CURRENT SEISMIC SAFETY REQUIREMENTS FOR LARGE DAMS AND THEIR IMPLICATION ON EXISTING DAMS

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ABSTRACT
Large dams, especially concrete dams, have been designed against earthquakes since the construction of Hoover dam in the 1930s. The design criteria and methods of analysis used in those times are outdated today and the actual seismic safety of these structures is not known in view of today’s requirements. Earthquake action was usually taken into account pseudo-statically, through an inertia force characterized by a seismic coefficient. Since 1989 the ICOLD guidelines consider two levels of seismic loading, namely the operating basis earthquake (OBE) and the safety evaluation earthquake (SEE). Various methods of dynamic analysis are now available to predict the dynamic response of dams to strong ground shaking. Rehabilitation planning and design must take into account these developments, even in regions of low seismicity and assess whether seismic upgrading of a particular dam is necessary from a seismic safety point of view. As an example the seismic safety assessment of the Mattmark fill dam is discussed, which is located in the area of highest seismicity in Switzerland with a peak ground acceleration of the SEE of 0.43 g.

INTRODUCTION
Large concrete dams have been among the first structures that have been designed against earthquakes since the 1930s, however, the knowledge about their seismic safety is actually poor, because most of them were designed using seismic design criteria and methods of dynamic analysis that are considered obsolete today. Earthquake loading was represented by a pseudo-static force expressed as the product of a seismic coefficient and the weight of the structure, acting in upstream-downstream direction plus a hydrodynamic force acting on the upstream face of the dam. This type of analysis did not consider the dynamic characteristics of the dam, although both, inertia forces and the hydrodynamic pressure were taken into account.

For embankment dams, the design practice was similar, but great efforts were made in understanding the dynamic behaviour of these dams after several failures had occurred due to liquefaction during major earthquakes in the 1960s and especially after the 1971 San Fernando earthquake in California. Earth dam design prior to the 1960s was mainly empirical using judgment guided by past experience. At those times, sites for dams were generally unproblematic and engineers were confident that they could build completely safe structures. Little attention was given to the consequences of a possible failure. The confidence in the ability to build safe dams was derived from the satisfactory performance of a large number of

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existing dams. However, there was a severe lack of precedence of dams which had been subjected to strong shaking by an earthquake (Sherard, 1967). Moreover, there were practically no quantitative data on the response of concrete and earth dams to strong ground motions due to the absence of instrumentation.

**THE CONCEPT OF SEISMIC COEFFICIENT**

During the 1960s, rapid progress was made in both analysis procedures and laboratory testing under dynamic loading. The first dynamic response analysis of an earth dam was made by Mononobe et al. already in 1936. They modeled the dam as an infinitely long symmetrical triangular section consisting of linear-elastic material and resting on a rigid foundation (Mononobe et al., 1936; Seed & Martin, 1966). However, general design practice at that time was to take account of the seismic loading of a dam by a seismic coefficient. This seismic coefficient was commonly between 0.10 and 0.15. The concept was that the seismic forces acting on the dam could be expressed by a static horizontal force expressed as a product of the seismic coefficient and the weight of the potential sliding mass in the dam body. If in a static stability analysis the factor of safety would approach unity, the dam would be considered close to failure and therefore unsafe. Ambraseys (1960) improved this approach by adopting seismic coefficients which were based on seismic response analyses of the embankment.

Seed & Martin (1966) developed dynamic seismic coefficients which took into the account the dynamic response of the dam to a ground motion input and also the shear wave velocity of the dam material. They started out with the Mononobe shear beam approach and a visco-elastic response analysis to determine the entire time history of accelerations and stresses developed in an embankment during the period of significant ground shaking. In this way, it was possible to express the stresses developed at different horizontal sections of an embankment in terms of actual seismic coefficients (as opposed to their pseudo-static version), which were dependent on the height of the dam and on the shear wave velocity of the embankment materials. These seismic coefficients did not exceed a value of about 0.4 for the cases analyzed.

