28 May 2010, 4:00 pm - 4:45 pm

Understanding Seismic Embankment Dam Behavior Through Case Histories

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Fifth International Conference on
Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
and Symposium in Honor of Professor I.M. Idriss
May 24-29, 2010 • San Diego, California

UNDERSTANDING SEISMIC EMBANKMENT DAM BEHAVIOR THROUGH CASE HISTORIES

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ABSTRACT

From the lessons learned from past earthquakes, it is noticed that modern embankment dams withstand the design earthquake without significant damages. In spite of this scenario it is important to prevent the occurrence of incidents and accidents of embankment dams during the earthquakes and so a deep understanding of the triggering factors is important. Well documents case histories from many parts of the world related embankment dams behaviour during recent earthquakes were carefully selected and are discussed. Based in the governed factors attention is given to the requirements for materials characterization, modelling, analysis, monitoring and safety evaluation. Ageing effects and rehabilitation of dams are analysed. The risks associated with dam projects are discussed. The benefits and concerns of dams are presented. It is important to develop new ways of thinking and strategies to address the future challenges.

I am very busy
I have already begun with my survey
And I began to write my next error.
Bertolt Brecht

INTRODUCTION

From a careful study of dam behavior during earthquakes occurrences the failure mechanisms are presented. Well documents case histories from many parts of the world related embankment dam behaviour during recent earthquakes were carefully selected and are discussed. The background of earthquake embankment dam engineering history is presented.

The seismic design and the analysis of dam stability during earthquakes are addressed. The reservoir triggered earthquakes and the causative factors are discussed. Dam monitoring and inspections of dams after earthquakes are presented. Ageing effects and rehabilitation of dams are analysed.

The risks associated with dam projects are discussed. The benefits and concerns of dams are presented. Some topics that deserved more consideration are introduced.

LESSONS FROM EMBANKMENT DAM PERFORMANCE DURING EARTHQUAKES

From a careful study of dam behavior during earthquakes occurrences the following failure mechanisms can be selected (Sêco e Pinto, 2001):

- Sliding or shear distortion of embankment or foundation or both;
- Transverse cracks;
- Longitudinal cracks;
- Unacceptable seepage;
- Liquefaction of dam body or foundation;
- Loss of freeboard due to compaction of embankment or foundation;
- Rupture of underground conduits;
- Overtopping due to seiches in reservoir;
- Overtopping due to slides or rockfalls into reservoir;
- Damages to waterproofing systems in upstream face;
- Settlements and differential settlements;
- Slab displacements;
- Change of water level due fracture of grout curtain;
- Movements on faults under or adjacent to dam.

A survey of dams behaviour during earthquakes carefully selected is presented in Annex 1.

Some interesting case histories are discussed subsequently, in order to absorb the lessons learned.

One of the early reports related with the behaviour of embankment dams during the occurrence of seismic effects describes the Sheffield Dam failure occurred during an earthquake near Santa Barbara, California in June 29, 1925.

The dam was constructed in the winter of 1917 with 219.50 m long and 76 m high. The body of the dam was composed of silty sand and sandy silt containing some cobbles and boulders, but the upstream slope was faced with a 1.20m thick clay blanket. No records of the degree of compaction are available but it was probably about 75 to 80 percent based on the standard AASHO compaction test (ICOLD, 1975).

The foundation soil consisted of a layer of terrace alluvium 1.20 to 3m thick, overlying sandstone bedrock.

There were no strong-motion instruments in existence at the time but on the basis of records obtained at distant stations, the earthquake has been assigned a magnitude of 6.3 with an epicenter located about 11.2 km north west of the dam site.

Lessons 1: Embankment dams with low degree of compaction are vulnerable to earthquakes.

During San Fernando earthquake, M= 6.6, February 9, 1971 a major slide has occurred in the upstream slope of the Lower San Fernando Dam.

Figs. 1 and 2 show dam views after the earthquake.

The dam was initiated by hydraulic fill method, with additional zones of compacted fill being added later. The slides movements were due an increase in pore pressure in the embankment due ground shaking with a loss of strength and liquefaction of the hydraulic fill (Seed et al, 1973).

Lesson 2: Hydraulic fills due the development of pore pressure are susceptible to liquefaction.

Oroville dam with 1707 m long and 235 m high was built in 1968 (Banerjee et al, 1979). The dam cross section is shown in Fig. 3. The dam has suffered August, 1, 1975 Oroville earthquake with 5.7 magnitude and has exhibited crest settlements of 9 mm and horizontal displacements of 15 mm.

The conventional pseudo-static analysis with a seismic coefficient of 0.1 g was performed during the design stage.

The Oroville earthquake disclose the existence of a previously unidentified fault in the vicinity of the dam. Due the concerns related the occurrence of a 6.5 magnitude earthquake with a hypocentral less than 8.5 km from the dam 2D and 3D finite element analyses were performed.

Lesson 3: Rockfill dams with compacted material exhibits a good behaviour during the occurrence of earthquakes. To correct simulate the dynamic resoponse of dams in steep triangular canyons a three-dimensional analysis is needed.

Fig.1 –San Fernando dam-dam view

Fig.2 –San Fernando dam –upstream slope slide

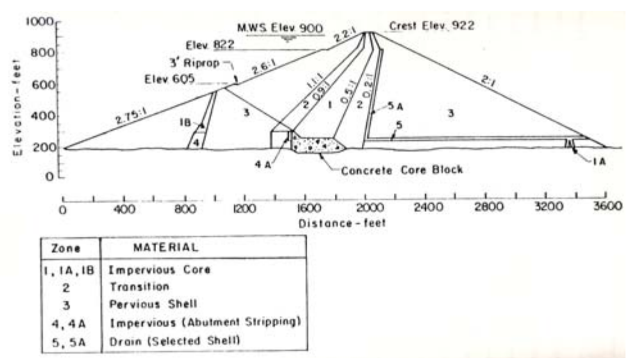
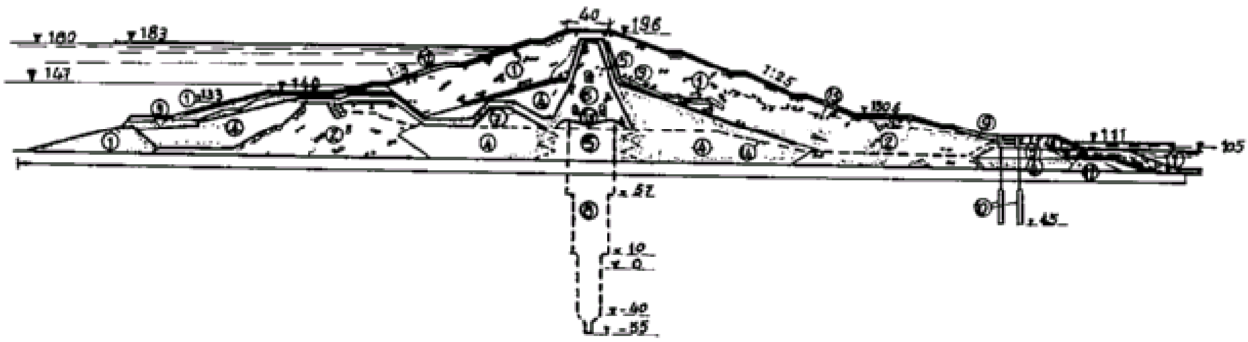


Fig.3 –Oroville dam cross section (after Banerjee et al, 1979).

High Aswan Dam (HAD) is a rockfill dam with clay core, rockfills shells and a wide grout curtain from the bottom of the clay to rock formation (Fig. 4). It is 111 m high and 3600 m long, the storage capacity is 162 km3 and is built on main river Nile (Shenouda, 1982).

For the design purposes it was considered that Aswan area was not seismic.



(1) Rockfill of muck.
(2) Sized stones sluiced with sand.
(3) Sized stones sluiced with silt and clay
(4, 4') Dune sand.
(5) Coarse sand.
(6, 7) Clay core and blanket.
(8) Grout curtain.
(9) Three layer filter.
(10) Drainage wells.
(11) D.S. prisms of fines.
(12) Protective layer of big blocks.
(13) Inspection galleries.
(14) Sluicing limits.

Fig.4 Cross section of dam (after Shenouda, 1982)

On 14 November 1981 a moderate earthquake of magnitude 5.3 occurred about 5,3 km Southwest of the dam. The sudden occurrence of this earthquake caused a significant concern due to the concentration of population in the valley and along downstream the dam. The evaluation of fault capability of releasing earthquakes in the Aswan area became a high priority problem (Shalaby, 1995).

High-gain seismographs and also a network of six portable seismographs surrounding the after shock zone were installed.

On July 1982 a telemeter network was installed.

The seismic monitoring and the telemetered network have shown a close association between the Kalabsha fault and the main shock of November 1981 and much of the subsequent local seismicity. It was also concluded that the risk of reservoir triggered seismicity was insignificant.

Seismic stability and potential deformations were assessed by a non linear finite element analysis. The results of the studies show that the occurrence of the largest potential earthquake would not jeopardize the safety and integrity of the dam and its appurtenant structures.

Lesson 4: The identification of tectonic mechanisms, location and description of faults and estimation of fault activity play an important role to assess the involved dam risk.

On May 2003 an earthquake of magnitude 6.8, with a depth of 10 km, occurred 40 km East Alger, provoking 2270 deaths. In Keddara rockfill dam, with 106m high (Fig. 5), located 30 km from the fault, only 1 longitudinal crack and 3 transverse cracks were observed (Fig.6). A value of 0.34 g was recorded in rock and the dam was designed for a acceleration value of 0.25g. No damages were observed in the gallery (Benlala, 2003).

Lesson 5: Well designed and constructed embankment dams exhibit a good behaviour for strong ground motions.

Fig .5 Keddara dam-downstream view

Fig. 6. Keddara dam-cracks observed at the crest

Matahina Dam is a rockfill embankment 80 m high, with a central core located in New Zealand.

The dam has leaked after first filling in 1967 due to core cracking, and was consequently repaired. In 1987 the dam was exposed to strong seismic shaking (peak horizontal crest acceleration 0.42g) due to M=6.3 earthquake, located at the Edgecumbe fault.

The dam is sited across the Waiohau active fault, 80 km long, with proven surface breaking during the Holocene.

For the dam safety evaluation and earthquake with M= 7.2 was selected considering the surface rupture of Waiohau fault, crossing the dam site. The value of 3.0 m in oblique was thus selected for the fault surface displacement as 2:1 (i.e. 2.7 m horizontal and 1.3 m vertical displacement). Such displacements would result in major cracking of the dam body inducing piping and internal erosion, as observed during the 1987 Edgecumbe earthquake.

