account of all of the earthquake scenarios that could affect the site, and describes the ground motion level that corresponds to a specified return period or annual probability of exceedance. The UHS represents the contributions from many different earthquakes, perhaps including small nearby earthquakes and more distant larger earthquakes, and may not be a realistic representation of the response spectrum of any individual scenario earthquake. For this reason, if it is desired to use ground motion time-histories to represent the response spectrum, it is then necessary to identify one or more scenario earthquakes that dominate the hazard for that return period in the ground motion period range of importance for the structure. This process, termed de-aggregation of the UHS, results in one or more earthquake scenarios, each having a specified magnitude, distance, and severity (described by the parameter epsilon).

The de-aggregation of the hazard varies with the return period and the ground motion period of interest, as shown in Figure C2.14. For short return periods, the hazard tends to be dominated by smaller, more distant earthquakes (top row of Figure C2.14) while for long return periods the hazard tends to be dominated by larger, more nearby earthquakes (bottom row). For short ground motion periods, the hazard tends to be dominated by smaller earthquakes (left side of Figure C2.14) while for long ground motion periods the hazard tends to be dominated by larger earthquakes (right side). Selection of time-histories therefore needs to take account of the return period of the UHS and the ground motion period of the structure that is to be analysed.

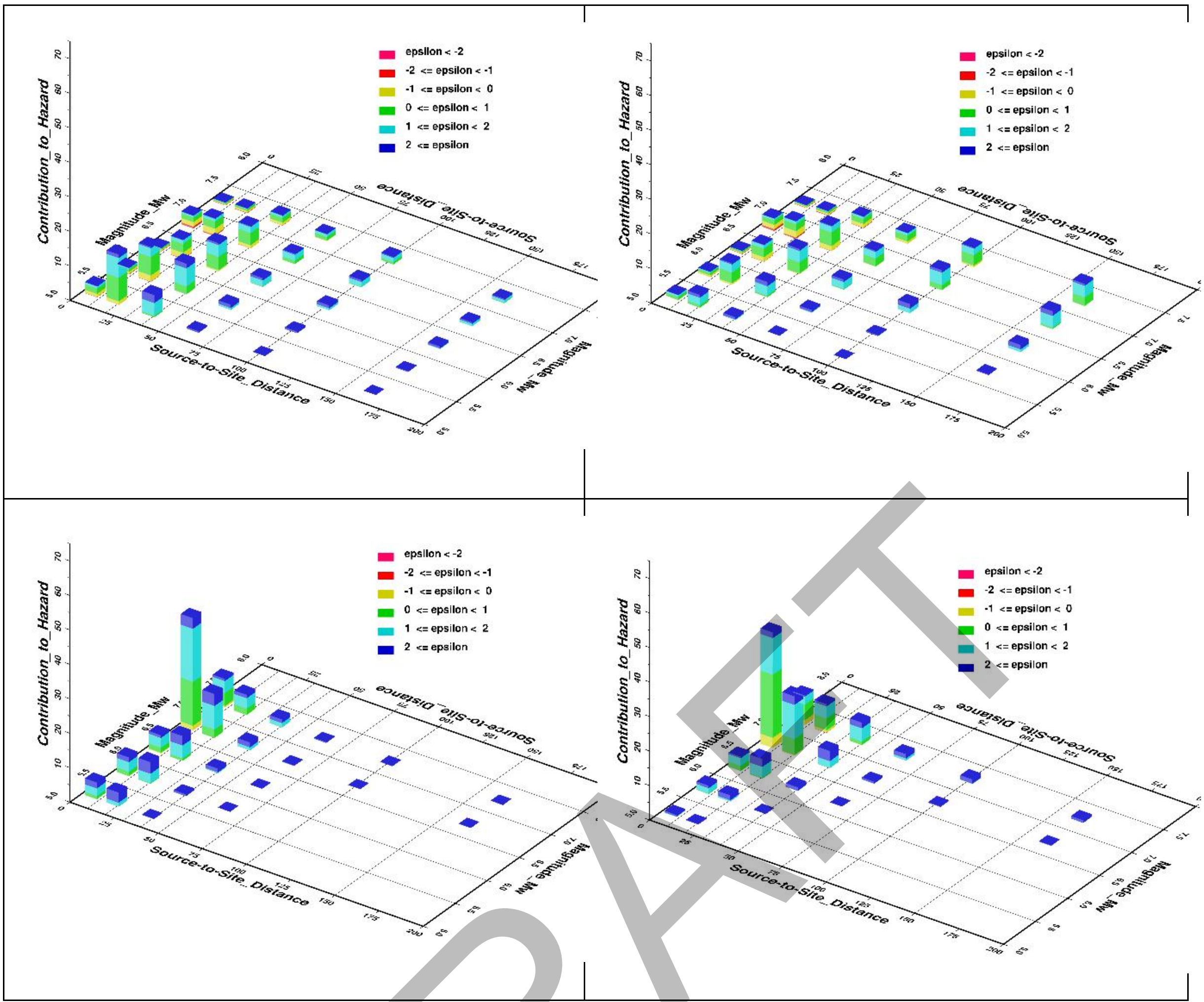


Figure C2.14. De-aggregation of the hazard for peak acceleration (left) and 1 s response spectral acceleration (right) for return periods of 500 years (top) and 10,000 years (bottom).

The de-aggregation of the UHS is used whenever it is necessary to represent the design response spectrum by a single earthquake. This is done whenever dynamic analyses of a structure are performed using a ground motion time-history.

We next consider the suitability of the UHS as a representation of the response spectrum of a single earthquake time-history. We show that there are two sources of over conservatism in the use of the UHS as a representation of a single earthquake time-history. In both cases, the UHS is too “broadband” (it is large over a range of periods that is unrealistically broad), rendering it sub-optimal for use as a target for scaling or spectral matching of time-histories to represent the design ground motions.

(e) Scenario Spectrum in Place of the UHS

As discussed above the UHS represents the contributions from many different earthquakes, perhaps including small nearby earthquakes and more distant larger earthquakes, and may not be a realistic representation of the response spectrum of any individual scenario earthquake. For example, small nearby earthquakes are expected to have relatively high ground motion levels at short periods and relatively low ground motion levels at long periods. Conversely, large distant earthquakes are expected to have relatively low ground motion levels at short periods and relatively high ground motion levels at long periods.

Instead, it is preferable to use ground motion time-histories that are scaled or spectrally matched to a set of scenario spectra representing a small set of magnitude and distance combinations that dominate de-aggregation of the UHS. These scenario spectra are in turn scaled so that they are enveloped by the UHS and collectively represent that broadband spectrum. One scenario from a small nearby earthquake may represent the short period part of the UHS, while another scenario from a larger more distant earthquake may represent the long period part of the UHS.

(f) Conditional Mean Spectrum in Place of the UHS

There is another source of over conservatism in the UHS that may become important when the scaled scenario spectrum lies considerably above the median level for that earthquake scenario. Baker (2011) showed that the UHS conservatively implies that large-amplitude spectral values will occur at all periods within a single ground motion time-history. An alternative, termed a Conditional Mean Spectrum (CMS), provides the expected (i.e., mean) response spectrum, conditioned on the occurrence of a target spectral acceleration value at the period of interest (Baker and Cornell, 2006; Baker, 2011). Baker (2011) shows this spectrum to be the appropriate target response spectrum for the goal described above, and it is thus a more appropriate target for scaling and spectrally matching ground motion time-histories as input to dynamic analyses. Baker (2011) demonstrates that the CMS spectrum maintains the probabilistic rigour of PSHA, so that consistency is achieved between the PSHA and the ground motion selection. This enables quantitative statements to be made about the probability of observing the structural response levels obtained from dynamic analyses that use this spectrum; in contrast, the UHS does not allow for such statements (Baker, 2011). Figure C2.15 shows an example.

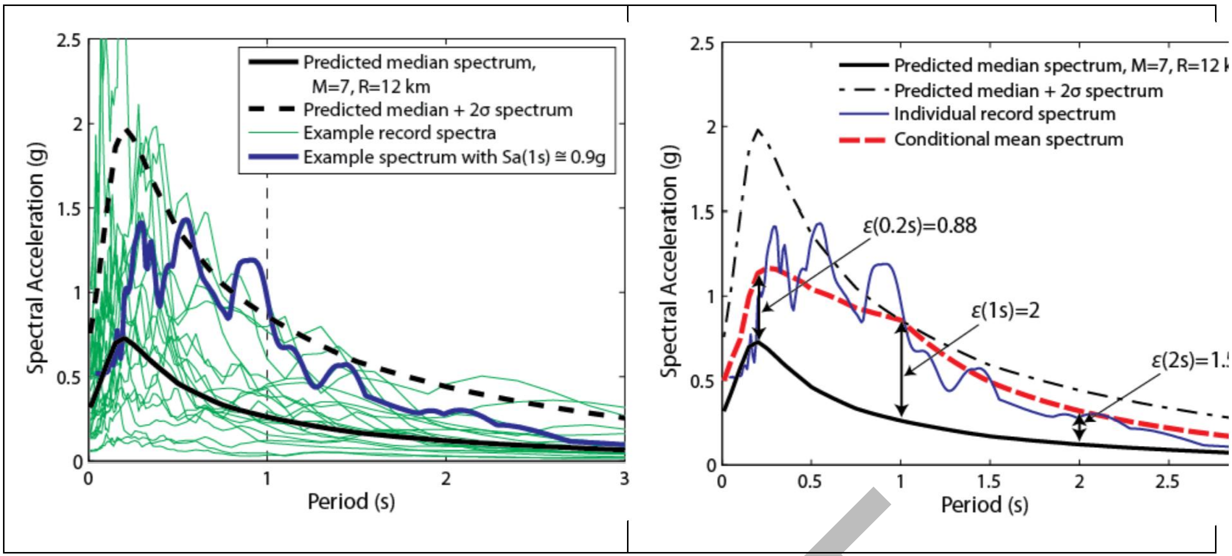


Figure C2.15 Left: Median (solid line) and median plus two standard deviations (dashed line) design spectrum for a magnitude 7 earthquake at a closest distance of 12 km, shown with recorded response spectra (green), one of which (blue) is close to the design spectrum at a period of 1 second. Right: Derivation of the conditional mean spectrum for a period of 1 second from a scenario spectrum. The conditional mean is derived from the correlation between response spectral values at adjacent periods. (Baker, 2010).

(g) Vertical Response Spectrum.

In dam engineering, the horizontal components of ground motions usually dominate the design. Accordingly, the vertical component of ground motion that is used in design should be compatible with the horizontal components; it should represent the characteristics of the vertical component that are expected to occur in conjunction with the horizontal components. Therefore, the vertical component is not derived from the PSHA – that would represent earthquake magnitudes and distances that are generally quite different from those of the horizontal component. Instead, the vertical component is derived from the de-aggregation of the horizontal component so that the vertical response spectrum represents the same combination of earthquake magnitude and distance as does the dominant contribution to the horizontal spectrum. This is accomplished by calculating the horizontal response spectrum using the de-aggregated magnitude and distance from the PSHA, and then scaling that spectrum by the vertical/horizontal ratio of a model such as Gulerce and Abrahamson (2011).

C2.5 Requirements of a Seismic Hazard Assessment

1. *Distributed Earthquake source models.*

In current earthquake source models for Australia, distributed seismicity is modelled in quite different ways that give rise to significant epistemic uncertainty. Distributed seismicity is represented by discrete seismic source zones (Brown and Gibson, 2004); by spatially smoothed seismicity (Somerville et al., 2009), and by a layered seismicity approach (Burbidge and Leonard, 2011), as described in more detail below. It is necessary to include these alternative source models in the seismic hazard analysis in order to account for this epistemic uncertainty. None of these three source models contains any fault sources; these need to be treated separately.

ES&S AUS5 Seismic Source Zone Model

The Environmental Systems and Services (ES&S) AUS5 source zone model originally developed by Brown and Gibson (2004) uses geological and geophysical criteria in combination with historical seismicity to identify zones of uniform seismic potential, and then uses historical seismicity to characterise the seismic potential of each zone by means of the a-values and b-values of the Gutenberg-Richter earthquake recurrence model, together with an estimate of the maximum magnitude of earthquakes in each zone. This approach has the advantage of allowing for the incorporation of geological and geophysical information as well as seismicity data in the identification of seismic source zones.

Risk Frontiers Spatially Smoothed Seismicity Model

Judgment is required in defining the source zone boundaries of models such as Brown and Gibson (2004), and it is unclear what would cause abrupt changes in seismicity levels across source zone boundaries. These considerations motivate the use of spatially smoothed historical seismicity to define the earthquake forecast, developed by Risk Frontiers (Hall et al., 2007). This approach gives a spatially continuous source model without boundaries except in b-value. The spatial smoothing approach has the advantages of simplicity and of avoiding uncertainty in the geological definitions of zones, but has the disadvantage of not making use of potentially informative geological data.

GA Layered Seismicity Model

Leonard et al. (2012) developed an earthquake source model for Australia that is based entirely on historical seismicity. They identify a set of background zones, a set of regional zones having higher seismic activity rates than the background zones, and hotspots that contain concentrations of earthquake activity within regional source zones. The 2012 Australian Seismic Hazard Map (Burbidge and Leonard, 2011) is composed of three layers of seismicity.

- Background Source Zones. There are two of these, one for the cratonic region of the western part of Australia, and another for the non-cratonic region of the eastern part of Australia. For each region there are two models: one that reflects the actual data and another that uses a minimum value (floor).
- Regional Source Zones. These represent the long-term earthquake activity above the background level in specified broad regions
- Hotspot Source Zones. These represent the short-term earthquake activity, including earthquake swarms and clusters, above the background and regional source zones, that have been occurring recently in local zones

Burbidge and Leonard (2011) propose a number of different ways in which the hazard maps from each layer can be combined. One way is to use the highest hazard value from among the three different layers at each site. In this approach, the total integrated seismic moment across Australia is not conserved, but the hazard in the areas of low seismic activity is allowed to increase without decreasing the hazard in areas of high seismic activity. This approach may be justifiable for application to the new building code provisions (AS1170.4), but it is not suitable for site-specific studies such as those for dams, where rigorous probabilistic analysis of hazard and risk is required. In applications to dams, it is preferable to use the version of the Background Source Zones that does not contain a floor (minimum value), together with the Regional Source Zones. If any of the Hotspot Source Zones contributes significantly to the seismic hazard at the site, they should also be included as source zones.

The three distributed earthquake source models all specify the maximum earthquake magnitudes in their recurrence models. The ES&S and RF models both assume a maximum earthquake magnitude of 7.5. The GA model has maximum magnitudes ranging from 7.3 to 7.7.

As shown in Figures C2.16 and C2.17 the models can give significantly different seismic hazard in some areas in Australia, and there is no systematic outcome, with one or the other model giving higher hazard. In view of this any seismic hazard assessment which uses only one source model is potentially unreliable.

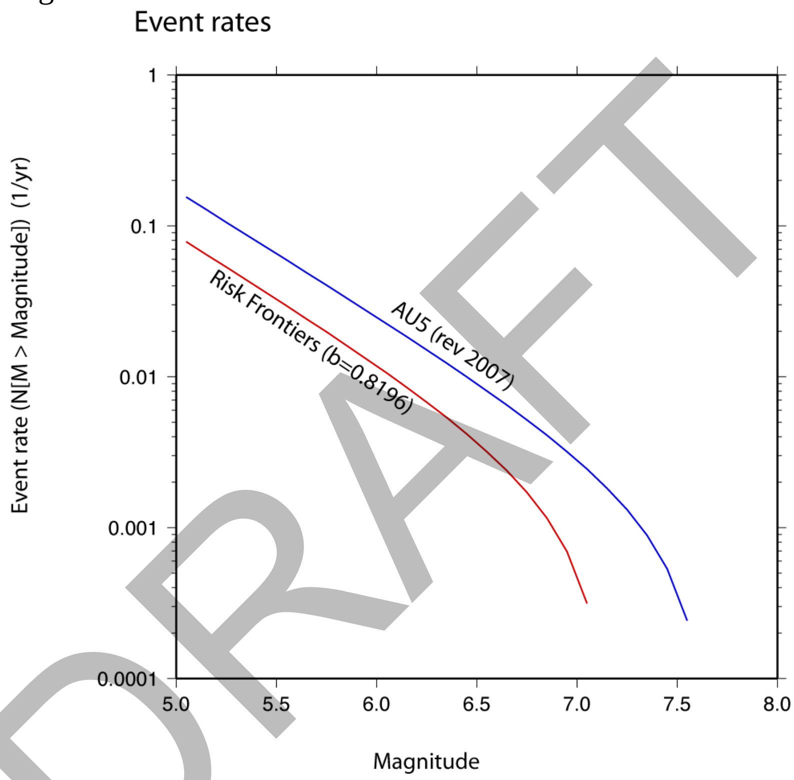


Figure C2.16. Example of differences in event rates for seismic hazard assessment from AUS5 discrete seismic source zones model and Risk Frontiers spatially smoothed seismic source model.

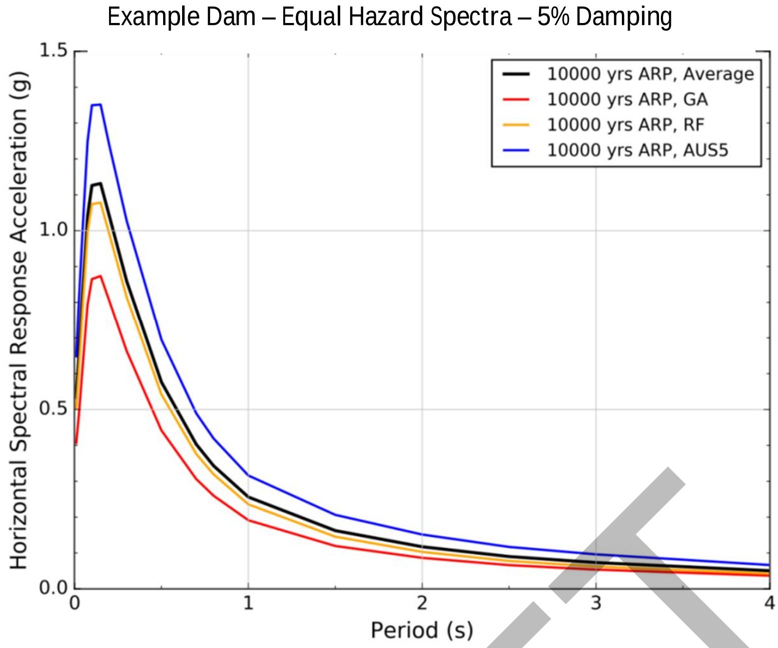


Figure C2.17 Seismic Response Spectra using the AUS5 and GA discrete seismic source zone models and the Risk Frontiers spatially smoothed seismic source model, and their average.

2. *Fault sources.*

(a) Active and Neotectonic Faulting in Australia

Fault sources here means active faults and neotectonic faults as defined in Section 2.2.

The long term uplift and subsidence caused by active faulting in geological time has a significant effect on drainage patterns in many parts of Australia, with the result that dams are frequently located in close proximity to active or neotectonic faults.

An Australia-wide assessment of faulting was made by Clark et al. (2011; 2012). They analysed a catalogue of over 200 neotectonic features, 47 of which are associated with named fault scarps. The data were derived from analysis of DEMs, aerial photos, satellite imagery, geological maps and consultation with state survey geologists and a range of other geoscientists. Verifying the features as active as defined for neotectonic faults is an ongoing process. The catalogue varies in completeness because sampling is biased by the available data bases, the extent of unconsolidated sedimentary cover, and the relative rates of landscape and tectonic processes.

Following Clark et al. (2011) and Sandiford et al. (2004), Seismologists consider neotectonic faults as potentially active if they have undergone displacement under the current stress regime in Australia, and hence may have the potential for displacement in the future. The age of the current stress regime in Australia is estimated to lie in the range of 5 to 10 million years (Sandiford et al., 2004).

In Australia, geological maps typically show numerous faults but do not indicate whether they are active in the current stress regime; most of them are probably not. For example, if these faults were previously active under a different stress orientation, it is possible that they are unfavorably oriented to undergo slip under the current stress orientation. However, if they are favourably oriented, then consideration should be given to the possibility that they have been reactivated under the current stress regime.

Clark et al (2011) list a number of faults which have been investigated and found to have had multiple surface rupture events over intervals of hundreds of thousands of years, with intervals of a few tens of thousands of year between event. In some cases these faults undergo a long period of quiescence, and may currently be in a quiescent phase. However, others may have had a large event within the past 35,000 years or may have the potential to generate a large earthquake within a 35,000 year time frame, and so may fit the definition of active fault that these guidelines has adopted. Figure C2.18 summarizes data for some of these faults which have resulted in surface rupture. Others which did not result in surface rupture are listed within the text of Clark et al (2012).

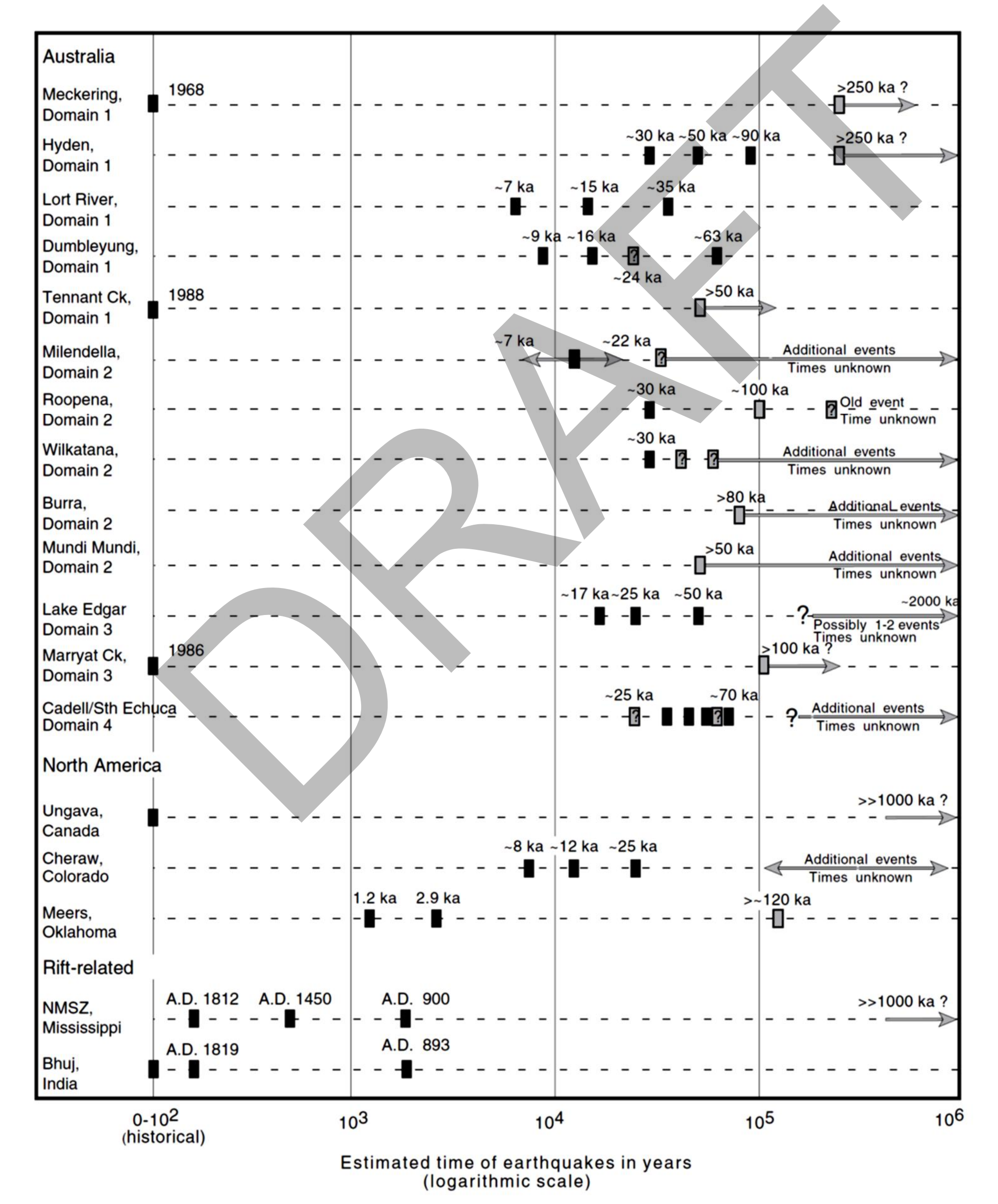


Figure C2.18 Compilation of surface breaking earthquake recurrence data. (Clark et al. 2012)

(b) Alternative Approaches to Incorporating Active and Neotectonic Faults

In view of the short time span of the historical earthquake catalogue in Australia, it is unclear how best to incorporate active and neotectonic faults into the earthquake forecast.