There is no relationship between the peak ground acceleration (PGA) and the seismic coefficient in an analysis. Marcuson (1981) suggested that appropriate pseudo-static coefficients for dams should correspond to one-third to one-half of the PGA including amplification and de-amplification effects, to which the dam is subjected. Similarly, from applying the Newmark method to a large number of accelerograms Hynes-Griffin & Franklin (1984) concluded that as long as the factor of safety is above 1.0 with a seismic coefficient of 0.5 PGA/g, unacceptable deformations would not occur.

Hence, pseudo-static analysis, because of its simplicity, may still have some merits today, at least for preliminary design. However, as soon as the materials in the dam or in its foundation show a tendency to build up significant pore water pressures, or lose more than about 15% of their strength during seismic loading in a laboratory test, the use of seismic coefficients is unsafe and must be strictly abandoned.

**DEFORMATIONS AS A PERFORMANCE AND SAFETY CRITERION**

Pseudostatic design methods using seismic coefficients selected empirically, have little rational basis and they may even be unsafe. When the factor of safety reaches 1.0, the slope of the dam is considered to have failed, i.e. a mass of soil will slide down the slope. However, since the inertia force is not acting continuously in the same direction, but oscillates, the movement will stop and continue only when the next cycle exceeds the resisting shear stress.
Newmark (1965) proposed therefore the important concept that the effects of earthquakes on embankment stability should be assessed in terms of deformations they produce rather than the minimum factor of safety.

Newmark’s method models the sliding mass of a dam slope by a block subjected to the acceleration time history of the selected earthquake. The block (or the soil body) will start to slide when the shear stresses exceed the shear resistance. The level of acceleration acting on this soil body at the onset of sliding is termed the yield acceleration. The displacements of the body can then be obtained by double integration of the acceleration time history. Newmark’s method could be verified from the analysis of instrumental displacement records obtained at the La Villita dam in Mexico (Elgamal et al., 1960). This dam had been shaken by strong earthquakes repeatedly.

Newmark’s method is only applicable for materials which do not lose strength in post-earthquake conditions which means that the slope is still stable after shaking has ceased. It is also useful for situations where no changes in pore-water pressure will occur, i.e. when the slope materials are cohesionless or dry.

**SEISMIC SAFETY ASSESSMENT FOR EXISTING DAMS**

Seismic safety assessment is an integral process consisting basically of:

(i) Seismic hazard assessment, which includes the seismotectonic features (i.e. fault movements), and the ground shaking (i.e. acceleration time histories) for different types of design earthquakes;

(ii) Seismic response analysis, which combines the model of the dam-reservoir-foundation system, the material properties and the method of analysis; and

(iii) Performance assessment which includes possible damage assessment or acceptance criteria

The various components of a seismic safety assessment for existing dams are shown in the flow chart of Fig. 1.

**Earthquake ground motion**

The earthquake ground motion can be represented best by acceleration time histories. However, if a probabilistic seismic hazard analysis is performed then the seismic hazard is represented by uniform hazard spectra and/or the PGA for different design earthquakes. From the uniform hazard spectra and the de-aggregation of magnitudes and epicentral distance, scenario earthquakes have to be determined. Again, the best representation of the scenario earthquake ground motion is by means of acceleration time histories. The time history of the outcropping rock obtained from the seismic hazard analysis is the input for any seismic analysis. Depending on the type of analysis further transformations of the ground motion are necessary especially in cases where wave radiation effects are taken into account.

With the publication of ICOLD Bulletin 72 “Selecting seismic parameters for dams” in 1989, it was realized that ground shaking which a dam must be able to resist can be much more severe than generally assumed in the past. The PGA was recognized as a realistic parameter to characterize earthquake ground motions and the seismic hazard at dam sites. State-of-the-art probabilistic seismic hazard analyses (PSHA) carried out for nuclear power plants in Switzerland have shown that the seismic hazard can be considerably larger than what has been assumed in the original design. The same will apply for dams in regions of low to moderate seismicity as a result of uncertainties accounted for in the seismic hazard analyses.
Improvements in the development of seismic hazard analyses have to focus on reducing the uncertainties and the introduction of so-called new generation attenuation (NGA) models (Stafford et al., 2008). Earthquake catalogues are often short in historical data, especially in regions with a low population density where most of the dams are located. There is a need for more research in paleo-seismicity to get better information on the characteristics of very strong earthquake ground motions.

