A Fresh Look at Earthquakes and Reservoirs

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SYNOPSIS

This paper seeks to address a wide range of topics which need to be borne in mind when designing or inspecting dams in seismic areas. The title refers to reservoirs rather than dams because there is much to consider other than the seismic behaviour of the dams themselves.

INTRODUCTION

ICOLD Bulletin 188 (2019)[1] and Foster, Fell and Spannagle (2000)[2] say that overall about 2.2 % of dam failures are due to seismic activity. The vast majority of these are small, homogeneous earth dams many of which have been in China, India and Japan.

In this paper comments are offered on the vulnerability of various types of dams to earthquakes as well as other events, such as landslides, which may be extremely serious whether triggered by earthquakes or by other causes.

AN OVERVIEW OF EARTHQUAKES

Although earthquakes can occur almost anywhere they are most frequent, and most severe, near the world’s tectonic plate boundaries. This is particularly the case around the rim of the Pacific Ocean, along the Sunda arc, in the Himalayas, through Iran and Turkey and in south-east Europe.

Whilst areas most prone to earthquakes are fairly easily identified precise timings and locations of large earthquakes are generally not predictable. A possible exception is the North Anatolian Fault where there were a series of large earthquakes progressing from east to west along the 1,500 km long right-lateral strike-slip fault between 1939 and 1999 (see Figure 1). There are concerns that the next event in the sequence may be close to the city of Istanbul with its population of over 15 million people.

There was a similar series of earthquakes on the North Anatolian fault in 967 – 1050 AD. Clearly one must look far enough back into the past to determine the risk. We cannot say that because there has been no activity for a couple of hundred years there is nothing to worry about. There was a gap of 203 years between major earthquakes in Guatemala in 1773 and 1976.

Significant earthquakes have taken place along the Jordan valley fault at an average interval of about 250 years although there have been gaps of 400 or 500 years. The last sizeable earthquake in 1546 destroyed the town of Nablus so another may be due fairly soon.

The San Andreas Fault in California is 1,200 km long and has many similarities to the North Anatolian Fault. It is also a right-lateral strike-slip fault with a slip rate of about 20 to 35 mm/year. The San Francisco earthquake of 18 April 1906 had M_W of 7.9 and is thought to have killed over 3,000 people. Some have suggested that the next major earthquake on the fault could be further south near the city of Los Angeles.

On 22 October 2012 six scientists and one ex-government official were convicted in Italy for manslaughter for failing to predict a major earthquake six days before the M_W 6.3 event at L'Aquila on 6 April 2009 which caused the deaths of 309 people. After a series of smaller events they had advised that a large earthquake was unlikely but possible, emphasizing the uncertainty of their knowledge. Their, highly controversial, six year prison sentences were eventually overturned on 10 November 2014.

Figure 1. Earthquakes on North Anatolian Fault in Turkey.

CALCULATION OF PEAK HORIZONTAL GROUND ACCELERATIONS

As noted below the most significant factors in determining the seismic response of concrete dams are thought to be the Peak Horizontal Ground Acceleration (PHGA) and the spectral acceleration at the natural frequency of the dam (Hansen and Nuss, 2011)[3].

At its simplest a rough estimate of the PHGA with a return period of 475 years can be made from the GEM map of global seismic hazards (Pagani et al, December 2018)[4]. Whilst this map gives a good overview of world seismicity it is not possible to derive from it the PHGA of events with much longer return periods (say 10,000 years).

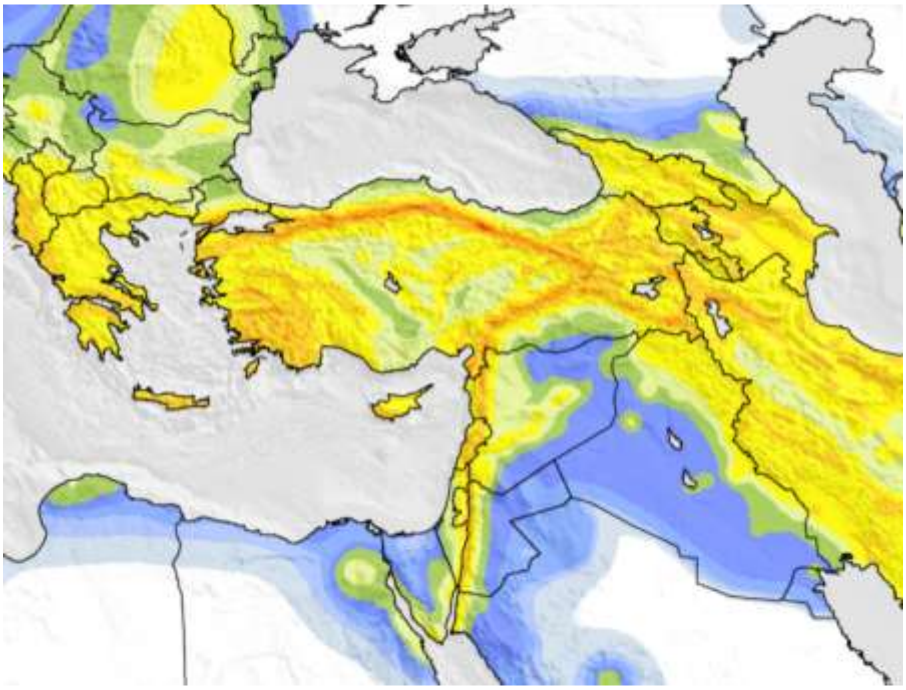

Figure 2. Seismicity of Turkey, Caucasus and Southeast Europe PHGA on rock surface for return period of 475 years (main fault zones; North Anatolian Fault, East Anatolian Fault and Dead Sea fault zone. Extract from GEM Map.

The preferred approach is to identify potential causative faults near the damsite and to determine the largest earthquake thought to be possible (or to have taken place historically) on those faults. It is then assumed that an earthquake of that magnitude could take place at the nearest point on the fault to the damsite (note: it will often be necessary to consider several faults and select that which gives the worst ground motion parameters at the site).

