DAM ENGINEERING-NEW CHALLENGES

P. S. Pinto
Professor of Geotechnical Engineering, University of Coimbra, Portugal
President of International Society for Soil Mechanics and Geotechnical Engineering
(ISSMGE)

P. Simão
Principal Research Engineer, National Laboratory of Civil Engineering (LNEC)

ABSTRACT

In this paper dam design criteria is addressed. Foundation studies for soil and rock materials are described. Alvito dam case study is analyzed. Lessons from dam behavior during earthquakes are presented. Analysis of dams stability during earthquakes by experimental and mathematical models is referred. Selection of design earthquakes by deterministic and probabilistic criteria is discussed. Las Cuevas Dam case study is presented. Hydrodynamic effects are discussed. Reservoir induced seismicity and prototype dynamic tests are treated. Ageing effects and rehabilitation of dams are discussed. Benefits and concerns of dams are referred. Some final considerations and topics for discussions are presented.

“Where love is great, the littlest doubts are fear;
Where little fears grow great, great love grows there.”
(Love, Shakespeare, Hamlet)

INTRODUCTION

Dam design criteria is addressed.
Foundation studies for soil and rock materials are described.
Alvito Dam case study is analysed.
Lessons from dam behavior during earthquakes are presented.
Analysis of dams stability during earthquakes by experimental and mathematical models is referred.
Selection of design earthquakes by deterministic and probabilistic criteria is discussed.
Las Cuevas Dam case study is presented.
Hydrodynamic effects are discussed.
Reservoir induced seismicity and prototype dynamic tests are treated.
Ageing effects and rehabilitation of dams are discussed.
Benefits and concerns of dams are discussed.
Some final considerations and topics for discussions are presented.
DAM DESIGN CRITERIA

Dam design requires the specification of all relevant conditions for “normal” and “unlikely but possible” behavior design. These sets of conditions are known as “utilization scenarios” and “hazard scenarios”, respectively (or simply “normal” and “extreme” conditions) (ICOLD, 1988).

Limit equilibrium analyses based in a safety factor do not provide any information on displacements.

Finite element analyses considering soil nonlinear behavior give the designer information about the distribution of stress and displacements.

Verification and Validation of Models

The need of model and code validation is getting increased attention.

Sargent (1990) has introduced the concepts of verification and validation and the relations established between the three entities: the physical problem; the conceptual model; and the computer model and its numerical implementation are illustrated in Figure 1.

Verification intends to ensure that the computer program is correct and its represents faithfully the conceptual model and validation applies essentially to the conceptual model, and its ability to reproduce satisfactorily the physical phenomena.

A slightly different terminology is adopted by ICOLD (1994a) that considers that the numerical modeling process for dams should be checked in order to avoid unreliable results considering the following aspects:

![Fig.1 Relations between physical problem, conceptual model and computer model (Sargent, 1990)]

i) Justification of the whole modeling method (the relevance to physical reality);
ii) validation of the computer code;
iii) quality assurance of the whole computation process.

Foundations Studies

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 pressuremeter tests (ICOLD, 2005a).

For the analysis of rock mass discontinuities the following numerical techniques are used: finite elements, finite differences, boundary elements and discrete elements.

To reproduce the opening of joints discrete element models are well suited for this type of study, as they represent the dynamic behavior of the jointed rock medium. The spacing and orientation parameters of discontinuous cross-joints are defined by a mean value and a random deviation. A coupled hydro-mechanical analysis can be performed under the assumption that blocks are impervious and fracture flow is governed by the cubic law (Lemos, 1999b).

Significant advances have been made in describing joint statistics and creating computer representations of rock masses with some measure of realism. The Veneziano polygonal model, which starts by generating lines by a Poisson process on each plane, where actual joint and rock bridge portions are then defined is shown in Figure 2.

Dershowitz and Einstein (1988) described several joint system models, which are intended to characterize, in aggregate way, the various joint characteristics. Individual joints are idealized as disks or polygons, distributed in a 3D space, either randomly or according to some spatial criterion. This model illustrated in Figure 3 generates the planes by Poisson process first, then find the intersections in order to define the joint and rock bridge portions on each plane (Lemos, 1999a).