Fig. 1: Basic components of a seismic safety assessment for existing embankment dams

Dam engineers are, however, ill advised when they believe that more sophisticated and complex seismic hazard studies will reduce the hazard levels and thus will improve the seismic safety of their dam. It is more important and effective to invest at least the same amount of effort and resources for studies on the resilient design of new dams or the performance assessment of existing dams. High values of PGA do not necessarily mean failure. For example, with other types of structures, like civil defense shelters, the PGA values used in analysis and design may be ten to thirty times higher than those employed by earthquake engineers. However, the frequency content and duration of ground shocks are different from those caused by earthquakes.

ICOLD Bulletin 72 of 1989, whose revision is under way, defines the following seismic design levels for the design of new dams and also for the safety evaluation of existing dams:

1. Operating Basis Earthquake (OBE) with the following criteria:
   - Return period: Approx. 145 years.
   - Performance of the dam: No structural damage, i.e. stresses in concrete dams must remain within allowable stresses.
   - Dam safety: not relevant for dam safety; the OBE represents a serviceability limit state and is basically an economical criterion, which is of main interest for the dam owner.
(2) Safety evaluation earthquake (SEE). This is the proposed terminology in the revised Bulletin 72. Other terms in use are: Maximum Credible Earthquake (MCE) and Maximum Design Earthquake (MDE). The following criteria apply:
- Return period: not specified, but typically 10,000 years.
- Performance of the dam: no uncontrolled release of water from the reservoir, structural damage accepted.
- Dam safety: The SEE is relevant for dam safety; it represents a limit state of ultimate load.

Seismic response analysis
The seismic response analysis for both concrete and embankment dams are well developed, especially the linear-elastic dynamic analysis. The numerical modeling of construction materials (concrete, etc.) and the dam foundation under severe dynamic loads causing large strains and cracking is a very difficult and costly task and considerable uncertainties remain in any dynamic analysis.

Performance/acceptance assessment
Performance criteria or specifications of acceptable damage under SEE ground motions are poorly defined and considering these in safety evaluations requires considerable engineering judgment. Basically, it must be shown that after the SEE the damaged dam can still retain the reservoir water. How can such damage be defined? So far very little work has been done in this area of dam safety assessment. Unlike the seismic hazard assessment which has been tackled by a large number of earth scientists, few engineers may be able to contribute to the damage assessment, mainly due to the lack of dams, which have experienced very strong ground shaking.

Rehabilitation needs and remedial actions
If the seismic safety assessment of a dam shows that the performance criteria are not satisfied (e.g. insufficient freeboard) or that the actual performance cannot be accepted from a dam safety point of view (e.g. excessive leakage with occasional discharge of solids) plans for rehabilitation or other risk-reducing measures should be established, for example, restrictions in the use of the reservoir.

ICOLD guidelines on seismic aspects of dams
At present, the following ICOLD guidelines are concerned with seismic aspects of dams (Wieland, 2003):
- Bulletin 112 (1998): Neotectonics and dams (i.e. dams on faults)
- Bulletin 120 (2001): Design features of dams to effectively resist seismic ground motion
- Bulletin 123 (2002): Earthquake design and evaluation of structures appurtenant to dams
- Bulletin in print (2008): Reservoirs and seismicity – state of knowledge (i.e. reservoir-triggered seismicity)
SEISMIC VULNERABILITY OF CONCRETE DAMS

The following lists of ICOLD recommendations for the design of new dams can also be helpful in identifying design deficiencies in the safety assessment of existing dam structures. Engineering judgment will then be needed to rate the severity of a particular deficiency and its probable impact during a strong earthquake.