The PHGA at the damsite can be calculated using suitable attenuation formulae which take account of magnitude, earthquake mechanism, rock properties (shear wave velocity in top 30 m), hypocentral distance and statistical confidence levels. Some experts advocate using only one, carefully chosen, attenuation formula appropriate to the area in question whereas others prefer to take the mean result returned by several formulae. Alternatively a number of attenuation formulae can be used with a suitable weighting given to each depending on their perceived applicability.

The above returns a deterministic Maximum Credible Earthquake (MCE) but probabilistic methods will often be used as well.

The PHGAs for lower return periods can be estimated using the following formula from Eurocode 8:

$$PGA_2 = (T_2/T_1)^{0.33} \times PGA_1$$

This formula implies a PHGA for the 10,000 year event which is 2.73 times that for the 475 year event. This will usually be very conservative so the formula should not be used for scaling up to obtain PHGAs for long recurrence period events and is not a substitute for a site-specific seismic hazard study (Lubkowski and Aluisi, 2012)[5].

RESERVOIR-TRIGGERED SEISMICITY

During impounding and/or during the first years of operation of large reservoirs reservoir-triggered seismicity (RTS) may occur, which is related to active faults in the reservoir region and/or the existence of faults with high tectonic stresses close to the strength of the fault. Such events are particularly associated with dams over 100 m high and reservoirs holding more than 500 Mm^3.

The first documented case of RTS was the case of Lake Mead (created by the 220 m high Hoover Dam in the USA) experienced after 1935. For a time it was an isolated case (although there probably were more RTS phenomena, especially on a microseismic level, which went unobserved). But by the late 1960s, a number of significant triggered events had accumulated, and some of them had quite serious consequences, so that the general interest in this phenomenon sharply increased.

From the very beginning, this subject has been controversial. A number of experts argued that it was not very credible that huge amounts of energy, corresponding to high magnitude earthquakes, can be released as a consequence of relatively small changes in the state of stresses at seismogenic depths, due to impounding.

The question of maximum magnitude which can be ascribed to RTS is difficult to clarify other than by relying on relative frequency of such cases compared to the number of large dams. In the case of general seismicity, the ceiling magnitude between 8 and 9 is generally accepted, as observed fact. Considering all accepted RTS cases magnitudes in the range of 6.0 to 6.3 have been recorded in only four cases. It is, therefore, logical to accept such a ceiling as a maximum possibility (ICOLD 2011)[6].

SEISMIC DESIGN CRITERIA

According to ICOLD Bulletin 148 (2016)[7] the following design earthquakes are needed for the seismic design of the different structures and elements of a large dam project where dam failure would present a great social hazard:

- Safety Evaluation Earthquake (SEE): The SEE is the earthquake ground motion a dam must be able to resist without uncontrolled release of the reservoir. It will normally be deterministically evaluated or have a very long return period, for example 10,000 years.
- Design Basis Earthquake (DBE): The DBE with a return period of 475 years is the reference design earthquake for the appurtenant structures. The DBE ground motion parameters are estimated based on a probabilistic seismic hazard analysis (PSHA).
- Operating Basis Earthquake (OBE): The OBE may be expected to occur during the lifetime of the dam. No damage or loss of service must happen. It has a probability of occurrence of about 50% during the service life of 100 years. The return period is taken as 145 years.
- Construction Earthquake (CE): The CE is to be used for the design of temporary structures such as coffer dams and takes into account the service life of the temporary structure.

It is worth recording that, when serving on Panels of Experts in highly seismic areas, the author has sometimes come under pressure from Clients and Designers to accept SEEs with return periods significantly lower than 10,000 years. In such cases it is worth calculating the probability that the SEE will be exceeded in a period of risk of, say, 100 years.

In one such case a return period of 3,000 years had to be accepted although the dam was for irrigation purposes and would, therefore, be full for only a short period each year increasing the effective return period of the SEE. The risk of exceedance of the adopted SEE in a period of 100 years would be 3.2 % (compared with 1 % for the event with a return period of 10,000 years). Anticipated settlement of the crest can be checked by full analyses or by using simplified methods such as those suggested by Charles, Abbiss, Gosschalk and Hinks (1991)[8].

In the case of another large concrete dam the Client has decided that the SEE should be chosen to have a value corresponding to a return period of 1,000 to 2,000 years (5 to 10 % probability of exceedance in a 100 year period of risk). Whilst no concrete dam (apart from Shih-Kang) is ever thought to have failed as a direct result of an earthquake the following have come very close to failure:

- Sefid Rud, Iran (1990)
- Hsinfenkiang, China (1962)
- Koyna, India (1967)

In view of the above ICOLD recommends that SEEs should have return periods close to 10,000 years. In the case mentioned above the author has recommended further studies in parallel with the concrete mix design and the geometric design of the dam.

DAM TYPES

General

The Pre-print of ICOLD Bulletin 188 (2019)[1] concludes that, for dams in general "*there is no significant effect of dam type on the failure ratio, except perhaps for rockfill dams with a somewhat larger failure ratio*".

The above comment refers to all types of failure and not just those caused by earthquakes. Looking only at failures caused by earthquakes (which represent about 2.2 % of the total) it seems logical to assume that seismic considerations will play a very important part in the choice of dam type as well as in the design itself.

The works of Foster, Fell and Spannagle (2000)[2] and Douglas, Spannagle and Fell (1998)[9] also give much useful data on the failure rates of various dam types. ICOLD Bulletin 183 (2019)[10] gives detailed guidance on the choice of dam type. Much useful information on the actual performance of dams in earthquakes is given in the USSD book on the subject (USSD, 2014)[11].

Methods of Analysis

Normally several acceleration time histories will be taken from historical events and scaled so that the maximum acceleration and the acceleration response spectra correspond to the Peak Horizontal Ground Acceleration (PHGA) obtained from the seismic hazard analysis. The time history for horizontal acceleration is usually applied non-synchronously with the vertical acceleration for which the peak value is often taken as 2/3 of the PHGA although higher ratios may apply near to the epicentre of earthquakes.

Both PGA and acceleration response spectra depend on the dynamic properties of the site, which is characterized by the shear wave velocity in the top 30 m. For hard sites the frequency range of the maximum spectral accelerations is between 5 Hz and 12 Hz and for softer sites the critical frequency range is between 3 Hz and 8 Hz (Charles et al, 1991)[8].