To investigate the nonlinear dynamic soil-structure interaction of a plane-strain earth dam founded on elastic halfspace, subjected to transient vertically incident SV waves a couple finite element-boundary element formulation was applied (Abouseeda and Dakoulas, 1996). This formulation can also be used to assess the relative importance of the effects of nonlinearity, soil structure-interaction, effect of a soft foundation layer, type of excitation (P, S, Rayleigh waves) and other parameters affecting the response of the dam.

The most dangerous manifestation concerning the dam stability and integrity is the surface fault breaking, intersecting the dam site.

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.

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 may 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 and Lun, 2003).

**Alvito Dam Case Study**

Alvito dam in the south of Portugal is a 50 m high and 1 500 m long earth dam. A cross-section of the dam is shown in Figure 4. The core of the dam was constructed of clayed material and the shells of schist material. A chimney - filter located downstream of the core is composed of sand.

A two dimensional plane strain analysis in total stresses of Alvito dam was done using hyperbolic stress - strain law. The construction stage was simulated in ten layers and the analysis of reservoir filling in three steps (Pinto, 1983).

Also an incremental stress-strain relationship (modified cam-clay model that takes into account the consolidation of partly saturated clay soils with varying permeability and compressibility of the pore fluid was developed.

The summary of parameters for the materials used in the analysis is given in Table 1 (Pinto, 1987).

The values of all the stress - strain parameters required for this model were obtained form the results of drained triaxial tests and one-dimensional consolidation tests performed on molded specimens. Special tests were conducted to quantity the variation of clay core permeability with degree of saturation, water content and void ratio.

The values of the vertical and horizontal internal displacements observed at the end of the construction period (from USBR devices and inclinometers) are compared in Figure 5 with the values predicted by the two rheological models. There is a good agreement in the core zone and a slight deviation in the downstream shell considering the degree of accuracy to be expected in this type of analysis.

**LESSONS FROM DAM BEHAVIOR DURING EARTHQUAKES**

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

- Sliding or shear distortion of embankment or foundation or both
- Transverse cracks
- Longitudinal cracks
- 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
Fig. 2 Veneziano polygonal model: a) 2D Poisson line process; b) marking of polygonal joints; c) 3D Poisson plane process (after Einstein, 1993)

Fig. 3 Dershowitz polygonal model: a) 2D Poisson plane process; b) Poisson line process formed by intersections, c) marking of polygonal joints (Dershowitz and Einstein, 1988)

Fig. 4 Alvito dam cross-section
Table 1 Summary of elasto-plastic Cam-Clay parameters for Alvito dam materials

<table>
<thead>
<tr>
<th>Parameter Name</th>
<th>Core material</th>
<th>Shell material</th>
<th>Filter material</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_t ) - Total unit weight (kN/m²)</td>
<td>20.0</td>
<td>22.5</td>
<td>18.4</td>
</tr>
<tr>
<td>( p_{ho} ) - Pressure at juncture (kPa)</td>
<td>1000.0</td>
<td>1000.0</td>
<td>1000.0</td>
</tr>
<tr>
<td>( p_r ) - Reference pressure (kPa)</td>
<td>296.9</td>
<td>99.8</td>
<td>0</td>
</tr>
<tr>
<td>( p'_{r} ) - Reference pressure (kPa)</td>
<td>296.9</td>
<td>99.8</td>
<td>0</td>
</tr>
<tr>
<td>( k ) - Slope of isotropic rebound curve</td>
<td>0.0058</td>
<td>0.0058</td>
<td>0.0059</td>
</tr>
<tr>
<td>( M_r ) - Strength parameter</td>
<td>0.79</td>
<td>1.17</td>
<td>1.55</td>
</tr>
<tr>
<td>( M'_r ) - Strength parameter</td>
<td>0.79</td>
<td>1.17</td>
<td>1.55</td>
</tr>
<tr>
<td>( e_0 ) - Initial void ratio</td>
<td>0.5</td>
<td>0.40</td>
<td>0.45</td>
</tr>
<tr>
<td>( S_0 ) - Initial degree of saturation (%)</td>
<td>67</td>
<td>70</td>
<td>70</td>
</tr>
<tr>
<td>( S_r ) - Threshold degree of saturation (%)</td>
<td>10</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>( k_{hs} ) - Horizontal saturated permeability (m/s)</td>
<td>( 5 \times 10^{-7} )</td>
<td>( 10^{-5} )</td>
<td>( 10^{-4} )</td>
</tr>
<tr>
<td>( k_{vs} ) - Vertical saturated permeability (m/s)</td>
<td>( 2 \times 10^{-8} )</td>
<td>( 10^{-5} )</td>
<td>( 10^{-4} )</td>
</tr>
<tr>
<td>( \sigma_t ) - Tensile strength (kPa)</td>
<td>10.0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