Brown and Gibson (2000) proposed a method which subtracts fault-related seismicity from the area source zone in which the fault occurs, and insert a fault source having that seismicity, using a Gutenberg-Richter recurrence model. This approach assumes that the fault seismicity is represented in the background seismicity of the area source.

An alternative approach (Somerville et al, 2008) is to add a fault source whose seismicity is based on slip rate, without modifying the background seismicity of the area source, using a characteristic earthquake recurrence model. This approach assumes that the fault seismicity is not represented in the distributed source zones, consistent with the Characteristic earthquake recurrence model, which is described next. This approach requires an estimate of the slip rate of the fault. The episodic nature of large earthquake ocurrence, described below, necessitates the careful consideration of the time period over which the slip rate of the fault should be estimated.

(c) Alternative Earthquake Recurrence models

The distribution of earthquake magnitudes in distributed earthquake source zones is usually assumed to follow the Gutenberg-Richter model (Figure C2.19, top left). However, the distribution of earthquake magnitudes on discrete active and neotectonic faults may be better represented by the characteristic recurrence model (Schwartz and Coppersmith, 1984), in which most of the fault slip is taken up in large earthquakes (Figure C2.19, bottom left and right).

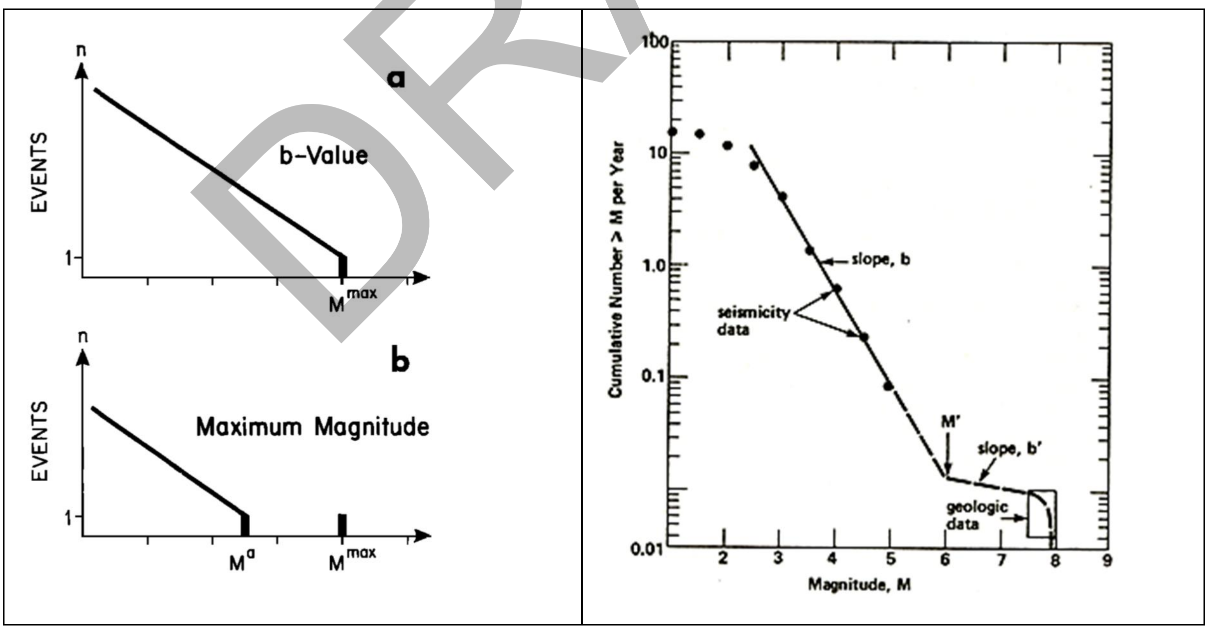


Figure C2.19. Left: Interval recurrence for the Gutenberg-Richter (top) and Characteristic earthquake recurrence models (bottom). (Wesnousky et al., 1983). Right: Cumulative recurrence for the Characteristic earthquake recurrence model. Source: Schwartz and Coppersmith, 1984.

If the characteristic recurrence model applies, then the recurrence rate of large earthquakes may be underestimated by the Gutenberg-Richter model that is derived from historical seismicity if it only contains small earthquakes, as indicated on the right side of Figure C2.19. For active faults, it is preferable to estimate the recurrence rate of large earthquakes from geological data, such as fault slip rates, rather than historical seismicity, because it is unclear whether the earthquake recurrence of fault sources follows the Gutenberg-Richter model.

(d) Alternative Recurrence Behaviour of Active Faults

Clark and Van Dissen (2006) and Clark et al. (2012) found that earthquake activity on faults in Australia is episodic, with clusters of earthquakes on a given fault occurring close together in time (several tens of thousands of years), separated by longer periods (several hundreds of thousands of years) of no large earthquake activity. This is inconsistent with the random temporal (Poisson) distribution of earthquakes that is usually assumed in seismic hazard analysis. Using the results of Clark et al. (2006), it may be possible to identify which faults are currently in an active phase and which are currently in an inactive phase. This could then be applied to the evaluation of the seismic potential of active faults in seismic hazard evaluations.

Clark et al. (2011) reviewed knowledge pertaining to the seismogenic deformation of the Australian continent over the last 5-10 Ma (the Neotectonic Era). Based upon perceived differences in character of the seismogenic faults across the continent, and guided by variations in the geologic and geophysical makeup of the crust, they propose six onshore neotectonic domains. A seventh offshore domain was defined based upon analogy with the eastern United States. These domains are characterised by different earthquake recurrence behaviour.

In practice unless there is definitive information to the contrary, faults identified as active as defined in Section 2.2 should be considered still active. If there are differences of view from Seismologists as to whether a fault is still active this can be managed by assigning a conditional probability to the occurrence of the earthquake motion to reflect these opinions.

(e) Modelling the Maximum Credible Ground Motion at the Dam site from an Active Fault

Where an active fault has been identified which could result in significant ground motions at the dam site its contribution to ground motion will have been considered in the PSHA as described above.

As detailed in Table 2.1 of Section 2.6, for deterministic seismic hazard for Extreme Consequence Category dams the maximum credible ground motions which might occur at the dam site from rupture of active faults are required

There are however uncertainties in doing this including whether the whole fault or only part ruptures: the magnitudes of the resulting earthquakes which might occur; the location of the focus of the earthquake; the ground motion models to be adopted. These should be discussed with the Seismologist so the Owner and Consultant understand the potential degree of conservatism inherent in the ground motion assessment.

It should be noted that in cases where the active fault is close to the dam site these ground motion estimates are likely to be greater than obtained from PSHA. This is because the PSHA will have assigned an annual probability to the occurrence of earthquakes on the fault while the process above takes no account of this. The process can be considered as equivalent to Probable Maximum Flood estimate which also has no annual probability assigned to it.

3. *Faults in the dam foundation*

In practice only active faults are likely to be of significance to the dam. However there may be neotectonic faults which warrant further investigation to determine if they are active.

4. *Ground motion prediction models*.

There are insufficient strong motion recordings from earthquakes in stable continental regions anywhere in the world, including Australia, to form a basis for the development of ground motion prediction models using regression analysis of recorded strong ground motions. Such analyses have only been feasible for crustal earthquakes in tectonically active regions, and while the ground motion prediction equations derived this way have been used extensively in Australia, the applicability of these models in Australia is still not well established. Accordingly, recent investigators (Liang et al., 2008; Somerville et al., 2009; and Allen, 2011), described further below, have used seismologically based methods to develop ground motion prediction models for Australia. In view of the significant differences in the ground motions that are predicted by the different ground models, it is necessary to include alternative ground motion prediction models in the seismic hazard analysis in order to account for epistemic uncertainty in which of these models is more applicable in Australia. The following briefly summarizes the available models (in 2016).

Liang et al. (2008).

Liang et al. (2008) estimated strong ground motions in southwest Western Australia using a combined Green's function and stochastic approach. This model is applicable to the Yilgarn craton, and may also be applicable to other cratonic regions of Australia, but is not applicable to non-cratonic regions, including Perth. This model uses epicentral distance as the distance measure. Figure C2.20 shows the crustal domains in Australia as defined by Clark et al (2012).

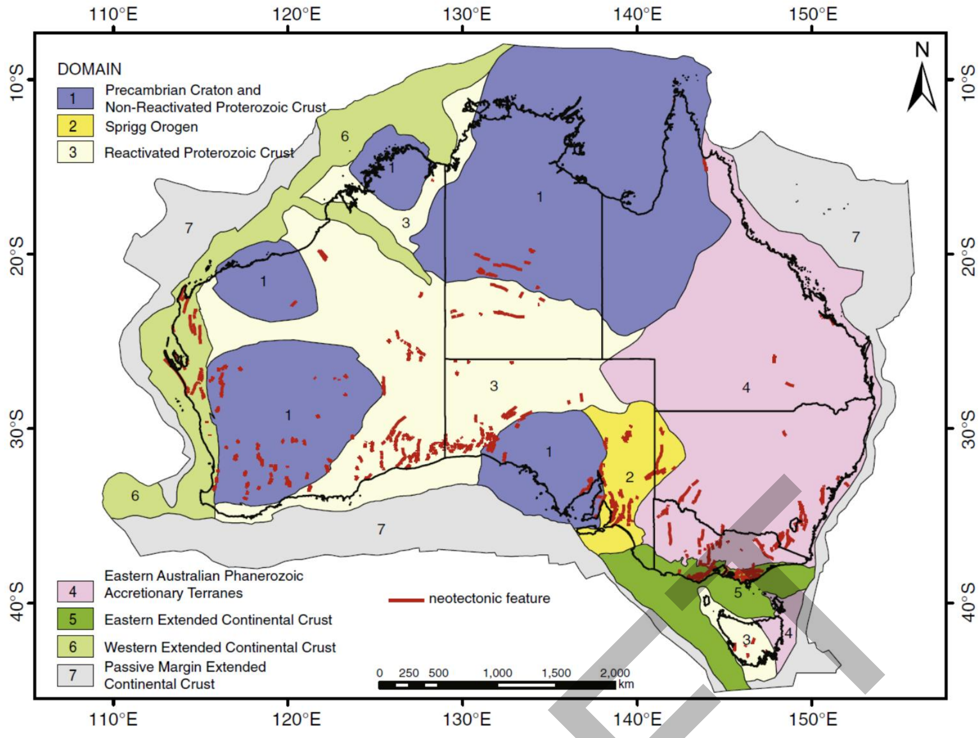


Figure C2.20 Neotectonic domains map of the Australian continent. Neotectonic features represent known or suspected features, primarily topographic fault scarps. (Clark et al, 2012)

Somerville et al. (2009).

Somerville et al. (2009) demonstrated their ability to simulate the recorded ground motions of small earthquakes that occurred in Eastern and Western Australia, and developed earthquake source scaling models for Australian earthquakes based on earthquake source modelling of the Mw 6.8 1968 Meckering and the Mw 6.25, 6.4 and 6.6 1968 Tennant Creek earthquakes. They then used a broadband strong ground motion simulation procedure based on the elasto-dynamic representation theorem and Green's functions calculated from crustal structure models for various regions of Australia to calculate ground motions for earthquakes in the magnitude range of 5.0 to 7.5. These ground motions were then used to develop ground motion prediction equations, which were checked for consistency with available data from Australian earthquakes at each step. These ground motion models predict response spectra for two crustal domain categories: Cratonic Australia and Non-Cratonic Australia. The cratonic regions of Australia include much of Western Australia (but not the coastal strip west of the Darling Fault, including Perth); south-central South Australia (including the site); the northern part of the Northern Territory; and north western Queensland (Clark et al, 2011). Non-Cratonic Australia consists of the remainder of Australia, including Eastern Australia and part of the coastal margin of Western Australia, which includes all of the state capital cities, is on Non-Cratonic Australia.

Allen (2012).

Allen (2012) developed a ground motion model for south eastern Australia based on the stochastic model, having calibrated the parameters of the stochastic model using recordings of small earthquakes in south eastern Australia (SEA). The source and attenuation parameters provided in Allen et al. (2007) were first reviewed and modified in light of additional available data and more rigorous statistical analysis. The dependence of stress drop on earthquake depth was examined, and options were provided for variable stress parameter values in the ground motion prediction equation. The near-surface, path-independent diminution parameter immediately beneath the station, k0, (Anderson and Hough, 1984; Campbell, 2009; Van Houtte et al., 2011) was also examined for average station conditions in SEA, in addition to the parameter's correspondence with a limited dataset of average shear-wave velocity measurements in the upper 30 m (Vs30) at seismic recording stations across Australia. These updated source and attenuation parameters were used as inputs to the stochastic finite-fault software package, EXSIM (Motazedian and Atkinson, 2005; Atkinson and Boore, 2006). Five percent damped response spectral accelerations were simulated for earthquakes of moment magnitude Mw 4.0 to 7.5. These stochastic data were then regressed to obtain model coefficients and the resulting ground motion prediction model was evaluated against recorded response spectral data for moderate-magnitude earthquakes recorded in south eastern Australia.

PEER-NGA West 2.

These are the most recently developed ground motion models for shallow crustal earthquakes in tectonically active regions. They were developed by five groups: Abrahamson et al. (2014), Boore et al. (2014); Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014), and are compared by Gregor et al. (2014). In view of the great care that was put into documenting the NGA West 2 metadata that describe the strong motion recordings, the vastly larger size of the data set that has been used, and the diligence that has been applied by the modellers, the NGA Program has resulted in a set of ground motion models that have a much more substantial basis than the earlier 1997 generation of models (Abrahamson and Shedlock, 1997).

Selection of Ground Motion Models for use in Australia

The NGA West 2 models are based mostly on a large global set of recorded ground motion data from tectonically active regions, but none of those data are from Australia. The Allen (2012) and Somerville et al. (2009) ground motion prediction models were both developed for Australia, but are not based on a large set of recorded strong motion data from Australia.

In view of this the NGA West 2 model should be included.

Judgments need to be made as to the relative weights that should be given to the Australian and global models. This is a matter for the Seismologist to determine, in consultation with the Owner and the Consultant, taking account of whether the site is in cratonic or non-cratonic conditions.

It can be expected that new models will be developed for Australian and relevant International conditions and given they are rigorously peer reviewed, they may be included in the PSHA with appropriate weighting.

Impact of Site Conditions on Ground Motion Level

Ground motion prediction models used in earthquake engineering are based on three main parameters: the magnitude of the earthquake, the distance of the earthquake from the site, and the site characteristics. It has long been known that site characteristics have a strong influence on ground motion level. Until recently, site characteristics have been represented by broad geological categories such as "rock" or "soil." In eastern Australia, it has been common to assume that the site characteristics of dam abutments can be represented by the "rock" site category in ground motion models such as Sadigh et al. (1997). However, this ground motion model was found in the course of the NGA Project to be representative of soft rock sites in California having an average shear wave velocity (Vs30) of only 520 m/sec, while many dams in Australia may be founded on hard rock having Vs30 of 1,000 m/sec or more. In view of this, re-evaluation of the seismic hazard which was carried out using Sadigh et al (1997) may identify a significant level of conservatism in those hazard assessments for those dams located on hard rock foundations.

Ground motion models, such as the NGA West 2 models (Gregor et al., 2014) have been developed that quantify site characteristics in a much more rigorous way. Specifically, these models specify the site characteristics using Vs30, which is the average shear wave velocity in the uppermost 30 meters below the ground surface. Amplification of ground motion is inversely proportional to Vs30. The amplification is roughly equal to the square root of the ratio of subsurface to surface shear wave velocity, and illustrated in Figure C2.21. Although Vs30 is not yet routinely measured in the foundation investigations for dams, it can usually be inferred from the P-wave velocities obtained from seismic refraction surveys which were often carried out as part of the original site investigation for the dam site.

The ground motion models for Australia that were developed by Somerville et al. (2009) and Allen (2011) do not have Vs30 as a variable, and instead are for rock site conditions. The Vs30 used by Somerville et al. (2009) is 865 m/sec, which is consistent with average rock site conditions in Australia. Similarly, the Vs30 of the Allen (2011) model is assumed to represent a Vs30 of 820 m/sec. The site amplification model of Campbell and Bozorgnia (2008) can and should be used to adjust the shear wave velocities represented in these models to that of a specific site where the Vs30 of the site is significantly different to that in the models.

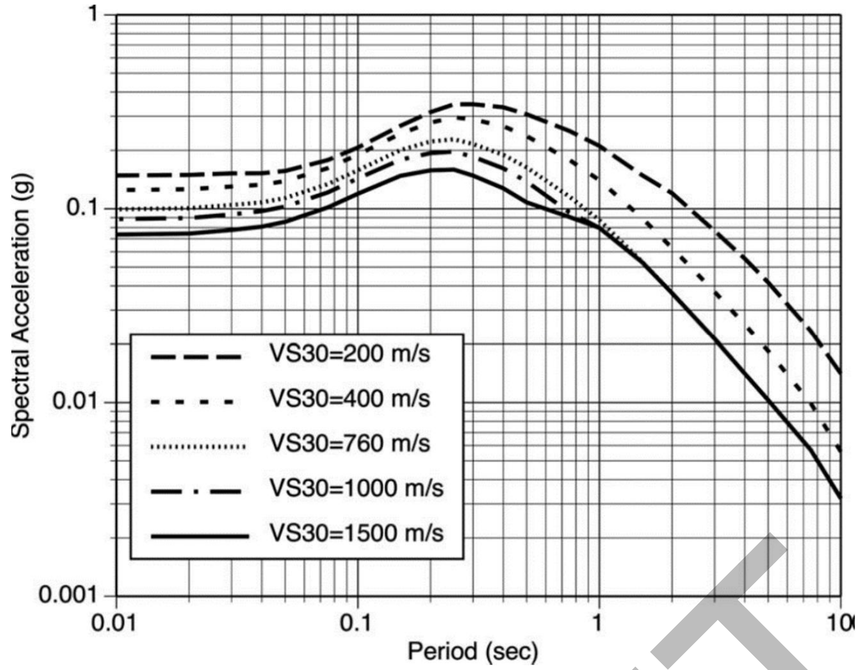


Figure C2.21 Dependence of response spectral acceleration on Vs30 for a magnitude 7 earthquake at a distance of 30 km. (Abrahamson and Silva, 2008).

5. *Shear wave velocity of the dam foundation bedrock*

The shear wave velocity can be measured directly or may be assessed indirectly from seismic refraction "P" wave velocities. Many dams have had seismic refraction ("P" wave) surveys carried out as part of the site investigations for the dam and these may be used to estimate Vs30.

If there are no data for the dam, a guide to what might be expected can be obtained from Table C2.4 and / or by consulting an experienced engineering Geologist. However in most cases if there are no data, or what data there are only old "P" wave data, site specific "S" wave testing should be carried out because the seismic hazard is significantly dependent on Vs30 and as can be seen from Table C2.4 unless measurements are made at the dam site the potential range of Vs30 is large.

Table C2.4 Shear wave velocity Vs30 (m/s) site classification applicable to Australian regolith conditions (Mcpherson and Hall, 2007)

Site Class	V_s^{30} (m/s)	Geological Materials
B	>760	Fresh to moderately weathered hard rock units (Plutonic & metamorphic rocks, most volcanic rocks, coarse-grained sedimentary rocks Cretaceous & older)
BC	555 - 1000	Highly weathered hard rock; some Tertiary volcanics
C	360 - 760	Sedimentary rocks of Oligocene to Cretaceous age; coarse-grained; sedimentary rocks of younger age; extremely weathered hard rock units
CD	270 - 555	Sedimentary rocks Miocene and younger age, unless formation is notably coarse grained; Plio-Pleistocene alluvial units; older (Pleistocene) alluvium, some areas of coarse younger alluvium
D	180 - 360	Younger (Holocene to Late Pleistocene) alluvium
DE	90 - 270	Fine grained alluvial, deltaic, lacustrine and estuarine deposits
E	<180	Intertidal and back-barrier swamp deposits

6. Effects of soil overlying bedrock

Current ground motion prediction models estimate the amplification of ground motions by shallow geology using Vs30, the time-averaged shear wave velocity over the upper 30 metres of the ground. Vs30 is a continuous variable that spans the range from soft soils, stiff soils, weathered rock, soft rock, to hard rock. Vs30 is specified in the range of 180 m/sec to 1100 m/sec (ranging from soft soil to hard rock). The data are insufficient to constrain amplification in rock above 1100 m/sec; it is expected to decrease slowly. Below 180 m/sec, the use of Vs30 is not viable and a site-specific soil response study is necessary. Such soils are described in United States codes as NEHRP sites E and F.

Use of Vs30 to estimate soil amplification at a specific site assumes that the average shear wave velocity, shear modulus reduction, and damping increase profiles represented in the strong motion data base used to develop the ground motion prediction models are a reasonably accurate representation of the specific profiles at the site. If this is not the case, then a site-specific analysis of the soil amplification should be done as described below. For most dam sites where the dam is founded on soft or deep valley alluvium, a site-specific analysis may be required.