The state of the art for the seismic analysis of concrete dams is given in the recent book by Løkke and Chopra (March 2019)[12]. The book describes the use of Finite Element techniques for dams going back to the work of early pioneers such as Zienkiewicz in the 1970s. In the title of their book Løkke and Chopra stress the importance of taking due account of non-linear behaviour of the concrete and also Dam-water-foundation interaction. They discuss two-dimensional and three-dimensional models.

Analyses of embankment dams will focus on the anticipated crest settlements. The simplified method of Makdisi and Seed (1978)[13] obtains yield accelerations from pseudostatic stability analyses and earthquake induced accelerations in the embankment are determined using dynamic response analyses. When the induced acceleration exceeds the calculated yield acceleration for a given potential sliding mass, movements are assumed to occur along the direction of the failure plane and the magnitude of the displacement is evaluated by a double integration procedure. Anticipated crest settlement can then be compared with freeboard.

Rockfill Dams with Clay Cores

Rockfill dams with clay cores have generally performed very well in earthquakes suffering only slight settlement. For example, the 102 m high **Yuvacik** dam in Western Turkey was only 10 km from the epicentre of a Magnitude 7.4 event on the North Anatolian Fault on 17 August 1999. The only permanent effect at the dam was crest settlement of 114 mm. There was no damage at appurtenant structures such as the intake to the gated spillway.

As one typically has more than 5 m freeboard at well-engineered large dams the conclusion is that settlement in earthquakes is unlikely to threaten the dam. However, there was a period when some rockfill dams were built without compaction. The 113.5 m high **Tikves** dam in North Macedonia, which was built without compaction of the rockfill, has settled by 2.5 m without a nearby earthquake. Further

Figure 3. Tikves dumped rockfill dam in North Macedonia.

significant settlement might be expected in a seismic event.

Concrete Faced Rockfill Dams

Concrete faced rockfill dams (CFRDs) are often seen as particularly suitable for seismic areas. Even if the slabs crack substantial quantities of water can leak through the rockfill without endangering the dam.

The 156 m high **Zipingpu** CFRD in China suffered a foundation acceleration estimated at 0.51 g in the Magnitude 7.9 Wenchuan earthquake of 12 May 2008 when the reservoir was about 30 % full. There was damage to the joints between the face slabs and some superficial damage to the slabs on the crest of the dam but the dam was not itself seriously threatened. Leakage increased from 10.4 l/s to 18.8 l/s and was turbid for a couple of days. There was 760 mm settlement including that in aftershocks.

The Longmenshan Fault, which was responsible for the earthquake, has the lowest long-term deformation rate compared with other major faults of the Qinghai-Tibetan plateau (Wieland and Chen 2009)[14]. With the exception of the M 7.5 Diexi earthquake of 1933, historic earthquakes within the Sichuan Province area have not exceeded Mw 6.5. The upper bound magnitude of the Yinxiu-Beichuan area has now been increased to 8.0.

The intensity of shaking in this case was dependent on the distance from the fault break rather than the epicentral distance. This will often be the case in large earthquakes where there is a long fault break – in this case 270 km long.

Concrete Gravity Dams

The most significant factors in determining the response of concrete dams is thought to be the PHGA and the spectral acceleration at the natural frequency of the dam (Hansen and Nuss, 2011)[3].

In earthquakes concrete gravity dams sometimes exhibit horizontal cracking towards the crest on the upstream and downstream faces.

The 103 m high **Koyna** Dam in India (a conventional concrete gravity dam) suffered such horizontal cracking in the magnitude 6.5 earthquake of 10 December 1967. There are three interesting things about this:

(1) Many people believe the earthquake was a case of reservoir triggered seismicity (see below).
(2) The dam height was increased during construction which led to increased weight near the crest of the non-overflow sections.
(3) The cracking did not extend to the central spillway blocks where there was less weight at a high level in the dam.

Some have expressed the opinion that if the earthquake had been of slightly longer duration the dam would have failed.

Hansen and Nuss (2011)[3] state that "*The threshold of no damage is project specific, but can quite probably be significantly higher than 0.3g for properly designed and constructed concrete gravity and arch dams".* The present author notes that the Sefid Rud buttress dam in

Iran came very close to failure in June 1990 when the PHGA was estimated to have been 0.714 g.

For concrete gravity dams it is worth noting that 3D effects in narrow valleys and curvature in plan can be beneficial in seismic events.

Roller Compacted Concrete (RCC) Dams

There are now approximately 1,000 RCC dams that are either complete or under construction. This form of dam has essentially superseded traditional concrete gravity dams and to a certain extent arch-gravity dams. In China there are also a significant number of RCC arch dams with heights up to 168 m (Wanjiakouzi). Quite high dynamic strengths can be achieved for suitably designed mixes.

The 95-m high **Platanovryssi** RCC gravity dam in northern Greece was designed for no cracking in the dam faces in the SEE with a PGA of 0.38g. This was, perhaps, a bit conservative as some cracking, of limited depth, is sometimes accepted in the SEE. It must, however, be

