Behaviour of embankment dams in earthquakes

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SYNOPSIS Severe earthquakes have occurred in recent years in various countries including China, Indonesia, Haiti, New Zealand, Chile, Pakistan and Japan. These have led to catastrophic events including tsunamis, severe damage and heavy loss of life.

Dams have generally behaved well in earthquakes and, although a number have suffered damage, the consequences have been relatively modest. This paper aims to look at the historical behaviour of embankment dams in earthquakes and its relevance to dam engineering in UK where future earthquakes are not expected to exceed Magnitude 6.0. Nevertheless, if occurring at shallow depth, such an earthquake could be quite damaging and could give rise to peak ground accelerations in excess of 0.3g.

The paper considers the 1991 BRE Guide to seismic risk to dams in the United Kingdom in the light of ICOLD Bulletin 148 (Selecting Seismic Parameters for Large Dams) which was approved at the Hanoi meeting in 2010.

INTRODUCTION

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 which have suffered the worst damage are:

- Dams where liquefaction of the dam or foundation has been a factor
- Tailings dams
- Dams built on active faults
- Small homogeneous dams (mostly in India and China).

This paper concentrates on the behaviour of earth embankment dams and, in particular, small homogeneous dams because these are more common in the UK than rockfill or concrete dams. The scope of the paper is, however, intended to cover the behaviour of earthfill dams around the world

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The Building Research Establishment Guide to Seismic Risk to Dams in the United Kingdom was published in 1991 and followed seven years later by the DETR Application Note. Since then all large dams in the UK have been subjected to an assessment of their ability to withstand seismic shaking. Only two dams (Argal in Cornwall and Upper Glendevon in Perth and Kinross) have had to be strengthened and these would probably have needed strengthening anyway for static loading.

EARTHQUAKES IN THE UK

The UK is located in the northwest portion of the Eurasia plate, far from any plate boundaries. Located within an intraplate area, the seismicity of the UK is characteristically low. A number of attempts have been made to identify seismotectonic regions within the UK, and include: Palaeozoic orogogenic episodes and basement provinces; zones of Paleogene-Neogene deformations resulting from sub-plate deformations; glacial rebound; major fault systems; mantle processes; and present-day crustal stresses related to fault displacements. Although the distribution of seismicity across the UK is not random, none of the above theories fully account for the distribution of seismicity recorded in the UK.

The largest recorded UK events have been offshore, with onshore events less than $5M_w$ (M_w=Moment Magnitude). In the UK, the maximum possible earthquake magnitude is difficult to define due to the available records and the fact that the length of the seismic cycle means that historical records may not have captured the largest possible event. In modern seismic hazard analysis approaches, a logic tree approach is therefore used, defining the maximum magnitude for a UK earthquake as: M_w=5.5 (20%), M_w=6.0 (50%), M_w=6.5 (30%).

A zoning map for the UK was produced for the BRE Guide to Seismic Risk to Dams in the UK (Charles *et al*. 1991). This map assesses hazard in a subjective way into high, medium and low classes, which are to be understood as entirely relative terms. Despite its informal nature, it still proves to be a reasonable depiction of relative hazard levels when compared to later quantitative maps.

The DETR Application Note to the BRE guide was published in 1998 (ICE 1998). This introduced two main changes. Firstly the zone map was replaced by a contour map giving Peak Ground Accelerations (PGAs) for 10,000 year return period events as a result of a nationwide study of seismicity (Musson and Winter, 1996). This generally provided higher PGAs for the 10,000 year return period (e.g. 0.32g in Zone A compared to the earlier recommendation of 0.25g). However, the second change was to reduce the return period for category IV dams to 10,000 years or Maximum Credible Earthquake (MCE). Previously the BRE Guide recommended a PGA of 0.375g based on a 30,000 year return period.

ICOLD suggests that two design earthquake scenarios should be provided for in the design of dams and appurtenant structures:

- The Safety Evaluation Earthquake (SEE), for which some damage can be accepted, but for which there should be no uncontrolled release of water from the reservoir
- The Operating Basis Earthquake (OBE), for which there should be no significant damage to the dam.

ICOLD suggests that for dams posing a great social hazard the SEE should be characterised by the MCE or by an event with a return period of about 10,000 years.