The proposed strengthening is shown in Fig. 7 and consists in excavating significant part of the downstream shoulder and on keeping the existing core. The post SEE leakage control is to be ensured by placing a wide zone of filter, transition and drainage materials with 5.0 m minimum of thickness. The new crest will be approximately 40 m with the crest being heightened by 3.0 m to accommodate any settlement due to shaking and maintain sufficient freeboard (ICOLD, 1999).

Lesson 6: Due the re-evaluation of design seismic action there is a need to strength the dams in order to accommodate the settlements and leakage.

A typical cross section of New Yamamoto dam, 42 m high, built in 1990 composed by shell materials, filter and clay core is shown in Fig. 8.

During the occurrence of Niigata earthquake 2004, the dam settled 0.8 m, i.e, 2%.. The following situations have occurred contamination of drain, liquefaction of the upstream and settlements. A pond at dam crest has occurred (Fig. 9), but the overall behaviour of the dam was satisfactory to retain the water (Matsumoto, 2006).

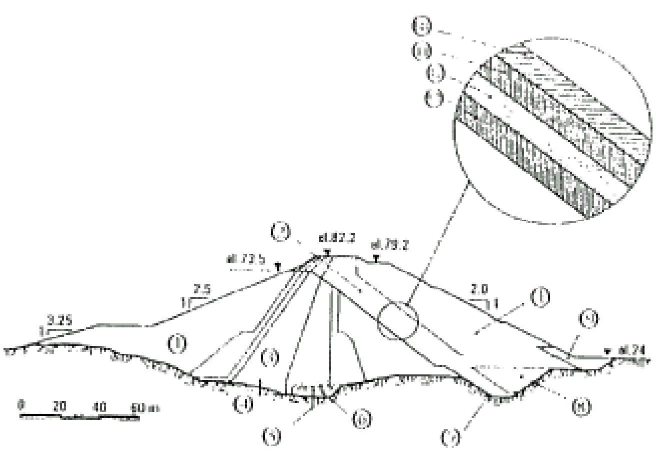


Fig. 7. Mathanina Dam (New Zealand)

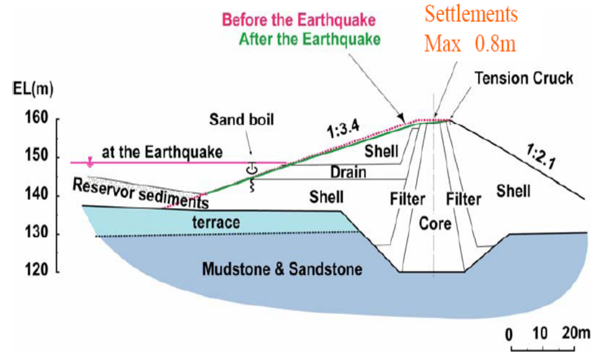


Fig.8. Yamamoto dam profile (after Matsumoto, 2006)

Fig. 9. Pond at dam crest (after Matsumoto, 2006)

Lesson 7: In spite of some high settlements that occurred during earthquakes the embankment dams still accomplish their function.

Aramos dam with 42 m high and 220 m long is located in Chile. The dam profile shown in Fig 10 has a core with fine soils and shells with gravelly sand. Due the existence of a foundation of sandy material a plastic concrete wall of 80 cm in thicken and a maximum of 22.5 m was built and the sectors of plastic wall that did not reach the foundation was injected (Verdugo and Peters, 2009).

During the construction of the embankment dam a bulldozer operating in the river bed sank showing that the ground was susceptible to liquefaction. A external evaluation of the project was performed and the recommended countermeasures have included a berm with 13m high confining the upstream shell and a battery of drainage columns between the toe of the dam and the spillway was installed.

During March 3, 1985 earthquake the dam exhibited an extremely good behaviour with a maximum settlement of 10 cm. A possible explanation is that the low SPT zones.

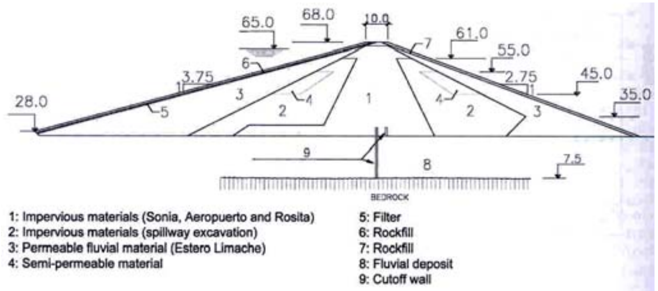


Fig. 10. Cross section of Aromos dam (after Verdugo and Peters, 2009)

are surrounded by stiffer zones that have reduced the disturbance of loose zones.

Lesson 8: Occurrence of liquefaction in a ground with heterogeneous conditions requires a deeper analysis and a better understanding of the interaction phenomena.

Tarbela dam with 143 m high built in Pakistan is an example of a dam that was designed without consideration of Darband fault which was revealed during the dam construction. So the review design with the dam cross sections showed in Fig. 11 has estimated a fault movement of 1,0m to 1,5m and the core dam was constructed of self healing material with a transition zone, a generous freeboard and a wide chimney drain on the downstream. A general view of the dam is shown in Fig. 12.

Tarbela dam was shaked by October 8, 2005 earthquake with a 7.5 magnitude. The pore pressures and seepage rise was observed in the dam right abutment. Relevant piezometers and seepage points were continuously monitored to know the trend which became normal after a few days.

Due to this event planning and installation of earthquake monitoring and strong motion recording instrumentation for dams and hydropower projects in northern areas of Pakistan was implemented.

Preparation of seismic provisions for Building Code of Pakistan which include revised seismic zoning map of Pakistan

-Catalogue of Historical earthquakes of Pakistan.

-A catalogue of available fault plane solutions of earthquakes in Pakistan.

Lesson 9: Dam behaviour during earthquakes contributes to update and implement national codes.

Zipingpu dam 156 m high and 663,7 m long is one of the largest CFRDs dam in China was built in 2006 and designed for a peak ground acceleration of 0,26g (Chen, 2008). A dam profile is shown in Fig. 13.

The dam was shaked during May 12, 2008 Wenchian earthquake (magnitude 8.0) and is located 17 km of the epicenter.

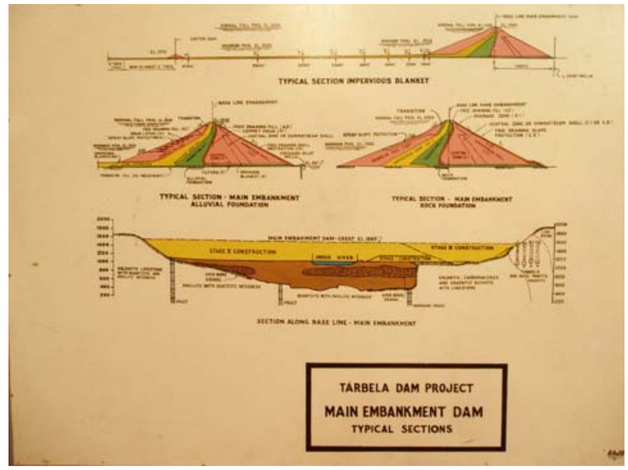


Fig- 11. Embankment dam-cross sections

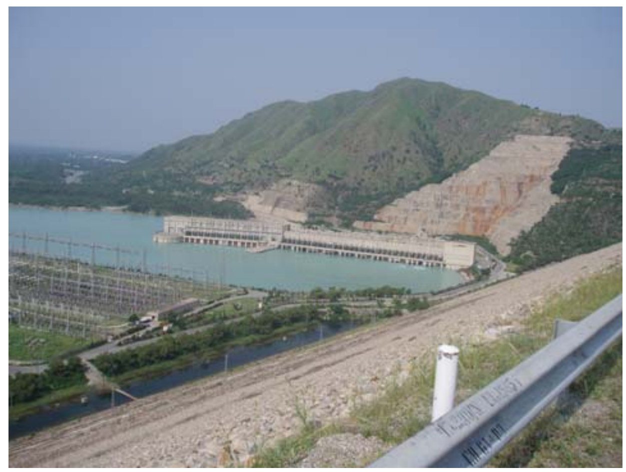


Fig. 12. Tarbela dam –general view

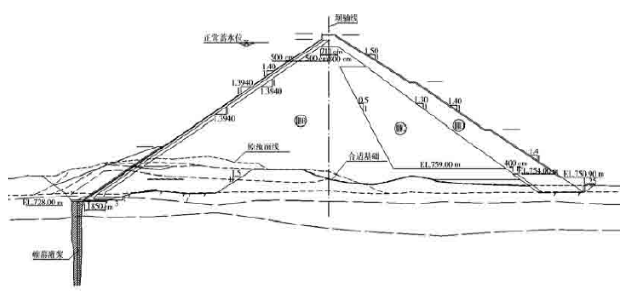


Fig 13. Zipingpu dam profile (after Chen, 2008)

During the earthquake the reservoir was low with a volume of 300 Mm 3, when under the normal conditions the reservoir

volume is 1100Mm3. Due to this situation is difficult to estimate the dam behaviour when the reservoir was full.

The crest of the dam and the concrete face were damaged. during the earthquake (Figs 14 and 15).

The dam crest settled 715mm and had a horizontal deflection of 180 mm.

Fig.14 Zipingpu dam crest damages (after Chen, 2008)

Fig. 15. Zipingpu dam joint damages (after Chen, 2008)

From the six strong motions instruments only 3 located at the crest were in good conditions at on the crest a peak acceleration of 2g was recorded

Lesson 10: In spite the apparent good behavior of CFRD dams during earthquakes there is still a lack of case histories of CFRD operating with full reservoir.

Figure 16 shows the location of dams including the main faults where strong motion records where obtained during Costa Rica earthquake January 8, 2009 6.2 magnitude. In this clear that Cariblanco and Toro projects are located very near of the epicentral area, sites (ICE, 2009).

Cipreses dam of Cariblanco project has no accelerograph installed, but is located very close (1.7 km) of the surge tank instrument and has shown cracks.

Toro II project has exhibits longitudinal cracks in the crest.

Lesson 11: Dams located near faults can exhibit a good behaviour for strong motions with only minor cracks.