Generally speaking, the larger the value of Vs30, the more likely it is that it provides an adequate representation of the amplification of the site while avoiding the uncertainties associated with nonlinear soil response analysis. Also, generally speaking, the lower the level of the ground motion that is input into the soil, the less nonlinear will be the response of the soil, and the more reliable will be the estimate of the soil amplification using Vs30.

However if the dam is founded on sediments or deep residual soils and completely weathered rock overlying the bedrock which are expected to have significant nonlinear response, then the response of these materials should be analysed in a separate study of nonlinear soil response using SHAKE, QUAD4M or similar programs that would not be part of the seismic hazard assessment. The Consultant who performs this analysis should specify the subsurface level (depth in the profile) and the associated (subsurface) Vs30, Z1.0 and Z2.5 at which the input ground motions are to be provided. In this case, Vs30 is the time-averaged shear wave velocity over a depth of 30 meters (m/s) below the selected subsurface level, and the Z1.0 and Z2.5 (depths to shear wave velocities of 1.0 and 2.5 km/sec) are also adjusted accordingly. The ground motions developed to represent free-field ground motions at this subsurface level are then used as input into the nonlinear analysis.

The analyses should use the non-liquefied properties of the soil.

One dimensional analyses such as SHAKE are potentially conservative for 2D and 3D structures such as embankment dams, so where the results are important to dam safety decision making it may be necessary to carry out 2D modelling such as QUAD4M or QUAKEW.

7. ***Response spectra***

No commentary.

8. ***Minimum Magnitude Earthquake***

Even small earthquakes can produce relatively large peak accelerations at close distances, but such earthquakes generally have low damage potential, and it is desirable to exclude earthquakes below a specified minimum magnitude to avoid such events contributing to an unrealistically high hazard level, especially for PGA and short period ground motions. The minimum magnitude considered by Burbidge and Leonard (2011) in the new draft seismic hazard maps of Australia for general building code applications is magnitude 4.5. For dams, it will be appropriate to use a higher minimum magnitude.

9. ***Quantification and reporting epistemic uncertainty***

Uncertainty in the inputs and models should be modelled and reported. There exists epistemic uncertainty (about the true state of nature) in earthquake source models (and the historical earthquake catalogues on which they are based); about how to incorporate active faults and in the source parameters of these faults; in ground motion prediction models; and in the shear wave velocity assigned to the dam foundation and the damping coefficients adopted. These uncertainties lead to uncertainty in the true value of the hazard at a specified return period. These uncertainties can be significant (Somerville and Thio, 2011). Figure C2.22 shows the results of a seismic hazard assessment where the mean (best estimate) of the hazard would result in the foundation largely being non-liquefiable, but the 85% fractile and 95th fractile would likely result in liquefaction, giving a totally different outcome for only marginally less likely loads.

Example Dam: Equal Hazard Spectra – 10,000 yr – 5%

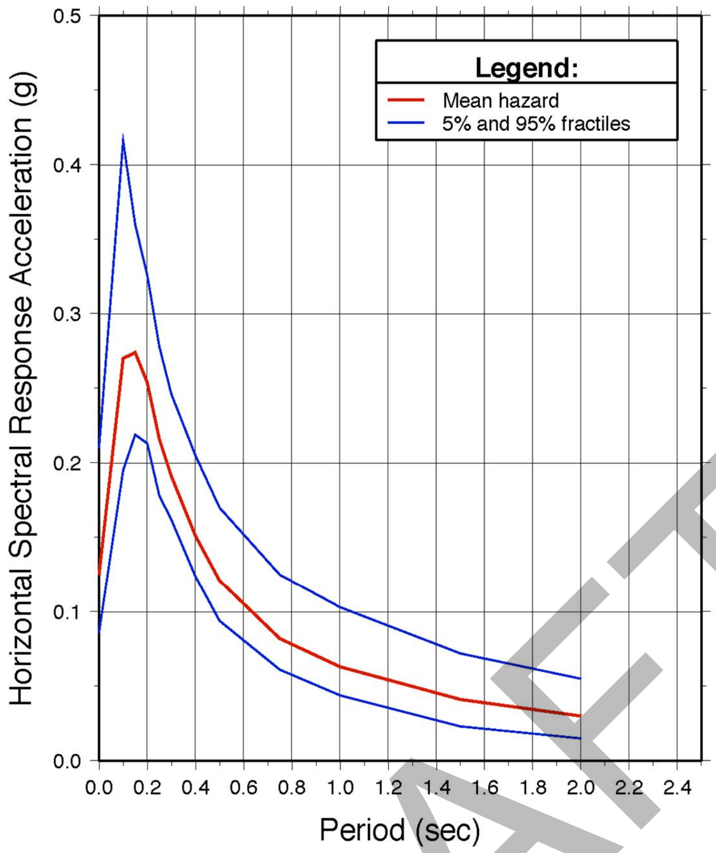


Figure C2.22 Example of horizontal spectral response acceleration showing the epistemic uncertainty in the true value of the hazard. The mean hazard is shown by the red line and the 5% and 95% fractiles are shown by the blue lines.

10. De-aggregation plots and selection of time-history motions.
Earthquake de-aggregation plots showing magnitude – distance contributions should be provided because these are a significant input into analyses for assessing embankment dam deformations and liquefaction. The de-aggregation of the hazard varies with the return period and with the period of the ground motion, so the magnitude and distance should be selected based on knowledge of the structure being analysed and the return period being considered.

11. ***Reservoir-induced seismicity*.**

See Sections C2.1.5 and C2.11.

In some situations for high and extreme consequence category dams a staged approach may be appropriate with a less detailed PSHA as described below for significant consequence category dams used initially, and the more detailed assessment carried out if it is recognised that the seismic hazard is critical.

For significant and low consequence category dams situations where a site specific and detailed PSHA may be required include dams which have potentially liquefiable foundations, or other features such as walls retaining the embankment which are sensitive to the seismic loading.

C2.6 Selection of Design Seismic Ground Motion - Deterministic Analysis Approach
Operating Basis Earthquake

Other stakeholders may for example include a local government council or other water supply authority which obtains their water from the dam Owner.

Safety Evaluation Earthquake

In assessing the required factor of safety to achieve a low likelihood of failure the effect of the 85th fractile (and possibly 95th fractile for Extreme Consequence Category dams) ground motions on the dam should be assessed. These are about 3 times and 10 times less likely than the median or 50th fractile ground motion. Alternatively the 1 in 30,000 AEP or 1 in 100,000 AEP ground motions may be used.

If the dam would fail leading to breach for these ground motions the likelihood of failure would probably be too high to satisfy ANCOLD (2003, 2016) tolerable risk criteria. Some additional information to assist in this assessment is given below.

For High B, High C , Significant and Low Consequence Category dams, if the structure is susceptible to liquefaction or has components which will fail at loads only a little greater than the loads in Table 2.1, check the design for the critical load and assess the adequacy of the design using risk assessment methods.

It should be noted that in Australia the MCE loading on known active faults may be significantly greater than the probabilistic loadings in Table 2.1 even though the probabilistic loadings include the effects of the active faults. This is because the MCE approach effectively ignores the AEP of the earthquake on the fault and this may be very low, so it does not have as much effect in the probabilistic assessment as it does in the MCE approach. In seismically active areas the reverse may apply.

If the MCE loading is significantly larger than the probabilistic loading detailed discussions should be held with the Seismologist to better understand the characteristics of the active fault. This may result in additional investigations being required such as trenching across the fault and dating the episodes of displacement. Once the MCE loading is finalised either adopt it and / or adopt a risk based approach.

If there is still some doubt about whether a fault is active or not, consult the Seismologist (s) and Geologist involved and assign a probability that the fault is active and apply this in the event tree logic in a risk analysis.

The use of a deterministic analysis approach requires that a factor of safety should be applied to estimated stresses and deformations within the dam and its foundations to give a low likelihood of the dam failing given the SEE loading. Table C2.5 shows the conditional probabilities of failure required to just achieve ANCOLD (2003) tolerable risk guidelines. To allow for the fact the tolerable risk criteria are for the sum of all potential failure modes, and the seismic loading component may only be a small part of the overall probability of failure, and to give some margin of safety that the risks are below the limit of tolerability, the conditional probability of failure for seismic loading for existing dams, given the SEE should be about 1 in 100, and for new dams or major augmentation, about 1 in 100 to 1 in 1000. This requires a significant factor of safety on stresses or deformations for the SEE load. It should be noted that individual risk is the controlling factor for High C, Significant and Low Consequence Category dams.

The requirement for a factor of safety on stresses or deformations applies to all consequence categories and is a critical requirement if a deterministic analysis approach is to be followed.
As there are no unique relationships between factor of safety and likelihood of failure the Consultant will have to apply engineering judgement to achieve the required low likelihood of failure.

Table C2.5 Indicative required conditional probabilities of failure given the ANCOLD (2013) Design Earthquake

Dam Consequence Category, or "class"	ANCOLD (2013) Design of Concrete Gravity Dams Recommended Design Loads	ANCOLD (2012) Consequence Categories Potential Life Loss (PLL)	ANCOLD (2003) Societal Risk Annual Probability of Failure Limit (a)	Conditional Probability of Failure To achieve ANCOLD(2003) Limit Criteria (a) (b)	Conditional Probability of Failure To achieve ANCOLD(2003) Limit Criteria (c)
Extreme	1:10,000 AEP	>50 100 1000	2E-05 1E-05 1E-06(d)	<0.2 <0.1 <0.01	<0.1 0.05 0.005
High A	1:10,000 AEP	1 to 50	1.0E-03 to 2E-05	< 0.2	< 0.1
High B	1:5,000 AEP	0.1 to 5	1.0E-02 to 2.0E-04	<1 (0.5,0.05)	<0.5 (< 0.25 to <0.025)
High C	1:2,000 AEP	<0.1 to 5	1.0E-02 to 2.0E-04	<0.4 (<0.2,0.02)	<0.2 (<0.1, 0.01)
Significant	1:1,000 AEP	<0.1 to 1	1.0E-02 to 1.0E-03	<1 (<0.1, 0.01)	(<0.5) (<0.05, 0.0005)
Low	1:1,000 AEP	<0.1	1.0E-02	<1 (<0.1, 0.01)	(<0.1) (<0.005, 0.0005)

Note

(a) The figures include the contribution to the annual probability of failure from other failure modes.

(b) Figures in brackets controlled by individual risk criteria, 1.0E-04 / annum for existing dams; 1.0E-05 for new dams or major augmentations. Figures assume vulnerability of the person most at risk = 1.

(c) These figures assume that the contribution to the risk from seismic loads is half the total.

(d) This assumes that the horizontal truncation in the societal risk plot is removed in ANCOLD (2016).

C2.7 Selection of Design Seismic Loading-Risk Based Analysis Approach

For the risk analysis the seismic loads should be partitioned taking account of the fragility of the structure. For example:

(a) If liquefaction is an issue, the earthquake loading with an AEP at which liquefaction is widespread would be used as a partition boundary.

(b) The AEP of the earthquake loading which results in critical retaining walls; e.g. between spillway and embankment; failing.

(c) The AEP of the earthquake loading which is likely to result in displacement of a concrete gravity dam.

The uncertainty in the earthquake loading should be considered and modelled in the risk analysis.

C2.8 Modelling vertical ground motions

Gulerce and Abrahamson (2011) provide ground-motion prediction equations (GMPEs) for the vertical-to-horizontal spectral acceleration (V/H) ratio, and the methods for constructing vertical design spectra that are consistent with the probabilistic seismic hazard assessment results for the horizontal ground motion component.

The proposed V/H ratio GMPE is dependent on the earthquake magnitude and distance, and accounts for the differences in the non-linear site-response effects on the horizontal and vertical components. This results in large V/H ratios at short spectral periods for soil sites located close to large earthquakes.

It is suggested that this method be used rather than using a constant ratio of vertical to horizontal ground motion as has been commonly done.

C2.9 Selection of Response Spectra and Time-History Accelerograms

The amount of damping assumed for the response spectrum and analyses will depend on the type of structure. For embankment dams the amount of damping will depend on the extent of straining within the dam. For example, the Makdisi-Seed method for estimating earthquake induced deformations in an embankment varies both the bulk modulus and the damping factor according to the shear strain in the embankment. For concrete dams, the USACE (1999) states: "Energy dissipation in the form of a damping ratio is included as part of the response spectrum curves. For the linear elastic or nearly elastic response during an OBE event, the damping value should be limited to 5 percent. For the SEE excitation, a damping constant of 7 or 10 percent may be used depending on the level of strains and the amount of inelastic response developed in the structure".

Suitable methods can be used to convert a response spectrum to account for higher damping factors than the one for which the response spectrum was prepared (e.g. Rezaein et al., 2014a, b).

At least four or five ground motion records are generally needed for advanced deformation analyses. A relatively large suite of records is required due to the range of deformation predictions that may occur even for a carefully selected set of motions. Since only 4 or 5 records are being used, the intent of the study is not to define the full range of potential displacements but to determine the average, expected response for the specified level of earthquake loading.

Three ground motion components should be provided for each record: two horizontal and one vertical. While vertical motions are often considered to have a modest effect on deformation predictions, advances in developing appropriate and consistent motions and the ease with which they can be included in many sophisticated analyses warrants their routine use in deformation analyses.

The suite of records should be obtained from different source earthquakes to reduce unintended bias in the record selection. The following criteria may be considered in the selection of earthquake record:

a) Records to be used for preparation of site specific time-histories should originate from a seismic event similar to the target design earthquake (e.g., magnitude, fault distance, and focal depth). The site condition for each record should reasonably correspond to the site condition for the target response spectrum. For example, it may be appropriate to use a record from a shallow, stiff soil site to represent soft rock conditions, but not a deep soil record.
b) The shape of the response spectrum for each record should reasonably match the target response spectrum over the frequency range of interest. This frequency range may be rather large and will typically include low frequencies (long periods).
c) Scaling factors may be applied to the record to provide a best fit to the response spectrum over the period range of interest (see Figure C2.23).

Alternatively, spectral matching programs such as RSPMatch can be used to more closely match the target response spectrum over a wide range of frequencies.

d) Spectral matching techniques should be carefully applied to preserve as much of the original character of the earthquake record as possible (e.g., relative magnitude and duration of velocity peaks). Although scaling factors are traditionally limited to values between 0.5 and 2.0, values outside of this range may be permitted in some cases (Watson-Lamprey and Abrahamson, 2006).

e) Additional criteria can be useful in defining an appropriate suite of ground motions, such as Arias Intensity or significant duration. Attenuation relationships are available for these parameters allowing their inclusion in deterministic or probabilistic hazard estimates (e.g., Watson-Lamprey and Abrahamson, 2006; Travasarou, et al., 2003; Kempton and Stewart, 2006].

f) Original earthquake records can be obtained from a number of online sources, including the COSMOS and PEER websites. Synthetic accelerograms should be considered when the design earthquake is not well-represented by the database of available records.

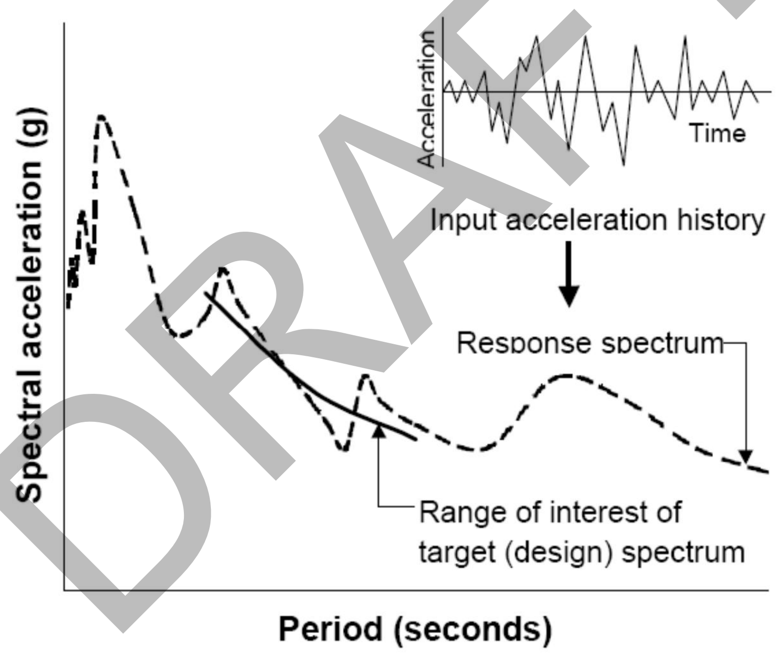


Figure C2.23 Scaling of the input time-history for best fit with target response spectrum over period range of interest (Perlea and Beaty, 2010).

C2.10 Earthquake Aftershocks

This is more likely to be an issue where an active fault or individual neotectonic fault is a major contributor to the seismic hazard.

An example of a dam which might be susceptible to aftershock seismic ground motions is a concrete face, asphalt face or other membrane face rockfill dam which has poor drainage capacity, so the rockfill may saturate from leakage through the damaged face slab or membrane after the main earthquake, leaving it with a low static factor of safety and susceptible to much larger deformations and damage during the aftershock.

Concrete dams and appurtenant structures such as walls retaining the embankment which may experience significant displacement during the primary ground motion, sufficient to compromise the foundation drainage or passive or post tensioned anchors should be considered for aftershocks.

C2.11 Seismic Ground Motions from Earthquakes Induced by the Reservoir
The following is adapted from ICOLD (2016a).

The Reservoir-Triggered Earthquake (RTE) represents the maximum level of ground motion capable of being triggered at the dam site by the filling, drawdown, or the presence of the reservoir. ICOLD Bulletin 137 on Reservoirs and Seismicity provides the state-of-knowledge on reservoir-triggered seismicity. Section C2.1.5 gives some details.

While there exist differences of technical opinion regarding the conditions which cause reservoir-triggered seismicity, it should be considered as a credible event if the proposed reservoir contains active or neotectonic faults within its hydraulic regime and if the regional and local geology and seismic record within that area are judged to indicate potential for reservoir-triggered seismicity. Even if all the faults within a reservoir are considered tectonically inactive, the possibility of reservoir-triggered seismicity should not be totally ruled out, if the local and regional geology and seismicity suggest that the area could be subject to reservoir-triggered seismicity.

Depending on the dam location and prevailing seismotectonic conditions, the RTE may represent ground motion less than, equal to, or greater than the OBE ground motion. RTE ground motion should in no case be greater than the Safety Evaluation Earthquake ground motion because the faults considered capable of triggering seismicity should be taken into consideration during the seismic hazard evaluation. However the result might be the premature triggering of seismic events due to the impounding of the reservoir that would have occurred naturally at some longer time in the future. It is therefore justified in the case of larger dams and storages located in seismically active regions and regions with high tectonic stresses to install a micro-seismic network and to monitor the seismicity prior to, during and after impounding.

This has been done on some Australian dams, e.g. Thomson and Dartmouth, and micro-seismicity was detected associated with faults in the reservoir as discussed in Section C2.1.5.

For existing dams there is unlikely to be any RTE from reservoirs which have been filled for several decades.

C3 ASSESSMENT OF EMBANKMENT DAMS FOR SEISMIC GROUND MOTIONS

C3.1 Effect of Earthquakes on Embankment Dams.

No commentary.

C3.2 Defensive Design Principles for Embankment Dams.

The concept of 'defensive design' of embankment dams for earthquake was developed by Sherard (1967) and Seed (1979) and endorsed by Finn (1993), ICOLD (1986, 1999a) and ANCOLD (1998).

C3.3 Seismic Deformation Analysis of Embankment Dams.

C3.3.1 The Methods Available and When to use them

There are a number of methods available for estimating the deformations which may occur in embankments and their foundations during and post seismic loading. These can be summarized as:

(a) *Screening and empirical database methods*. These are applicable to embankments and their foundations which do not liquefy, or experience significant loss of strength due either to build up of pore pressure or strain weakening. These should only be applied if the criteria listed in Section C3.3.2 are satisfied, and should only be relied upon if the estimated deformations are much less than what is tolerable; e.g. crest settlements are much less than the available freeboard.

(b) *Simplified methods for estimating deformations during earthquakes*. These methods are also only applicable to embankments and their foundations which do not liquefy. They assume the post-earthquake deformations are negligible and the deformations during the earthquake are due to the action of the horizontal inertia forces induced by the earthquake. They are commonly based on the Newmark (1965) principle.

(c) *Post-earthquake deformations for liquefied conditions*. These methods assume that the deformations are primarily caused by gravitational forces acting on an embankment following the earthquake, and allow for the reduction in strength caused by liquefaction or strain weakening of other soils. They may be carried out using limit equilibrium and or static numerical methods.

(d) *Advanced numerical methods for estimating deformations during and post-earthquake for non-liquefied and liquefied conditions*. These cover a wide range of sophistication and complexity and include dynamic analyses using total and effective stress methods, linear and non-linear models, and varying degrees of refinement of how pore pressures are developed and coupled to deformations.

Perlea and Beaty (2010) give an overview of the methods and their application.

The Guideline and the information presented below give guidance on which methods should be used.

It should not be assumed that the screening and simplified methods are conservative and where estimated deformations are near to the tolerable deformations more advanced methods should be used or measures taken to reduce deformations.

For risk based methods these calculations will in principle need to be done for a range of seismic loads. In practice for many dams deformations will be very small even for the largest loads so it will be unnecessary to do the calculations for the smaller loads.

For deterministic methods only the OBE and SEE load will need to be analysed subject to the qualifications in Sections 2.6 and C2.6.

The screening, database and simplified methods are not applicable to tailings dams constructed from hydraulically placed tailings because the tailings are not compacted to the degree required for conventional earthfill and rockfill, and will as a result be more subject to densification during seismic loading, and may be subject to pore pressure generation and liquefaction.

C3.3.2 Screening and Empirical Database Methods

Screening Method

Perlea and Beaty (2010) report that the USACE do not require deformation analyses for low to moderate height dams < 60m high if all the following criteria are met:

1. Dam and foundation materials are dense, not subject to liquefaction, and do not include sensitive clays.
2. The dam is well built and densely compacted to at least 95% of the laboratory maximum dry density, or to a relative density greater than 80%.
3. The slopes of the dam are 3:1 (H:V) or flatter, and/or (*the slopes are steeper but*) the phreatic line is well below the downstream face of the embankment.
4. The predicted peak horizontal ground acceleration (PGA) at the base of the embankment is no more than 0.20g. Compacted clay embankments on rock or stiff clay foundations may offer additional resistance to deformations. Somewhat higher allowable PGA values may be justifiable for these dams on a case-by-case basis, although the PGA criterion should not exceed 0.35g (USBR, 1989).
5. The static factors of safety for all potential failure surfaces involving loss of crest elevation (other than shallow surficial slides) are greater than 1.5 under the loading and pore-pressure conditions expected immediately prior to the earthquake.
6. The freeboard at the time of the earthquake is at least 3 to 5 percent of the dam height plus alluvial foundation, and not less than 0.9 m. Special attention should be given to the presence and suitability of filters for dams with modest freeboard.
7. There are no appurtenant features related to the safety of the dam that would be harmed by small movements of the embankment.
8. There have been no historic incidents at the dam that may indicate a limitation in its ability to survive an earthquake.