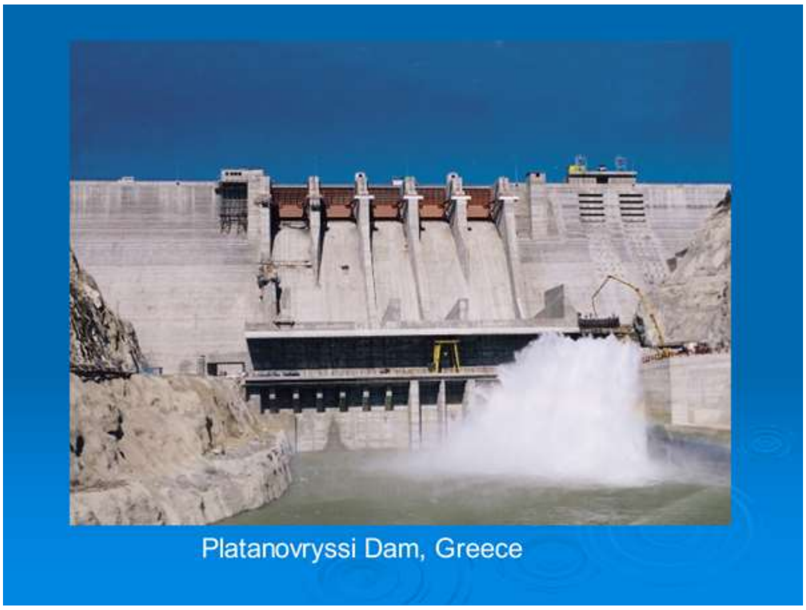

Figure 4. Platanovryssi RCC dam in Greece

remembered that there will already be pre-existing thermal stresses dating from the initial cooling of the dam. The mixture proportions of the RCC at Platanovryssi are unusual as a Class C (high-lime) flyash was used and in spite of the high dynamic loading the Portland cement content was only 50 kg/m³ while the flyash content was 255 kg/m³. The average core compressive strength as an age of 91 days was 29.6 MPa and the average vertical direct tensile strength across the horizontal joints 1.77 MPa. The average cohesion of the joints was 4.04 MPa and the average vertical compressive modulus 23 GPa.

The 105-m high **Changuinola-1** RCC arch-gravity dam in Panama was designed for a PGA of 0.40g[15]. The dam consists of an arch-gravity section in the middle of the dam with gravity sections on either abutment. The dam was subjected to a reasonably severe earthquake during construction but not since it had been completed and the instrumentation installed.

The 132-m high **Shapai** RCC arch dam was designed for a PGA of 0.138g. It was only 36 km from the epicentre of the Wenchuan earthquake on 12 May 2008. It is estimated that it was subjected to a PGA of 0.5g. The reservoir was almost full at the time and the dam body was completely undamaged although the Powerhouse, some kilometres downstream of the dam, was very severely damaged due to a landslide and rockfalls.

Figure 5. Changuinola Arch-Gravity RCC Dam, Panama (105 m high)

Arch Dams

Arch dams have behaved well in earthquakes. The Ambiesta arch dam in northern Italy is 59 m high and was only 22 km from the epicentre of the Gemona-Friuli earthquake of 6 May 1976 (magnitude 6.5). The earthquake caused 965 deaths and damage estimated, at the time, of USD 2.8 billion. A maximum acceleration of 0.33 g was measured at the right abutment.

Neither the Ambiesta dam nor 13 other concrete arch dams in the area suffered damage from the event; this includes the 136 m high Maina di Sauris dam some 43 km from the epicentre.

As mentioned above the Shapai RCC arch dam in China suffered bedrock acceleration of about 0.5 g. The reservoir was almost full at the time but there was no visible damage to the dam (although there was damage to the intake tower).

The 113 m high Pacoima arch dam in California suffered cracking at the left abutment in a magnitude 6.6 event in 1971.

It is worth noting that, while seismic compressive stresses are not usually a problem with gravity dams, they may become important in the design of arch dams.

Concrete Buttress Dams

Buttress dams do not have a particularly good reputation with regard to earthquakes. This is largely because of serious cracking at **Sefid Rud** dam in Iran and **Hsingfengkiang dam** in China.

Fig. 6. Sefid Rud buttress dam was damaged near the crest due to ground shaking during the magnitude 7.5 Manjil earthquake of June 20, 1990.

Despite the concerns of analysts about out-of-plane vibrations of the monoliths both of these dams seem to have cracked in response to transverse (upstream/downstream) accelerations.

Hsingfengkiang dam is 105 m high and suffered horizontal cracking 16 m below the crest in an event of magnitude 6.1 in 1962 (see above).

The Sefid Rud buttress dam in Iran is 106 m high and was affected by the Manjil earthquake of June 1990 which had a magnitude of about 7.5. It has been estimated that the PHGA at the dam would have been about 0.71 g. Major cracks about 10 mm wide developed along horizontal construction joints near to the change of slope on the downstream profile. Unlike at Hsingfengkiang, the level of the cracks varied from monolith to monolith. At monolith 15 there was a 20 mm displacement of the crest of the dam towards the downstream side, with corresponding leakage through the cracks along horizontal lift joints. There was also some relative movement between buttresses.

Earthfill Dams

The first point about earthfill dams is that one must be very concerned about the risk of liquefaction in the dam or foundations.

It is often said that "no well-built embankment dam has ever failed due to seismic action".

Of course it is all a matter of definition. What is the definition of 'failure' and what is the definition of 'well built' ?

In fact a lot of embankment dams have failed as a result of earthquakes.

145 dams failed in Japan in the Nihon-kai-Chubu earthquake in 1983 where the definition of failure was:

- Sliding of slope
- Longitudinal crack more than 50 mm wide
- Transverse crack
- Crest settlement more than 300 mm
- Leakage of water

An unknown number of these "failures" would have involved a catastrophic release of water and many would probably have required reconstruction of the dam. In this it is worth noting that irrigation reservoirs may only be full for a short time at the start of the irrigation season each year and that "failure" will often not lead to a catastrophic release of water.

Until 11 March, 2011 no people are thought to have died from the failure or damage of a large water (rather than tailings) storage dam due to earthquake. However, during the magnitude 9.0 Tohoku earthquake in Japan in 2011 an 18.5 m high embankment dam failed and the flood wave created by the release of the reservoir caused the loss of eight lives

330 earthfill dams were damaged in China by the Tangshan earthquake in 1976.

There are other dams that could be mentioned including the 245 damaged in the Bhuj earthquake in India on 26 January, 2001. Damage at some of the dams was quite serious although the earthquake fortunately took place when water levels were low.

Figure 7. Damage in Bhuj earthquake of 26 January, 2001 (Courtesy Prof S.K.Jain)

Many of the dams damaged, in China, India and Japan, were of only modest height.

The reference to well-built dams not failing seems to go back to H. Bolton Seed's 1979 Rankine lecture[16] but it is worth quoting his words in full. What he said was:

"*Virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking, say with a peak acceleration of about 0.2 g, with no detrimental effects".*

Many of the dams mentioned above probably suffered accelerations well in excess of 0.2 g but the reference to well-built dams needs to be noted. The time to decide whether a dam was well-built may be after the earthquake rather than before it.