\( p_{o}^{avg} \) - Average stress \( (p_o + \sigma_3)/2 \) (kPa)

\( \lambda \) - Slope of virgin isotropic consolidation curve

\( p \) - mean effective stress (kPa)

\( v \) - Poisson ratio

ANALYSIS OF DAM STABILITY DURING EARTHQUAKES

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 (Údias and Arroyo, 2005). The explosive theory was also proposed by Newton’s Optics (1718).
Fig. 5 Values predicted by the two rheological models

- Disruption of dam by major fault movement in foundation
- Differential tectonic ground movements
- Failure of spillway or outlet works
- Piping failure through cracks induced by ground motions
- Liquefaction of embankment or foundation.

The damage modes listed are not necessarily independent of each other.

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 viewpoint. Also Kant and Rosseau defended the optimist position.

Recent Earthquakes-Dam behavior

During the Committee Seismic Aspects on Dams meeting in Seul, in 2004, the following reports were presented (ICOLD, 2004).

D. Babbitt reported on the performance of dams during the San Simeon earthquake in California of December 22, 2003 (Magnitude 7.1). The maximum peak ground acceleration (PGA) estimated at the Las Tablas Creek embankment dam site was 0.36 g, which resulted in minor crest cracks and a spillway rock slide. At the 61 m high San Antonio embankment dam the seepage almost doubled after the earthquake and returned to normal in the following days (PGA ca. 0.25 g).

M. Lino reported on the February 2004 earthquake the North of Morocco, magnitude 6.5, ca. 600 people killed. Two dams were affected by this earthquake, i.e. (i) a 30 m high CFRD built on a 30 m thick soil layer with epicentral distance of ca. 10 km experienced a PGA of 0.24 g; minor cracks were observed at the crest, and (ii) an RCC dam ‘built in a raw manner’ and located at an epicentral distance of 30 km behaved well.

N. Matsumoto reported on the M = 8.0 Tokachi-Oki earthquake of September 26, 2003 in Japan (epicenter ca. 60 km offshore of Hokkaido island): PGA recorded at the ground surface
(mainly alluvion) at epicentral distances of 100 to 150 km ranged from 0.2 to 0.8 g. At a depth of 50 to 100m (rock) the PGA was about ½ to 1/3 of that at the ground surface. 45 dams were inspected immediately after the earthquake but no major damage was found. At Takami dam two longitudinal cracks were found at the crest with a depth of less than 1 m, which were repaired.

On May 2003 an earthquake of magnitude 6.8, with a depth of 10 km, occurred 40 km East Alger, provoking 2270 deaths. In Kedda rockfill dam, with 106m high, located 30 km from the fault, only 1 longitudinal crack and 3 transverse cracks were observed. 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).

**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 (Portugal, 1999).

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

The existent equipments in LNEC are shown in Figures 6 and 7.

**Mathematical Models**

The following dynamic analysis of embankment dams is used (Pinto et. al, 1995):

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.