The words in italics have been added for clarity.

This approach may be used as a screening method. However for the SEE most dams in Australia will have a PGA greater than 0.2g (0.35g).

Empirical Database Methods

There are a number of these methods which are developed by gathering deformation and other data on embankment dams which have been subject to earthquake, and relating the amount of deformation and cracking to the earthquake loading experienced by the dam. Of these the Swaisgood (1998) method is based on a large database and is referred to in Perlea and Beaty (2010) as being used by USACE, and is in use in Australia.

The Pells and Fell (2002, 2003) method uses a larger database and also records the amount of damage as evidenced by longitudinal and transverse cracking observed. It also gives guidance on for what seismic loading (measured by Mw and PGA) transverse cracking can be expected. This is important for considering concentrated leak erosion following the earthquake.

Both methods are described in Fell et al (2005, 2015). They only apply to dams where there is no potential for liquefaction or significant strain weakening. They should be applied conservatively, e.g. use upper bound estimates, allowing for the scatter in the data used to form the plots. They are for many dams quite sufficient provided the dams are well constructed and there is a large margin between estimated settlement and the freeboard.

Hynes-Griffin and Franklin (1984) Pseudo-Static Seismic Coefficient Method.

This method is based on Newmark sliding block analyses of 349 horizontal components of natural earthquakes and 6 synthetic records for a range of yield accelerations. From this permanent displacements u in cm were plotted versus N/A where N is the yield acceleration and A is the peak value of the earthquake acceleration at the base of the dam.

Hynes-Griffin and Franklin (1984) then used an arbitrary limit of 100cm (one metre) displacement as representing tolerable displacements, and from Figure C3.1, used the upper bound plot of displacements, which gave N/A = 0.17. After allowing for amplification factor of 3 between the base acceleration and that at the crest of the dam, they concluded that a factor of safety of 1.0 with a seismic coefficient of one half of the peak ground acceleration at the base of the dam, would assure that deformations would not exceed 1 metre. Their suggested method is:

(a) Carry out a conventional pseudo-static stability analysis using a seismic coefficient equal to one-half the predicted peak ground acceleration.
(b) Use a composite S-R strength envelope (Effective stress strength at low stresses, undrained strength at high stresses) for pervious soils and R undrained strength for clays, multiplying the strength in either case by 0.8.
(c) Use a minimum factor of safety of 1.0.

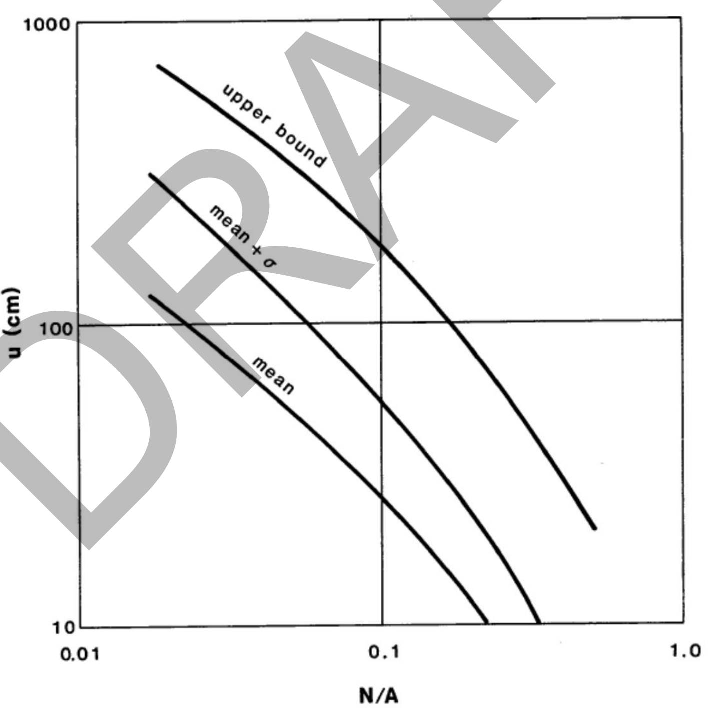


Figure C3.1. Permanent displacement u versus N/A based on 354 accelerograms (Hynes-Griffin and Franklin, 1984)

It should be noted that this method cannot be used in reverse. That is, just because the factor of safety is less than 1.0, it should not be assumed that the deformations will be greater than 1 metre. This is because the upper bound deformation curve in Figure C3.1 has been used in the method.

In practice for most dams with static factors of safety about 1.5, the SEE PGA will result in factors of safety using this method less than 1.0 so the method is not particularly useful. It is included here because the background to the method is often not appreciated by practitioners, and to avoid its misuse.

C3.3.3 Simplified Methods for Estimating Deformations during Earthquakes

These methods are all based on the Newmark (1965) approach. He introduced the basic elements of a procedure for evaluating the potential deformations of an embankment under earthquake loading. In this procedure, sliding of a soil mass along a failure surface was likened to slipping of a block on an inclined plane. Newmark (1965) envisaged that failure would initiate and movements would begin when the inertia forces exceed the yield resistance, and that movements would stop when the inertia forces were reversed. Thus, he proposed that once the yield acceleration and the acceleration time-history of a slipping mass are determined the permanent displacements can be calculated by double integrating the acceleration history above the yield acceleration as shown in Figure C3.2.

Newmark's approach is limited in application to compacted clayey embankments and dry or dense cohesionless soils that experience very little reduction in strength due to cyclic loading. The approaches using this principle should not be applied where embankments or their foundations are susceptible to liquefaction or strain weakening because they will significantly underestimate displacements.

There are a number of methods based on the Newmark (1965) principle:

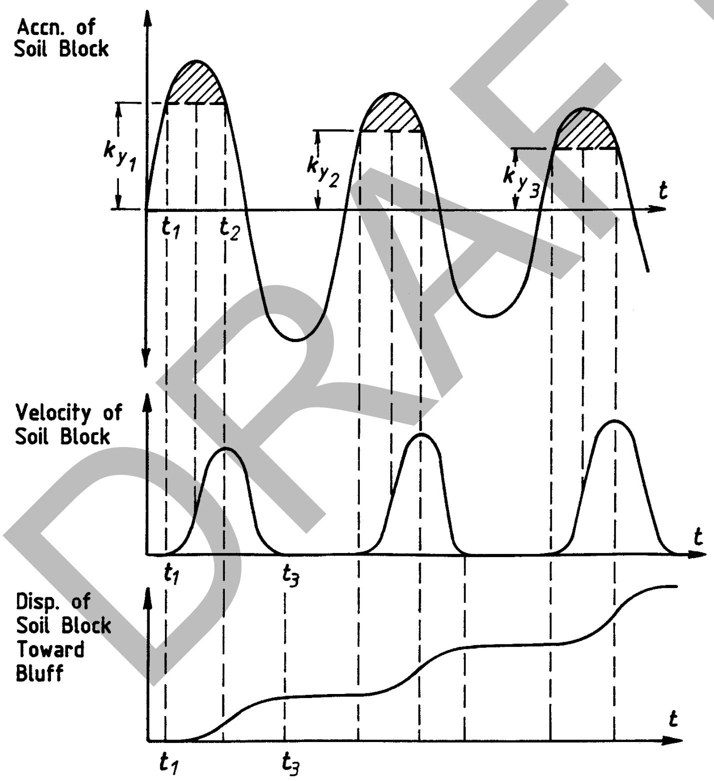


Figure C3.2. Double integration method for determination of the permanent deformation period of an embankment (Newmark, 1965).

Makdisi and Seed (1978) Method

The Makdisi and Seed (1978) approach is based on Newmark's method, but modified to allow for the dynamic response of the embankment as proposed by Seed and Martin (1966). The approach was developed from a series of deformation analyses performed on a large number of embankments subjected to earthquake loading.

The approach has been widely used and accepted among practicing engineers. However it is limited in application to dams not susceptible to liquefaction or strain weakening in the embankment or its foundations. Perlea and Beaty (2010) also warn the method may not be appropriate for severe ground shaking because it does not model strength losses. They also point out it was based on few earthquake records.

Bray and Travasarou (2007) used this method as well as their own (described below) and for the dams they analysed the Makdisi and Seed (1978) method generally underestimated the actual displacements.

Bray and Travasarou (2007) Method.

A simplified procedure was proposed by Bray and Travasarou (2007) for estimating earthquake induced permanent displacements in earth dams using a Newmark-type model. This procedure is based on the results from a set of simplified non-linear analysis using 688 recorded ground motions from 41 earthquakes. The flexibility of the dam system, and the interaction between yielding and seismic loading, were considered by using a non-linear coupled stick-slip deformable sliding model. The flexibility of the dam structure is captured through an estimate of the initial fundamental period *Ts*.

Key parameters of this procedure are the yield acceleration *ky* (in g), the initial fundamental period of the embankment, *Ts*, and the value of spectral acceleration for a damping of 5% and a degraded response period equal to 1.5*Ts*.

Bray and Travasarou (2007) claim the method provides improved characterisation of the uncertainty involved in the estimate of seismic displacement. It can be also be incorporated into a probabilistic framework.

C3.3.4 Post-earthquake Deformations for Liquefied Conditions

C3.3.4.1 Limit Equilibrium Analysis.
The post liquefaction stability is assessed as follows (Fell et al, 2005, 2015):

(a) Determine the zones which have liquefied (i.e. FS < 1.0) under the earthquake loading using the methods in Section C3.4.4.
(b) Determine the liquefied shear strength ($S_{u(LIQ)}$) for these zones using the methods described in Section C3.4.5.
(c) For potentially liquefiable soils with a factor of safety against liquefaction greater than 1.0, determine the residual excess pore pressure as detailed in Section C3.4.6.1.
(d) For clay soil zones in the embankment and foundation assign strength and pore pressure consistent with the soil's behaviour in static loading after being cracked and disturbed by the earthquake. If the clay is contractive in nature, use undrained strengths. If it is dilative on shearing, use effective stress strengths c′, f ′. Usually there will be some cracking and loosening, and if so fully softened strengths, would apply.. Some apply an arbitrary 10% or 15% loss of strength.
(e) For well compacted free draining rockfill filters and dense sands and gravels adopt the effective stress strengths c′, f ′, with no change in the pore pressures. If large deformations are expected in the earthquake the dense granular materials may have loosened and will have a strength approaching the critical state strength, rather than the peak strength. In practice this can be accommodated by a small reduction from the expected peak strengths.

The analysis is done with conventional limit equilibrium analysis methods. No loading from the earthquake is applied, since this is a post-earthquake analysis.

If the liquefied zone is subject to flow liquefaction and the post-earthquake factor of safety is significantly less than 1.0, large, rapid deformations and flow sliding can be expected. If the factor of safety is only marginally less than 1.0, or marginally above 1.0, deformations may not be so large as to lose freeboard between the dam crest and the reservoir level.

In considering these factors of safety account should be taken of the uncertainty in the assessment of the extent of liquefied zones and the liquefied strength. Perlea and Beaty (2010) indicate that a factor of safety in excess of 1.2 to 1.5 may be required to achieve tolerable displacements.

Because of the uncertainty in what post liquefaction factor of safety will give tolerable displacements, this approach and the simplified method described in Section C3.3.4.2 should only be used as described in the Guideline.

C3.3.4.2 Simplified Deformation Analysis.

An approximate estimate of the deformations can be obtained using the Khalili et al. (1996) method which is detailed Fell et al (2005, 2015).

C4.3.4.3 Static Numerical Deformation Analyses.

Indicative estimates of deformations can be obtained by performing a static deformation numerical analysis which incorporates the earthquake induced pore pressures and the liquefied strength of the liquefied soils (Finn, 1993). The analysis is often performed in two stages. In the first stage, the numerical model is initialised to the pre-earthquake conditions of the dam by simulating the current in-situ stresses. Then, in the second stage, the earthquake induced pore pressures and residual strengths of the liquefied soils are incorporated into the model to simulate post-liquefaction conditions.

This type of analysis is also referred to as uncoupled deformation analysis and generally leads to conservative estimates of post liquefaction deformations, as it does not allow for dissipation of earthquake induced pore pressures with time. More accurate estimates of post liquefaction deformations can be obtained using fully and semi-coupled methods of analysis, as discussed in the following sections.

However for many projects the uncoupled deformation analyses are sufficient. As for the more simplified methods, account should be taken of the uncertainty in the assessment of the extent of liquefied zones and the liquefied strength. These are best accounted for in a risk framework by assigning a likelihood to a range of liquefied strength, and analysing for each liquefied strength.

C3.3.5 Advanced Numerical Methods for Estimating Deformations During and Post-Earthquake for Non-Liquefied and Liquefied Conditions

The dynamic numerical codes used in practice may be divided into two main categories: total stress codes, and effective stress codes (Zienkiewicz et al., 1986 and Finn, 1993). They are reviewed in Zienkiewicz et al., (1986), Finn (1988,1993, 2000), Marcuson et al. (1992), and Perlea and Beaty (2010). A brief discussion of some of the more frequently used codes within each category is provided in the following sub-sections.

C3.3.5.1 Total Stress Codes

The total stress codes, as can be inferred from the classification, are based on the total stress concept and do not take account of pore pressures in the analysis. Therefore they should only be used in situations where the seismically induced pore pressures are negligible. The total stress codes may be divided into two main categories: (1) codes based on the equivalent linear (EQL) method of analysis, and (2) fully non-linear codes.

(a) Equivalent linear analysis (EQL)

The earlier total stress codes were based on the EQL method of analysis developed by H.B. Seed and his colleagues in 1972. EQL is essentially an elastic analysis and was developed for approximating non-linear behaviour of soils under cyclic loading. Typical of the EQL codes used in practice are: SHAKE (Schnabel et al., 1972), QUAD-4 (Idriss et al., 1973), QUAD4M (Hudson et al 1994) and FLUSH (Lysmer et al., 1975). SHAKE is a one dimensional wave propagation program and is used primarily for site response analysis. QUAD-4, QUAD4M and FLUSH are two-dimensional versions of SHAKE and are used for seismic response analysis of dams and embankments.

Given the elastic nature of the EQL analysis, however, these codes cannot take account of material yielding and material degradation under cyclic loading. Therefore, they tend to predict a stronger response than actually occurs. Also, they cannot predict the permanent deformations directly. Indirect estimates of permanent deformations can however be obtained using the acceleration or stress data obtained from an EQL analysis and the semi-empirical methods proposed by Newmark (1965) and / or Seed et al. (1973).

(b) Fully non-linear analysis

More accurate and reliable predictions of permanent deformations can be obtained using the elasto-plastic non-linear codes. Typical of the elasto-plastic non-linear codes used in the analysis of embankments are DIANA (Kawai, 1985), ANSYS (Swanson, 1992), FLAC (Cundull, 1993), etc. The constitutive models used in these codes vary from simple hysteretic non-linear models to more complex elasto-kinematic hardening plasticity models. Compared to the EQL codes, the elasto-plastic non-linear codes are more complex and put heavier demand on computing time. However, they provide more realistic analyses of embankments under earthquake loading, especially under strong shakings. Critical assessments of non-linear elasto-plastic codes can be found in Marcuson et al. (1992) and Finn (1993, 2000).

C3.3.5.2 Effective Stress Codes

Effective stress codes allow modelling pore pressure generation and dissipation in materials susceptible to liquefaction and thus to obtain better estimates of permanent deformations under seismic loading. The effective stress codes may be divided into three main categories: fully coupled, semi-coupled and uncoupled.

(a) Fully coupled codes

In the fully coupled codes, the soil is treated as a two-phase medium, consisting of soil and water phases. Two types of pore pressures are considered, transient and residual. The transient pore pressures are related to recoverable (elastic) deformations and the residual pore pressures are related to non-recoverable (plastic) deformations. A major challenge in fully coupled codes is to predict residual pore pressures. The residual pore pressures, unlike the transient pore pressures, are persistent and cumulative and thus exert a major influence on the strength and stiffness of the soil skeleton. The transient pore pressures are cyclic in nature and their net effect within one loading cycle is often equal to zero. An accurate prediction of residual pore pressures requires an accurate prediction of plastic volumetric deformations. In the fully coupled codes this is often achieved by utilizing elasto-plastic models based on kinematic hardening theory of plasticity (utilizing multi-yield surfaces) or boundary surface theory with a hardening law. These models are very complex and put a heavy demand on computing time.

Generally speaking, fully coupled prediction of pore pressures under cyclic loading is very complex and difficult. The validation studies performed on a number of these codes suggest that the quality of response predictions is strongly path dependent. When the loading paths are similar to the stress paths used in calibrating the models, the predictions are good. As the loading path deviates from the calibration path, the predictions become less reliable. Apart from the numerical difficulties, part of this unreliability is also due to the poor or less than satisfactory characterization of the soil properties required in the models. For instance, because of sampling problems, it is often very difficult to accurately determine volume change characteristics of loose sands as required by these models. In general, the accuracy of pore pressure predictions in fully coupled modes is highly dependent upon the quality of the input data.

Fully coupled codes include DNAFLOW (Prevost, 1981, 1988), DYNARD (Moriwaki et al., 1988); PLAXIS (2012); ABAQUS (2012) and FLAC (Itasca, 2011).

As described in Perlea and Beaty (2010), FLAC contains a number of general purpose constitutive models but also provides for the use of user-defined constitutive models. These include a bounding surface hypo-plasticity model (Wang and Makdesi, 1999) by AMEC Geomatrix Inc.; FLAC-UBCSAND, (Beaty and Byrne, 2008) and PM4Sand (Dafalais and Manzari, 2004; Boulanger, 2010; Boulanger and Ziotpoulou, 2012; Ziotopoulou and Boulanger, 2012). PLAXIS and ABAQUS also offer user defined constitutive models such as UBCSAND.

(b) Semi-coupled codes

Compared to the fully coupled codes, the semi-coupled codes are more robust and less susceptible to numerical difficulties. However they are theoretically less rigorous. In these codes empirical relationships, such as those proposed by Martin et al. (1975) and Seed (1983), are used to relate cyclic shear strains/stresses to pore pressures. The empirical nature of the pore pressure generation in these codes generally puts less restriction on the type of plasticity models used in the codes. The semi-coupled codes are in general less complex and computationally demanding. The parameters they require are often routinely obtained in the laboratory or in the field.

Semi coupled codes include DESRA-2 (Lee and Finn, 1978), DSAGE (Roth, 1985), and TARA-3 (Finn et al., 1986). FLAC (Cundall, 1993, Itasca, 2011) can also be used in this mode as in the URS cycle weighted model described in Perlea and Beaty (2010)

C3.3.5.3 Summary

More advanced dynamic methods are potentially expensive and should only be done by very experienced persons who understand the limitations of the analysis and the need to use well considered properties. The analysis methods are controlled by the quality of data put into them, the limitations of the methods themselves and particularly of those doing the analysis.

They are in most cases only to be contemplated for dams and foundations subject to liquefaction, and where the extra refinement of the analyses are warranted; e.g. where the simplified methods are not really applicable, and / or are resulting in marginal assessments on whether remedial works are required.

Any non-linear deformation analysis needs to be documented in sufficient detail that it can be reasonably scrutinized. All input parameters, all numerical details (boundary conditions, damping parameters, etc), and enough output plots to evaluate reasonableness of initial conditions and dynamic responses. These should be reviewed by person's expert in the practice.

Perlea and Beaty (2010) give an overview of USACE practice and examples of use of many of the programs described above. Stark et al (2012), Friesen and Balakrishnan (2012) give useful examples.

C3.4 Assessment of the Effects of Liquefaction in Embankment Dams and Their Foundations

C3.4.1 Overall Approach

No commentary.

C3.4.2 Definitions and the Mechanics of Liquefaction

It should be noted that there are no universally accepted definitions for liquefaction. Those provided in the Guideline have fairly wide acceptance.

The definitions for liquefaction, initial liquefaction, flow liquefaction and temporary liquefaction are based on USNRC (1985), Robertson and Fear (1995), Robertson and Wride (1997), Yamamuro and Lade(1997).

It is important to recognize that these phenomena apply to monotonic (static) as well as cyclic loading and are apparent in contractive soils (e.g. in loose fills) , both cohesionless and those with very low plasticity.

Robertson and Fear (1995) and Robertson and Wride (1997) defined the cyclic liquefaction and cyclic mobility terms.

Idriss and Boulanger, (2008) defined cyclic softening.

The mechanics of liquefaction are described in USNRC (1985) and Fell et al (2005, 2015). However the description in Idriss and Boulanger (2008) is more complete and up to date.

It is essential that those carrying out liquefaction assessments read Idriss and Boulanger (2008) because it will assist them understand the background to the simplified procedure for liquefaction assessment and the way laboratory performance has been used to develop the method alongside field data.

C3.4.3 Methods for Identifying Soils which are Susceptible to Liquefaction

(a) Geology and age of the deposit.

Idriss and Boulanger (2008) refer to Youd and Perkins (1978) and indicate that the soils which are most susceptible to liquefaction are non-plastic or very low plasticity fill, alluvial, fluvial, marine and deltaic soils. They indicate soils are most susceptible when they are recently deposited, becoming more resistant to liquefaction as they become older.

The most susceptible soils are Holocene or younger soils, Pleistocene soils are low or very low susceptibility except for loess.

Youd et al (2001) discuss the effects of the age of the deposit and conclude that for man-made structures such as fills and embankment dams, aging effects are minimal, and corrections for age should not be applied in calculating liquefaction resistance.

Andrus et al (2009) and Hayati and Andrus (2009) present methods for correcting for the age of the soils being assessed for liquefaction. These are discussed further in Section C3.4.4.3. These include data showing liquefaction has occurred in Pleistocene sands.

It is concluded that geological age alone cannot be used as a means of screening for liquefaction susceptibility. It may be taken as additional information to be considered in assessing the likelihood of liquefaction.

(b) Soil gradation, plasticity, moisture content

There are a number of methods available to identify soils which are susceptible to liquefaction based on the particle size gradation, plasticity and in-situ moisture content of the soil. These include:

(i) Seed et al (2003) criteria
(ii) Robertson and Wride (1998) soil behaviour type index Ic for Cone Penetration Tests
(iii) Boulanger and Idriss (2006)
(iv) Robertson (2010) for Cone Penetration Tests

It is recommended that at least two and preferably more of the methods be used to assess the likelihood the soils are potentially liquefiable. A suggested approach for doing this is given in Table C3.1.