Well-designed dams with wide filters are generally considered good for earthquakes. Unfortunately the filters tend to be expensive and there is not a lot of published advice on their desirable thickness.

It is worth mentioning that in the 1906 San Francisco earthquake, which had a magnitude of 8.25, there were 33 earth dams within 56 km of the fault and 15 within 8 km. It seems likely that all these dams were subjected to ground motions having peak ground accelerations greater than 0.25 g and that those within 8 km probably experienced accelerations greater than about 0.6 g. Yet none of these old dams suffered any significant damage.

In his 1979 Rankine lecture H.B. Seed pointed out that the slopes were typically 1:2 to 1:3 and that the dams had generally been compacted by moving livestock or by teams and wagons. He added that they were all constructed of clayey soils on rock or clayey soil foundations. Two dams were built largely of sand but this was apparently not saturated.

The Sharredushk dam in Albania failed after a modest earthquake on 18 March 2009 (M=4.1) when freeboard was reduced from 1.5 or 2.0 metres to only 0.1 m. The contents of the reservoir were not, however, lost.

Figure 8. Sharredushk Dam after earthquake of 18 March 2009 (courtesy Tim Hill).

Finally the Earlsburn Dam in Scotland failed on the evening of 23 October 1839 some 8 hours after an earthquake thought to have had a magnitude of 4.8. The dam was an embankment of peat and earth with a narrow central core of silty clay. The core extended down to rock but most of the dam, which was 6 m high, was only founded on peat.

In the section on seismic seiches below there is discussion of the repeated overtopping of the Hebgen dam in Montana to a depth of about one metre. This earthfill dam has a concrete corewall as does the Sorpe dam in Germany which withstood heavy bombing in WWII. Statistical evidence suggests that earthfill dams with concrete corewalls are particularly robust (Foster, Fell and Spannagle, 2000)[2]

Figure 9. Sorpe dam after heavy bombing in WWII.

Dams with upstream Asphaltic Concrete Membranes

The 53 m high Winscar Dam, in an area of low seismicity in UK, was built with an upstream asphaltic concrete membrane and was completed in 1975. By 2001 water seeping through a crack in the membrane was causing the loss of between 4,000 and 6,000 m^3 of water per day. The leakage was stopped in 2002 by installing a PVC membrane on the upstream face.

Minamikawa Saddle Dam is a 19.5 m high asphalt faced rockfill dam that was complete in 1987 and damaged in the M_W = 9.0 Tohoku earthquake of 11 March, 2011. The earthquake caused a temporary increase in leakage from 9 l/min to 87 l/min, a crack in the asphalt face, and a maximum crest settlement of 100 mm. A peak horizontal acceleration of 0.27g was measured on the foundation of the main dam, 1 km away. (USSD, 2014)[11].

These cases illustrate the value of PVC membranes for remedial works but do not inspire confidence in the use of upstream asphaltic membranes particularly in remote areas where high standards of workmanship cannot be guaranteed. Where clay is not available for the core of a dam a vertical, or near vertical, asphaltic core may be a better solution.

Dams with Asphaltic Concrete Cores

Dams with vertical asphalt concrete cores and filters (or with cores slightly inclined upstream) are a common form of construction, particularly where clay is not available (ICOLD, 2018)[17]. Since 1978 nearly all Norwegian dams have been built in this way (Saxegaard H. 2000)[18] and many dams of this type have been built in Austria, China, Canada, Brazil, Germany, Japan and the USA. The method can be employed in wet weather and the corewall adjusts to the deformations in the embankment and in the dam foundation.

It is not difficult to achieve air porosity less than 3 % which makes the asphalt concrete virtually impervious . It is required to drill samples out of the core during construction to document that the porosity is below that specified limit , but hardly ever has the core construction slowed down the raising of the embankment. The filling and compaction of the shells takes longer than building the core. The highest Asphalt Core Dam is now about 175 m, recently completed in China.

The corewall is protected from weathering and from impact or sabotage and is said to be specially suited on compressible foundations where CFRD and RCC may not be suitable. Having noted the above it is observed that, when once built, the core will not be accessible for repairs so a high initial standard of workmanship is needed.

An undocumented rule-of-thumb has evolved which calls for thickness at any level of at least 1 % of the head difference between the upstream and downstream sides of the core at that level. However Norwegian experience suggests that this is unduly conservative and that a minimum core thickness of 0.5 m, and no more than 1.0 m, may be appropriate unless there are very special circumstances, for instance in extreme earthquake regions or for embankments on compressible, erratic foundations (Høeg, 1993)[19].

During an extreme earthquake, it has been suggested that the induced permanent shear displacements for an embankment dam may become so large that a narrow core is sheared off and a gap opens. For such an eventuality it would be advisable to have a relatively fine-grained material next to the asphaltic concrete core. It is essential that the downstream shell and toe is designed with adequate drainage capacity to handle accidental leakage and prevent dam failure even if the temporary water loss is dramatic. (quoted by Høeg, 1993)[19].

Figure 10. Simultaneous compaction of asphaltic core and filters (Høeg, 1993)

Dams with upstream geomembranes

PVC geomembranes have been used to provide upstream water barriers in new dams. The deformability and tensile resistance of such materials allows them to withstand high stresses without failure. A significant example is the 97 m high Olivenhain RCC Dam in California, part of the Emergency Storage Project protecting the San Diego area against a disruption in water deliveries.

The Olivenhain Dam is less than 50 km from the Elsinore, Rose Canyon and Coronado faults, and a little more than 100 km from the San Andreas fault. The Olivenhain Dam design earthquake was a magnitude 7.25 event on the Rose Canyon fault at a minimum distance of 17.8 km. A dynamic analysis was made for the highest section of the dam under a postulated Maximum Credible Earthquake (MCE) loading with maximum peak ground acceleration of 0.386g. Since the primary purpose of Olivenhain Dam is to provide a reliable water supply following a severe earthquake, the selection criteria for the upstream facing system placed emphasis (higher weighting factor) on seismic stability and seepage control.