![Fig. 6 Shaking table](image-url)
Simplified procedures to assess deformations were proposed by Newmark (1965), Sarma (1975) and Makdisi and Seed (1977) 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, 1979).

Several finite element computer programs assuming an equivalent linear model in total stress have been developed for 1D (Schanabel et al., 1972), 2D (Idriss et al., 1973; Lysmer et al., 1974) and pseudo 3D (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) (Pinto, 1999).
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 (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 (Prevost, 1993) and the two-surface model (Mröz et al., 1979).

Endochronic models have been refined by the inclusion of jump-kinematics hardening to satisfy Drucker's postulate and achieve closure of the hysteresis loops (Bazant et al., 1982).

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.

**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 (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 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 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, 1990a).

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).
Las Cuevas Dam – Case Study

In general for embankment dams 2 D analyses are performed, but for narrow canyons where the ratio crest length to height L/H is less than 3, a 2 D model act as a kind of low pass filter in the frequency domain and the vibration in dam axis direction would induce the vibration in UD direction due to the 3D effect of valley (Pinto et al., 1992; Sato and Obnuchi, 2000; Jafarzadeh, 2003).

Las Cuevas dam is an earthfill embankment to be constructed in Venezuela, on the Doradas river, in a zone of very high seismicity. The dam is 100m high, 260m long at the crest, the total embankment volume will be $4.5 \times 10^6$ m$^3$ and will impound a reservoir of $1.2 \times 10^9$ m$^3$. The cross section is shown in Figure 8 and the upstream slope will be 1:1.8 and the downstream slope 1:1.8. The dam has a sloped narrow core of clay material and two shells of silty-sand material. A chimney-filter located downstream of the core is composed of sand.

The bedrock is composed from a sand-stone with high permeability and for this reason an extensive foundation treatment was recommended.

![Fig. 8 Las Cuevas dam cross-section](image)

Las Cuevas Dam is located close to active faults of Caparo and Bocono where generation of very strong motions were recorded.

The characteristics of the main sources of earthquakes are synthesized in Table 2.

The first vibration mode is shown in Figure 9.

Figure 10 presents the maximum acceleration response in both horizontal and vertical directions for selected points located in the upstream shell, core and downstream shell for the near earthquake.

The shear stress time histories for six selected elements located at the upstream shell, core and downstream shell for the near earthquake are shown in Figure 11.

Table 2 - Selection of design earthquakes
The shear strain potential distribution is presented in Figure 12. It is clear that relatively high values of shear strain potential are concentrated near the surface of the upstream shell.

Since, for dams in narrow canyons, the response of the structure is of a three-dimensional nature and as the crest length to height ratio L/H=2.6 a three dimensional analysis was performed.

The computer program DYSE52 developed at LNEC was used and Figure 13 shows the finite element model. The structure was discretized in 555 isoparametric finite element (8 mode cubic element and 6 mode prismatic element) with a total of 648 modal points.

The materials that incorporate the dam were considered continuous and isotropic with the shear modulus and Poisson ratio obtained from the last iteration of the 2D dynamic analysis. A damping viscous coefficient of 10% was assumed.

The first twenty vibration modes were determined with fundamental frequencies between 1Hz and 2Hz.

The first vibration mode is shown in Figure 14.
A comparison between natural frequencies computed from 2D and 3D analyses has shown that for 3D analysis the fundamental natural frequencies were 25% higher than those computed from a plane strain analysis.

It was found that a plane strain analysis of the maximum section gave values for the shear stresses that were 50% higher than those computed from a 3D analysis of the dam.

**Hydrodynamic Effects of Reservoir**

Seed, et al. (1985) have disregarded hydrodynamic effects of the reservoir water upon the concrete face considering that this effect is no important for earthfill dams where slopes are with 1(V): 3 (H). But for concrete faced rockfill dams (that will soon reach 200m height), located in high seismicity zones, where slopes are with 1 (V): 1.3 (H) and water pressures are applied directly to the concrete facing, these effects have deserved more attention (Bureau et. al., 1985).