Large concrete dams, which were exposed to a strong earthquake, have experienced severe cracking but none of these dams has violated the SEE criterion, i.e. there was no uncontrolled release of reservoir water.

Arch dams

There are several design details that are regarded as contributing to the very favourable seismic performance of arch dams (ICOLD, 2001), i.e.:

- Development of a design with regular and smooth geometry (symmetry is desirable)
- Maintenance of continuous compressive loading along the foundation, by shaping of the foundation and also designing a plinth structure to support the dam and transfer load to the foundation, if found necessary.
- Limiting the crest length to height ratio, to assure that the dam does not distort excessively in the higher modes of vibration during earthquake shaking and that it carries a substantial portion of the applied seismic forces by arch action.
- Providing contraction joints with adequate interlocking
- Improving the dynamic resistance and consolidation of the foundation rock by appropriate excavation, grouting, etc.
- Provision of well-prepared lift surfaces to maximize bond and tensile strength.
- Increasing the crest width to reduce high dynamic tensile stresses in the crest region.
- Minimizing unnecessary mass in the upper portion of the dam that does not contribute effectively to the stiffness of the crest.

Gravity and buttress dams

The structural features which are considered to improve the seismic performance of gravity and buttress dams are as follows (ICOLD, 2001):

- Maintenance of low concrete placing temperatures to minimize initial heat-induced tensile stresses and shrinkage cracking
- Development and maintenance of a good drainage system.
- Providing well-prepared lift surfaces to maximize bond and tensile strength
- Utilization of shear keys in vertical construction joints
- Minimizing of discontinuities in the dam body, to prevent local stress concentrations.
- Increasing the crest width to improve the dynamic stability of the dam crest
- Avoiding a break in slope on the downstream faces of gravity dams to eliminate local stress concentrations.

SEISMIC VULNERABILITY OF EMBANKMENT DAMS

The only dams that are known to have failed completely as a result of seismic shaking are tailings and hydraulic fill dams, or also relatively small earthfill embankments of older and, perhaps, inadequate design and construction. One of the most dangerous consequences of the dynamic loading of an embankment dam is liquefaction of foundation or embankment zones containing cohesionless materials of low relative density.
The main recommendations for design and construction of embankment dams subject to severe earthquake shaking are as follows (ICOLD 2001):

- Foundation must be excavated to very dense soil or rock; alternatively the loose foundation materials must be densified or removed.
- Fill materials which tend to build up significant pore water pressures during strong shaking must not be used.
- All zones of the embankment must be thoroughly compacted to prevent excessive settlement during an earthquake.
- All embankment dams, and especially homogeneous dams, must have high capacity internal drainage zones to intercept seepage from any transverse cracking caused by earthquakes, and to assure that embankment zones designed to be unsaturated remain so after any event that may have led to cracking.
- Filters must be provided on fractured rock to preclude piping of embankment materials into the foundation.
- Wide filter and drain zones must be used.
- The upstream and/or downstream filter and transition zones should be self-healing, and of such gradation as to also heal cracking within the core.
- Sufficient freeboard should be provided in order to cover the settlement likely to occur during the earthquake and possible water waves in the reservoir due to mass movements, etc.
- Since cracking of the crest is possible, the crest width should be wider than normal to produce longer seepage paths through any transverse cracks that may develop during earthquakes.

**EXAMPLE: MATTMARK DAM**

Mattmark dam is a 117 m high embankment dam with a sloping core, constructed in the 1960s and located in the southern Swiss Alps (see, e.g. Gilg, 1974). This region has the highest seismicity in Switzerland. A planned increase of the reservoir level by 2 m gave rise to a safety evaluation, also under dynamic loading, following the new guidelines of the Swiss Dam Safety Authority. The dam was designed using pseudo-static analysis according to the state-of-practice of the early 1960s. The seismic coefficient was 0.1 and resulted in a safety factor of 1.03 for a load combination including rapid draw-down. The dam has never been exposed to a strong earthquake.