Table C3.1 Suggested method for assessing the likelihood that a soil is potentially liquefiable

Result of assessment	Likelihood this strata is potentially liquefiable
Three methods indicate liquefiable	Highly likely to certain
Two methods indicate liquefiable	Likely
Only Seed et al (2003) indicates liquefiable, others do not	Likely to Possible Depending if Zone A or Zone B
Only Robertson and Wride (1998) indicates liquefiable, others do not.	Unlikely
Three methods indicate non liquefiable	Liquefaction can be discounted

It should be noted that where soils plot in Zone B of the Seed et al (2003) method they should be considered as potentially susceptible to liquefaction and /or cyclic softening. If their behaviour becomes critical to the assessment of the safety of the dam either the soil should be assumed to be liquefiable or undisturbed samples should be subject to cyclic loading tests rather than relying on semi-empirical methods to predict behaviour.

The following points should be noted:

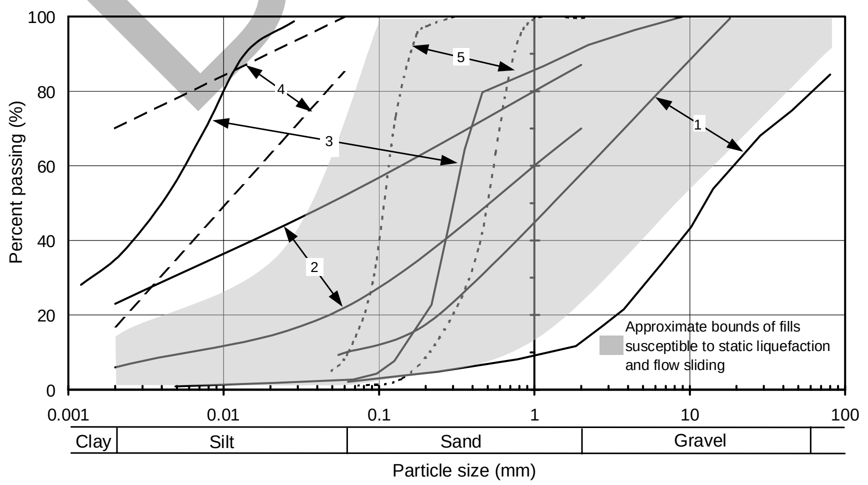
(a) The Seed and Idriss (1982) Chinese Criteria method has been used extensively in Australia but often the last criteria that the natural moisture content is ≥ 0.9 times the liquid limit has been ignored which mean that lower moisture content soils have been incorrectly identified as potentially liquefiable.

(b) Seed et al (2003) and Boulanger and Idriss (2007), Idriss and Boulanger (2008) all indicate that the Chinese Criteria (Seed and Idriss, 1982) are outdated and should not be used. It is recommended that this advice be heeded.

(c) Seed (2010) discusses the Boulanger and Idriss (2006) method and plots field case data which he indicates shows the Boulanger and Idriss (2006) criteria are non-conservative, with case data showing liquefaction of soils occurring with PI> 7 contrary to the Boulanger and Idriss (2006) criteria. Given these data it would appear that caution should be applied in relying on the Boulanger and Idriss (2006) criteria alone to identify soils which may liquefy.

(d) Fell (2012) notes that for one dam where foundation liquefaction was thoroughly assessed using SPT and CPT data the Robertson and Wride (1998) method identified a number of strata as non-liquefiable where the Seed and Idriss (1982) and Seed et al (2003) methods both indicated the soil was potentially liquefiable. Given this caution should be applied in relying only on the Robertson and Wride (1988) or Robertson (2010) CPT based methods and possibly other CPT and CPTU based methods alone.

(e) Particle size distribution alone is not able to identify soils which are susceptible to liquefaction. Figures 11 and 12 in ANCOLD (1998) are outdated and should not be relied upon. Figure C3.3 is a better guide.



1 - Coarse grained coal mine waste (Dawson et al 1998; Taylor 1984; Bishop et al 1969; Hutchinson 1986)
2 - Loose silty sand fills, Hong Kong (upper and lower quartile of pre 1977 fills (HKIE 1998))
3 - Hydraulically placed mine tailings and fills in dam embankments (various published sources)
4 - Sensitive clays (indicative limits from: Lefebvre 1996; Bentley & Smalley 1984; Mitchell & Markell 1974; Hutchinson 1961, 1965)
5 - Sub-aqueous slopes, natural and fill slopes (Koppejan et al 1948; Kramer 1988; Sladen & Hewitt 1989; Cornforth et al 1974)

Figure C3.3: Particle size distributions of material types susceptible to liquefaction and flow sliding (Hunter and Fell 2003)

C3.4.4 Methods for Assessing Whether Liquefaction may occur

C3.4.4.1 Introduction

Whether soils are likely to liquefy under seismic loading is almost always carried out using the "simplified method" first developed by Seed and Idriss (1971). In the ANCOLD (1998) guideline the method referenced was Seed and De Alba (1986) which was a refinement of the Seed and Idriss (1971) method. Since then there have been further developments in the details of the methods but the basic approach remains the same as described in Section C3.4.4.2.

Youd et al (2001) published a summary report on the 1996 NCEER and 1998 NCEER/NSF Workshops on valuation of Liquefaction Resistance of Soils. This was a consensus of the 20 experts who attended the Workshops and has been widely used for assessing liquefaction since it was published, including by the Australian dam community.

Seed et al (2003) prepared a University of California at Berkeley Report which presented updated methods based on enlarged databases of cases, and using a probabilistic approach.

Idriss and Boulanger (2008) from University of California at Davis produced Monograph MNO-12 for the Earthquake Engineering Research Institute (EERI). This reviewed the available literature and updated many aspects of the method.

Seed (2010) issued a University of California at Berkeley Geotechnical Report which was critical of a number of aspects of Idriss and Boulanger (2008).

Idriss and Boulanger (2010) responded to this criticism and provided more details on the basis of their EERI Monograph.

As a result of the differences of opinion EERI set up an ad hoc committee to review what they termed the "strong differences of opinion, often personalised and polarized". This is reported in Finn et al (2010).

Youd (2010) provided some personal insights to the discussion.

Cox and Griffiths (2011) contributed by comparing the use of the Youd et al (2001), Seed et al (2003) and Idriss and Boulanger (2008) approaches on three sites.

As a result of these developments it has been necessary in preparing these guidelines to review these documents and provide guidance on which method or methods should be used, and details which need to be considered.

Unlike the ANCOLD (1998) guideline details of the suggested method are not included here. Those doing liquefaction assessments should instead work from the paper, Monograph or Report so they have available the background to the methods.

It has been proposed by Finn et al (2010) that a Workshop similar to the 1996 Workshop be convened. Practitioners should seek that publication if and when it is produced. In the meantime they should keep up to date with the literature particularly that in high quality refereed journals.

Since the drafts of these guidelines were prepared there have been a number of publications by Idriss, Boulanger, and others. These update and refine the methods but do not significantly alter the outcomes so no attempt has been made to review them here.

C3.4.4.2 Outline of the "Simplified Method" for Assessing Liquefaction.

The method requires the calculation or estimation of two variables for evaluation of liquefaction resistance of soils:

(a) The Cyclic Stress Ratio, CSR, which is a measure of the cyclic load applied to the soil by the earthquake.

(b) The Cycle Resistance Ratio, CRR, which is the capacity of the soil to resist liquefaction. The CRR is estimated from Standard Penetration Tests (SPT), Cone Penetration Tests (CPT) or, less frequently, the shear wave velocity.

(c) If the CSR is greater than the CRR, liquefaction is likely to occur.

The Cyclic Stress Ratio, CSR, is calculated from:

$$CSR = (\tau_{av} / \sigma'_{vo}) = 0.65(a_{max} / g)(\sigma_{vo} / \sigma'_{vo})r_d$$

Where a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake; g = acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively and r_d = stress reduction coefficient.

The CRR for M7.5 earthquakes can be estimated from the curves in Figure C3.4. It is usually recommended that the SPT Clean Sand Base Curve be used, with correction for fines content to give $(N_1)_{60CS}$. There are similar plots for CPT and shear wave velocity.

In this figure, $(N_1)_{60}$ is the SPT blow count normalised to an effective overburden pressure of 100 kPa, a hammer energy of 60%, borehole diameter, rod length and sampling method.

$(N_1)_{60}$ is given by:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$$

Where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B =correction factor for borehole diameter; C_R = correction factor for rod length; C_S = correction for samplers with or without liners.

To allow for fines content the $(N_1)_{60}$ values are adjusted to $(N_1)_{60cs}$ using:

$$(N_1)_{60CS} = \alpha + \beta(N_1)_{60}$$

α and β vary with fines content.

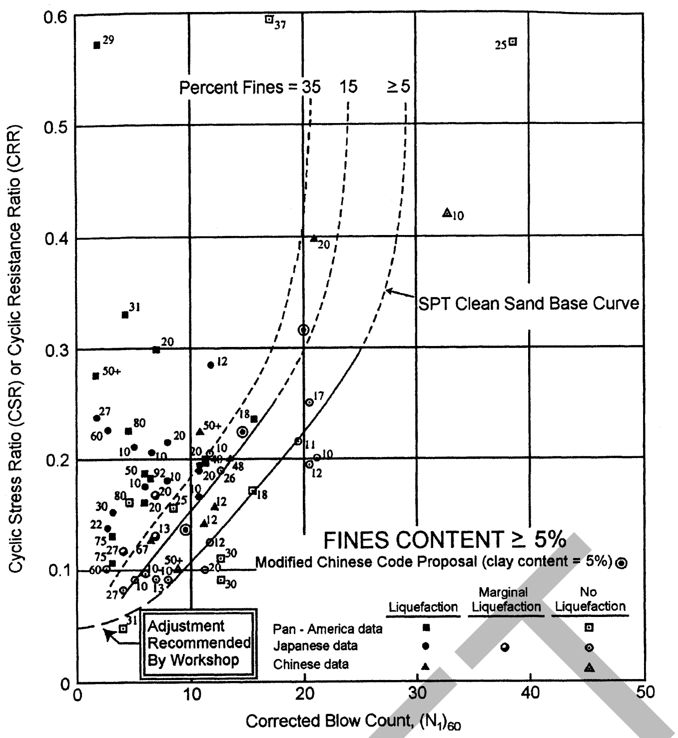


Figure C3.4 SPT Clean-sand Cyclic Resistance Ratio (CRR) curve for Magnitude 7.5 Earthquakes with data from liquefaction case histories (Youd et al., 2001, modified from Seed et al., 1985).

Magnitude scaling factors MSF

The procedures outlined above give $CRR_{7.5}$, cyclic resistance ratios for M7.5 earthquakes.

Smaller magnitude earthquakes giving the same peak horizontal acceleration are less likely to initiate liquefaction, because the earthquake will have fewer cycles of motion. Larger magnitude earthquakes are more likely to initiate liquefaction. This is allowed for by using a magnitude scaling factor (MSF) in the equation for factor of safety (FS) against liquefaction:

$$FS = (CRR_{7.5}/CSR)\ MSF$$

Where CSR = calculated cyclic stress ratio generated by the earthquake shaking; and $CRR_{7.5}$ = cyclic resistance ratio for magnitude 7.5 earthquakes.

Corrections for overburden stresses, static shear stresses K_s and K_a

The method of assessment of liquefaction potential described above is for horizontal or gently sloping ground and for depths less than about 15 m.

For assessments of liquefaction for embankment dams, the confining stresses may be higher than this and the dam will impose static shear stresses, which may alter the liquefaction potential.

These are allowed for by using the factors K_s and K_a to allow for the high overburden stresses and static shear stress respectively. They are applied by extending the equation above to:

$$FS = (CRR_{7.5}/CSR).MSF.\ K_s.\ K_a$$

There are three most commonly used methods based on SPTs and CPTs:

- Youd et al (2001)
- Seed et al (2003), Cetin et al (2004) and Moss et (2006)
- Idriss and Boulanger (2008), Boulanger and Idriss (2012)

Much of the discussion about the method relates to how the stress reduction factor r_d, magnitude scaling factor MSF, and corrections for high overburden stresses and static shear stresses K_s and K_a are treated. There are also differences of view regarding the CRR plot.

C3.4.4.3 Discussion of Differences between the available SPT and CPT Based Methods

Those who wish to understand the background to the methods should read the references in Section C3.4.4.1.

Cox and Griffiths (2011) used the three SPT based methods to analyse three case studies. They found that they did not give greatly differing factors of safety against liquefaction in the upper 24m of the soil profiles. They found that the Idriss and Boulanger (2008) method typically yields the highest factor of safety at depth, generally ranging from 5-15% higher than the others, but the differences were much greater at raw blow counts greater than 23. This was attributed to Idriss and Boulanger (2008) using higher K_s values but more importantly higher C_N values. These combined with the shape of the CRR curve at high $(N_1)_{60CS}$ to give the larger differences.

They found that in the upper soil profile the individual inputs to the methods varied quite a lot but the effects were often balanced by other factors. In particular they found that the quite different r_d values did not result in greatly different factors of safety.

This is consistent with the advice from Idriss and Boulanger (2008) that because the case histories used in the development of their method were analysed using their methods for accounting for depth effects, earthquake magnitude and fines content, it is important that when using their method the same methods as they used for these effects are adopted.

It is apparent that this would also apply if using the Seed et al (2003) method. In particular values of r_d , C_N , and K_s should not be substituted from one method to another.

Overall it is concluded that:

1. All three SPT based methods may be used and be expected to result in similar factors of safety against liquefaction in the upper 15 metres.
2. The Idriss and Boulanger (2008), Boulanger and Idriss (2012) SPT based method incorporates many refinements developed since the Youd et al (2001) method was published and its use is preferred. However where K_s values are critical to the outcome both the Idriss and Boulanger (2008) and Youd et al (2001) / Hynes and Olsen (1999) values should be used, and where the Idriss and Boulanger (2008) values are controlling the outcome of the assessment of the likelihood of liquefaction expert advice should be sought.
3. As can be seen in Figures C3.5 and C3.6 there is not a large difference between the NCEER/NSF Workshops (Youd et al (2001) and Idriss and Boulanger (2004, 2008) plots. Within the context of the uncertainty in ground motions the differences are small and will make little difference in a risk based analysis. The Cetin et al (2004) (Seed et al 2003) curve plots to the right of the others. This apparent significant difference between the three authors can partly be explained by the way in which the database has been analysed by Cetin et al (2004). However Idriss and Seed (2010, 2012) present a discussion of the data points which control the position of the curve. They cast doubt on the validity of 8 key points on the grounds of incorrect assignation of liquefaction / no liquefaction of four points and significant numerical errors between the r_d values used to develop their correlations and the r_d values computed using their applicable equation for four others. In view of this the Idriss and Boulanger (2004,

2008), Boulanger and Idriss (2012) method is preferred.

4. There is less information available to make an assessment of CPT based methods. However on what is presented all three CPT based methods may be used for assessing factors of safety against liquefaction in the upper 15 or 20 metres.
5. It is apparent there is considerable uncertainty in the estimation of K_a. It is suggested that Idriss and Boulanger (2008) be used but marginal cases should be assumed liquefiable, or laboratory tests and numerical analysis of stresses should be carried out to assess the effects.
6. The simplified methods are not considered reliable below about 15 or 20 metres. Most of the uncertainty discussed above is below that depth. For important decisions relating to marginally liquefiable soils below that depth expert advice should be obtained.

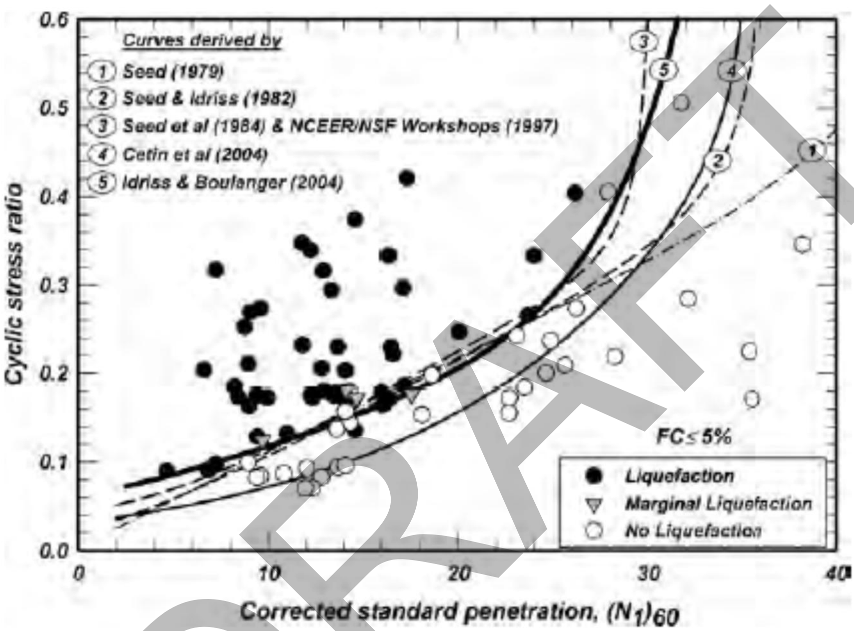


Figure C3.5 Curves relating cyclic resistance ratio (CRR) to $(N_1)_{60CS}$ values for earthquake magnitude Mw =7.5. (Idriss and Boulanger (2008).

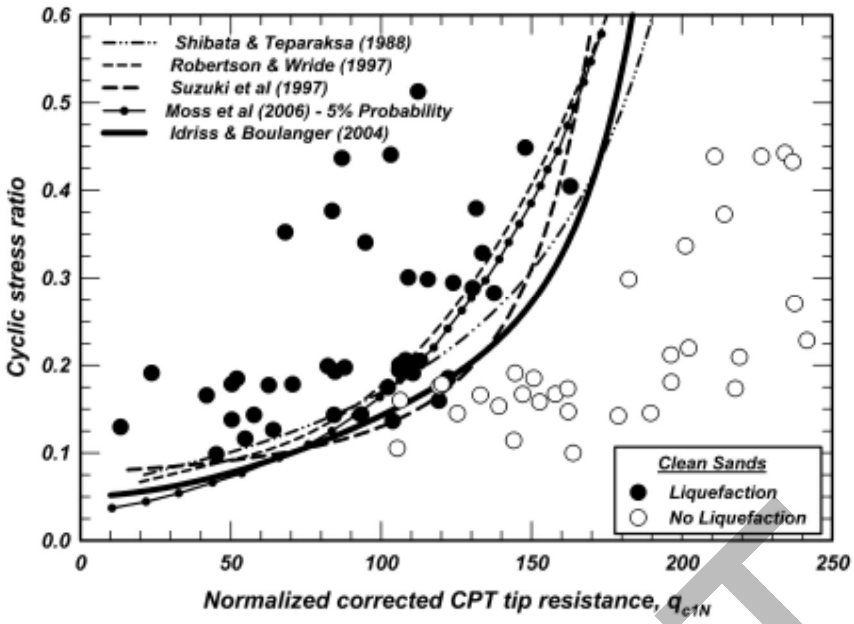


Figure C3.6 Curves relating Cyclic Resistance Ratio (CRR) to $(qc1N)_{cs}$ values for earthquake magnitude Mw =7.5. (Idriss and Boulanger (2008)).

7. There is considerable uncertainty in many of the parameters used in these methods. Users should not regard the outcomes as clearly liquefiable or not liquefiable for soils plotting near the boundaries. This can be readily accounted for in a risk based framework as these ANCOLD Guidelines are recommending.

C3.4.4.4 Shear Wave Velocity (V_S) Based Methods

Youd et al (2001), Seed et al (2003) and Idriss and Boulanger all refer to Andrus and Stokoe (2000). Kayen et al (2012) have gathered more case data and have developed a probabilistic approach based on this.

Idriss and Boulanger (2008) point out the relative lack of sensitivity of V_S to changes in relative density. A relative density range of 30% to 80% results in about 7 x difference in SPT “N” value, 3 x in CPT cone resistance and only 1.4 x in V_S. As a result greater weight should be placed on SPT and CPT based methods.

This fundamental constraint means the method is mostly restricted to use in gravelly soils where SPT and CPT samplers are affected by the gravel particles.

C3.4.4.5 Allowance for the Age of the Soil Deposit

There are a number of authors who have presented data to support the conclusion that the age of the soil deposit has an effect on the resistance of the soil to liquefaction. These include Youd and Perkins (1978), Seed (1979), Arango et al (2000), Leon et al (2006), Andrus et al (2009) and Hayati and Andrus (2009).

Andrus et al (2009) present an alternative method based on measured to estimated shear velocity V_S. They include data which shows how the shear wave velocity V_S and CPT and SPT can give quite different assessments of liquefaction potential in Pleistocene and Tertiary sands.

It is suggested that these methods may be considered as additional information where SPT and CPT based methods are showing marginal potential for liquefaction in Pleistocene and Tertiary sands. However the methods are very approximate and not too much reliance should be made on them.

They should also be considered for application in assessing the liquefaction potential of recently deposited mine tailings and dredged fills, as there is evidence the resistance of these to liquefaction may be lower than for the soils in the databases upon which the CRR are calculated.

C3.4.4.6 Use of Becker Penetration Test

In sandy gravel and gravelly sand soils the SPT sampler may hit gravel size particles and register high blow counts even if the soil is not dense. In USA and Canada for these soils the Becker Penetration Test is sometimes used.

The Becker Penetration sampler is 168mm diameter compared to 35mm diameter in the SPT sampler, and is driven by a down-hole diesel hammer. A modern improvement of the equipment is described in Ghafghazi et al (2014a, b) and DeJong et al (2014, 2016a)

Some methods for interpreting the results are given in Sy and Campanella (1994), and Ghafghazi et al (2014a, 2016b).

C3.4.5 Methods for Assessing the Strength of Liquefied Soils in the Embankment and Foundation

C3.4.5.1 Introduction

As discussed in Idriss and Boulanger (2008), Seed et al (2003) and Seed (2010) the residual shear strength that the liquefied soil mobilizes in the field is affected by void redistribution, particle intermixing and other field mechanisms which are not replicated in the laboratory.

As a result they conclude that the only practical methods for estimating the liquefied strength S_r is by back analysis of case studies and relating these to SPT and CPT data. However there are not a large number of cases available, and often for these the SPT and CPT data is variable and sometimes of limited quality, and / or the geometry of the embankment and the stratigraphy of the foundation is complicated. Olsen and Stark (2002) also describe some of these issues. As a result considerable judgement is exercised by the authors in developing the methods, and the results are uncertain, with a wide range of strengths possible.

The issue is further complicated because there are two schools of thought:

(a) Those such as Seed et al (2003), Seed (2010) who believe that the liquefied residual strength S_r should be related to the SPT $(N_1)_{60CS}$ value directly with no allowance for the effective overburden stress. This is termed a “critical state” based method.
(b) Those such as Olsen and Stark (2002) who believe the residual strength ratio S_r / σ'_{vc} should be used and related to the SPT $(N_1)_{60CS}$ value. That is the residual strength is controlled by the effective overburden stress. Idriss and Boulanger (2008) and Robertson (2010) have developed such methods. Figure C3.7 shows the Idriss and Boulanger (2008) data and recommended curves. This is termed the “normalised strength ratio” approach.