Both Westergaard method and Galerkin formulation were used to analyze reservoir interaction (Pinto, 1996b and Câmara, 1999).

Hydrodynamic effects induced by the impounded water may have significant influence on the response of a dam subjected to earthquake excitation.

Nevertheless the studies related with the dynamic analyses of embankment dams have been intensely developed the effects of water deserve more attention due the following reasons:

The increasing interest in building concrete faced rockfill dams that will soon reach 200 m height.

For concrete face rockfill dams where slopes are with 1(V): 1,3 (H) and water pressures are applied directly to the concrete facing, concentrating the loads in that area.

Nevertheless the good observed behaviour of concrete face rockfill dams in areas of relatively low seismicity there is a lack of information about the behaviour of these dams in zones where strong earthquakes have occurred.

Westergaard method employing the concept of added-mass and using a finite element analysis was used to study a concrete face rockfill dam with 100 m high (Figure 15).

A mesh with 70 isoparametric of 8 nodes and 229 nodal points is presented in Figure 16. An interface element with thickness was considered between the concrete face and the rockfill material.

The construction stage was simulated in 7 layers and the reservoir filling in six steps.

The non-linear parameters for hyperbolic model were considered.

The characteristics of design earthquakes are synthesized in Table 3.

For the vertical component a value of 2/3 of horizontal component was assumed.

The shear distribution at the end of reservoir filling is presented in Figure 16.

A parametric study was done and the following parameters were analysed: (i) effect of the upstream concrete face; (ii) interaction of the reservoir; (iii) effect of the vertical component.
Due to the lack of space only the main results are presented in Table 4.

Table 3  Design earthquakes characteristics

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Near source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period of life (years)</td>
<td>100</td>
</tr>
<tr>
<td>Probability of exceeding the ground acceleration (gal)</td>
<td>10</td>
</tr>
<tr>
<td>Ground acceleration (gal)</td>
<td>406</td>
</tr>
<tr>
<td>Duration of earthquakes</td>
<td>21</td>
</tr>
</tbody>
</table>

From the study the following conclusions were obtained:

Fig. 10 Maximum accelerations responses

Frequencies
The frequencies related with the 1st vibration mode present the following variation: (i) they decrease when the two components of seismic acceleration are incorporated; (ii) they decrease with the effect of reservoir interaction; and (iii) they increase with the effect of the upstream concrete face.

Shear stresses

(i) The maximum shear stress values for the elements of upstream concrete face are located in the last third of the dam;

(ii) The simultaneous occurrence of horizontal and vertical components of the earthquake provokes an increase of the maximum shear stresses;

(iii) The maximum shear stresses increase with the effect of reservoir interaction;

(iv) The effect of the upstream concrete face provokes a decrease of the maximum shear stresses values.

The equivalent number of uniform cycles presents the following path; (i) it decreases when the two components of seismic acceleration are incorporated; (ii) it decreases with the effect of reservoir interaction; and (iii) it increases with the effect of the upstream concrete face.

Accelerations

(i) The nodal accelerations increase from the base to the crest of the dam;

(ii) The nodal accelerations increase when the two components of seismic acceleration are incorporated;

(iii) The nodal accelerations increase with the effect of reservoir interaction;

(iv) The effect of the upstream concrete face provokes a decrease of the acceleration values.

Only some results will be presented here for analysis nº 8.

The shear stress time histories for three selected elements located at upstream face (Figure 16) are shown in Figure 17.

Figure 18 presents the maximum acceleration response in both horizontal and vertical directions for selected points located at the upstream face (Figure 15).

RESERVOIR TRIGGERED SEISMICITY

Man - made earthquakes caused by the filling of reservoirs have drawn the attention of designers concerned with dam safety.

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.
Fig. 11  Shear stress time histories

Fig. 12  Shear potential distribution
Fig. 13  3D finite element mesh

Fig. 14  3D first vibration mode

Fig. 15  Concrete face rockfill dam section
Fig. 16  Finite element mesh

Fig. 17  Shear stress – time story for finite elements n° 13, 33 and 53
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.