A typical section of Mattmark dam is shown in Fig. 2. A special feature are the rather complex foundation conditions with various glacial deposits having a maximum depth of about 90 m. Sealing of these deposits was accomplished by a multiple-row curtain consisting of clay-cement and bentonite-silicate grouts (Gilg, 1974).

The seismic evaluation parameters used are the Safety Evaluation Earthquake (SEE) defined in the Swiss guidelines as an earthquake with a return period of 10,000 years, and an elastic response spectrum taken from Eurocode 8, Part 1. The SEE was available in terms of an MSK-intensity with a value of 9.3, which was converted to peak horizontal and vertical ground accelerations of 0.42 g and 0.28 g, respectively. The ground motion was then expressed by three statistically independent spectrum compatible accelerograms of different duration.

The dynamic material properties were obtained from cyclic triaxial testing of materials taken from the dam core and from published relationships of the strain-dependent shear modulus.
and damping ratio. The main objectives of the cyclic triaxial tests were (1) to observe the development of excess pore water pressures during cyclic loading and (2) to enable an estimate of the residual deformations caused by the cyclic loading. In order to perform the tests under realistic loading conditions, the cyclic shear stress amplitude was determined from a two-dimensional dynamic finite element analysis which yielded the cyclic shear stress on the horizontal plane of each element. Seismic loading was then modelled by taking a number of stress cycles with constant amplitude of 0.65 times the maximum shear stress amplitude of the calculated stress history and a frequency of 0.1 Hz (Wieland & Malla, 2002).

The results obtained from the cyclic triaxial tests revealed that the predominantly granular material exhibited dilatant behaviour and the development of pore water pressures ranged from negative to a maximum of about 20 kPa. Liquefaction or significant loss of strength during a strong earthquake could therefore be excluded for the Mattmark dam material. The residual deformation at the end of cyclic loading (usually limited to 50 cycles) could, however, be substantial, i.e. 5% or more (Brenner et al, 2005).

The analysis of the response to seismic loading determined the deformation (settlement) due to densification (since there is no liquefaction) and the deformation caused by a body sliding on a surface which passes through the entire core. For the densification analysis it was assumed that the additional deformation could be estimated from the reduced stiffness of the dam material which results from the dynamic loading (strain dependent moduli). The second analysis was carried out by the Newmark method. It was found that shallow slip surfaces intersecting the crest or the core only partially, could produce large displacements, i.e. on the order of up to 3 m. On the other hand, soil bodies moving along slip surfaces passing through the entire width of the crest did not displace more than about 0.8 m. Hence, the total vertical deformation, relevant to the available freeboard, amounted to only about 1.6 m. With a freeboard of 5.0 m, there is therefore still ample reserve to avoid overtopping.

Fig. 2: Typical section of Mattmark dam
The re-evaluation of the seismic safety of Mattmark dam could therefore demonstrate that none of the performance/acceptance criteria was violated and that increasing the reservoir level could proceed without the necessity of remedial works.

CONCLUSIONS
Following ICOLD guidelines, current seismic safety requirements demand that new and as well also existing dams can withstand the SEE without causing an uncontrolled release of the reservoir water. There is a significant number of large dams all over the world, both in regions of high and moderate to low seismicity, which were designed for ground motions of considerably lower intensity than the SEE. A re-analysis of these dams using state-of-practice procedures is needed to check whether they can satisfy the performance criteria. Rehabilitation planning of old dams must therefore take into account these new guidelines, also in regions of low seismicity.

Attention shall be paid to the dynamic characteristics of construction materials, especially those of older dams for which little experience may be available

The dam safety assessment shall not focus only on the seismic hazard, although new geological investigations may reveal a hazard enhancement.

To cope with uncertainties in the assessment of the behaviour of dams under very strong ground shaking, i.e. next to a fault zone, or even within a fault zone, a resilient and conservative design is the right answer.

REFERENCES


ICOLD, 1989. Selecting Seismic Parameters for Large Dams, Guidelines, Bulletin 72, Committee on Seismic Aspects of Dam Design, ICOLD, Paris