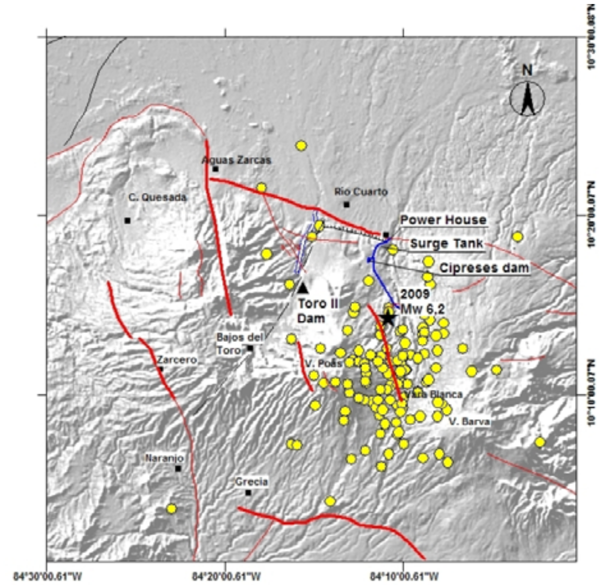


Fig. 16 Epicentral location (black star), aftershocks and other small events until January 16 (yellow circles). Main faults in the region (in red) and location of Cariblanco and Toro projects (after ICE, 2009)

Lesson 12: The current state of the art and state of practice allow the design and construction of embankment dams that exhibit a good performance record in regards of earthquake shakes with less than 1% dam failures.

BACKGROUND OF EARTHQUAKE EMBANKMENT DAM ENGINEERING HISTORY

Pre-Historic (before 1940)–This period was characterized by the development of historical earthquakes and Paleoseismicity, the use of empirical methods, the knowledge was primary and parcelled. The measurement of the destructiveness of the earthquake was based in human reaction and observed damage and use of Mercalli scale. Investigation of the earthquake induced damage due to Great San Francisco earthquake (1906) was performed by Sano (1916). For the assessment of seismic behavior of embankments dams Mononobe & Matsuo (1929) and Okabe (1924) methods were proposed.

Classic Period (1940-1983) with the attempt to organize as scientific discipline, records of typical earthquakes e.g. El-Centro earthquake (1940), the use of magnitude for the physical measure of size of the earthquake and several scales based on the amplitude of seismograph records. After Niigata and Alaska earthquakes in 1964 the first studies of liquefaction evaluation of sands and silty sands came out. Use of geophysical tests namely refraction tests, up-hole and downhole tests. Use of laboratory cyclic tests namely reasonant column tests, simple shear tests and triaxial tests for soil behavior and definition of shear modulus and damping ratio. Developments of pseudo-static methods for embankments (Ambraseys, 1960) and simplified methods for assessment of displacements (Newmark, 1965, Sarma, 1975, Makdisi and Seed, 1977). Implementation of codes in total stress SHAKE in 1971 and QUAD 4 in 1974.

Modern Period (1983-1995) characterized by the definition of seismic action using strong ground motions parameters PGA, PGV and PGD, response spectra and use of deterministic and probabilistic methods. Development of laboratory and field tests with more automation in operation, more accurate measurements, reduced costs in maintenance and production of data processing techniques with high resolution and degree of reliability, use of seismic arrays and SASW. Use of physical models e.g. shaking table, reaction walls, centrifuge tests, calibration chambers and prototype tests. Proposals for liquefaction assessment of dam materials were presented. Developments of mathematical models for dynamic analysis and codes in effective stress using plasticity models e.g. DIANA, DYNAFLOW, TARA among others. First stage of development of codes and standards. Lessons from Mexico earthquake (1985), Loma Prieta earthquake (1989) and Northridge earthquake (1994) were taken into account.

Actual Period (after 1995) with the implementation of cyclic triaxial tests and torsional shear tests. Combination of laboratory and field tests to assess design parameters. Development of more realistic coupled models, using boundary elements and discrete elements, incorporating non linear behavior, ageing, thermal effects and 3D analyses.
Verification, calibration and validation of computer codes(ICOLD, 1993). Prediction of residual strength and allowable deformation of soils exploring aerial photographs. Implementation of instrumentation and monitoring to assess seismic behavior of structures. Great emphasis on diffusion of knowledge by journal, conferences, codes of practice and development of networks. Use of case histories for a better understanding of seismic behavior of embankment dams and calibration of predictions (Sêco e Pinto, 2009b). Developments of techniques for remediation and rehabilitation of embankment dams.

SEISMIC DESIGN

Introduction

According to Aristotle (384-322 B.C.) in his book Meteorologica earthquakes were produced by the dried exhalations (spirits or winds) in caves inside the earth which trying to escape make the earth shake

Martin Lister in England and Nicolas Lemery in France in 17th century were the first to propose that earthquakes were produced by large explosions of inflammable material formed by a combination of sulfur, coal, niter and other products accumulated in the interior of earth (Udias and Arroyo, 2005). The explosive theory was also proposed by Newton`s Optics (1718) and the modern scientific ideas consider the earthquake a natural phenomenon.

In France the world was considered a good place in which everything that happened was viewed to be "for the best" and earthquake was considered with optimism. Voltaire in his novel Candide presented a hard attack to this optimistic view point. Also Kant and Rosseau defended the optimist position.

Selection of Design Earthquakes

The selection of seismic design parameters for dam projects depends on the geologic and tectonic conditions at and in the vicinity of the dam site (Sêco e Pinto, 2004).

The regional geologic study area should cover, as a minimum, a 100 km radius around the site, but should be extended to 300 km to include any major fault or specific attenuation laws.

The probabilistic approach quantifies numerically the contributions to seismic motion, at the dam site, of all sources and magnitudes larger than 4 or 5 Richter scale and includes the maximum magnitude on each source.

The dam should be designed for Design Earthquake (DE) and Maximum Design Earthquake (MDE). Both depend on the level of seismic activity, which is displayed at each fault or tectonic province (Wieland, 2003).

For the OBE only minor damage is acceptable and is determined by using probabilistic procedures (SRB, 1990).
For the MDE only deterministic approach was used (ICOLD, 1983) but presently it is possible to use a deterministic and probabilistic approach. If the deterministic procedure is used, the return period of such an event is ignored, if the probabilistic approach is used a very long period is taken (ICOLD, 1989).

2 levels for seismic activity, namely MCE (Maximum Credible Earthquake) considering a return period of 500-1000 years and DBE (Design Basis Earthquake) for a return period of 145 years, with a probability of exceeding in 100 years less than 50%, were proposed by ICOLD(1089).

ICOLD (2002) has considered 3 levels of seismic action, namely: MDE (Maximum Design Earthquake), MCE (Maximum Credible Earthquake) and OBE (Operating Basis Earthquake). Four hazard classes were defined, namely: Low with PGA<0,10g, Moderate with 0.10<PGA<0.25g, High with PGA>0.25g (no active faults within 10 Km) and. Extreme with PGA>0.25g (active faults within 10 Km).

In Eurocode 8 (1998a), in general, the hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock or firm soil called "design ground acceleration" expressed in terms of: a) the reference seismic action associated with a probability of exceeding (P_{NCR}) of 10 % in 50 years; or b) a reference return period (T_{NCR}) = 475 years. The seismic action to be taken into account for the "damage limitation requirement" has a probability of exceedance, of 10% in 50 years and a return period of 95 years (Seco e Pinto, 2009a).

Tectonic Conditions

Within this framework the tectonic conditions should include tectonic mechanisms, location and description of faults (normal, stryke and reverse) and estimation of fault activity (average slip rate, slip per event, time interval between large earthquake, length, directivity effects, etc), these factors are important to assess the involved risk.

The foundation properties for soil materials are estimated by geophysical tests (crosshole tests, seismic downhole tests and refraction tests), SPT tests, CPT tests, seismic cone and pressurometer tests (ICOLD, 2005a).

Following (ICOLD, 1998) an active fault is a fault, reasonably identified and located, known to have produced historical fault movements or showing geologic evidence of Holocene (11 000 years) displacements and which, because of its present tectonic sitting, can undergo movements during the anticipated life of man-made structures.

The current practice is the deterministic approach in which the seismic evaluation parameters were ascertained by identifying the critical active faults which show evidence of movements in Quaternary time (ICOLD, 1998).

Dense recording GPS arrays with sampling rate allow determining deformation rates in seismic active regions. Intrinsic properties of rock at depth have to be obtained in situ by deep drilling into active faults. Computational with high resolution model for stress and deformations in communicating fault systems should be developed. A better exploration of microtremors technique, directivity effects and attenuation laws is needed.

Surface fault breaking i.e. surface slip along an identified fault zone under the dam is considered as the most dangerous tectonic scenario for dam safety.

With the tendency of less favourable dam sites these tectonic conditions are getting increase attention.

The active tectonic movements result on fault breaks and in creep movements. Also block movements have to be considered in the near field of major faults.

Following Sherard et al. (1974) a concrete dam on active faults or near major active faults is not advisable and if a site with fault movements can not be avoided it is recommended to build an embankment dam

Evaluation of the displacement that could occur along the fault during the lifetime of the dam and the selection of the design details to ensure safety against fault displacement are still difficult problems to be solved (Wieland et al, 2008).

Potentially Liquefiable Soils

Empirical liquefaction charts are given with seismic shear wave velocities versus SPT values to assess liquefaction.

The new proposals integrate: (i) data of recent earthquakes; (ii) corrections due the existence of fines; (iii) experience related with a better interpretation of SPT test; (iv) local effects; (v) cases histories related to more than 200 earthquakes; and (vi) Bayesian theory.

For liquefaction assessment by shear wave velocities two methodologies are used: (i) methods combining the shear wave velocities by laboratory tests on undisturbed samples obtained by tube samplers or by frozen samples; (ii) methods measuring shear wave velocities and its correlation with liquefaction assessment by field observations.

It is important to refer that Eurocode 8 (1998b) considers no risk of liquefaction when the ground acceleration is less than 0.15 in addition with one of the following conditions: (i) sands with a clay content higher than 20 % and a plasticity index > 10; (ii) sands with silt content higher than 10% and N_1 (60)>20; and (iii) clean sands with N_1 (60)>25.

For post liquefaction strength relationships between SPT and CPT tests and residual strength were proposed by several authors.

Also to assess the settlement of the ground due to the liquefaction of sand deposits there are some proposals based on the knowledge of the safety factor against liquefaction and the relative density converted to the value of N1.

The remedial measures against liquefaction can be classified in two categories (TC4 ISSMGE, 2001; INA, 2001): (i) the prevention of liquefaction; and (ii) the reduction of damage to facilities due to liquefaction.

For the selection of the remedial measure it is important to consider: (i) Potential efficiency; (ii) Technical feasibility; (iii) Impact on structure and environmental; (iv) Cost-effectiveness; and (v) Innovation (Sêco e Pinto, 2008).