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 registred any increase in seismicity due to impounding of waters.

Table 5 presents some examples of dam sites where induced earthquakes with magnitude higher than 5 in the Ritcher scale have occurred (Pinto, 1996b).

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 Figure 19 taken from USSD Report (1997).
Table 4 Parametric study

<table>
<thead>
<tr>
<th>Ex.</th>
<th>Acceleration</th>
<th>Analyse n°</th>
<th>Frequence (Hz)</th>
<th>Element</th>
<th>Maximum shear stress (kPa)</th>
<th>Uniform number of cycles</th>
<th>Nodal points</th>
<th>Horiz. accel. (m/s²)</th>
<th>Vert. accel. (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>1</td>
<td>1.64</td>
<td>9 25 41</td>
<td></td>
<td>219.8 252.5 173.8</td>
<td>31 29 31</td>
<td>53 105 157</td>
<td>4.2 4.4 7.2</td>
<td>0.44 1.8 4.6</td>
</tr>
<tr>
<td>H + V</td>
<td>2</td>
<td>1.58</td>
<td>9 25 41</td>
<td></td>
<td>273.4 278.8 192.4</td>
<td>22 22 27</td>
<td>53 105 157</td>
<td>5.1 5.9 7.4</td>
<td>3.1 6.0 8.6</td>
</tr>
<tr>
<td>H</td>
<td>3</td>
<td>1.57</td>
<td>9 25 41</td>
<td></td>
<td>328.3 299.7 206.8</td>
<td>15 24 31</td>
<td>8.4 7.8 20.0</td>
<td>5.2 9.1 10.0</td>
<td></td>
</tr>
<tr>
<td>H + V</td>
<td>4</td>
<td>1.52</td>
<td>9 25 41</td>
<td></td>
<td>363.0 331.6 242.8</td>
<td>14 18 21</td>
<td>1.1 1.7 4.6</td>
<td>8.6 9.1 10.0</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>5</td>
<td>1.74</td>
<td>13 33 53</td>
<td></td>
<td>187.4 203.7 106.5</td>
<td>39 24 23</td>
<td>1.1 1.7 4.6</td>
<td>1.1 1.7 4.6</td>
<td></td>
</tr>
<tr>
<td>H + V</td>
<td>6</td>
<td>1.69</td>
<td>13 33 53</td>
<td></td>
<td>213.7 209.1 122.5</td>
<td>32 22 15</td>
<td>2.9 5.4 8.0</td>
<td>2.9 5.4 8.0</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>7</td>
<td>1.69</td>
<td>13 33 53</td>
<td></td>
<td>224.9 221.3 114.3</td>
<td>31 23 15</td>
<td>6.3 6.1 7.3</td>
<td>6.3 6.1 7.3</td>
<td></td>
</tr>
<tr>
<td>H + V</td>
<td>8</td>
<td>1.64</td>
<td>13 33 53</td>
<td></td>
<td>259.2 220.3 142.1</td>
<td>22 26 12</td>
<td>6.7 7.8 7.5</td>
<td>2.7 7.8 7.5</td>
<td></td>
</tr>
</tbody>
</table>

H - horizontal acceleration  V - vertical acceleration

Monitoring the RTS activity is recommended for large dams and reservoirs (Oliveira, 1988). 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, 2005c).
Table 5 Examples of dams with induced seismicity.