ICOLD also suggests that an appropriate return period for an OBE is 145 years and that the dam and appurtenant structures should remain functional with the damage easily repairable. The consequences of exceeding the OBE are normally economic and ICOLD suggests that the circumstances of particular cases may justify use of a more severe event for the OBE.

Work undertaken by British Geological Survey, (BGS) (Musson and Sargeant, 2007) for the purposes of seismic zoning of the UK within the context of Eurocode 8 derived new seismic hazard maps, which take advantage of recent advances in the modelling of strong ground motions. The source model used in the analysis attempted to express both the tectonics and seismicity of the UK. The model was based on crustal divisions, especially with respect to the overall kinematic processes that are expected to play a controlling part in determining the distribution of seismicity.

The values provided by the BGS study are intended to give a general indication of the expected hazard level for return periods of 475 years and 2,500 years and depict horizontal peak ground accelerations for sites located on rock foundations.

The UK National Foreword to Eurocode 8, defines the UK as an area of very low seismicity and therefore there is no statutory requirement from BS EN 1998-1 (EC 8 as it applies to the UK) to consider seismic loading. However the UK National Foreword advises that there may be circumstances in which an explicit seismic design is warranted (Booth et al, 2008). Similarly, there are no statutory requirements in the UK to consider seismic loading for dams. However, it is current general practice for Panel Engineers to require a seismic assessment following the guidelines of Charles *et al* (1991).

The PGAs shown on the BGS hazard map are generally quite low, compared to previous hazard maps. The PGA reported are the geometric mean of the two horizontal components, whereas, previously the larger of the two horizontal components was generally reported. This accounts for a reduction in acceleration by a factor of 1.15.

The recent BGS hazard maps are not recommended for use in the seismic assessment of dams, except perhaps for preliminary assessments. Firstly, the maps are for return periods that do not necessarily correspond with the return periods in the BRE Guide or those identified by ICOLD. Perhaps more importantly, the maps do not allow for the influence of local faults. In addition, most contoured maps depicting PGA are for structures founded on rock; an increase in this value is generally required for structures on soil foundations and for topography. Other anomalies occur with respect to the contouring algorithm, which may give unreliable results when a site location lies close to a contour line.

ICOLD guidelines on seismic hazard analysis for large dams recognise the use of both deterministic and probabilistic approaches for the computation of earthquake design parameters. However, it is worth noting that the guidelines recognise a number of benefits in using a probabilistic approach over a deterministic approach, including:

- Provision of design parameters with a specified probability of occurrence (as discussed above)
- The explicit use of all earthquake data within the catalogue in the computation of seismic hazard, rather than just a single earthquake at a specified distance from the site, as is the case for a deterministic analysis.

ICOLD also suggest that for extreme and high category risk category dams that the PGA should be calculated as the mean PGA plus one standard deviation (*i.e.* the 84th percentile), whereas the contoured maps provide only the mean values.

The current best practice for structures which warrant a seismic analysis is to undertake site-specific studies. This is particularly preferable where the dam is located near an active fault zone.

As discussed in the introduction, fault ruptures have been known to cause damage to dams. However, in the UK, there have been no known surface ruptures caused by earthquakes. After the 1901 Inverness earthquake, a crack was observed in the towpath of the Caledonian Canal at Dochgarroch; but one can be fairly certain that this was a ground movement effect and not a fault displacement. It is expected that any surface fault displacements from British earthquakes will be zero. The larger events for example, in the Great Glen area, have had focal depths typically around 10km; the maximum rupture dimension for a $5M_L$ (M_L=Local Magnitude) earthquake is probably <1km, so the chances of the rupture plane intersecting the surface are minimal.

Although, within geotechnics, the term 'liquefied' was probably first used to describe the 1918 failure of the Calaveras Dam in California by Hazen (1920), in the UK, there has only been one observation of what has been generally accepted to be a result of liquefaction. Therefore, in the UK, studies of liquefaction potential for return periods of less than 475 years are typically not pursued (Booth *et al*, 2008). However, for hydraulically placed fills and for events of lower probability the possibility of liquefaction failures cannot be ruled out.