More recently it is recognized that gravelly material can liquefy.

The behavior of Keenleyside dam with foundation composed of sands and gravel was investigated. Due many uncertainties in the assessment multiple methods both field tests (SPT tests, Becher Penetration tests, shear wave velocities) and laboratory tests (triaxial and permeability tests) were used (Yan & Lun, 2003).

Performance Basis Design

The new trend for performance basis design is to consider 2 levels of seismic actions and to analyse the situation when the limit of force balance is exceeded for high intensity ground motions associated with a very rare seismic event (Sêco e Pinto, 2009b).

For the design two basic requirements are defined: (i) Non collapse requirement (ultimate limit states) i.e. after the occurrence of the seismic event the structure shall retain its structural integrity, with respect to both vertical and horizontal loads, and adequate residual resistance, although in some parts considerable damage may occur, (ii) Minimization of damage (serviceability limit state) after seismic actions with high probability of occurrence during the design life of the structure some parts can undergo minor damage without the need of immediate repair. The structure shall be designed and constructed without the occurrence of damage and the associated limitations of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself(Towhata, 2008).

Acceptable level of damage in performance based design shown in Table 1 are specified by a combination of structural and operational damage (Iai, 2009).

Table 1. Acceptable level of damage in performance based design

Acceptable level of damage	Structural	Operational
Degree I: Serviceable	Minor or no damage	Little or no loss of serviceability
Degree II. Repairable	Controlled damage	Short term loss of serviceability
Degree III: Near Collapse	Extensive damage in near collapse	Long term or complete loss of serviceability
Degree IV Collapse	Complete loss of structure	Complete loss of serviceability

ANALYSIS OF DAM STABILITY DURING EARTHQUAKES

Introduction

Principles are the fundamental of dam safety. The dam shall safely retain the reservoir and any stored solids and pass environmentally acceptable flows as required for all loading conditions ranging from normal to extreme loads, commensurate with the consequences of failure.

Practices and Procedures suggest methodologies that may be used to meet the Principles.

Table 2 presents the seismic criteria probabilistic approach recommended by Canada

Table 2. Seismic Criteria-Probabilistic approach recommended by Canada

Consequence Class of Dam	EQ Design Ground Motion (EDGM-Mean Annual Exceedance Probability
Low	1/500
Significant	1/1000
High	1/2500
Very High	1/5 000
Extreme	1/10 000

New Zealand and UK have suggested a return period of 10 000 years for SEE for Extreme and High risk dams, 3000 years for Moderate risk and 1000 years for Low risk.

Experimental Models

Experimental methods are used to test predictive theories and to verify mathematical models. Nevertheless some limitations they are useful for physical modeling in geotechnique.

The most popular techniques for embankment dams are shaking table and centrifuge models.

Japan E-Defense 3D Shaking table with 15m long 20m wide with payload capacity of 1200 t0ons and with maximum accelerations of 9 m/s2 is one of the largest facilities in the world (Tokimatsu, 2007).

2D shakers were installed in UC Davis and RPI geotechnical centrifuges with robots.

A review of the existing testing facilities for earthquake research in order to address new scientific topics in earthquake engineering was performed by Taucer (2005).

Mathematical Models

The following dynamic analysis of embankment dams is used (Sêco e Pinto et. al, 1993):

i) pseudo-static analyses;
ii) simplified procedures to assess deformations;
iii) dynamic analysis.

The pseudo-static analyses assume a rigid or elastic behavior for the material (Ambraseys, 1960) and have the limitation that the seismic coefficient acts in one direction for an infinite time.

Simplified procedures to assess deformations were proposed by Newmark (1965), Sarma (1975), Makdisi and Seed (1977) and Bray (2007) and have given reasonable answers in areas of low to medium seismicity.

Newmark´s original sliding block model considering only the longitudinal component was extended to include the lateral and vertical components of earthquake motion by Elms (2000).

The use of dynamic pore pressure coefficients along with limit equilibrium and sliding block approaches for assessment of stability of earth structures during earthquakes was demonstrated by Sarma and Chowdhury (1996).

For large dams where strong earthquakes have occurred more sophisticated methods were used (Seed, 1980, ICOLD, 2001).

Several finite element computer programs assuming an equivalent linear model in total stress have been developed for 1D –SHAKE code (Schanabel et. al., 1972), 2D –LUSH code

(Idriss et. al., 1973; Lysmer et al., 1974) and pseudo 3D TLUSH code (Lysmer et al., 1975).

Since these models are essentially elastic the permanent deformations cannot be computed by this type of analysis and are estimated from static and seismic stresses with the aid of strain data from laboratory tests (cyclic triaxial tests or cyclic simple shear tests) (Sêco e Pinto, 1993).
For embankment dams a value of 5% axial strain is used as allowable deformation.

To overcome these limitations, nonlinear hysteretic models with pore water pressure generation and dissipation have been developed using incremental elastic or plasticity theory.

The incremental elastic models have assumed a nonlinear and hysteretic behavior for soil and the unloading-reloading has been modeled using the Masing criterion and incorporate the effect of both transient and residual pore-water pressures generated by seismic loading implemented in TARA 3 and DESRA codes (Lee and Finn, 1978; Finn, 1987).

For the models based on the theory of plasticity two particular formulations appear to have a great potential for multidimensional analysis: the multi-yield surface model implemented in DYNAFLOW code (Prevost, 1993) and the two-surface model (Mröz et al., 1979).

A modified cam-clay model for cyclic loading taking into account that when saturated clay is unloaded and then reloaded the permanent strains occur earlier than predicted by the cam-clay model was proposed by Carter et al. (1982). The predictions exhibit many of the same trends that have been observed in laboratory tests involving the repeated loading of saturated clays.

For the definition of the constitutive laws the following laboratory tests are used for embankment dams: resonant column tests, cyclic simple shear tests, cyclic triaxial tests and cyclic torsional shear tests.

For medium embankment dams a conventional pseudo-static analysis method is used to evaluate the seismic behavior of dams (Ambraseys, 1960; Seed and Martin, 1966), but for dams over 100m high a dynamic analysis including computational analysis (modal analysis), model tests, field measurements and prototype tests is recommended (ICOLD, 1975).

A flowchart that integrates stability analysis of dams, monitoring and case histories is shown in Fig.17.

RESERVOIR TRIGGERED SEISMICITY

Man - made earthquakes caused by the filling of reservoirs have drawn the attention of designers concerned with dam safety(ICOLD, 2008a).

The reservoir triggered earthquake (RTS) is linked to dams higher than about 100 m or to large reservoirs (capacity greater than 500 x 10^6 m^3), rate of reservoir filling and to new dams of smaller size located in tectonically sensitive areas. This means that the causative fault is already near to failure conditions and so the added weight stresses and pore pressures propagation due to reservoir impounding, can trigger the seismic energy release.

The earthquakes that have occurred around the few dams by mere accident cannot definitely be attributed to dam or water load, which is insignificant, compared to the earth mass.

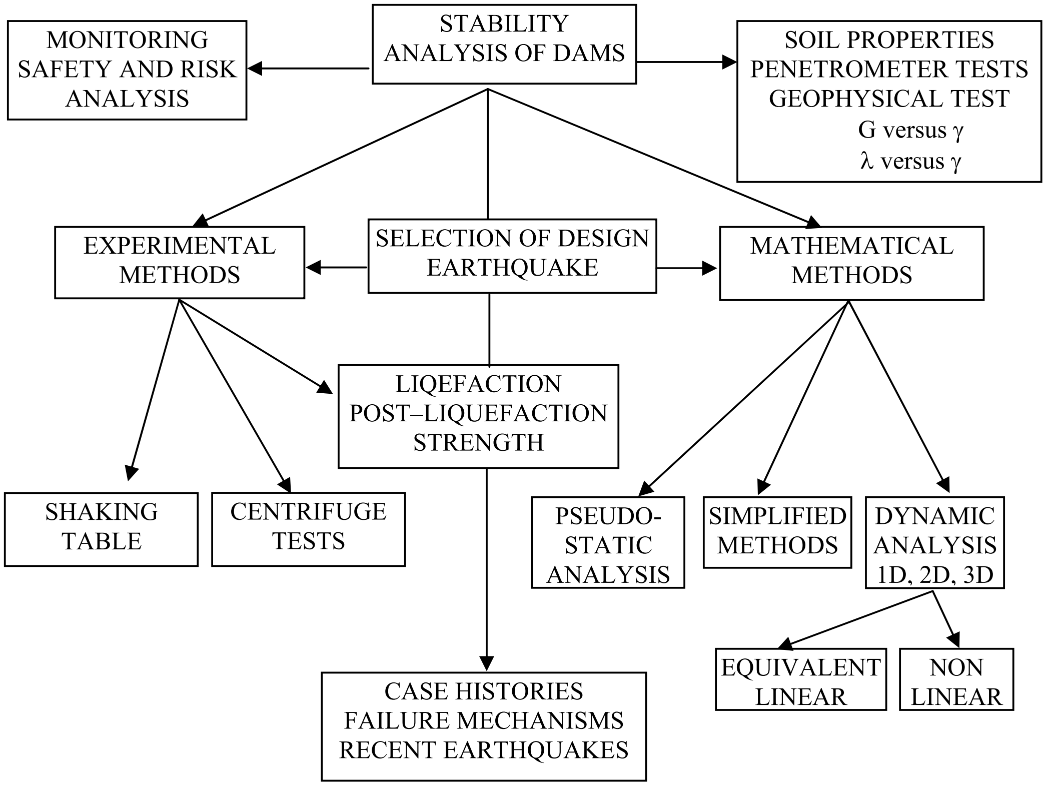


Fig. 17. Flowchart for embankment dams

The detection of reservoir induced seismicity may be performed in two phases (ICOLD, 1999): (i) phase 1 includes on historical seismicity and surveys of reservoir and surrounding geological structures, aiming at identification of possible active faults; and (ii) the second phase is carried out starting at least one or two years prior to the impounding with the installation of a permanent network of seismometers and other measures such as precise levelling, use of instrumentation to detect active fault movements, and reservoir slope stability studies.

Seismological observations established at Bhakra, Pong and Ramanga dams in the Hymalayan terrain have not registed any increase in seismicity due to impounding of waters.

Table 3 presents some examples of dam sites where induced earthquakes with magnitude higher than 5 in the Ritcher scale have occurred (Sêco e Pinto, 2006).

The question of maximum magnitude to be ascribed to reservoir triggered seismicity is difficult to clarify but it seems in the range of 6 to 6.3.