<table>
<thead>
<tr>
<th>DAM</th>
<th>Country</th>
<th>Type</th>
<th>Height (m)</th>
<th>Reservoir volume ($x 10^6$ m$^3$)</th>
<th>Year of impounding</th>
<th>Induced seismicity</th>
<th>Prior seismicity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marathon</td>
<td>Greece</td>
<td>gravity</td>
<td>63</td>
<td>41</td>
<td>1930</td>
<td>5</td>
<td>1938 moderate</td>
</tr>
<tr>
<td>Hoover</td>
<td>U.S.A.</td>
<td>Arch-gravity</td>
<td>221</td>
<td>36703</td>
<td>1936</td>
<td>5</td>
<td>1939 ---</td>
</tr>
<tr>
<td>Kariba</td>
<td>Zimbabwe/Zambia</td>
<td>arch</td>
<td>128</td>
<td>160368</td>
<td>1959</td>
<td>5,8</td>
<td>1963 low</td>
</tr>
<tr>
<td>Haifengkig</td>
<td>China</td>
<td>buttress</td>
<td>105</td>
<td>10500</td>
<td>1959</td>
<td>6,1</td>
<td>1962 aseismic</td>
</tr>
<tr>
<td>Koyna</td>
<td>India</td>
<td>gravity</td>
<td>103</td>
<td>2708</td>
<td>1964</td>
<td>6,5</td>
<td>1967 low</td>
</tr>
<tr>
<td>Kremasta</td>
<td>Greece</td>
<td>embankment</td>
<td>165</td>
<td>4750</td>
<td>1965</td>
<td>6,3</td>
<td>1966 moderate</td>
</tr>
<tr>
<td>Roi Constantin</td>
<td>Greece</td>
<td>embankment</td>
<td>96</td>
<td>1000</td>
<td>1969</td>
<td>6,3</td>
<td>1966 moderate</td>
</tr>
<tr>
<td>Oroville</td>
<td>U.S.A.</td>
<td>embankment</td>
<td>236</td>
<td>4298</td>
<td>1967</td>
<td>5,7</td>
<td>1975 moderate</td>
</tr>
<tr>
<td>Nurek</td>
<td>Tajikistan</td>
<td>embankment</td>
<td>330</td>
<td>11000</td>
<td>1972</td>
<td>5</td>
<td>1977 moderate</td>
</tr>
<tr>
<td>Tarbella</td>
<td>Pakistan</td>
<td>embankment</td>
<td>143</td>
<td>14300</td>
<td>1974</td>
<td>5,8</td>
<td>1996 low</td>
</tr>
<tr>
<td>Aswan</td>
<td>Egypt</td>
<td>embankment</td>
<td>111</td>
<td>163000</td>
<td>1974</td>
<td>5,3</td>
<td>1981 aseismic</td>
</tr>
</tbody>
</table>

PROTOTYPE DYNAMIC TESTS

Full scale and man-made vibration tests accompanied by appropriate geophysical measurements can provide vital information regarding the dynamic properties of the materials (Pinto, 1991).

These tests play a fundamental role, providing reliable reference data on the seismic behavior of a dam, but the stress level is always very low with respect to that induced by an earthquake. So these tests can not reproduce adequately the non linear behavior of the dam.

For concrete dams forced vibration tests still maintains an important role for the validation of the analytical model.

Table 6 lists the dams, which were subject to one or more types of dynamic excitation.

The dam response to such vibrations was recorded on seismometers placed at various locations of the dam. By analyzing the recorded motions, natural frequencies and associated 3D mode shapes, as well as damping ratios were obtained.
Table 6 lists the dams, which were subject to one or more types of dynamic excitation.

The dam response to such vibrations was recorded on seismometers placed at various locations of the dam. By analyzing the recorded motions, natural frequencies and associated 3D mode shapes, as well as damping ratios were obtained.
Table 6 List of dams subjected to dynamic excitation