EMBANKMENT DAMS

The failure of the Earlsburn Dam near Stirling occurred about eight hours after an earthquake of Magnitude 4.8 on 23 October 1839. The failure has been documented by Musson of the British Geological Survey (Musson, 1991). The epicentre of the earthquake was at the village of Comrie about 32km from the dam site and the depth of the earthquake was about 18km. This implies a PGA of about 0.04g at the dam.

A comprehensive survey was carried out on 74 embankments severely damaged during the Magnitude 6.6 Oga earthquake in Japan in 1939. The heights of the embankments ranged from 1.5m to 18m and the intensity of shaking in the zone of severe damage was approximately 0.3g to 0.4g. Twelve of the dams failed completely and there were also 40 reported cases of slope failures. The conclusions of the survey are worth summarizing:

- The majority of damaged and failed embankments consisted of sandy soils
- No complete failures occurred in embankments constructed of clay soils
- There were very few cases of dam failures during the earthquake; most failed either a few hours or up to 24 hours after the earthquake.

The last point shows that events at Earlsburn were fairly typical in that failure took place some hours after the earthquake.

Damage to earthfill dams has sometimes occurred in association with quite low peak ground accelerations. For example the Paiho main dam in China suffered damage in the 1976 Tangshan earthquake even though the peak ground acceleration recorded at the toe was only 0.05g

Another dam that was damaged in an earthquake with a fairly low PGA at the site was the 24m high Sharredushk dam in Albania. This was damaged by a Magnitude 4.1 earthquake on 18 March 2009. The reservoir was full at the time of the earthquake and the epicentre was only 9.5km from the dam site. Freeboard was reduced from 1.5m or 2.0m to only 0.1m but the contents of the reservoir were not lost. The focal depth was 10km and the peak ground acceleration at the site was estimated as 0.07g.

Figure 1. Sharredushk Dam in Albania after earthquake of 18 March 2009.

A total of 245 dams were damaged in the Bhuj earthquake in India on 26 January 2001. The earthquake had a surface wave Magnitude M_S of 7.9 (M_S=Surface Wave Magnitude). Major damage was ascribed almost entirely to liquefaction and occurred for PGA exceeding 0.2g with SPT results of N< 20.

Figure 2. Cracks in earthfill dam following Bhuj earthquake of 26 January 2001

The 1906 San Francisco earthquake had a Magnitude of 8.25 when there were 33 earth dams within 56km of the fault and 15 within 8km. It seems likely that all these dams were subjected to ground motions having peak ground accelerations greater than 0.25g and that those within 8km probably experienced accelerations greater than about 0.6g. Yet none of these old dams suffered any significant damage. In his 1979 Rankine Lecture, Seed pointed out that the slopes were fairly steep (typically 1 on 2 to 1 on 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 San Andreas Dam also withstood the 1906 earthquake with remarkably little damage. The dam is composed of two embankments separated by higher natural ground through which the fault passes. In the 1906 event, displacement of 1.8m to 2.4m took place along the fault in the vicinity of the dam. The dam withstood the shaking without major damage although there was longitudinal cracking along the embankments and transverse cracking at the abutments.

Upper Crystal Springs dam, under the same earthquake, suffered an offset movement of 2.4m in the embankment. When the earthquake occurred the water level was the same on both sides of the dam.

The Santa Barbara earthquake of 29 June 1925 caused the complete failure of the 8m high Sheffield dam in California. The water level was 4.5m below the crest at the time. The earthquake had a Magnitude of 6.3 and the epicentre was about 11km from the dam site. Both the dam and its foundation were predominantly silty sand. A 100m length of the dam moved bodily 30m downstream releasing 200,000m^3 of water. The maximum ground acceleration has been estimated at 0.15g. Liquefaction has been suggested as the cause of failure, although this explanation is not universally accepted. Had the reservoir been full at the time of the earthquake the situation would have been even more serious.