An interesting overall picture is shown in Fig. 18 taken from USSD Report (1997)Monitoring the RTS activity is recommended for large dams and reservoirs. In order to distinguish between background seismicity and RTS monitoring before impounding is recommended. For obtained reliable epicentral locations and hypocentral depths, a local array of stations is required (ICOLD, 2005b).

The International Symposium on Reservoir Induced Seismicity 95 held in Beijing, China, referred to 120 RIS cases from 29 countries with 22 in China, 18 in USA and 12 in India.

Most of the existing reservoirs are aseismic, i.e. no correlation with triggering.

Out of the existing 45000 dams and reservoirs for only 120 a correlation has been reported with a relevant seismic event.

The seismic phenomena are taking place in the brittle and fractured part of the earth crust in which water is circulating and that such phenomena are spent in the underlying plastic mass.

Table 3 - Examples of dams with induced seismicity

DAM	Country	Type	Height (m)	Reservoir volume (x 10^6 m^3)	Year of impounding	Induced seismicity		Prior seismicity
						M	year	
Marathon	Greece	gravity	63	41	1930	5	1938	moderate
Hoover	U.S.A.	arch-gravity	221	36703	1936	5	1939	---
Kariba	Zimbabwe/ Zambia	arch	128	160368	1959	5,8	1963	low
Haifengkiang	China	buttress	105	10500	1959	6,1	1962	aseismic
Koyna	Índia	gravity	103	2708	1964	6,5	1967	low
Kremasta	Greece	embankment	165	4750	1965	6,3	1966	moderate
Roi Constantine	Greece	embankment	96	1000	1969	6,3		moderate
Oroville	U.S.A.	embankment	236	4298	1967	5,7	1975	moderate
Nurek	Tajikistan	embankment	330	11000	1972	5	1977	moderate
Tarbella	Pakistan	embankment	143	14300	1974	5,8	1996	low
Aswan	Egypt	embankment	111	163000	1974	5.3	1981	aseismic
Polyphyton	Greece	embankment	112	2244	1974	6.7	1995	aseismic
Mornos	Greece	embankment	126	640	1961			aseismic

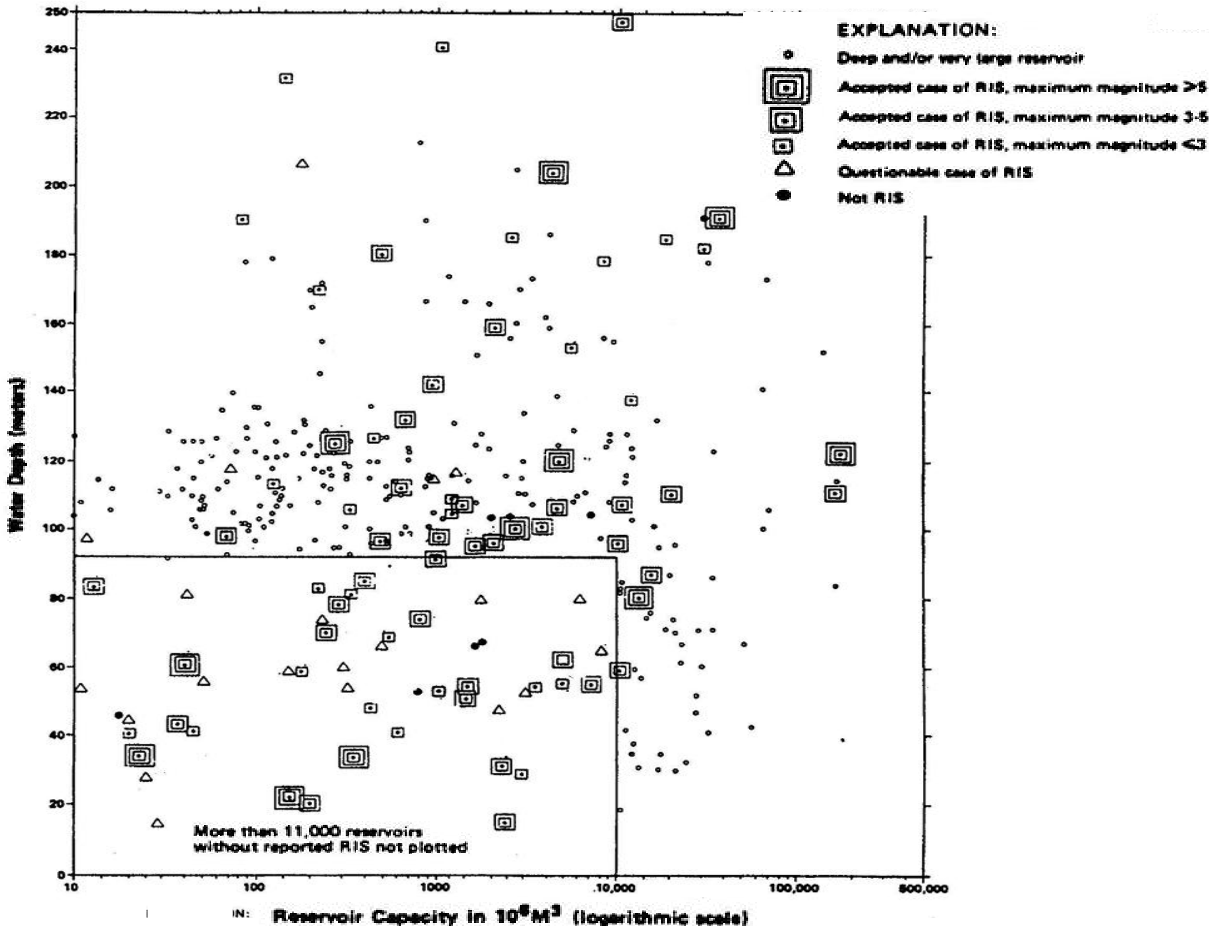


Fig.18. Scattergraph of RTS cases (after USSD, 1997)

The difference between a reservoir triggered earthquake and a natural earthquake is that the reservoir triggered earthquake has a relatively high likelihood of occurring within the first few years after the filling of the reservoir or when the reservoir reaches the maximum level.

These earthquakes have a shallow focus and their epicentres are closed to the dam sites or reservoirs.

MONITORING AND INSPECTIONS

The detailed definition of the monitoring scheme cannot be made on the solid basis of the features of the dam, because many external factors are to be taken into account when safety problems are considered.

The risk factors are classified in three classes, which are referred respectively to actions, to the structure or to values affected by hazards. The arithmetic average of all indices falling in a given class forms an overall risk factor for the class; in this way we define, respectively, an environmental factor E, a reliability factor F, a potential human/economic hazard factor R. Lastly a global risk index a_g, is developed by taking the product of the three partially factors E, F, R, (ICOLD, 1981).

Experience has shown that the rational and systematic control of dam safety should consist of several tasks:

- frequent visual inspection by staff in charge of the observation system;
- periodic visual inspection by specialist;
- regular instrumentation measurements;
- data validation;
- data storage;
- visual inspections;
- safety evaluation;
- corrective actions.

Visual inspections are compulsory after exceptional occurrences, such as important earthquakes, big floods and total or nearly total drawdowns of the reservoir (ICOLD, 1988; Sêco e Pinto, 1993).

There are two steps in performing dam inspection: i) an immediate inspection by the dam operator; (ii) follow-up inspection by dam engineering professional (ICOLD, 2008b).

During inspections the following aspects deserve attention:
Dam Body - (i) upstream face (slope protection, vegetative growth, settlement, debris, burrows and unusual conditions); (ii) downstream face (signs of movement, seepage or wet areas, vegetative growth, condition of slope protection, burrows or unusual conditions); (iii) crest (surface cracking, settlement, lateral movement, camber).

Spillway – (i) approach channel (vegetation, debris, slides, slope protection); (ii) control structures (apron, crest, walls, gates, bridge, chute, stilling basin, outlet channel).

Outlet Works – (i) inlet works, emergency control facility, outlet conduit, service control facility, stilling basin.

Reservoir - Log Boom, landslides, other.

Access Road - Condition of pavement, ditches, bridge.

If an earthquake of moderate or high Richter magnitude occurs an immediate inspection of the dam shall be done following these procedures (ICOLD, 1988):

1) If the dam is damaged to the extent that there is increased or new flow passing downstream immediately implement failure or impending failure procedures as previously planned.
2) If abnormally reduced flow is present at the upstream end of the storage, immediately inspect the river course for possibility of upstream damages due to landslide. If such is the case, implement failure or impending failure procedures.
3) Make an estimate of the characteristics of the earthquake.
4) Immediately conduct a general overall visual inspection of the dam.
5) If visible damage has occurred but has not been serious enough to cause failure of the dam, quickly observe the nature, location and extent of damage and report all the information to the supervisory office for a decision on further actions.
6) Make additional inspections at any time because of possible aftershocks.
7) During inspection the following aspects deserve attention: (i) cracks, settlements and seepage located on abutments or faces of the dam; (ii) drains and seeps for increased flow or stoppage of flow; (iii) outlet works or gate misalignment; (iv) visible reservoir and downstream areas for landslides, new springs and sandboils and rockfalls around the reservoir and in downstream areas; (v) for tunnels and conduits, observe whether silt, sand, gravel, rock or concrete fragments are being carried in the discharge stream.
8) Continue to inspect and monitor the facilities for at least 48 hours after the earthquake because delayed damage may occur.
9) A secondary inspection 2 weeks to a month after the initial inspection should be made.
10) A schedule of very frequent readings should be followed for at least 48 hours after the earthquake.
11) If failure is imminent, warning to downstream residents is essential. All measures should be used to reduce storage in the reservoir.

Sketches, photographs, videos, may help to describe the nature and extent of any damage.

If an earthquake is observed at or near a dam with a Ritcher magnitude greater than and within a radial distance shown in Table 4 immediately conduct a general overall inspection of the dam and major appurtenant structures.

Table 4-Reccommended inspections based in magnitude versus distance

Magnitude	Distance (Km)
>4,0	25
>5,0	50
>6,0	80
>7,0	125
>8,0	200

If the dam is damaged with increased new flow passing downstream or there are signs of imminent failure immediately implement Emergency Action Plan.

Following Australia dam guidelines for all dams that have experienced a MMI of 4 or greater the response required has adopted the guidelines of Table 5.

Table 5 (ICOLD, 2008b)

Response Level	Likely Impact	Response Required
A	<MMI 4	Inspect dam at next routine inspection
B	MMI 4	Inspect dam within 18 hours
C	MMI 5	Inspect dam within 6 hours
D	MMI 6	Inspect dam immediately
E	MMI 7 or greater	Inspect dam immediately

Related earthquakes alarm systems, shake maps are generated automatically.