C3.4.5.2 Discussion

There is no consensus on which of the two approaches is best.

Idriss and Boulanger (2008) provide both methods without guidance on which to adopt but say they believe the residual strength ratio method provides a better representation of the potential effects of void redistribution than the liquefied residual strength, while it is recognised that neither correlation fully accounts for the numerous factors that influence void redistribution processes.

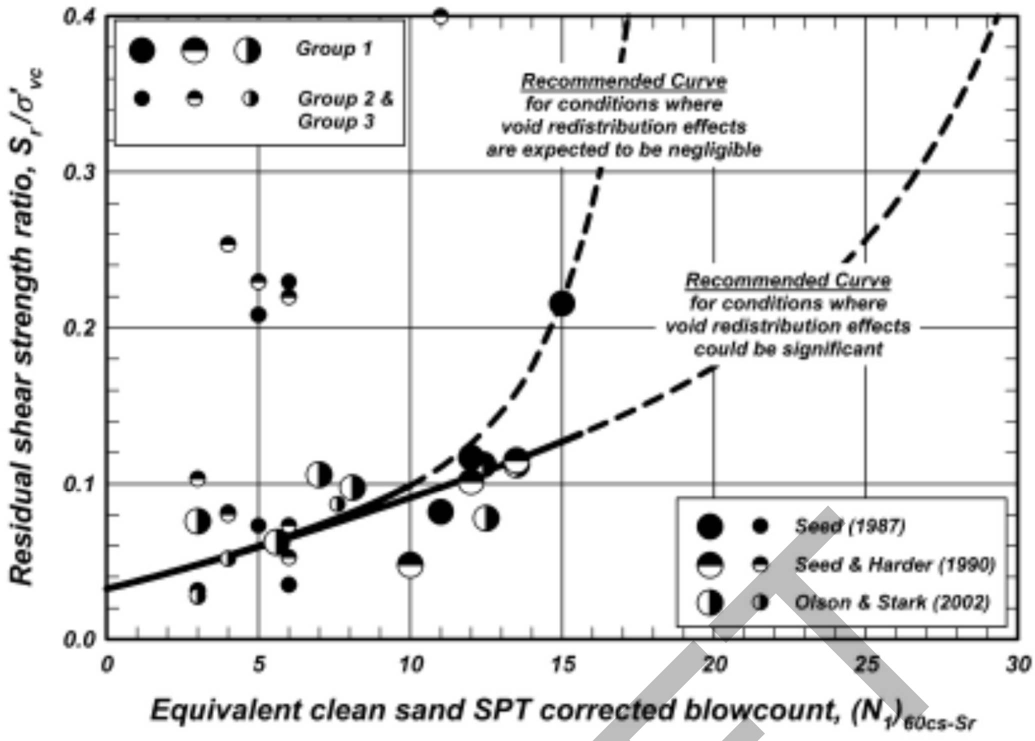


Figure C3.7. Liquefied residual strength ratio S_r/s'vc versus SPT $(N_1)_{60CS}$ value Idriss and Boulanger, 2008).

Seed (2010) strongly criticises the Idriss and Boulanger (2008) method, in particular the "recommended curve for conditions where void ratio redistribution effects are expected to be negligible". He argues that three of the data points use incorrect and over stated $(N_1)_{60CS}$ ratios. He argues that the case data does not support this curve.

Youd (2011) discusses this and says that Idriss and Boulanger unequivocally state that the upper curve is based on laboratory data and was not guided by the points with incorrect strength in any way. He also says that a transformation from contractive to dilative behaviour occurs at about a corrected SPT blow count of 15, and that should be accompanied by a major increase in shear strength. He indicates he believes Idriss and Boulanger are "on the right track".

The Robertson (2010) CPT based method has the limiting value of liquefied undrained strength ratio is assumed to be 0.4 at $Q_{1n,cs}$ =70. This is based on the soils being dilative at $Q_{1n,cs}$ > 70 so not being subject to flow liquefaction.

Baziar and Dobry (1995), Ishihara (1993) and Cubrinovski and Ishihara (2000a, b) have investigated the boundary of flow liquefaction conditions. Figure C3.8 summarizes the outcomes. These support the use of larger liquefied strengths for soils with high $(N_1)_{60}$ values at least above $(N_1)_{60}$ = 15.

Hence if the liquefied strength of such soils is controlling the assessment of the safety of the dam, and large costs are potentially involved in remedial works, it may be warranted to seek expert advice from one of the experts in the field.

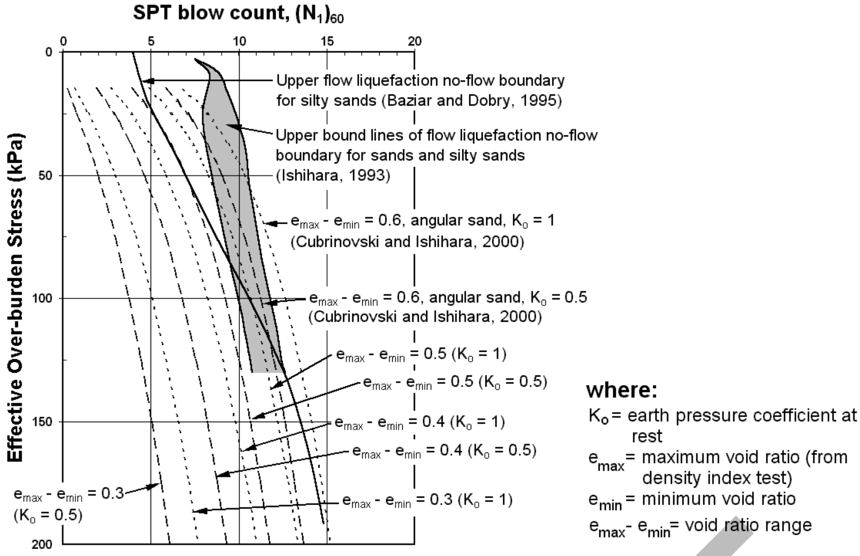


Figure C3.8. Comparison of flow liquefaction boundaries in terms of SPT $(N_1)_{60}$ for sands and silty sands from monotonic laboratory undrained tests and earthquake triggered field case (Hunter and Fell, 2003).

Given the lack of consensus on which approach to use (Critical state or normalised strength ratio) it is suggested that both methods be used for assessing the liquefied residual strength, and the results accounted for assuming they are equally valid.

Either the Idriss or Boulanger (2008), or Seed and Harder (1990) / Seed (2010) or Olsen and Stark (2002) methods may be used. These may be compared to the Robertson (2010) method.

The majority of foundation liquefaction cases in Australia will satisfy the requirements for Idriss and Boulanger (2008) "recommended curve for where void redistribution effects could be significant" because the liquefiable layer is overlain by a lower permeability strata. In view of this and the reservations of Seed (2010) it is suggested that only this curve (not the upper one) be used unless expert advice is sought.

An important issue is what liquefied residual strength to assume for design of stabilising berms for existing dams, or for the design of new dams.

Olsen and Stark (2002) indicate that in their view, at least for silty sands with > 12% fines, it is reasonable to allow for the increase in S_r which would be indicated by the increase in $\sigma_{vc'}$ from the berm. For high risk projects they suggest laboratory consolidation tests be carried out to confirm that it is parallel to the steady state line which is implicit in the assumption that S_r /σ_{vc}' is constant. NSFW (1998) caution against the use of S_r /σ'_{vc} ratios, particularly for clean sands. Given this and the uncertainty in which approach is most valid it would appear unwise to allow for the strength increase from the increased effective stress of the berm and the S_r obtained without the berm should be assumed to apply after the berm is built, unless expert opinion advises otherwise.

In all cases it should be recognised that the data indicates strength lower than the suggested design values are possible, so either a significant factor of safety is allowed for or the uncertainty modelled in risk based assessments.

C3.4.6 Methods for Assessing the Post-Earthquake Strength of Non-Liquefied Soils in the Embankment and Foundation

C3.4.6.1 Saturated Liquefiable Soils

Soils which are potentially liquefiable but which have a factor of safety > 1.0 for the seismic load being considered will generate pore pressures under the cyclic loading. These pore pressures should be allowed for in post-earthquake stability analyses. Idriss and Boulanger (2008) refer to Marcuson et al (1990) for level ground conditions but warn that pore pressures will be higher for situations where static shear stresses are present.

The Marcuson et al (1990) data may be used to give a guide to what may happen. If these become critical, and the project is sufficiently important seek expert advice.

C3.4.6.2 Cyclic softening in Clays and Plastic Silts

Boulanger and Idriss (2006), Idriss and Boulanger (2008) and Bray and Sancio (2006) describe how to estimate the effects of cyclic softening of clays and plastic silts. The discussion relates to saturated soils with over-consolidation ratios of 1 to 4. Such soils do occur in association with liquefiable soils, and may also occur as mine tailings.

The boundary between which this behaviour and liquefaction occurs is discussed in Section C3.4.3.

C3.4.6.3 Well Compacted Plastic and Non-Plastic Soils

Fell et al (2005, 2015) suggest that:

(a) For clay soil zones in the embankment and foundation assign strength and pore pressures consistent with the soil's behaviour in static loading after being cracked and disturbed by the earthquake. If the clay is contractive in nature, use undrained strengths. If it is dilative on shearing, use effective stress strengths c', ϕ'. Usually there will be some cracking and loosening, and if so fully softened strengths, would apply (e.g. $c' = 0$, $\phi' = \phi'$peak). Some apply an arbitrary 10% or 15% loss of strength.

(b) For well compacted free draining rockfill filters and dense sands and gravels adopt the effective stress strengths c', ϕ', with no change in the pore pressures. If large deformations are expected in the earthquake the dense granular materials may have loosened and will have a strength approaching the critical state strength, rather than the peak strength. In practice this can be accommodated by a small reduction from the expected peak strengths.

C3.4.7 Site Investigations Requirements and Development of Geotechnical Model of the Foundation

The following are some suggestions regarding site investigations of foundations of dams for assessing the likelihood and consequences of liquefaction.

(a) *Review of available data.*

Gather together all available data on the foundations including for existing dams, site investigations and laboratory testing carried out prior to construction; geological mapping, reports, and photographs taken during construction; monitoring data; and site investigations and laboratory testing carried out since the dam was constructed.

(b) *Preliminary geotechnical model and planning site investigations.*

Draw plans and sections of the available data, taking account of the depositional environment of the foundation soils.

From this develop an interpretive geotechnical model of the strata in the foundation, and summarize the properties in a form useful for liquefaction assessment.

Assess the adequacy of the available data and plan site investigations to supplement this data.

(c) *Site investigation methods.*

Foundations are best investigated by both boreholes in which SPT tests are carried out on potentially liquefiable soils, and undisturbed thin wall tube samples taken of other strata; and by Cone Penetration Tests (CPT, CPTU). This is desirable because CPT are best able to detect layering in the strata, but SPT and laboratory tests are required to provide data for some parameters, and to make available two methods for assessing the liquefaction potential and post-earthquake liquefied strengths.

Some of these (SPT and CPT) should be done adjacent each other for correlation purposes.

Boreholes should where practicable be drilled using wash boring methods with casing and drilling mud support. To prevent "blow in" at the base of the hole with resulting loosening of the soil in which SPT are carried out.

Wash boring should not be used for holes drilled through the core of a dam because hydraulic fracture may be induced by the drill water or mud. If hollow flight augers are used (as will be necessary for holes drilled through an existing dam) take measures to prevent "blowing" of the strata at the base of the augers with subsequent reduction in SPT blow counts.

Sonic drilling may be used but may result in densification of the soil below the casing.

Take samples from all SPT tests in potentially liquefiable strata and test for moisture content and fines content (% passing 0.075mm sieve). For representative samples test for Atterberg limits.

Record the level of the water table during drilling, and preferably install piezometers to monitor groundwater pressures at the time of the investigations and as they fluctuate with river flows.

For SPT tests in gravelly soils take blow counts for each 75mm as an aid to detecting interference by gravel particles and allow adjustment to allow for this. For gravelly soils consider the use of shear wave velocity in addition to boreholes with SPT.

The energy rating of the SPT hammer can be anything from around 55% to 90% to the energy rating of the hammer(s) used for the investigation should be measured during the site investigations. It is not sufficient to use an energy rating for the hammer determined elsewhere because it varies with the site conditions and the drill equipment.

Past practice has been to not measure the energy rating and assume 60%. This is not considered good practice and is potentially conservative in many cases so the expenditure to measure the energy rating is warranted.

For CPT and CPTU tests in layered soils adjust the data for the effects of soft and stiff layers as detailed in Figure 7 of Youd et al (2001).

It should be noted that it is almost impossible to take undisturbed samples of non-plastic soils such as those potentially liquefiable soils in the foundations or mine tailings. The sampling process disturbs and potentially compresses the soil. Even specialised samplers are likely to result in disturbance. It is for this reason most analyses are based on in-situ testing.

(d) Geotechnical model

Draw plans and sections of the available data including the data from the new investigations, taking account of the depositional environment of the foundation soils.

From this develop an updated interpretive geotechnical model of the strata in the foundation, and summarize the properties in a form useful for liquefaction assessment.

Take particular attention to the continuity and aerial extent of potentially liquefiable strata.

(e) *Laboratory cyclic shear testing.*

For many of the dynamic numerical analyses methods cyclic simple shear and cyclic triaxial tests will be required to provide inputs into the models. Expert advice should be obtained on the nature and detailed requirements for such tests.

C3.4.8 Liquefaction Analysis

The analysis of liquefaction potential is best carried out as follows:

(a) Assemble the SPT and CPT /CPTU data for potentially liquefiable strata into spreadsheets. In so doing assess the data for quality and inconsistencies and remove or "tag" as unreliable poor quality or inconsistent data.
(b) For SPT include all "N" in the spreadsheet. For CPT / CPTU enter data at 50 mm intervals as recorded.
(c) Analyse the data to determine if the soils are potentially liquefiable using the criteria described above. Exclude from further considerations soils which are not liquefiable, or which are above the water table, and will not be saturated under any condition.
(d) For the potentially liquefiable strata, asses the factor of safety against liquefaction for a number of peak ground accelerations within the range to be assessed for the site.
(e) From these data, plot plans and sections showing contours of the Annual Exceedance Probability (AEP) of liquefiable zones.

When assessing whether the soil may be saturated a cautious approach should be adopted taking into account the following:

(i) For some dams the tailwater level may vary throughout the year and during floods. As a result some soils in the foundation and in the embankment which are potentially liquefiable may be below the water table part of the time. It may be appropriate to assume the soils in the water table range are sufficiently close to fully saturated that they may liquefy.
(ii) Soils in the capillary zone above the phreatic surface may be sufficiently saturated to liquefy. However the height of capillary rise is not likely to be large for sandy soils.
(iii) As noted in Section 3.1, for tailings dams using upstream or centreline construction, there is a potential for liquefaction of loose to medium dense partly saturated tailings where perched water tables or nearly saturated zones may exist above the measured phreatic surface. This may lead to a series of persistent saturated layers within the tailings.
(iv) Hossain (2010), Yang et al (2004), Nakazawa et al (2004) and Tsukamoto et al (2002) present information which shows that the cyclic resistance of partially saturated sands is higher than saturated sand, and that these can be related to the compressional ("P" wave) velocity.

C3.4.9 Assessment of the Likelihood and the Effects of Cracking of Embankment Dams Induced by Seismic Ground Motions

Fell et al (2008), Section 5.5 describes a method for estimating the likelihood of transverse cracks in the embankment caused by earthquake. It relies on the damage classification method of Pells and Fell (2002, 2003).

Fell et al (2008) uses these data to estimate the likelihood of failure of the dam given the cracking.

These methods are empirical, and very approximate.

C3.5 Methods for Upgrading Embankment Dams for Seismic Ground Motions

C3.5.1 General approach

No commentary

C3.5.2 Embankment Dams not Subject to Liquefaction

No commentary.

C3.5.3 Embankment Dams Subject to Liquefaction

Mitchell (2008), Idriss and Boulanger (2008) and Seed et al (2003) discuss the various ground improvement methods and their limitations. These are summarized in Table C3.2.

The most widely used methods in Australia have been "stone" columns, deep soil mixing, and removal and replacement.

"Stone" columns are the most common but there are detailed design matters which need to be considered. These include:

(a) If the columns are situated on the downstream side and there is no cutoff through the alluvium upon which the dam is constructed, the columns may acts as drains for seepage water. In this case they should be backfilled with sandy gravel designed to act as a filter to the surrounding soil, or backward erosion may initiate into the columns. This can create problems with the construction of the columns as the backfill is low permeability. For some projects the backfill has been designed as a "some erosion" filter, using the Foster and Fell (2001) method.

(b) The composite strength of the columns and the surrounding soil is estimated allowing for the relative areas of the columns and the surrounding soil. The strength of the surrounding soil should be taken as the liquefied strength unless it can be demonstrated that the soil between the columns is sufficiently permeable to allow dissipation of pore pressures built up during the cyclic loading. This can be assessed by the method of Seed and Booker (1977). Idriss and Boulanger (2008) indicate that method has been refined by Onoue (1988), Iai and Koizumi (1986) and Pestana et al (2000), but they caution that uncertainties in the hydraulic conductivities of the surrounding soil (particularly as it is disturbed by construction of the columns) and the columns themselves make it difficult to have confidence of efficient drainage except in relatively high permeability soils.

(c) Seed et al (2003) discuss the effect of the stiffer stone columns attracting the cyclic shear stresses and potentially reducing the build-up of pore pressure in the liquefiable soil. They caution against relying upon this because the columns still flex if they are longer than three times their diameter.

Some examples of seismic upgrades of dams are given in Davidson et al. (2003), Toose et al. (2007), Mejia (2005), Dise et al. (2004) and Luehring et al. (2001)

Table C 3.2. Summary of ground improvement methods and their application for remedial works for liquefiable foundations (Fell et al, 2015)

Method	Method of Improvement	Soils for which it the method is applicable	Limitations and Comments
Vibroflotation	Densification, increased lateral stresses.	Sand, sand with some silt, gravelly sand	Ineffective if silt content > 20% approx. May not penetrate gravelly strata.
Vibro-replacement e.g. stone columns	Densification, increased lateral stresses, reinforcement, increased drainage.	Sand, silt, clay	Ineffective densification if silt content > 20% approx. May not penetrate gravelly strata. Columns may be constructed by driving casing, removing soil from within casing, and compacting “stone” as casing is withdrawn. Treatment must penetrate through liquefiable soil or treatment will be ineffective at the base.
Dynamic compaction	Densification, increased lateral stresses.	Sand, silty sand	Ineffective if silt content > 20% approx. Effective only in the upper 10m. Quality control difficult. Vibration of adjacent structures.
Compaction grouting	Densification, increased lateral stresses, reinforcement.	Sand, silt, clay	Ineffective at depths < 6m. Augers must penetrate through liquefiable soil or treatment will be ineffective at the base. Quality control may be difficult.
Deep soil mixing	Reinforcement, reduce earthquake induced strains and pore pressures,	Sand, silt, clay	Augers must penetrate through liquefiable soil or treatment will be ineffective at the base. Quality control may be difficult. Soil-cement elements may be brittle.
Jet grouting	Reinforcement, reduce earthquake induced strains and pore pressures,	Sand, silt, clay	Uses very high pressures so should not be used where hydraulic fracture may damage the embankment core. Difficult to control diameter of treatment.
Vertical drainage	Drainage	Sand, sand with some silt	Ineffective in silty sands and silts
Permeation grouting	Reinforcement	Sand, gravel, sandy gravel, gravelly sand	Particulate grouts e.g. microfine cement will not penetrate fine sands because the sand acts as a filter. Chemical grouts expensive and some toxic. Quality control difficult.
Removal and replacement	Removes liquefiable soil	All soils	Dewatering, stability during construction may be problems. Gives high degree of confidence in final product.

C3.6 Flood Retarding Basins

No commentary.

C4 ASSESSMENT OF CONCRETE DAMS FOR SEISMIC GROUND MOTIONS

C4.1 Effect of Earthquakes on Concrete Dams

C4.1.1 General

It was stated in ANCOLD (1998) "*Guidelines for Design of Dams for Earthquake*" and also ANCOLD (2013) "*Guidelines on Design Criteria for Concrete Gravity Dams*" but is worth repeating here, that concrete dams do fail. However, there are no recorded cases where concrete dams have completely failed under earthquake loading, with the sudden release of the reservoir. However, a number of concrete dams have suffered very serious damage, including cracking right through the concrete. The two best known dams are Koyna Dam (a 103m high gravity dam in India) which suffered major cracking through the upper part of the dam when subjected to a M6.5 earthquake in 1967 and Sefid Rud Dam (a 106m high massive buttress dam in Iran) which suffered major cracking in the buttresses when subjected to a M7.3-7.7 earthquake in 1990. A number of other concrete dams of all types have also suffered damage under earthquake loading. Generally, the magnitude of the earthquake causing damage has been M5.3 or greater. Damage has ranged from displaced copings and minor cracking to major cracks with consequent leakage from the dam. Hansen and Nuss (2011) and Wieland and Chen(2010) give information on the experiences of concrete dams subjected to earthquake loadings. Arch dams have tended to perform well under earthquake loading. The 113m high Pacoima Dam (a double curvature arch dam) in California was subjected to a ground shaking of approximately 0.5g (horizontal and vertical) during the Northridge Earthquake in 1994. The base accelerations were strongly magnified to 2g horizontal and 1.4g vertical on the top of the left abutment: the dam suffered only minor damage.

The M7.6 Chi Chi Earthquake in Taiwan (1999) caused severe damage to the Shih Kang Dam (a gated concrete gravity structure) due to a fault rupture under the dam causing a 9m vertical differential displacement of the dam. The rest of the dam which was subjected to just the earthquake shaking, performed well. Notwithstanding, the large differential displacement of the dam, there was no uncontrolled loss of storage so technically, the dam did not fail.

Major fault displacements directly under a concrete dam are very difficult to design for. Consequently, it is not included in the scope of these guidelines. It is covered by ICOLD (1998).

ANCOLD(1998) included a section "*Analysis and Design of Concrete Dams*". However, that section was limited in that it effectively covered only concrete gravity dams. Since that time, there have been a number of earthquake analyses carried out in Australia, of other types of concrete dams (e.g. Bendora Dam – a double curvature arch dam in the ACT; Carcoar Dam – a double curvature arch dam in NSW; and Oberon Dam – a buttress dam in NSW; Moogerah Dam – a double curvature arch dam in Queensland; Mt. Bold Dam-a gravity-arch dam in SA, see McKay and Lopez (2015); Clover and Junction Dams-slab and buttress dams in Victoria). As there were no relevant local guidelines to cover these dams, resort had to be made to reference material from overseas organisations (e.g. Federal Energy Regulatory Commission (FERC) and the US Army Corps of Engineers (USACE) in the USA).