The dam was required to remain watertight even in case of a seismic event. An external geomembrane liner against formed RCC, and a reinforced conventional concrete face cast-in-place after RCC placement, received the highest score among the 16 possible alternatives.The geomembrane liner alternative was selected due to a less adverse impact on overall construction schedule and technical merit considerations (Kline et al., 2002)[20]. The exposed upstream geomembrane system was installed in 2003.

Figure 11. Exposed PVC geomembrane at Olivenhain Dam (courtesy Carpi Tech)

With the reservoir about 50% full, a magnitude 5.2 earthquake hit the San Diego region in June 2004, centered about 60 km from the dam. Significant shaking was reported, and a peak horizontal acceleration of 0.18g recorded. No damage to the dam was observed and seepage did not increase following the event. The reservoir filling was completed in January 2005. The geomembrane system continues to exceed performance specification for seepage.

Other examples of upstream PVC membranes adopted in seismic areas to provide a water barrier are related to new rockfill dams, where the PVC membrane can be either exposed and installed on extruded porous concrete curbs, like at Sar Cheshmeh tailings dam raising in southern Iran, or covered by an upstream granular cover, like in the lower 65 metres of an 80 m high cofferdam in Tajikistan. A zigzag membrane core has been used in a 50 m high cofferdam in Ethiopia.

Dams built with Hydraulic Fill

The upstream slope of the Lower San Fernando Dam in California failed due to liquefaction during the February 1971 San Fernando earthquake which had a magnitude of 6.6. The collapse occurred shortly after earthquake shaking ended. Fortunately the water level in the reservoir was low at the time as the crest settled 8.5 metres. More than 80,000 people living downstream had to be evacuated for 4 days.

The Krasnodar Dam on the Kuban River in southern Russia is 11.5 km long and impounds a reservoir with a capacity of 2.9 km^3. Seismicity at the site was originally thought to be quite low but the dam is not far from Georgia where seismicity is much higher. In view of this, a site investigation was carried out which shows that a moderate risk of liquefaction exists in the core of the hydraulic filled part and a high potential for liquefaction in the foundation of the dam. In case of an earthquake with a peak ground acceleration of 0.3 g, the dam would experience strong disorders due to settlements, but probably not a complete failure. Rehabilitation of the drainage system has been advocated (Droz and Acs, 2003)[21].

Tailings Dams

Tailings dams may be very dangerous as was shown when the Brumadinho Tailings dam in Brazil collapsed on 25 January 2019 with the loss of about 300 lives. Whilst this particular failure has not been linked to seismic activity tailings dams do not have a good record of behaviour in earthquakes (Hinks and Gosschalk, 1993)[22]. This is, in no small part, due to failures in Japan and also in Chile where 4 dams failed between 1928 and 1965 (see Table 1).

Table 1. Failure of Tailings Dams in Chile due to Earthquakes

Dam	Height m	Date	Magnitude M	Damage
Barahona Tailings dam	63	1928	8.0	Catastrophic failure: 54 killed
El Cobre Tailings dams	32 – 35	1965	7.4 to 7.6	2 dams failed. 350 to 400 killed
Cerro Negro Tailings dam		1965	7.0 – 7.25	1 dam failed

In the case of failures due to earthquakes, where the loading is rapid and unexpected, the initial design of the tailings storage facility is the most important management consideration. The design needs to be fit for purpose – for example, tailings storage-facility studies indicate that the upstream method of dam construction is more susceptible to instability from seismic loading compared to the downstream method (ICOLD, 2019)[23].

LIQUEFACTION

As noted in the section on Hydraulic Fill the upstream slope of the 38 m high Lower San Fernando Dam in California failed due to liquefaction during the 1971 San Fernando earthquake. The collapse occurred shortly after earthquake shaking ended and was attributed to loss of strength in the hydraulic fill due to liquefaction.

Figure 12. Lower San Fernando dam after earthquake of 9 February, 1971. (Source: Earthquake Engineering Research Center, University of California, Berkeley)

Liquefaction can occur in saturated, low density soils with coarse grained sands (say around 0.07 to 0.6 mm) being particularly vulnerable (although gradings between 0.02 and 2.0 mm, and even gravelly soils, have been mentioned. Green and Bommer (2019)[24] note that liquefaction has been observed in earthquakes with magitudes as low as about 4.5 but conclude that " *M = 5.0 is the lower bound for liquefaction triggering*" for risk to sites suitable for building structures. However, for other infrastructure (eg for pipelines and levees) magnitudes as low as 4.5 may need to be considered.

The most common measures of liquefaction potential have been the Standard Penetration Test (SPT) N value and the Cone Penetration Test (CPT) tip resistance. It has been found that the relationship between CPT and SPT data is a function of mean grain size (Idriss and Boulanger, 2008)[25].

DAMS ON ACTIVE FAULTS

It was demonstrated by the vertical movement of about 9 m between bays 16 and 18 at the 21.4 m high Shih-Kang gravity dam in Taiwan in the M_W = 7.7 Chi Chi earthquake of 21 September 1999 that significant movements on an active fault beneath a dam can destroy the structure (Hansen and Nuss, 2011)[3].

For this reason engineers have tended to discard entirely the idea of building dams on sites with known active faults within the dam footprint.

Figure 13. Shih-Kang dam after the earthquake of 21 September 1999.

During construction of the 100 m high Clyde dam in New Zealand in 1982 to 1993 a potentially active fault was found running underneath the dam footprint. This led to redesign of the dam to incorporate a slip joint deemed capable of accommodating 1 to 2 metres of potential ground movement.

A particularly interesting case, with which the author has been involved, is the design of the 153 m high Rudbar-Lorestan dam in western Iran. This was originally conceived as an RCC dam. It was about 1.6 km from the Saravand-Baznavid fault on which there had, further north, been an earthquake of M_w = 7.4. The chosen PGA for the SEE was 0.66 g.