On 17 August 1959 the 35m high Hebgen dam in Montana was subjected to an earthquake quoted as having a Magnitude of 7.5 to 7.8. The embankment was built of rolled fill comprising gravelly clay of medium plasticity. It has a thick central concrete core. One of the main faults passed along the shore of the reservoir about 215m from the dam. Along the fault there was a maximum vertical displacement of the ground surface of 4.5m to 5.5m. The dam did not fail although it settled by up to 1.2m on either side of the core wall which was left standing proud of the dam. The core wall was itself somewhat cracked and the spillway was damaged. It is worth noting that the event caused a seiche in the reservoir which caused overtopping of the dam to a depth of one metre for 10 minutes. This happened at least four times as the seiche moved up and down the reservoir.

The San Fernando earthquake in California on 9 February 1971 had a Magnitude of 6.6. The relatively modern earth dams in the area performed well but the 40m high Lower San Fernando Dam, which was constructed of hydraulic fill, was severely damaged by the motion which was estimated to have a peak ground acceleration of 0.55g to 0.6g. Failure seems to have been initiated by liquefaction of the hydraulic sand fill in the lower section of the upstream shoulder.

The earthquake caused a major slide in the upstream shoulder of the dam taking out the crest and the upper 9m of the downstream slope. Failure seems to have been initiated by liquefaction of hydraulic sand fill in the lower section of the upstream shoulder.

It was fortunate that the water level in the reservoir was about 10m below the crest at the time of the earthquake. After sliding, only about 1.5m of badly cracked material remained above the water level. Some 80,000 people living downstream of the dam were evacuated until the water level was reduced.

The 24 m high Upper San Fernando dam was also affected in the same earthquake. The crest settled by 900mm and moved downstream by 1.5m.

Figure 3. Lower San Fernando Dam after the 1971 earthquake.

This incident at the Lower San Fernando Dam illustrates the dangers of using poorly compacted sand fill in dams that may be subject to earthquakes. The most easily liquefiable sands have a particle size distribution between 0.07mm and 0.6mm.

The Krasnodar Dam in the south of Russia is 40m high, 11.5km long and is built of hydraulic fill. It was built to boost rice production in the Soviet Union. It impounds a reservoir with a surface area of 413km² and a capacity of 2,914Mm³. The city of Krasnodar with a population of 745,000 is just downstream.

When this dam was studied there was concern about the relative proximity of a highly seismic area in the Caucasus to the south. The epicentre of the Magnitude 6.9 Spitak earthquake on 7 December 1988 was a few hundred miles away. There was no doubt that the dam would be very vulnerable to an earthquake but could a sizeable earthquake happen near enough to Krasnodar to cause a problem? Nobody knew the answer to this question although the seismic zoning was increased at about the same time that the problem was being studied. Eventually the Swiss Government offered funding. It is understood that a Safety Evaluation Earthquake with a PGA of 0.3g was derived and it was concluded that drainage and other measures were needed at an estimated cost of $56m. As a dambreak was estimated to cost $3bn this was not disproportionate.

In the 1976 Tangshan earthquake in China, some 330 dams were damaged including the Paiho main dam. The earthquake had a Magnitude of 7.8. The 22m high Douhe dam was one of the others damaged in the same event. There was extensive longitudinal cracking, crest settlement and heaving of the toes of the embankment. The damage was attributed primarily to liquefaction of the saturated silts in the foundation.

Three tailings dams (not water storage dams) failed in Chile in 1965 during earthquakes of Magnitude 7.0 to 7.25. Over 200 people were killed. This followed another tailings dam failure in Chile in 1928 when 54 people were killed. The earthquake Magnitude in that case was 8.0.

During the March 1985 Valparaiso earthquake in Chile, which had a Magnitude of 7.8, embankment dams generally performed well and exhibited minor longitudinal and transversal cracking.. The exceptions are La Marquesa and La Palma dams which completely collapsed due to liquefaction in the foundations. The liquefaction occurred in loose sand layers near the base of the embankments. The dams were low (around 10m) and the horizontal movements were substantial.

The Wenchuan Earthquake of 12 May 2008 occurred in Sichuan Province, China. The earthquake had a Magnitude of 8.0 and focal depth of 15km. The maximum recorded peak ground accelerations were 0.98g (horizontal) and 0.97g (vertical). 2,666 dams were affected or damaged by the earthquake, although none collapsed entirely. Earthfill and rockfill dams generally performed well. However, after the Wenchuan event, the China General Design Institute for Hydropower Projects provided a supplementary guideline that dams higher than 200m shall be checked for the maximum credible earthquake (assumed to have a return period of 10,000 years).