<table>
<thead>
<tr>
<th>Nº</th>
<th>Name</th>
<th>Type</th>
<th>Country</th>
<th>Height (m)</th>
<th>Type of Recorded Vibrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Santa Felicia</td>
<td>ED</td>
<td>U.S.A.</td>
<td>83</td>
<td>Seismic, ambient, forced, hydrodynamic</td>
</tr>
<tr>
<td>2</td>
<td>Brea</td>
<td>ED</td>
<td>U.S.A.</td>
<td>27</td>
<td>Seismic</td>
</tr>
<tr>
<td>3</td>
<td>Carbon Canyon</td>
<td>ED</td>
<td>U.S.A.</td>
<td>33</td>
<td>Seismic</td>
</tr>
<tr>
<td>4</td>
<td>Bouquet</td>
<td>ED</td>
<td>Yugoslavia</td>
<td>60</td>
<td>Forced</td>
</tr>
<tr>
<td>5</td>
<td>Mavroro</td>
<td>ED</td>
<td>Japan</td>
<td>56</td>
<td>Forced</td>
</tr>
<tr>
<td>6</td>
<td>Kisenyama</td>
<td>RD</td>
<td>Japan</td>
<td>95</td>
<td>Seismic, ambient, forced</td>
</tr>
<tr>
<td>7</td>
<td>Shimokotori</td>
<td>RD</td>
<td>Japan</td>
<td>119</td>
<td>Seismic, ambient, forced</td>
</tr>
<tr>
<td>8</td>
<td>Nikappu</td>
<td>RD</td>
<td>Japan</td>
<td>103</td>
<td>Seismic, ambient, forced</td>
</tr>
<tr>
<td>9</td>
<td>Talaragi</td>
<td>RD</td>
<td>Japan</td>
<td>65</td>
<td>Seismic, ambient, forced</td>
</tr>
<tr>
<td>10</td>
<td>Sannokai</td>
<td>ED</td>
<td>Japan</td>
<td>37</td>
<td>Seismic</td>
</tr>
<tr>
<td>11</td>
<td>Ainono</td>
<td>ED</td>
<td>Japan</td>
<td>41</td>
<td>Seismic</td>
</tr>
<tr>
<td>12</td>
<td>Ushino</td>
<td>RD</td>
<td>Japan</td>
<td>21</td>
<td>Seismic</td>
</tr>
<tr>
<td>13</td>
<td>Kamishibha</td>
<td>AD</td>
<td>Japan</td>
<td>110</td>
<td>Forced</td>
</tr>
<tr>
<td>14</td>
<td>Mainadisauro</td>
<td>AD</td>
<td>Italy</td>
<td>136</td>
<td>Forced</td>
</tr>
<tr>
<td>15</td>
<td>Pacoima</td>
<td>AD</td>
<td>USA</td>
<td>128</td>
<td>Forced</td>
</tr>
<tr>
<td>16</td>
<td>Cabril</td>
<td>AD</td>
<td>Portugal</td>
<td>136</td>
<td>Forced</td>
</tr>
<tr>
<td>17</td>
<td>Aguieira</td>
<td>MAD</td>
<td>Portugal</td>
<td>89</td>
<td>Forced</td>
</tr>
</tbody>
</table>

ED - earth dam            RD - rockfill dam            AD - arch dam            MAD - multi arch dam

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, 1993b).

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 rockfill 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.

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 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 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.

**FINAL REMARKS AND TOPICS FOR DISCUSSION**

In spite of the impressive progress that has been made in the last years on dam engineering there is still space for questions that remain without a definitive answer. Some questions that deserve further discussion are outlined below.
The floor discussions that we hope lively will give us the unique opportunity to share our experience and will also contribute to the advancement of the knowledge.

i) The complexity of computer codes demands validation of models. What is the best approach?
ii) It seems that the criteria to select the recurrence periods for MCE and OBE are not well defined. Are the ICOLD suggestions well accepted?
iii) The use of discrete element models to reproduce the opening of joints rock medium during earthquakes.
iv) Interplay between ageing effects and maintenance and monitoring related with the safety of dams.
v) The use of geomembranes for dam rehabilitation related with: limitation of specific height of dam, the use of a second watertight barrier in large structures and the risks of leaving the geomembranes uncovered;
vi) Co-operation among engineers, scientists, environmentalists and stakeholders for new dam projects due the important role of social, political and environmental factors;
vii) Continuing education is highly recommended to follow the very fast developments of earthquake geotechnical engineering and particularly of dam engineering (Sêco e Pinto, 2000).

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”.

REFERENCES


ICOLD (2004) Seismic Aspects on Dams Committee meeting in Seul. Reports.


Lemos, J.V. (1999b) Discrete Elements Analysis of Dam Foundations, Memória 817, LNEC.