It was felt possible to design the dam for shaking as a result of an earthquake on the main fault but there were concerns about possible movements along secondary faults, and other discontinuities, in the footprint of the dam. One of the secondary faults, at the left abutment, is shown in Figure 14. The investigation of this fault showed that it had experienced maximum horizontal displacement of up to 1.4 m in a single event. Based on additional seismotectonic studies and interpretation it was decided that a clay core rockfill dam with wide filter zones was to be preferred.

Figure 14. Secondary fault at left abutment of Rudbar-Lorestan dam in Iran

Figure 15. Rudbar-Lorestan Dam under construction.(courtesy Martin Wieland)

APPURTENANT STRUCTURES AND EQUIPMENT

Care needs to be taken in the seismic design of safety-critical components such as spillways and intake towers. The latter, in particular, may be tall with much weight at the top and the likelihood of interaction between the structure and water in the reservoir. Experience suggests that heavy reinforcement may be needed at the base of the tower to withstand high local stresses.

Ground shaking affects all civil structures (above and below ground) and safety-critical hydro-mechanical and electro-mechanical components of a large storage dam at the same time. The need for aseismic design will be particularly important for components which may be subjected to amplified accelerations on the crest of the dam. Hansen and Nuss (2011)[3] note that crest accelerations at Pacoima Arch dam (USA) and Kasho Gravity dam (Japan) exceeded 2.0 g .

OTHER HAZARDS

Following the Wenchan earthquake construction equipment could not be transported to several dam sites for somel months because access roads were blocked by rockfalls. Therefore, it has to be assumed that a damaged dam has to remain safe for several months after an earthquake before it can be rehabilitated or transformed into a safe state.

LANDSLIDES

Potential landslides into reservoirs or into river valleys, so that they form temporary dams, can be extremely dangerous whether they are triggered by earthquakes or by some other cause.

A total of 30 large landslide dams were created by the Wenchuan earthquake of 12 May 2008 ($M_W = 7.9$) in China. Of these the largest was Tangjiashan which was 124 m high and had a crest width of more than 300 metres. Its volume was more than 20 Mm^3.

The landslide impounded a 320 Mm^3 reservoir 6km upstream of Beichuan city, which was heavily damaged and, today, serves as a large open air earthquake museum. Beichuan city was rebuilt about 25 km away from the old, destroyed, city.

There was no road access after the earthquake so initial information came from satellite photography. Access by helicopter was first obtained eight days after the earthquake. A channel had to be constructed to release the water which was flowing into the reservoir at a maximum rate of 170m^3/s. As this lake threatened more than a million people downstream of the landslide dam, it was decided to bring in equipment using a large Russian helicopter with a lifting capacity of 13 tonnes. In this way 24 excavators were lifted in, 13 bulldozers and 8 tipper trucks. The necessary work was done in 10 days.

Good geological records were used to predict that the relief channel would only scour down to a certain level. In fact flows reached 6,500 m^3/s which was slightly more than the flood with a return period of 200 years.

*Figure 16**.** Tangjiashan landslide dam (Photo by Mr. Li Gang)*

Perhaps the best known case is the 270 Mm^3 landslide that entered the reservoir behind the 262 m high Vajont arch dam in Italy on 9 October 1963. A wave with a volume of at least 50 Mm^3 overtopped the dam and, although the dam did not fail, caused the deaths of almost 2,000 people downstream. There were small earthquakes associated with the slide but the main cause was thought to be geotechnical.

Another huge landslide, which was triggered by an earthquake of magnitude 7.0 in February 1911, created a natural barrier known as the Usoi dam. The dam is about 567 m high and is on the Murghab River in a remote region in Central Tajikistan. Due to its remote location it was some time before the local authorities became aware of the dam, which now impounds Lake Sarez with a volume of more than 16 km^3. The volume of the dam itself has been estimated as 2 km^3 (Droz P and Spasic-Gril L , 2006)[26].

The area is highly seismic and there are fears that a future earthquake could breach the dam endangering more than five million people downstream. Other perceived threats are that water seeping through the dam might cause internal erosion or that a partially detached rock mass of up to 3 km^3 might fall into the lake and cause failure of the dam.

The Tapovan dam in Uttarakhand, India was washed away on 7 February, 2021 with the loss of about 200 lives. The landslide upstream that caused this disaster is not thought to have been triggered by an earthquake but nevertheless illustrates again the potential risks posed by landslides and glacial lake outbursts.

ROCKFALLS

A major feature of the magnitude 7.9 Wenchuan earthquake of 12 May, 2008 was the rockfalls which obliterated many kilometres of main roads, damaged power plants and destroyed transmission lines. The rockfalls would have made access to many dams impossible for long periods of time.

Figure 17. Typical rockfall during Wenchuan earthquake in China on 12 May, 2008.

As noted rockfalls seriously damaged hydropower stations in the Wenchuan earthquake. This was particularly the case at the Shapai hydropower scheme where the dam itself performed well.

SEISMIC SEICHES

Long-period reservoir oscillations (seismic seiches) are not thought to have caused overtopping at many dams partly because, where they have occurred, the reservoirs have rarely been full. This was the case at the Yuvacik Reservoir in western Turkey in the earthquake of 17 August 1999. (M_W 7.4 and epicentral distance 10 km). It has been calculated that, if the reservoir had been full at the time of the earthquake, the dam would, because of the camber, have been overtopped to a depth of 1.68 m at the abutments and 0.25 m at the centre-line (Halcrow Group Ltd, 2006)[27].

At the 35 m high Hebgen earthfill dam, with central concrete core, in Montana, USA there was a notable seismic seiche on 17 August 1959 in an earthquake of Magnitude 7.5. One of the main faults passed within 215 m of the dam. It was observed that water flowed over the crest to a depth of about 1.0 m for about 10 minutes before receeding and traveling to the other end of the reservoir.

There can be no doubt that water flowed over the dam at least four times (Sherard, Woodward, Gizienski and Clevenger, 1963)[28] which may be partly explained by the fact that the dam itself suffered 1.2 m settlement in the earthquake. A blog by the Berkeley Seismology Laboratory on the 60th anniversary of the event states that in the days following the earthquake the dam was "on the verge of collapse".