AGEING EFFECTS

Ageing is defined as a class of deterioration associated with time-related changes in the properties of the materials of which the structure and its foundation are constructed in normal conditions. And so these deteriorations occur more that 5 years after the beginning of operation (ICOLD, 1993a).

Inspection, testing and monitoring of the works are the methods used to obtain the knowledge required to exercise control. A direct evaluation of ageing is possible by monitoring changes in structural properties, and indirect evaluation is available by monitoring the effects and consequences of these changes and of the actions causing them.

Piping in the foundation and in the body of fill dams has caused a number of failures.

The progress in safety of dams is due the improvements of design and construction, but possibly even more to maintenance and monitoring and in particular to proper visual inspections and careful follow-up of increases in leakage that have prevented many failures and reduced the consequences of others.

REHABILITATION OF DAMS

Due to ageing effects and retrofit of dams this topic is getting an increasing attention.

Anastassopoulos et al. (2004) describe the behavior of Thissavros rockfil dam, with 172m high and 480m long, constructed in Greece. The bedrock is composed by gneiss, partially schistose, granitic gneiss and layers of mica schist.

Based on morphological criteria the project assumed the existence of several large dormant landslides at the dam site.

Two areas of instability developed at the site: (i) right bank slide where a monitoring system composed by inclinometers, piezometers and survey monuments was installed to assess the remedial measures that have included major excavation, removing of unstable mass, toe buttressing and drainage by a system of galleries; and (ii) left bank where jet grouting and toe load berm were used.

Perlea et al. (2004) conducted numerous seismic retrofit solutions to reinforce the strength of a liquefiable sand deposit in the foundation of a major embankment dam with 42 m high and 1650m long.

The following methods of foundation soil stabilization were evaluated: (i) removal and replacement of liquefiable material; (ii) dynamic compaction (heavy tamping); (iii) densification by vibrocompaction; (iv) compaction grouting; (v) jet-grouting; (vi) soil mixing, (vii) densification by stone columns; (viii) gravel drains; (ix) enlargement embankment; and (x) foundation seepage cutoff.

The authors have considered that the best alternative solution for stabilization of the upstream slope was jet grouting from a platform built on the lower portion of the slope and for stabilization of the downstream slope was deep soil mixing.

The use of geomembranes for the rehabilitation of dams is a topic of great interest. Following ICOLD (1991) more that 70 dams located in 24 countries have used geomembranes.

The causes of dam deterioration are related with irregular settlement of the fill or foundation, poor concrete quality and shrinkage cracks.

The following agents are related with the dangers to which the geomembranes are exposed:

- falling rock at mountain site
- blows from heavy floating objects
- ultraviolet radiations
- willful damage.

It is considered that grout curtain in the rock foundation is not vulnerable to earthquakes. Dynamic soil-structure interaction, joint movements and fissures in the dam foundation due earthquakes can provoke local damages in grout curtain and additional grouting works are necessary to rehabilitate foundation drainage system (Wieland, 2005).

It is well accepted that due limited resources available to face maintenance of old dams there is a need to develop a rational plan for rehabilitation based on trough scientific research. There is a need to increase research and development for abetter effectiveness and efficiency of the investment in maintenance and safety of dams.

RISK ANALYSES

The findings of dam failures statistical analysis of data show that (ICOLD, 1995):

(i) the percentage of failure of large dams has been falling over the last four decades, 2,20 % of dams built before 1950 failed, failures of dams built since 1951 are less than 0.5 %;

(ii) most failures involve newly built dams. The greatest proportion 70% of failures occur in the first ten years and more especially in the first year after commission.

At the end of XX century, one billion people was living downstream of dams. It seems that millions may be at risk within the next 50 years as a result of dam failures. Although the annual failure probability of dams is lower than 10^{-6} in most cases, it may be higher for dams in seismic areas subject to sudden failures such as tailing dams and hydraulic fill dams.

The potential risk associated with dams consists of structural components and socio-economic components. The structural components of potential risk depend mostly on storage capacity and on the height of the dam, as the potential downstream consequences are proportional to the mentioned values (ICOLD, 1989). Socio-economic risks can be expressed by a number of persons who need to be evacuated in case of danger and by potential downstream damage.

The structures following EC8 are classified in 4 importance categories related with the size, value and importance for the public and on the possibility of human losses in case of collapse. To each important category an important factor is assigned. The important factor γ_f =1.0 is associated with a design seismic event having a reference return period of 475 years. The importance category varying I to IV (with the decreasing of the importance and complexity of the structure) are related with the importance factor γ_f assuming the values 1.4, 1.2, 1.0 and 0.8, respectively.

Risk management comprises the estimation of the level of risk and exercising adequate control measures to reduce the risk when the level is not tolerable (Caldeira et al, 2005). The essence of risk management and the role of quantitative risk assessment (QRA) within the context of risk management are shown in Fig. 19 (Ho et al., 2000).

ICOLD has introduced the potential risk of dam associated with capacity, height, evacuation requirements and potential downstream damage considering these 4 hazard classes (ICOLD, 2009).

Tables 6 and 7 are convenient to define risk associated with dams. Four risk factors are separately weighted as low, moderate, high or extreme.

Table 6. Risk Factor (ICOLD, 2009)

Risk Factor	Extreme	High	Moderate	Low
Capacity (hm3)	>120 (6)	120-1 (4)	1-0-1 (2)	<0.1 (0)
Height (m)	>45 (6)	45-30 (4)	30-15 (2)	<15 (0)
Evacuation Requirements	>1000 (12)	1000-100 (8)		None (0)
Potential Downstream Damage	High (12)	Moderate (8)	Low (4)	None (0)

The weighting points of each of the four risk factors, shown in brackets in Table 6 are summed to provide the Total Risk Factor as

Total Risk Factor= Risk Factor (capacity)
+ Risk Factor (height)
+ Risk Factor (evacuation requirements)
+ Risk Factor (potential downstream damaged).

The link between the Total Risk Factor and Risk Class is giving in Table 7.

Table 7. Risk Class (ICOLD, 2009)

Total Risk Factor	Risk Class
0-6	I(Low)
7-18	II(Moderate)
19-30	III(High)
31-36	IV(Extreme)

There is a rich discussion related Failure Modes and Effect Analysis (FMEA), Failure Mode, Effects and Critically Analysis (FMECA), Event Tree Analysis (ETA), Fault Tree analysis (FTA) (ICOLD, 2005b).

Structural Reliability Methods permit the calculation of failure probabilities of the mechanisms. Probabilities are calculated using the methods of the modern reliability theory such as Level III Monte Carlo, Bayesian theory, Level II advanced first order second moment calculations.

Dam owners, regulatory authorities and consultants have been carrying out risk analyses for many years. Its purpose is to identify the main real risks associated with each type and height of dam for all circumstances and can be conducted: (i) in extensive risk analysis of very large dams, to substantiate

Fig. 19. Framework for risk management (after Ho et al, 2000)

reliably the probabilities chosen in fault trees using Monte Carlo simulation technique; (ii) in simplified risk analysis of smaller dams, to focus low-cost risk analysis on a few main risks; (iii) and in identifying possibilities for reducing these risks through low-cost structural or non-structural measures (Lempérière, 1999).

The main components of risk management are risk assessment (risk analysis and risk evaluation), risk mitigation and control (risk reduction, emergency actions) and decision (Seco e Pinto, 2002).

Consideration of human behavior is essential when assessing the consequence of failures: well organized emergency planning and early warning systems could decrease the number of victims and so the study of human behavior plays an important role in assessment of risk analysis (Sêco e Pinto, 1993).

We should never forget the contribution of Voltaire and the book Candide published in 1759, after the Lisbon earthquake (1755), for the change from the intellectual optimism and potential fatalism that is a necessary condition for the construction of future scenarios in a risk analysis context.

The results of a risk analysis can be used to guide future investigations and studies, and to supplement conventional analyses in making decisions on dam safety improvements. Increasing confidence in the results of risk analyses can lead to a better cost-effective design and construction, satisfy our personal needs providing a better insight of the different factors of the design and give more confident to our decisions (Sêco e Pinto, 2002).

A probabilistic risk assessment addresses three fundamental questions (Salmon and Hartford, 1995): (I) what can go wrong? (ii) how likely is it?; (iii) what damage will it do?
In general, a society risk of 0.001 lives per year per dam appears to be acceptable. Assuming that the combined probability of failure to a PMF and MCE is approximately 1/100 000 per year, a loss of life of up to 100 people would result in an acceptable risk of 0.001 lives per year per dam.

The past practices of US Army Corps of Engineers, US Bureau of Reclamation and BC Hydro are shown in Fig. 20 along with a risk line of 0.001 (or 10^{-3}) lives per year per dam (Salmon and Hartford, 1995).

There are several uncertainties in seismic hazard and seismic input in material properties, in structural modelling, in dynamic analysis and in performance criteria.

Due to large uncertainties in predicting the seismic behaviour of dams it is recommended to increase the resilience to earthquake loading instead reducing the uncertainties in seismic hazard, or material properties or using more sophisticated methods of seismic analysis.

First order methods such as the First Order Second Moment (FOMS) and the First Order Reliability Method (FORM) have received significant exposure (e.g. Low, 1997; Nadim, 2002; Duncan, 2000) in recent years as relatively simple methods for estimating the probability of events occurring in geotechnical analysis.

The basic objective is as follow: given statistical data (mean and standard deviation) for key geotechnical input parameters (e.g. strength parameters c` and tan, seepage parameters k, settlement parameters E) what are the statistics (mean and standard deviation) of the key output quantities (e.g. factor of safety FS, flow rate Q, settlement d).

In the case of the output parameter, if these statistics are combined with an assumed probability density function, the probability of events such as slope failure, excessive flow rates, excessive settlements, etc, can be estimated.

While these methods are relatively easy to implement and give useful qualitative and sensitivity information about the

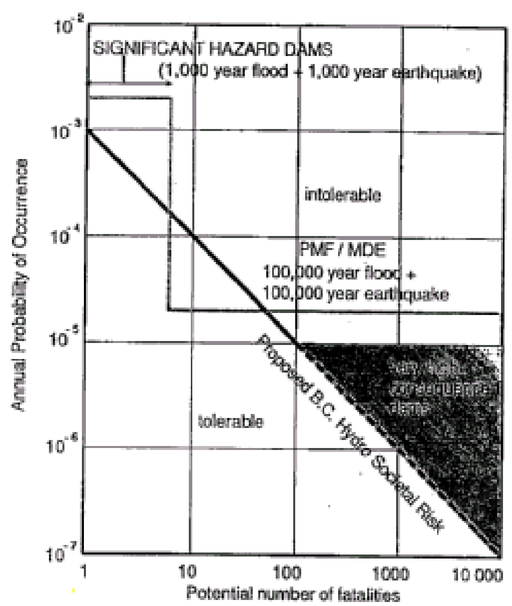


Fig. 20. Incremental hazard criteria (after Salmon and Hartford, 1995

input and output parameters, they are based on an underlying assumption of a Taylor Series truncated after the linear terms-hence first order.