ANCOLD (1998) also had a number of other shortcomings in today's (2016) context:

- They were oriented to safety reviews and to standards based acceptance criteria;
- Acceptance criteria only given for gravity dams (see discussion above);
- There was a detailed description given for a popular simplified methodology for analysing 2d gravity dams (fenves & Chopra (1986) method) which was

probably unnecessary as documentation for this methodology is readily available via the internet;
- little information was given regarding analysis required for risk assessments (i.e. analysis required to help estimate conditional probabilities of failure of concrete dams);
- advances in computer software for undertaking sophisticated analysis of all types of concrete dams both in 2D and 3D space since ANCOLD (1998) was published; and,
- recent publication of the updated ANCOLD "*Guidelines on Design Criteria for Concrete Gravity Dams*", (ANCOLD, 2013).

The Chapter of the Guidelines and this Commentary relating to concrete dams attempts to resolve the above shortcomings by:

- emphasising the strategy of increasing the complexity of analysis only as the need requires;
- ensuring that the level of sophistication of the analysis is consistent with the available information;
- providing more information on the analysis of arch and buttress dams;
- providing more guidance on non-linear methods in recognition of more sophisticated software available.

C4.1.2 ICOLD
It should be noted that ICOLD has a number of Guidelines relating to earthquake analysis and design of concrete dams. A number of relevant Guidelines published this millennium are given below:

- Bulletin 62A (ICOLD, 2016b): Inspection of dams following earthquakes - guidelines
- Bulletin 148 (ICOLD, 2013): Selecting seismic parameters for large dams- guidelines
- Bulletin 120 (ICOLD, 2001): Design features of dams to effectively resist seismic ground motion
- Bulletin 123 (ICOLD, 2002): Earthquake design and evaluation of structures appurtenant to dams
- Bulletin 155 (ICOLD, 2013): Guidelines for use of numerical models in dam engineering

The above Bulletins should generally be considered as background information.

C4.2 Defensive Design Principles for Concrete Dams
No commentary.

C4.3 General Principles of Analysis for Seismic Ground Motions

C4.3.1 Gravity Dams
ANCOLD (2013) includes a short section on the design and analysis of gravity dams for earthquake loading. That section is very general in its nature and points to these Guidelines for information regarding gravity dams subjected to earthquake loading.

Concrete gravity dams including roller compacted concrete (RCC) gravity dams can generally by analysed in the first instance, as two dimensional structures – see Section C4.3.4 below for exceptions. This means that relatively simplistic analysis methods can be used to estimate the effects of earthquake loadings on these dams. These include the methodology of Fenves and Chopra (1986) which will be discussed in Section C.4.4.3.

C4.3.2 Arch Dams
Arch dams include single curvature arch dams (most of which were built in this country, from the end of the 19th century to the beginning of the 20th century), double curvature arch dams (which can range from thin to thick cross sections) and arch gravity dams. The principles discussed in this chapter also apply to gravity dams which are in relatively narrow, steep sided valleys and are therefore should be considered as three dimensional structures. Due to the relatively

flexible nature of arch dams (especially thin double curvature arch dams) and their inherent 3D nature, it can be necessary to consider factors not applicable to gravity dams or possibly, buttress dams. These include: water compressibility; dam-water interaction; wave absorption at the storage boundary and dam/foundation interaction. It is usual practice to assume that the foundation rock is massless and compressibility of the water and dam-water interaction ignored if the fundamental frequency of the storage is greater than that of the dam and foundations alone. Duron et al (1994) give a procedure for calculating the fundamental frequency of the storage.

C4.3.3 Buttress Dams

Buttress dams can range from mass buttress dams which are akin to concrete gravity dams to thin buttress dams which are essentially reinforced concrete structures.

Slab and buttress type dams can be sensitive to cross valley shaking. The reinforced concrete slabs and the buttress corbels supporting them have to be able to handle upstream/downstream shaking. However, the relatively thin buttresses and the strutting system between the buttresses have to be able to handle the cross valley shaking as well as upstream/downstream shaking. The interface between the slabs and their supports has to be able to safely transmit the shear forces between them.

Applying concrete buttressing to the downstream face of an existing concrete gravity dam is a common form of raising/strengthening those dams (e.g. Burrinjuck Dam in NSW, the spillway at Hinze Dam in Queensland, Wellington Dam in WA). This buttressing can be either continuous (Hinze and Wellington Dams) or intermittent (Burrinjuck Dam). Analysis of these dams for earthquake loadings (as well as static loads applied post buttressing) has to consider the stress state of the dam prior to applying the buttressing (i.e. how composite action is set up between the original and new concrete) as well as any other locked in stresses that could exacerbate cracking when earthquake stresses are added.

C4.3.4 Other Types

Arch gravity dams and gravity dams built in narrow, steep sided valleys will be generally reviewed according to the concrete gravity guidelines. However, 3D analysis will generally be required in addition to any simplified 2D analysis to ensure that undesirable stability conditions/stress regimes are not developed in the dam.

C4.4 Seismic Structural Analysis of Concrete Dams

C4.4.1 Material Properties

C4.4.1.1 Concrete

The compressive and tensile strengths of concrete increase as the rate of loading increases. The dynamic compressive and tensile strengths of concrete can therefore be expected to be greater than the static strengths. As dynamic compressive stresses are rarely of concern, the allowable compressive stress for static loading can be used also for dynamic loading. Raphael (1984) states that the apparent tensile strength of concrete under seismic loading which should be used with linear finite element analyses is given by:

$$f_r = 0.65\, f_c^{2}$$

where f_c is the concrete compressive strength in MPa and f_r is the apparent seismic tensile strength in MPa. Values given by this formula are some 50% greater than the apparent tensile strength for static loading. Raphael suggests that f_r is twice the splitting strength of the concrete under static loading. Clough and Ghanaat (1993) suggest that the apparent dynamic tensile strength is about 25% greater than the measured static value which gives apparent tensile strength about 20% of the standard compressive strength. They further suggest that there may be a 15 to 20% loss of strength across lift joints. These figures may be even lower for poorly constructed or defective lift surfaces. However, the peak dynamic tensile stresses only exist during a fraction of a response cycle. Even though these peak stresses may greatly exceed the tensile strength of the concrete, any cracking that might be initiated will not have time to fully develop. It is well recognised that a single spike of excessive localised tension should not be taken to represent dam failure.

In consideration of the above however, these guidelines recommend that for sound lift surfaces, the apparent tensile strength to be used is 16% of the standard compressive strength.

For dynamic modulus of elasticity, Clough and Ghanaat (1993) suggest a value 25% greater than the static value and these guidelines recommend this be adopted. For existing dams, the elastic modulus of the concrete mass may be determined using geophysical means (e.g. derived from measured shear wave velocity). Values obtained should be compared with static and dynamic small sample laboratory test values for credibility.

C4.4.1.2 Foundations

The Young's modulus of deformation for a jointed rock mass can be estimated from the Geological Strength Index (GSI) using the method proposed by Hoek et al. (1998) and Hoek and Brown (1997. Fell et al (2015) describe these and the method by Douglas (2003). The dynamic modulus may be higher than the static modulus as discussed in Clough and Ghanaat (1993) and Scott and Von Thun (1993) and may, as for concrete, be obtained by geophysical means, or by relation to the static modulus.

A dam's foundations will normally contain joints, shears, and bedding. Consequently, it will not be possible to transmit tensile stress within the foundations and the allowable tensile strength for the foundations will therefore normally be assumed to be zero. However, if extensive site investigation and strength testing is able to prove that the foundations for a particular dam site are massive rock, e.g. massive sandstone, are capable of transferring tensile stresses, then the tensile strength of the rock may be included for small concrete structures less than about 5m high

C4.4.2 Loads

C4.4.2.1Static

Static loads applied to a dam for seismic and post-earthquake analyses for all types of concrete dams will generally be the same as those defined in the ANCOLD (2013)

C4.4.2.2 Uplift

No commentary.

C4.4.2.3 Silt

No commentary.

C4.4.2.4Seismic

General

Earthquake ground motion occurs in all directions. Consequently, it is recorded in 3 orthogonal directions: 2 horizontal and one vertical. For 2D linear elastic models, sometimes only a single response spectrum (see below for a discussion on response spectra) is available. In this case, it is necessary to consider what vertical earthquake motion might be concurrent with the horizontal shaking. In the past, the vertical acceleration used in analysis has been taken as a proportion (e.g. half to 2/3) of the peak ground acceleration. USACE (2005) gives the following multipliers for vertical spectral acceleration to horizontal spectral acceleration depending on the distance of the site from the earthquake generating fault:

- <= 10km distance: $Sa|_V/Sa|_H = 1.00$
- 25 km distance: $Sa|_V/Sa|_H = 0.84$
- >= 40km distance: $Sa|_V/Sa|_H = 0.67$

It is ANCOLDs preferred approach to develop a vertical to horizontal ratio specific for the site as described in Section C2.8.

Inertia

Inertial loads are those loads caused by earthquake accelerations acting on the mass of the dam. The acceleration at a particular level in the dam will generally be an amplification of the ground acceleration due to the flexibility of the dam and its response to the earthquake shaking. This acceleration is applied to the incremental mass of the dam and added water mass (where added mass is used as against coupled hydrodynamic pressures - see Hydrodynamic Loads) to obtain the inertial loads. For superposition methods, there will be a set of inertia loads for each mode of vibration.

Hydrodynamic

Gravity and Buttress Dams:
For gravity and buttress dams which can be considered sufficiently rigid or for initial analysis of more flexible dams, the water can be assumed to be incompressible. The Westergaard (1933) pressure distribution is therefore valid. This pressure distribution is considered equivalent to a parabolic added mass of water fixed to the face of the dam. The incremental mass of water (m_i) in Kg is given by:

$$m_i = \frac{7}{8}\rho_w\sqrt{H(H-y_i)}\mathrm{A}_i$$

where:
H = depth of water (m)
ρ_w = density of water (Kg/m^3)
y_i
= height of incremental added mass of water, (m) = above the base of the dam
A_i =
incremental surface area on face of dam (m^2)

The above only applies for a dam having a vertical upstream face and a straight axis. For a dam having a sloping or curved upstream face, the acceleration normal to the face is used when calculating the hydrodynamic force. In this case Zanger (1952) should be used for rigid dams.

The Westergaard pressure distribution has been the standard method for many years. However, consideration should also be given to alternative hydrodynamic pressure distributions that are more relevant to the slope of the dam's upstream face and the conditions at the bottom of the storage. For example, the hydrodynamic pressure distribution of Fenves and Chopra (1986) takes into account the depth of the storage with respect to the height of the dam and the energy absorbing characteristics of the storage bottom.

USBR (2011) presents a discussion on a number of methods for estimating hydrodynamic pressures. The reference is specifically for spillway gates but is relevant for concrete dams as well.

Arch Dams:
For thin arch dams, the assumption of incompressible water and rigid dam do not necessarily hold true. If the fundamental frequency of the dam (without water) is approximately equal to the fundamental resonant frequency of the water in the storage, then the dam/storage interaction should be considered in more detail. The fundamental resonant frequency (f_1^y) in Hz of the storage is given by:

$$f_1^y = \frac{C}{4H}$$

where:
$H = depth\ of\ the\ storage$ (m)
$C = velocity\ of\ sound\ in\ water$ (m/s)

For the case where the dam is flexible and the water is compressible, finite element modelling should include the storage modelled using fluid elements, in addition to the dam. A valuable discussion on this matter is given in FEMA (2014).

C4.4.2.5 Dynamic Earth Pressures
No commentary.

C4.4.2.6 Load Combinations
No commentary.

C4.4.3 Methods Available and When to Use Them
Earthquake analysis of a concrete dam may be carried out in the frequency domain or in the time domain. In the former, a response

spectrum is used which has been derived for the dam site from available seismic data. In the latter, a time-history for ground acceleration or velocity is used that again, is site specific.

A response spectrum is usually prepared by a specialist Seismologist as described in Section 2.9 . The response spectrum is normally a plot of spectral acceleration versus natural frequency or natural period of a single degree of freedom oscillator (e.g. a lumped mass on top of a vertical, massless cantilever). Sometimes however, it is presented as a tri-partite plot giving spectral acceleration, spectral velocity and spectral displacement all plotted against frequency or period. The response spectrum gives an indication as to how ground accelerations are magnified through the height of a dam. For a more detailed description of response spectra and their theoretical background, the reader is referred to Clough and Jenzien (2003).

Time-histories of ground acceleration or ground velocity can be either recorded time-histories or synthetic time-histories. Recorded time-histories have to be derived from earthquakes having a peak ground acceleration (PGA) relevant to the annual exceedance probability (AEP) earthquake used to analyse the dam. For extreme earthquakes (e.g. the SEE), recorded time-histories will generally not be available. However, synthetic time-histories can be generated from recorded time-histories including ones from overseas earthquakes. As described in Section 2.9 two methods for generating these synthetic time-histories can be used: amplitude scaling or spectral matching. Advice should be obtained from a specialist Seismologist regarding the method appropriate for a particular dam.

There are a number of methods available to analyse concrete dams ranging in increasing level of sophistication and complexity. These are detailed in Table C4-1 of the Guideline, which is taken from ICOLD (2013):

Table C4.1 Methods of analysis of concrete gravity dams for seismic ground motions. (ICOLD, 2013)

Initial static conditions and loads Initial state of the dam (cracking , joints)… Static load conditions including uplift pressure Selection of design earthquake ground motion characteristics		
Level of Seismic Analysis	**Input**	**Output**
<u>Level 0: preliminary screening</u> Relative evaluation of seismic vulnerability of a portfolio of dams	PGA or effective acceleration, seismic coefficient (maps), smooth design spectra	Damage indices PGA, C, PSa(ξ,Ti)
<u>Level 1: pseudo-static analysis with constant seismic coefficients</u> Equivalent static forces, rigid body equilibrium - 2D gravity method	Effective acceleration, seismic coefficient (maps), Westergaard added mass	Pseudo-static max. stress and velocity

Level of Seismic Analysis	Input	Output
Level IIa - Pseudo-dynamic analysis (Chopra 1988) Standard cross-section gravity dams equivalent static forces rigid body equilibrium 2D gravity method	Smooth design spectra, hydrodynamic interaction, foundation interaction	SRSS (CQC) of max. stress and velocity
Level IIb - Linear response spectra (modal) analysis Classical modal analysis - Arbitrary cross section	Smooth design spectra, reservoir added mass, Westergaard or fluid elements, Foundation model	SRSS (CQC) of max. stress and velocity
Level IIIa - Linear time-history analysis - frequency domain Linear elastic FE analysis EAGD84, EACD-3D	Spectrum compatible accelerograms, analytical reservoir compressible model, analytical foundation model with visco-elastic half space	Time-history envelopes of stress, velocity, acceleration, displacement
Level IIIb - Linear time-history analysis - time domain Linear elastic FE analysis 2-D, 3-D)	Spectrum compatible accelerograms, added Westergaard mass (or added mass using an alternative method e.g. Fenves & Chopra, 1986) for reservoir or fluid elements, foundation model	Time-history envelopes of stress, velocity, acceleration, displacement
Level IVa - Non-linear time-history analysis - Time domain Finite element analysis fracture analysis (cracking) Level IVb - Non-linear hybrid time-frequency domain (HFTD methods) - EAGD slide Solution of non-linear frequency dependent equations of equilibrium	Spectrum compatible accelerograms, added Westergaard mass (or added mass using an alternative method e.g. Fenves & Chopra, 1986) for reservoir or fluid elements, foundation model, fracture material properties (Kic, Kiic, Gf)	Cracking response, stability of cracked components

These ANCOLD Guidelines are generally consistent with this table by ICOLD. Further details on numerical methods appropriate to the earthquake analysis of concrete dams can be found in the ICOLD Guidelines as well as USBR (2006) and FEMA (2014).

C4.4.4 Analysis in the Frequency Domain

Linear Elastic

(a) The simplest method (item (i) above) for undertaking a pseudo-static stability analysis of a concrete gravity dam is to use the method of Fenves and Chopra (1986) with a site specific response spectrum for the size (AEP/PGA) earthquake being examined. In general terms, the natural frequency or period of the dam is calculated for a number of modes of vibration. The spectral acceleration is then determined for these frequencies or periods, from the design response spectrum for the dam site. Detailed documentation on the method is readily available via the internet. The method can be worked through on a spreadsheet and assumes a triangular dam on a rigid foundation to calculate the fundamental natural period. This natural period is then modified using supplied charts to estimate the effective fundamental natural period and damping factor accounting for the elastic modulus of the concrete in the dam relative to that of the rock foundation, the depth of water against the dam and the amount of seismic wave reflection at the bottom of the storage. The generalised mass of the dam and the earthquake force coefficient (which allows the multi degree of freedom dam to equate to a single degree of freedom oscillator) are then calculated. The hydrodynamic effect of the water is added as a series of lumped masses using Westergaard's Theory to the lumped masses of the dam (see Sub-section C4.4.2.4). Inertial forces at the levels of the lumped masses are then calculated, with an adjustment that allows for higher modes of vibration. These forces can then be added to the static forces on the dam, in a normal 2D cantilever stability analysis of the dam to estimate extent of cracking and sliding stability. A set of inertial forces should be calculated for the dam with the storage at minimum draw-off level to estimate the extent of cracking from the downstream toe/face. This cracking may limit the allowable cracking from the upstream heel/face for either flood or earthquake cases, especially if cohesive strength is being relied upon.

(b) Most commonly available finite element analysis (FEA) software packages appropriate for concrete dam analysis include the ability to determine the natural frequencies and mode shapes for a structure and to carry out a response spectrum analysis such that stresses induced by earthquake loading can be determined.

(c) The most recent review of pseudo dynamic analyses using finite element methods is given in FERC (2016). This reference inter aila, compares Westergaard's Method for hydrodynamic pressures with other researchers.

The extreme response values for the various modes of vibration are combined using the square root, sum of the squares method (SRSS) or complete quadratic combination method (CQC) combination. In turn, they are further combined using the appropriate combination method to include the effects of all three components of the earthquake ground motion. The resulting overall dynamic response values obtained will have no sign and may be considered to be either positive or negative. The SRSS method usually provides a conservative estimate of the maximum response when the vibration periods of the dam structure are well separated. However, due to the method ignoring the correlation between the adjacent modes, the total response for the closely spaced vibration periods is underestimated. In that case, it is better to use the CQC method.

Unless the foundations of the dam are significantly stiffer than the concrete in the dam (and can therefore be considered as being rigid), it is essential that the foundations are included in any FEM. Usual practice is to give the foundation rock zero density and appropriate stiffness parameters.

C4.4.5 Analysis in the Time Domain

Linear Elastic

Many commonly available finite element analysis (FEA) software packages include the ability to determine the earthquake response of a dam using either direct integration or superposition methods. The former method is more accurate but requires greater computer resources. The method is less conservative than methods in the frequency domain. Linear elastic analysis in the time domain should be undertaken prior to any non-linear analysis. As noted in Section C4.4.4 it is important to include the foundation rock in any FEM, with time-history analysis, the damping factor has to be built into the analysis. This is different to a response spectrum analysis where the input data (the response spectrum) accounts for the damping.

Non-linear

A publication by the USBR (2006) on the state of practice for the non-linear analysis of concrete dams provides up to date advice on dynamic material properties, loads and load combinations, damping, fluid elements, appropriate FEA methods, FEM philosophy and ways of dealing with cracking.

A publication by FEMA (2014) provides advice on analyses that might be carried out as part of a risk assessment for a concrete dam.

Consideration should be given to the direction that a crack might propagate. For cracking at the base of a dam, the crack is likely to propagate along the foundation interface. For cracking through the concrete of the dam itself, a crack is likely to be horizontal along a lift surface where the tensile strength across the lift surface is less than that of the concrete. However, where there is no discernible difference in tensile strength between the lift surface and the concrete in general, the crack could propagate in a downward direction as well as in the downstream direction (following the principal stresses). This will affect sliding and overturning stability and should therefore be taken into consideration. Constitutive models in some FEA code exist for doing this. Where horizontal cracking is assumed, gap elements that allow frictional resistance (but only defined or zero tensile strength) should be used.

In modelling the dam's foundations, appropriate consideration should be given to the geological/geotechnical model, especially with regard to continuous defects along which movement might occur.

It is essential that damping factors and stiffness moduli are appropriately modelled such that the hysteretic behaviour of the dam and foundations is properly captured.

C 4.5 Approach to Analyses and Acceptance Criteria

C4.5.1 During the Earthquake

C4.5.1.1 Gravity Dams

FERC (2016) states that stresses induced during extreme earthquake loading will induce stresses exceeding relevant material strengths. Consequentially, no acceptance criteria are given for the behaviour of a gravity dam during the course of extreme earthquake shaking. FERC emphasises the need for post-earthquake static analysis. ANCOLD (2013) and this guideline take the approach that pseudo-static methods are useful and in general likely to be conservative, so if factors of safety are ≥ minimum required in Tables 6.1 and 6.2 of that Guideline the behaviour of the dam for that seismic ground motion is likely to be satisfactory.

The USACE (2007) requires that during extreme earthquake shaking, acceptance based on linear elastic time-history analysis depends upon the demand/capacity ratio (DCR) for the dam and the cumulative time excursions of non-linear behaviour. The demand is defined as "*the ratio of stress demands to the static tensile strength of the* *concrete*". The DCR is not to exceed 2 for a very limited cumulative non-linear duration and reduces to 1 for a cumulative non-linear duration of 0.75 second. The DCR method is further described in USACE (2003b).

C4.5.1.2 Arch Dams:
The same principles apply to arch dams as apply to gravity dams. In addition, consideration has to be given to whether or not to specifically model the storage (as against using empirical formulae to estimate hydrodynamic pressure distributions) in any finite element, time-history analysis. This will depend on the relative fundamental frequencies of the dam and the storage – refer Section 4.4.2.4. If an arch dam has massive gravity abutments (thrust blocks) finite element modelling has to be such that stresses at the interfaces between the ends of the arch and the abutment units can be easily integrated to produce thrust and shear forces acting on the abutment units. These forces are then used to carry out 3D stability analysis of the gravity abutment units. The acceptance criteria for abutment blocks may be taken as for concrete gravity dams.

The post-earthquake factors of safety should be as given in ANCOLD (2013) for extreme load cases.

Additional details on the evaluation of arch dams can be found in Jonker (2014).

C4.5.1.3 Buttress Dams:
Massive buttress dams (e.g. Burrinjuck Dam in NSW) should use the acceptance criteria given for gravity dams. The following discussion relates to buttress dams that are more structural in nature (e.g.thin slab and buttress dams).