It is worth quoting the simple empirical formula given in the Russian SNIP 11-7-81 (Moscow 1991)[29]. It would give an amplitude of 3.4 m for I = 10, which is possibly too much. Impulse waves due to mass movements will usually be more critical.

$$\Delta h = 0.4 + 0.76 (I – 6)$$

Where I is the earthquake intensity on the Medvedev, Sponheuer, Karnik scale and Δh the amplitude in m.

The maximum water waves in reservoirs recorded during the March 11, 2011 Tohoku earthquake in Japan (magnitude 9.0) was less than half a metre.

During the Great Assam earthquake ($M_W = 8.6$) of August 15, 1950, unusual waves were observed in at least 37 localities in fjords and lakes in Norway. In most places the waves were standing waves, with periods of 1 to 3 minutes and amplitudes of 50 to 1,000 mm. and began when the acceleration at the seismological observatory in Bergen surpassed 0.02 g in the east-west direction and 0.04 g in the vertical direction.

There were also small seiches, with maximum amplitudes of about 50 mm in reservoirs at Margate, Chichester and Portsmouth in the UK as a result of the same earthquake.

CONCLUSIONS

Dams are generally robust structures designed to have substantial factors of safety under normal operating conditions. It is, therefore, not surprising that most have behaved well under seismic loading. However a number have suffered significant damage and a few have failed completely. Those most at risk are thought to be:

- Dams where liquefaction of the dam or foundation is possible.
- Tailings dams
- Dams built on active faults
- Small homogeneous dams (mostly in India, Japan and China).

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REFERENCES

1. ICOLD Bulletin 188 (Preprint), "Dam Failures – Statistical Analysis", Paris, 2019

2. Foster M, Fell R. and Spannagle M, "The Statistics of Embankment Dam Failures and Accidents", Canadian Geotechnical Journal, 37 pp 1000 to 1024, 2000.

3. Hansen K.D. and Nuss L.K., "Lessons Learned from the Earthquake Performance of Concrete Dams", Water Power and Dam Construction, September 2011

4. Pagani M. et al, GEM Global Seismic Hazard Map. December 2018. (https://maps.openquake.org/map/global-seismic-hazard-map/#5/-15.221/33.115)

5. Lubkowski Z.A and Aluisi B, "Deriving S_S and S_1 Parameters from PGA Maps", Lisboa 15 WCEE, 2012.

6. ICOLD Bulletin 137 "Reservoirs and Seismicity", Paris, 2011

7. ICOLD Bulletin 148, "Selecting Seismic Parameters for Large Dams", Paris, 2016.

8. Charles J.A, Abbiss C.P, Gosschalk E.M and Hinks J.L " An Engineering Guide to Seismic Risk to Dams in the United Kingdom", Building Research Establishment, 1991

9. Douglas K. , Spannagle M. and Fell R. "Estimating the Proability of Failure of Concrete and Masonry Gravity Dams", University of New South Wales, 1998.

10. ICOLD Bulletin 183 (Preprint). "Selection of Dam Type", Paris, 2019

11. United States Society on Dams, "Observed Performance of Dams During Earthquakes", Volume III, February 2014.

12. Løkke A. and Chopra A.K. "Direct-Finite-Element Method for Nonlinear Earthquake Analysis of Concrete Dams Including Dam-Water-Foundation Rock Interaction", PEER Report No 2019/02, University of California, Berkeley, March 2019 .

13. Makdisi F.I and Seed H.B, "Simplified Procedure for Estimating dam and embankment earthquake induced deformations", ASCE Journal of Geotechnical Engineering Division , 1978. 104 (GT7), pp 849 – 867.

14. Wieland, M. and Chen, H., "Lessons learnt from the Wenchuan earthquake", Int. Journal Water Power and Dam Construction, September, 2009.

15. Erlendsson G , Cameron-Ellis D, Shaw Q.H. W and Dunstan M.R.H, "The development of the design of Changuinola 1 dam", Sixth International Symposium on Roller-compacted concrete dams, Zaragoza, Spain, October 2012.

16. Seed, H.B. (1979), " Considerations in the earthquake-resistant design of earth and rockfill dams", Geotechnique 29, No 3 pp 215 – 263.

17. ICOLD Bulletin 179 (Preprint), " Asphalt Concrete Cores for Embankment Dams", Paris, 2018

18. Saxegaard H. “Asphalt Concrete Cores for Embankment Dams”, International Water Power and Dam Construction, February 2000

19. Høeg K. “Asphalt Concrete Cores for Embankment Dams”, Norwegian Geotechnical Institute, 1993

20. Kline, R. A., Holderbaum, R.E., Tarbox, G. S., and Hartman, R. J. “Design of Roller-Compacted Concrete Features for the Olivenhain Dam”, Proceedings of 22nd Annual USSD Congerence, USA 2002

21. Droz P and Acs A.M. “Cost Benefit Analysis of the Rehabilitation of Krasnodar Dam in Russia”, Proceedings of ICOLD Conference in Montreal, Q81 – R5, 2003

22. Hinks J.L, and Gosschalk E.M “Dams and Earthquakes – A Review” Dam Engineering Vol IV Issue 1, Reed Business Publishing, 1993.

23. ICOLD Bulletin 181 (Preprint) “Tailings Dam Design – Technology Update”, Paris, 2019

24. Green R.A and Bommer J.J “What is the Smallest Earthquake Magnitude that needs to be Considered in Assessing Liquefaction Hazard ?”, Earthquake Spectra, Vol 35, No 3, pp 1441 – 1464, August 2019.

25. Idriss I.M. and Boulanger R.W. 2008 “Soil Liquefaction during Earthquakes”, Earthquake Engineering Research Institute, 2008.

26. Droz P and Spasic-Gril L. “Lake Sarez Risk Mitigation Project: A Global Risk Analysis”, Q86-R75 ICOLD Conference, Barcelona, 2006.

27. Halcrow Group Ltd. “Yuvacik Dam – Review Panel Report”, 2010

28. Sherard J.L., Woodward R.J., Gizienski S.J. and Clevenger W.R, “ Earth and Earth-Rock Dams, John Wiley and Sons, New York, 1963.

29. SNIP 11-7-81 “Hydro-Engineering Structures in Seismic Areas”, Moscow, 1991

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