A fully probabilistic assessment of sliding displacement incorporating the aleatory variability in the earthquake ground motion prediction was proposed by Rathje and Saygili (2008). The product of this analysis is a displacement hazard curve which provides the annual rate of exceedance for a range of displacement levels. The different deterministic and probabilistic methodologies to predict the siding displacement of a slope are shown in Fig. 21.

Warning Systems

For warning systems there two possibilities approaches: direct and indirect monitoring.

For example in the direct approach a potential sliding area is monitored by simple displacement instrumentation and when a predicted threshold value of displacement is exceeded the people of the valley is evacuated**.**

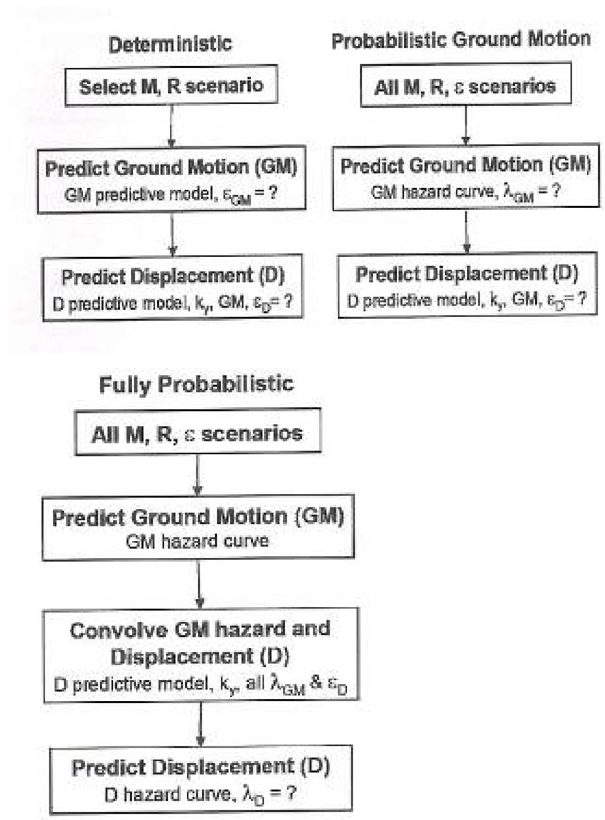


Fig. 21. A fully probabilistic assessment of sliding displacement (after Rathje and Saygili, 2008)

An example of indirect warning system is the city of Hong Kong where an early warning system has been used for over 15 years and people were educated to recognize report landslide symptoms (cracking, reactivation of spring lines, surface runoff, etc).

The possible avenues for warming systems are shown in Fig. 22.

BC Hydro dam safety program has five basic components: surveillance, emergency preparedness planning, dam safety reviews, deficiency investigations and capital improvements (Stewart, 2000).

Past practice has been to require relatively large increases in reliability (decreases in probability of failure) when the consequences exceed some fixed criteria such as one expected fatality or six expected fatalities.

Fig 23 shows the risk analysis proposed by AGS (2000).

BENEFITS AND CONCERNS OF DAMS

The benefits of dams are demonstrated with the multipurpose uses of dams for water supply, irrigated agriculture, electric energy generation, flood control, recreation and other usages.

Importance to the environmental and social aspects of dams and reservoir is increasing. Construction of dams is no longer acceptable without a careful analysis of mitigation and adverse impacts. It is important to build dams in harmony with the environment and therefore economic development and environmental protection must proceed hand in hand.

Social and economic impacts of large dam projects vary greatly in different geographic, political, and economy contexts (ICOLD, 1992). Social and economic considerations must be brought into the planning process early to permit major process layout and design elements.

In the Stockholm Conference on World Environmental held in 1972, hunger and poverty were identified as the major

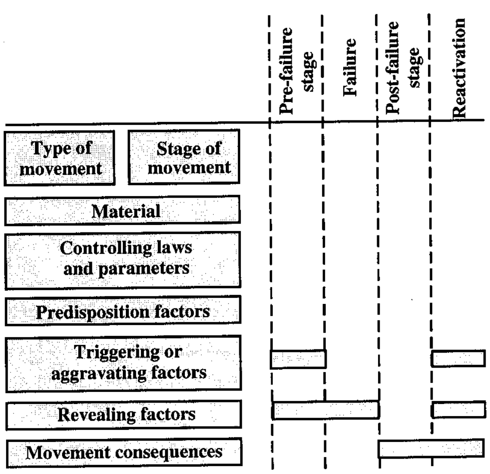


Fig. 22 Possible avenues for warning systems

reasons for environmental degradation. Inadequate and uneven distribution of rainfall, drought and floods, lower irrigation intensity and instability of agricultural, poor health status are all factors contributing to hunger and poverty.

One of the predominant concerns about reservoirs is re-settlement. Following ICOLD (1997) involuntary settlement must be handled with special care, managerial skill and political concern based on comprehensive social research and sound planning for implementation.

The implementation of resettlement planning needs to take into account: (i) opinion surveys and talks to people about resettlement rights; (ii) identification of entitled families; (iii) site selection; (iv) allocation of funds, (v) preparation of agricultural land; (vi) road construction, provision of water supply and other infrastructure; (vii) tendering of bids for resettlement housing construction; (viii) transportation of settlers, (ix) training and agricultural extension services; and (x) rehabilitation programs.

A study carried out in respect of a number of major dams built for multipurpose projects indicates that the population displaced on account of construction of dams varies between 0,5% to 4% of the population benefited by the irrigation facilities and a tiny fraction of the percentage of those benefited by electricity (Naidu, 1999). The rate of beneficiaries to affected persons is better than 200:1.

Statistical analysis of a number of major projects also indicates that forest area submerged is just 1-2% of the area to be irrigated by those projects.

A detailed listening of over 80 potential impacts on the natural re-environment (flora, fauna and aquatic fauna), social economic and cultural aspects, land, dam construction activities, sedimentation of reservoirs, downstream hydrology, water quality, tidal barrages, climate and human health was presented by Veltrop (1998).

Technical feasibility and economic justification of new dam projects are now second to social, political and environmental considerations and requires cooperation among engineers, scientists, environmentalists and stakeholders.

NEW CHALLENGES- LESSONS FOR TOMORROW

The following topics deserve more consideration and can be considered new challenges for a better understanding of seismic embankment dams behaviour:

Liquefaction

i) The use of Becker hammer and geophysical tests to assess the liquefaction of gravelly materials; ii) Determination of residual strength of soil; iii) Evaluation of liquefaction consequences and post earthquakes displacements; iv) Mitigation methods with use of microorganisms.

Embankment Dams

(i) Coupled models with non linear analyses and pore water pressure generation and dissipation models; (ii) Hydrodynamic effects of reservoir associated with dynamic foundation-structure interaction (Seco e Pinto, 2001); (iii) Failure of tailing dams that currently reach more than 200m high and reservoirs with more than one billion tons of slimes

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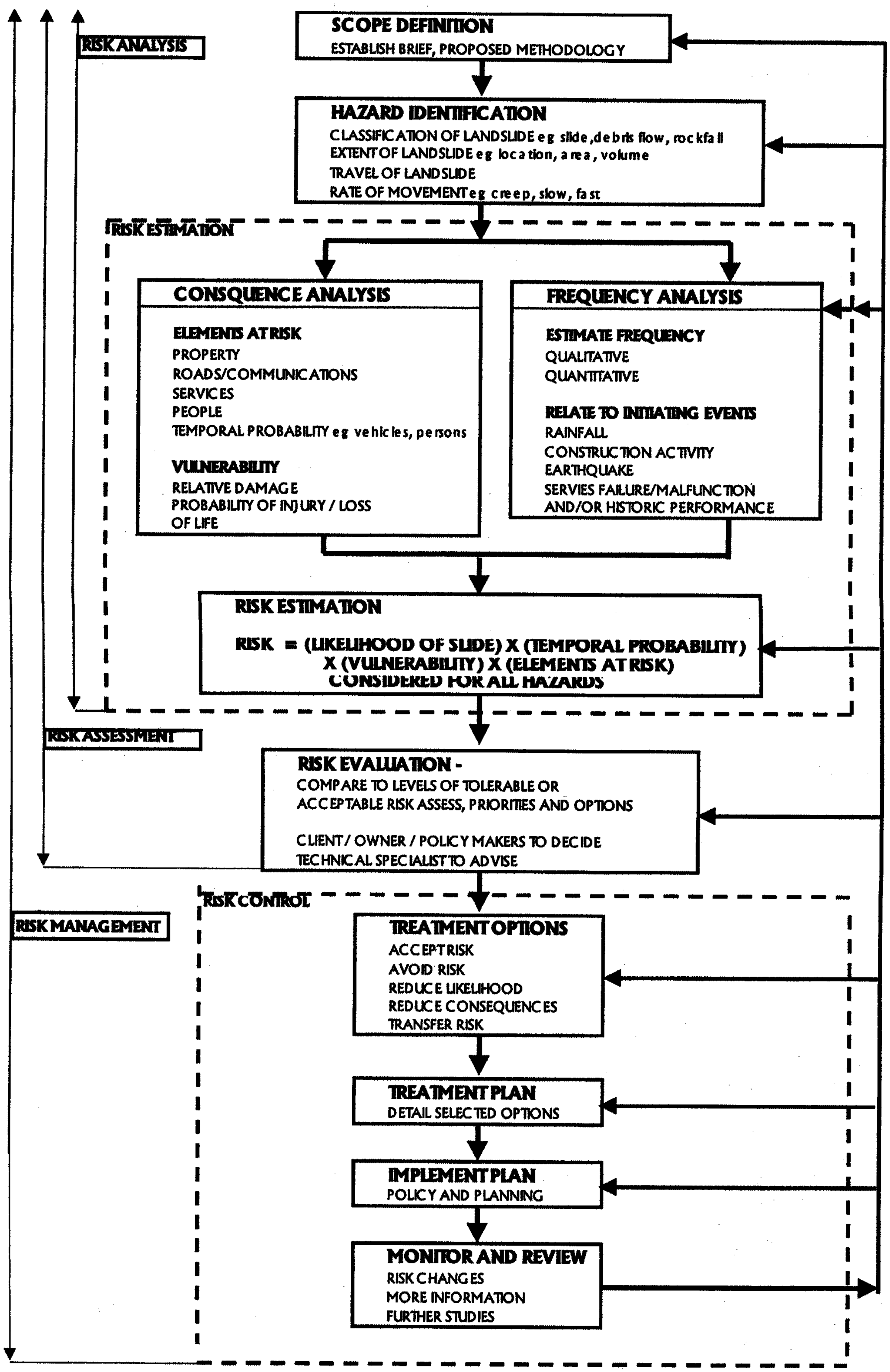

Fig. 23. Risk analysis proposed by AGS (2000)

due the occurrence of liquefaction and the increase of the resistance due to ageing effects of the deposits.