During 2010 Maule earthquake in Chile, which had a Magnitude 8.5 and caused considerable damage due to ground shaking and tsunami, embankment dams only suffered minor cracking of the crest.

The effects of the Tohoku earthquake and tsunami in Japan on 11 March 2011 are well known. The cause was a Magnitude (M_W) 9.0 thrust earthquake at a depth of 32km about 70km east of the coast of Japan. The event caused heavy loss of life and a number of nuclear accidents at the Fukushima 1 Nuclear Power Plant. In all 252 dams were inspected after the earthquake. The 18.5m high Fujinuma irrigation dam in Sukagawa failed completely 20 to 25 minutes after the earthquake despite efforts by locals to repair leaks. At least eight people were killed.

Figure 4. Fujinuma Dam after Tohoku earthquake of 11 March 2011.

CONCLUSIONS

The dams to have failed in the largest numbers in earthquakes are probably small dams built of homogeneous materials. Many such dams have failed in China and India.

The worst damage to embankment dams has often been associated with liquefaction of embankment materials or of the foundations. Loose sandy materials with particle size lying between 0.07mm and 0.6mm are particularly susceptible to liquefaction. Apart from barrages (many of which were destroyed in the 1976 Tangshan earthquake in China) the foundations of concrete dams are not likely to be liable to liquefaction as they are likely to be founded on rock

Material placed as hydraulic fill appears to be particularly vulnerable to earthquake damage. Whereas clay dams on clay foundations generally perform well when subjected to earthquakes.

Relatively low ground accelerations (of the order of 0.05g) have been known to have serious effects on some earthfill dams.

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REFERENCES

Booth E.D. Skipp B. and Watt P. (2008). *Establishing the need for seismic design in the UK.* ICE Research Enabling Fund Report ICE-02

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

Hazen A. (1920). Hydraulic-fill dams. *Transactions of the American Society of Civil Engineers 83*: pp1717–1745.

Hinks J.L. and Gosschalk E.M. (1993). Dams and Earthquakes – a review. *Dam Engineering*, Vol IV, Issue 1, February, pp 9 – 26.

Hinks J.L., Lewin J. and Warren A. (2003). Extreme Events and Reservoir Safety. *Dams and Reservoirs* vol 13, no 1.

Hinks J.L. (2009). China Shares Experience and Lessons from the Wenchuan Earthquake. Part Two – Observations on the Behaviour of Dams. *Hydropower and Dams,* vol 16, 3, pp 107– 110.

International Commission on Large Dams, (approved 2010 and awaiting publication), Bulletin 148, *Selecting Seismic Parameters for Large Dams.*

Krinitzsky E.L. and Hynes M.E. (2002). The Bhuj, India, earthquake: lessons learned for earthquake safety of dams on alluvium. *Engineering Geology 66*, pp 163 – 196.

Musson R.M.W. (1991). The Earl's Burn Dam Burst of 1839: An Earthquake Triggered Dam Failure in the UK? *Dams and Reservoirs*, Vol 1, No 1, May.

Musson R.M.W. and Sargeant S.L. (2007). *Eurocode 8 seismic hazard zoning maps for the UK.* British Geological Survey, Technical Report CR/07/125.

Musson R.M.W. and Winter P.W. (1996). *Seismic Hazard of the UK.* Report for DTO by AEA Technology, Warrington.

Seed H.B. (1979). 19th Rankine Lecture: Considerations in the Earthquake Resistance Design of Earth and Rockfill Dams. *Geotechnique*, vol 29 no 3, pp215-263.

Wieland M. Bozovic A. and Brenner R.P. (2008). Dam design – the effects of active faults. *International Water Power and Dam Construction.*

Wieland M. and Chen H. (2009). Lessons learnt from the Wenchuan earthquake. *International Water Power and Dam Construction*, September, pp 36 – 40.

Zhang C. (2011) Performance of dams during the Wenchuan 5-12 Earthquake and lessons learned from the event. *Journal of Earthquake and Tsunami*, Vol. 5, No. 4 (2011) page 309–327,