While the interaction between the face slabs and the buttresses is generally statically determinate, the interaction between the buttresses and the struts between the buttresses is generally statically indeterminate. That is, there is a significant degree of redundancy in that part of the structure. Consequently, a number of struts could potentially fail without causing failure of the structure and leading to an uncontrolled loss of storage. While some structural elements in the dam may fail, overall failure of the dam (breaching) will not necessarily occur. It may be necessary to remove structural elements that are likely to fail in order to see how loads re-distribute in the dam and whether they are likely to lead to further failure of structural elements. Consideration should also be given to the ductility of various critically loaded structural elements. A number of slab and buttress dams have relatively light reinforcement in the struts for example and the cracking moment for the strut is greater than the moment capacity for the cracked, reinforced concrete section inferring a non-ductile cross section..

C4.5.2 Post-Earthquake

C4.5.2.1 Gravity Dams:
No commentary.

C4.5.2.2 Arch Dams:
Due to the usually small distance from the upstream heel to the downstream toe (compared to a gravity dam), the change in uplift force from a fully drained uplift pressure distribution (or linear from the upstream heel to the downstream toe) to a cracked uplift pressure distribution (full headwater pressure within the cracked zone) will be relatively small. However, consideration should be given to the reduced sliding resistance at the foundation interface caused by cracking in the arch. The stability of the abutment sections of the arch or of any gravity abutments should be analysed for post-earthquake loadings taking any cracking occurring during the earthquake into account.

Linear elastic FEA may be undertaken using very low elastic modulus elements along the crack(s).

The abutments of an arch dam should be analysed considering any redistribution of stresses within the arch due to cracking. Consideration should be given to the 3D analysis of gravity abutment units and of rock wedges in the abutment foundations that have viable release surfaces.

C4.5.2.3 Buttress Dams:

Carry out a static post-earthquake stability and strength analysis of the buttress dam considering structural cracking that may have occurred in face slabs and buttresses and structural that might have occurred in the struts between buttresses. Residual shear strength parameters should be used where appropriate. Consideration should be given to each individual buttress foundation as localised foundation properties could lead to differential sliding deformations which in turn, could lead to differential structural effects within the various parts of the buttress dam. Reference should be made to FERC (1997, "Other Dams") and to USACE (2007). It should be noted that the uplift pressure under the whole of a buttress foundation will be determined by the tailwater level.

C4.5.3 General

In the analysis, uplift pressures are likely to have been represented in the finite element model, by a set of self-equilibrating pressures on a thin layer of elements at the base of the dam. It should be remembered that the stresses obtained from the finite element analysis in this case, will be total stresses. In determining the effective stress distribution through the dam, the distribution of seepage (pore) pressures through the dam will have to be considered.

Post-earthquake analysis would usually be required when the dynamic analysis of the dam and its foundations during earthquake shaking indicate:

- Cracking propagates such that post-earthquake sliding stability will be reduced due to reduced cohesion (if cohesion is being relied upon).
- Cracking is such that shear strengths will reduce to residual strengths post-earthquake.

C5 ASSESSMENT OF TAILINGS DAMS FOR SEISMIC GROUND MOTIONS

C5.1 Some General Principles

Figures 19.17 to 19.24 of Fell et al (2005, 2015) show schematically the methods of construction.

C5.2 The Effects of Earthquakes on Tailings Dams

See ANCOLD (2012) for more information.

C5.3 Defensive Design Principles for Tailings Dams

These Guidelines caution against the use of upstream construction methods for High and Extreme Consequence Category tailings dams and require practitioners taking this approach to use conservative assumptions based on the following information:

(a) There have been a number of failures of such dams due to seismic loads.
(b) It is very difficult to ensure that quality control of placement of the tailings during the operation of the TSF (often many years) will be sufficient to give the required low likelihood of failure. One loose saturated layer in otherwise properly densely placed tailings is sufficient to lead to liquefaction.
(c) There are large uncertainties in predicting the liquefied strengths and deformations of liquefied tailings.
(d) It would be contradictory to say it is good practice to site conventional dams to avoid liquefaction, and not recommend against upstream construction for High and Extreme Consequence Category tailings dams.

This ultimately is a matter for State Regulators to decide. ANCOLD counsel that if upstream or centreline construction is used for high and extreme consequence category dams persons expert in liquefaction of tailings be involved; conservative lower bound strengths be adopted for design; and assumptions made for design confirmed by testing during operation of the dam.

Small raises of the tailings dam can usually be designed so there is sufficient rockfill or other non strain-weakening or cracking material in the starter dams so that adequate post-earthquake stability can be provided and deformations limited to tolerable amounts with a high degree of confidence. Essentially these become centreline construction raises.

C5.4 Design Seismic Loads and Analysis Method

This Guideline, which draws no distinction between conventional and tailings dams, recommends different design earthquake loadings during operation to what is recommended in Table 7 of ANCOLD (2012a) . The reasons for this are:

(a) The OBE should be determined by the Owner in consultation with the Consultant in consideration of dam safety and business risks.
(b) The OBE in Table 7 of ANCOLD (2012a) were not consistent with usual practice in that it is seldom that an OBE with a frequency as low as 1 in 1000 AEP is adopted. OBE of 1 in 50 and 1 in 100 are so low a load as to make them not significant.
(c) The SEE (MDE) for TSF should be consistent with those for conventional dams. ANCOLD (2012a) has a lower SEE for Low Consequence Category dams (1 in 100).

The post closure design earthquake loading is shown in ANCOLD (2012a) as the MCE for all Consequence Categories of TSF. This was in recognition of the long life after closure of a tailings dam compared to say 100 years for a conventional dam.

ANCOLD (2012a) also recommended the 1 in 1000 AEP ground motion for OBE to better capture the potential for liquefaction of tailings because they may liquefy at 1 in 1000 AEP ground motions and not at 1 in 500 AEP ground motions.

These issues have been considered for this Guideline and this Guideline concludes:

(a) In Australian conditions it is seldom possible to identify the location and activity of all faults which may contribute to the seismic hazard so the design seismic loading should be expressed in probabilistic terms, not MCE. This is as is done in Table 2.1 of this Guideline. For Extreme Consequence Category dams there is also a requirement for MCE.
(b) The requirement of ANCOLD (2012a) for MCE post closure for Low and Significant Consequence Category tailings dam could be considered onerous. It is inconsistent with the requirement for conventional dams.
(c) The argument that tailings dams are required to operate post closure for an indefinite period and therefore should be designed for the largest credible earthquake has some merit but to apply it to all tailings dams contradicts normal risk based management because the consequences of failure for Low and Significant Consequence Category tailings dams are lower than for High and Significant Consequence Category tailings dams. Also the principles of tolerable risk criteria are that they are annual, not summed over the life of a structure, so the life of the structure is not a factor.
(d) In any case many conventional dams are likely to operate for centuries.
(e) The OBE should not be used as a de facto SEE to pick up liquefaction issues. This Guideline specifically says to consider liquefaction for the SEE of Significant and Low Consequence Category dams and to use the higher loadings if liquefaction is assessed to be an issue.
(f) The principle that OBE is for Owners to decide should stand.

Based on these factors it is concluded that the post closure design seismic loading should be as for the SEE, but that the SEE should be re-evaluated at the time of closure to allow for any developments in understanding of seismic hazard and for any changes in the consequences of failure, and that allowance be made for potential increases in the PAR post closure.

C5.5 Seismic Deformation Analysis of Tailings Dams not Subject to Liquefaction

Section 6.1.5.1 and Figure 6 of ANCOLD (2012a) incorrectly indicate that the Swaisgood (1998) and Pells and Fell (2003) methods can be applied to TSF which experience liquefaction. Those methods are based on databases which exclude liquefaction cases. Swaisgood (1998) does include hydraulic fill dams but his plots do not include those which have liquefied.

C5.6 Assessment of the Effects of Liquefaction in Tailings Dams and Their Foundations

As described in ANCOLD (2012) Section 6.1.5, tailings are usually stratified, so under-drains may not result in complete drainage of the tailings. Perched water tables with layers of saturated tailings should be assumed to be present unless extensive investigations prove otherwise.

C5.7 Methods for Upgrading Tailings Dams for Seismic Loads

In mining operations there will often be a ready and relatively inexpensive supply of waste rock to construct berms to improve post-earthquake stability and reduce deformations to tolerable amounts.

The geochemistry of the rock should be considered to ensure that there are no acid mine generation issues.

C6 ASSESSMENT OF APPURTENANT STRUCTURES FOR SEISMIC GROUND MOTIONS

C6.1 The Effects of Earthquakes on Appurtenant Structures

It is important that their functional and structural integrity of appurtenant structures is retained in the event of a notable earthquake. The importance of the role of the appurtenant structures was particularly highlighted in the Wenchuan earthquake in China in 2008, as discussed in Wieland and Chen (2010), which included examples of:

- Outlet works control panels toppling leaving the outlet works inoperable;
- Penstock failure and powerhouse flooding due to rock falls; and
- Large scale rock falls and landslides leaving dams inaccessible for a number of months following the earthquake event.

It is recommended that the reader also refers to ICOLD which has a number of Guidelines relating to earthquake analysis and design of appurtenant structures including ICOLD (2002) '*Bulletin 123: Earthquake design and evaluation of structures appurtenant to dams'*.

Regardless of the method of analysis selected, the final evaluation of seismic safety of an appurtenant structure should be based on engineering judgement and experience with similar structures, keeping in mind that each structure and its immediate environment are unique and may not be duplicated elsewhere.

C6.2 Intake/Outlet Towers

C6.2.1 General

The apparent simple geometry of intake towers is often complicated by the presence of various openings and appendages. There are also often changes in wall thickness at various elevations. Sometimes, the access bridge that provides access to the tower will structurally interact with it. There can be extreme variability in the amount of reinforcement within existing towers not designed for seismic conditions with steel to concrete areas often ranging from 0.2 percent to 2 percent, a variation factor of about 10. In many existing towers, minimal or no confinement (transverse) steel is provided, which is a very important aspect in the earthquake structural engineering. There are many cases where there will be a strong and a weak axis of the intake tower depending upon the shape of the tower and the intake and outlet arrangement. It is recommended that the tower is assessed in both directions. All of this variability in the towers underscores the diversity of design encountered and the need to consider each intake tower individually, based on the actual configuration and applicable seismic loads.

In the assessment it needs to be considered that the frequency of the tower will change following the inclusion of added masses for either or both the external and internal water.

It is to be noted that in general, intake towers have historically performed well during earthquakes, with minimal reported instances of tower damage or failure.

C6.2.2 Defensive Design Principles

No commentary.

C6.2.3 Performance Requirements

No commentary.

C6.2.4 Analysis Procedures

Response Spectrum

This type of analysis assesses the maximum response of the structure to earthquake excitation by combining the maximum responses from individual modes and multi-component inputs. This method uses an added mass representation of hydrodynamic effects due to surrounding (outside) water and contained (inside) water (in the case of wet towers). In addition, it includes the effects of the tower/foundation interaction.

The determination of the hydrodynamic mass of the water should be done using Goyal and Chopra (1989a,b,c,d,e). The Square-Root-of-the-Sum-of-Squares (SRSS) method is appropriate to combine the multidirectional loading components. Although, the Complete Quadratic Combination (CQC) method can be used as an alternative to the SRSS method. A damping ratio of 5% is typically adopted for concrete structures, for steel structures 2% damping is typically adopted. The accuracy of the results depends on the number of vibration modes considered and the methods of combination used for the modal and multi-component earthquake responses. The response spectrum analysis procedures are detailed in full in USACE (2003a) and USACE (2007).

The response spectrum analysis is adequate for towers whose responses to earthquakes are within the linear elastic range. All towers should be designed to remain in the linear elastic range for the OBE. However, it will not generally be necessary for the tower to behave elastically during the SEE. Therefore, in order to ensure that the tower can survive intense ground shaking due to the SEE with limited damage, it should possess a ductility capacity greater than ductility requirements imposed by the ground motion. This is expressed as a demand capacity ratio (DCR), which is the ratio of the computed section force demand to the section force capacity. For an SEE the minimum DCR should be taken as equal to 2 for flexure and 1 for shear.

Linear Time-History Analysis

Linear time-history analysis involves computation of the complete response history of the structure to earthquakes. The procedure for this analysis is similar to that described for the response spectrum procedure, except that earthquake demands are in the form of acceleration time-histories, rather than response spectra and the results are in terms of displacement and stress (or force) histories. The results of such analysis serve to demonstrate the general behaviour of the seismic response, and can provide some estimate of the expected inelastic behaviour or damage when the non-linearity is considered to be slight to moderate. Time-history analysis provides valuable time-dependant information that is not-available in the response spectrum analysis. Especially important is the number of excursions beyond displacement levels where the structure might experience strength degradation (strain softening).

Linear time-history analysis involves computation of the complete response history of the structure to earthquakes. USACE (2007) provides details for the assessment and criteria using linear time-history analysis. The basic procedure is to perform a linear time-history analysis with appropriate amount of damping to obtain bending moment DCR ratios for all finite elements. Initially a damping ratio of 5 percent is used and then increased to 7 percent if DCR ratios are approaching 2 and to 10 percent if they exceed 2. After adjusting the damping the damage is compared against the criteria.

From the outcomes of the assessment the damage is considered moderate and acceptable if the following conditions are met:

- Bending moment DCR ratios computed on the basis of linear time-history analysis remain less than 2;
- Cumulative duration of bending moment excursions above DCR ratios of 1 to 2 fall below the acceptance curve provided in USACE (2007); and
- The extent of yielding along the height of the tower (i.e. plastic hinge length for DCR ratios of 1 to 2) is limited and falls below the acceptance curve provided in USACE (2007).

If the DCR ratios exceed 2.0 or the cumulative duration and the yield lengths rise above the acceptance curves, the damage is considered to be severe and would need to be assessed using non-linear analysis procedures.

Non-linear Time-History Analysis
The non-linear analysis of towers is typically complicated and time consuming. Until only recently this type of analysis has been considered non-practical. However the increase in capabilities of computers and the further understanding of the performance of these structures in earthquakes is leading to more emphasis being placed on these types of analyses. For details on non-linear analysis of intake tower refer to USACE (2007).

Stability Assessment - Sliding

Response Spectrum Method
The sliding stability for the response spectrum method is expressed in terms of a factor of safety. The response spectrum analysis procedures described for the Strength Assessment above are adopted to determine the inertial forces on the structure.

Linear Time-History Analysis
The results of the linear elastic time-history analysis can be used to compute time-history or instantaneous sliding factor of safety along a sliding plane. The procedures for this assessment are provided in USACE (2007). Within this analysis, the stability is maintained and sliding does not occur if the factor of safety is greater than 1. However, a factor of safety of less than 1 indicates a transient sliding, which if repeated numerous times, could lead to excessive or permanent displacement. If this is the case then the magnitude of the sliding displacement may need to be evaluated using non-linear analysis.

Non-linear Time-History Analysis
In the non-linear time-history analysis an assessment can be made of the total permanent sliding displacement of the tower. It then needs to be assessed whether the permanent displacements meet the criteria for the SEE.

Stability Assessment - Rotational

A structure will tip about one edge of its base when earthquake plus static overturning moment (M_o) exceed the structure restoring moment capacity (M_r), or when the resultant of all forces falls outside the base. Rocking responses to ground motions may include:

- No tipping because $M_o > M_r$
- Tipping or uplift because $M_o > M_r$, but no rocking due to insufficient ground motion energy
- Rocking response ($M_o > M_r$) that will eventually stop due to the energy loss during impact
- Rocking response that leads to rotational instability (considered unlikely)

Tipping Potential Evaluation
When towers are subjected to large lateral forces produced from an earthquake, the tower may start to tip and start rocking when the overturning moment becomes so large that the structure breaks contact with the ground. The assessment of the tipping potential for the tower, either a rigid or flexible tower, is provided in USACE (2007). If it is determined that no tipping occurs then the tower is considered stable. If tipping occurs then the rocking block analysis should be conducted.

Rocking Block Analysis
A tower would eventually overturn if the moment $M_o > M_r$ is applied and sustained. However, the overturning moments typically only occur for a fraction of a second in each cycle in a large earthquake, with intermediate opportunities to unload. Therefore, although rocking occurs tower structures are unlikely to become unstable. USACE (2007) provides the procedures for assessment of the tower rocking.

C6.3 Spillways
Gated spillway structures may be unstable during seismic loading due to a combination of large water loads concentrated near the top of the structure and the relative lack of mass

involved with this type of structure. Increasing the mass at the base of the structure, anchoring the structure, or other measures, may be necessary to stabilise the structure during seismic loading.

C6.4 Spillway Gates
No commentary.

C6.5 Outlet Works - Water Conduits, Gates and Valves

C6.5.1 General

World-wide experience with the performance of tunnels during earthquake has been very good. Even soft ground tunnels have performed well as long as their design incorporated some degree of articulation and flexibility. This has been attributed to decrease of the inertial effects versus kinematic ones in their seismic response and also due to decreasing rate of seismic intensity in the ground depth. However, particular attention is required at tunnel portals. Since portals are surface features, they are usually situated in weak rock, experience surface wave reflections, and are exposed to blockage by earthquake-induced rock falls and landslides. Most cases of historic earthquake damage to tunnel systems have been at tunnel portals. Particular consideration also needs to be given where tunnels go from within rock to within an embankment.

A comprehensive study of the effects of earthquake and tunnels can be found in Dowding and Rozen, 1978, and Owen and Scholl, 1981. Further information on the assessment procedures for tunnels for earthquakes is provided in Wang (1993).

C6.5.2 Defensive Design Principles

No commentary.

C6.5.3 Performance Requirements

No commentary.

C6.5.4 Analysis Procedures

Guidance is provided in FEMA (2005) regarding the seismic assessment of outlet conduits. This document suggest that unless the conduit is founded upon deep soil layers, seismic ground motions for the bedrock are typically assumed to act on the conduit. For a conduit founded on soil the ground motion should allow for amplification or de-amplification effects. For a structure with a fundamental frequency less than 33Hz, it is suggested that a pseudo-static approach is generally suitable. Where structures have a higher fundamental frequency, a response spectrum or time-history analysis may be required.

Information on the assessment and performance of tunnels for earthquakes is also provided in Wang (1993). Dynamic loads on the conduit are to include those induced from the earthquake effect on the overlying dam.

Earthquakes can cause substantial hydrodynamic forces in penstocks and pressure tunnels depending upon the foundation conditions and length of the penstock or tunnel. High dynamic pressures can occur in short sections of tunnel such as those through the body of a dam. The reader is referred to Weiland (2005) which provides analysis procedures for the hydrodynamic loads.

C6.6 Retaining Walls

C6.6.1 General

No commentary.

C6.6.2 Defensive Design Principles

No commentary.

C6.6.3 Performance Requirements

No commentary.

C6.6.4 Analysis Procedures

Depending on the magnitude of wall movements the backfill material is said to be in yielding, non-yielding, or intermediate state. Accordingly, the available methods of design and analysis of the backfill soil pressures also fall into similar categories. Further information on the analysis procedures is provided in USACE (2007).

Dynamic pressures of yielding backfill. Yielding backfill condition means wall movements due to earthquake ground motions are sufficient to fully mobilize shear resistance along the backfill wedge creating limit state conditions. The dynamic earth forces will then be proportional to the mass in the failure wedge times the ground accelerations. When designing retaining walls with yielding backfill conditions for earthquake ground motions, the Mononobe-Okabe (Mononobe and Matuo 1929; Okabe 1924) approach and its several variations are often used. This is the most common approach used in the assessment of retaining walls, refer to USACE (2007).

Dynamic pressures of non-yielding backfill. For massive structures with soil backfill, it is unlikely that movements sufficient to develop backfill yielding will occur during an earthquake. In this situation the backfill soil is said to be non-yielding and is treated as an elastic material. If idealized as a semi-infinite uniform soil layer, the dynamic soil pressures and associated forces for a non-yielding backfill can be estimated using a constant-parameter single degree of freedom model or a more elaborate multiple degree of freedom system. The dynamic soil pressures for a more general non-yielding backfill soil can be determined by finite-element procedures.

Intermediate case. The intermediate case in which the backfill soil undergoes non-linear deformations can be represented by finite element procedures using a soil-structure interaction computer program. The foundation and backfill soil are represented using plane-strain 2-D soil elements whose shear modulus and damping vary with level of shearing strains, and the non-linear behaviour is approximated by the equivalent linear method.

C6.7 Parapet Walls

Parapet walls should be designed for Static and dynamic loads to satisfy maximum concrete and steel reinforcement stresses which satisfy AS3600 (Concrete Structures).

Detailing of the wall, and how the embankment is zoned to support the wall is critical to the performance under seismic ground motions.

C6.8 Mechanical and Electrical Equipment

No commentary.

C6.9 Access Roads and Bridges

No commentary.

C6.10 Reservoir Rim Instability

The assessment of landslides is a specialist area and suitably qualified engineering geologists and geotechnical engineers should be consulted.

The following references are relevant to the assessment:

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Effect of earthquakes on landslides:

- Keefer (1984)

Methods for assessing whether landslides will travel slowly or rapidly:

- Glastonbury and Fell (2008a, 2008b, 2010)
- Fell et al (2007)
- Glastonbury et al (2002)

Methods for predicting impulse waves resulting from landslides into reservoirs:

- Heller et al (2009)
- Panizzo et al (2005)

It should be noted that only slides which move rapidly after failure, and as they enter the reservoir will result in waves. The references above give guidance on whether a slide will be rapid.

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APPENDIX A MODIFIED MERCALLI SCALE 1956 VERSION (Richter, 1958; Hunt, 1984).

Intensity	Effects
I	Not felt. Marginal and long period effects of large earthquakes.
II	Felt by persons at rest, on upper floors, or favourably placed.
III	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognised as an earthquake.
IV	Hanging objects swing. Vibration like passing of heavy trucks or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV wood walls and frames creak.
V	Felt outdoors, duration estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swings, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.
VI	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, etc. off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken (visibly, or heard to rustle – CFR).
VII	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices (also unbraced parapets and architectural ornaments – CFR). Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII	Steering of motor cars affected. Damage to masonry C, partial collapse. Some damage to masonry B, none to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX	General panic. Masonry D destroyed; masonry C heavily damaged, sometimes with complete collapse; masonry B seriously damaged. (General damage to foundations – CFR). Frame structures, if not bolted, shifted off foundations. Frames cracked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas sand and mud ejected, earthquake fountains, sand craters.
X	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, lakes, etc. Sand and mud shifted horizontally on beachheads and flat land. Rails bent slightly.
XI	Rails bent greatly. Underground pipelines completely out of service.
XII	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.

Note: Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

– Masonry A: Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces;
– Masonry B: Good workmanship and mortar; reinforced, but not designed to resist lateral forces;
– Masonry C: Ordinary workmanship and mortar; no extreme weaknesses such as non-tied-in corners, but masonry is neither reinforced nor designed against horizontal forces;
– Masonry D: Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally;
– CFR indicates additions to classification system by Richter (1958).