Lessons for Tomorrow

Today there is a need to work in large teams exploring the huge capacity of computers to analyze the behavior of large dams. Innovative methods and new solutions require high reliable information and teams integrating different experts, namely seismologists, geologist, geophysics, geotechnicians and structures engineers.

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A joint effort between Owners, Decision-Makers, Researchers, Consultants, Professors, Contractors and General Public to face this challenge is needed.

It is important to understand the concepts of vulnerability and resilience. Vulnerability is associated with two dimensions, one is the degree of loss or the potential loss and the second integrates the range of opportunities that people face in recovery. This concept received a great attention from Rousseau and Kant (1756). Resilience is a measure of the system`s capacity to absorb recover from a hazardous event. Includes the speed in which a system returns to its original state following a perturbation. The capacity and opportunity to recolate or to change are also key dimensions of disaster resilience. The purpose of assessing resilience is to understand how a disaster can disturb a social system and the factors that can disturb the recovery and to improve it.

It is important to stress that a better understanding of embankment dams during the occurrence of earthquakes can only be achieved by a continuous and permanent effort in order to be up-to-date with the last developments in earthquake engineering. It is important that engineers educate themselves and the Public with scientific methods for evaluating risks incorporating the unpredictable human behavior and human errors in order to reduce disasters.

From the analysis of past dam incidents and accidents occurred during the earthquakes it can be noticed that all the lessons have not deserved total consideration, in order to avoid repeating the same mistakes. We need to enhance a global conscience and develop a sustainable strategy of global compensation how to better serve our Society. The recognition of a better planning, early warning, quality of evacuation that we should take for extreme events which will hit our civilization in the future. Plato (428-348 BC) in the Timaeus stressed that destructive events that happened in the past can happen again, sometimes with large time intervals between and for prevention and protection we should followed Egyptians example and preserve the knowledge through the writing.

We should never forget the 7 Pillars: Practice, Precedents, Principles, Prudence, Perspicacity, Professionalism and Prediction. Following Thomas Mann we should enjoy the activities during the day, but only by performing those will allow us to sleep at the night.

Also it is important to narrow the gap between the university education and the professional practice, but we should not forget that Theory without Practice is a Waste, but Practice without Theory is a Trap. Kant has stated that *Nothing better that a good theory,* but following Seneca *Long is the way through the courses, but short through the example*. I will add through a careful analysis of Case Histories.

Within this framework all the essential steps of good dam analyses, whatever the type of material is involved shall be performed with a sufficient degree of accuracy that the overall results can be extremely useful in guiding the engineer in the final assessment of seismic stability. This final assessment is not made by numerical results but shall be made by experienced engineers who are familiar with the difficulties in defining the design earthquake and the material characteristics, who are familiar with the strengths and limitations of analytical procedures, and who have the necessary experience gained from studies of past performance

In dealing with these topics we should never forget the memorable lines of Hippocrates:

- “The art is long
- -and life is short
- experience is fallacious
- -and decision is difficult”.

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ANNEX 1

Table A1 - Behaviour of embankment dams during earthquakes

Dam	Country	L(m)	H (m)	Construction Year	Dam characteristics	Construction Technique	Earthquake characteristics	Dam behaviour	Reference
Baihé	China	960	60		Zoned with sandy gravel materials and slope clay core		Tangshan 1976 I= 6	Slope of the upstream shell	Wenshao (1987)
Cogoti	Chile	159	84	1939	Rockfill with upstream concrete membrane	Dumped rockfill	Illapel 1943 M= 8.3	Crest settlements 0.38 m	Cook (1984) Seed et al. (1978)
Douhe	China	6000	22	1970	Homogeneous		Tangshan 1976 I= 6 horiz. accel. 0.4 g	Longitudinal cracks crest settlement due foundation liquefaction	Liu et al. (1979) Shen et al. (1981)
El Caracol	México		126	1985	Zoned with rockfill shells and clay core	Compacted fill	1985 Earthquake September 19 M= 8.1	Transverse and longitudinal deformations, crest settlement 160 mm	Ulloa (1987)
El Infiernillo	México	350	148	1963	Zoned with rockfill shells and clay core	Compacted fill	1979 and 1981 Earthquakes M=7.6	Longitudinal cracks 0.60 m depth	Tamura (1986) Resendiz et al. (1982)
Gokçe	Turkey		50				August 17, 1999, M= 7.4		
Kuzuryu	Japan	355	128	1964	Zoned with rockfill shells and sloping clay core	Compacted fill	1969 Earthquake M= 6.6		Nose and Baba (1980)
La Marquesa	Chile	220	10	1943	Zoned with silty shells and impervious core	Compacted fill 85-88% Modified Proctor	1985 Earthquake March 3 M= 7.8	Upstream and downstream shells slopes. Liquefaction of sandy material	Retamal et al. (1989)
La Villita	México	420	60	1968	Zoned with rockfill shells and central clay core		1979 and 1981 Earthquakes M= 7.1-7.6 a_{max}= 0.31 - 0.38g	Longitudinal cracks with 150 m length and 0.50 m depth	Tamura (1986) Resendiz et al. (1982)
Leyroy Anderson	U.S.A.	370	72	1950	Zoned with rockfill shells and impervious core	Compacted core rockfill shells without compaction	Morgan Hill 1984 amax= 0.42g	Longitudinal cracks with 300 m length and 2 m depth	Gazetas (1987)
Long Valley	U.S.A.	200	60	1941	Homogeneous with silty sand material with gravels	Compacted fill 93% modified AASHO	1980 Earthquake May 27 M= 6	Springs on downstream toe, cracks	Seed (1980) Lai and Seed (1985)
Mahio	Japan		106	1961	Zoned with rockfill shells and central core	Compacted core rockfill shells without compaction	Nagano Prefecture September 14 1984 M= 6.8	Settlements of upstream shell	Yonezawa et al. (1987)

Malpaso	Peru	152	78	1936	Rockfill with upstream concrete membrane	Dumped rockfill	1938 Earthquake October 10 I= 6.6 Mercalli modified scale de Mercalli	Crest settlement (76 mm) and downstream displacements 51 mm	Ambraseys (1960)
Matahina	New Zeland	400	86		Zoned with rockfill shells and impervious core	Compacted fill	1987 Earthquake March 2 M=6.3	Upstream shell has settled 800 mm, downstream shell has settled 100 mm with 250 mm tilting for downstream	Matsumoto et al. (1985) Gillon (1988)
Miboro	Japan	405	131	1960	Rockfill	Compacted fill	1961 Earthquake M= 7.2 acceleration= 0.25g	Settlement of 30 mm displacement for downstream of 50 mm	Nose and Baba (1980)

Minase	Japan	665	---	1964	Rockfill with upstream concrete membrane	Dumped rockfill	Niigata 1964 M= 7.5	Damages of the membrane joints, cracks on the crest, increase of seepage	Matsumoto et al. (1985)
Oroville	U.S.A.	1707	235	1968	Zoned with gravel shells and slope core	Compacted fill	Oroville 1975 M= 5.7	Crest settlements 9 mm	Banerjee et al. (1979)
S. Fernando	U.S.A.	664		1940	Homogeneous with sandy silty and clay sandy materials	Hydraulic fill	S. Fernando Fev. 9 1971 M= 6.6	Longitudinal cracks Liquefaction	Seed et al. (1973) ICOLD (1975)
Leyroy Anderson	U.S.A.	370	72	1950	Zoned with rockfill shells and impervious core	Compacted core rockfill shells without compaction	Morgan Hill 1984 a_{max}= 0.42g	Longitudinal cracks with 300 m length and 2 m depth	Gazetas (1987)
Long Valley	U.S.A.	200	60	1941	Homogeneous with silty sand material with gravels	Compacted fill 93% modified AASHO	1980 Earthquake May 27 M= 6	Springs on downstream toe, cracks	Seed (1980) Lai and Seed (1985)
Mahio	Japan		106	1961	Zoned with rockfill shells and central core	Compacted core rockfill shells without compaction	Nagano Prefecture September 14 1984 M= 6.8	Settlements of upstream shell	Yonezawa et al. (1987)

Malpaso	Peru	152	78	1936	Rockfill with upstream concrete membrane	Dumped rockfill	1938 Earthquake October 10 I= 6.6 Mercalli modified scale de Mercalli	Crest settlement (76 mm) and downstream displacements 51 mm	Ambraseys (1960)
Matahina	New Zeland	400	86		Zoned with rockfill shells and impervious core	Compacted fill	1987 Earthquake March 2 M=6.3	Upstream shell has settled 800 mm, downstream shell has settled 100 mm with 250 mm tilting for downstream	Matsumoto et al. (1985) Gillon (1988)
Miboro	Japan	405	131	1960	Rockfill	Compacted fill	1961 Earthquake M= 7.2 acceleration= 0.25g	Settlement of 30 mm displacement for downstream of 50 mm	Nose and Baba (1980)

Sheffield	U.S.A.	220	.5	1923	Homogeneous with silty sand and upstream concrete membrane	Compacted fill	Santa Barbara June 19 1925 M= 6.3	Dam failure	Seed et al. (1969)
Tarumizu	Japan	256	3	1976	Zoned with rockfill shells and central clay core material	Compacted fill	Miyagi-Ken-Oki 1978 M= 7.4	No apparent damages Calculated crest acceleration 0.36g	Yanagis-awa and Fukui (1980)
Vidra	Romania		23		Zoned with rockfill shells and central clay core material		Vrancea March 4, 1977 M=7.2 a_h= 0.2g	No apparent cracks	Priscu (1979)
Wangwu	China	761	0		Zoned with sandy shells and clay core	Dumped fill	Bohai Wan 1969 I= 6	Liquefaction and slope of upstream shell	Wenshao (1987)

H - dam height   L - dam length   I - earthquake intensity   M - earthquake magnitude