United States Society on Dams

Observed Performance of Dams During Earthquakes

Volume III

February 2014

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USSD

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Prepared by the USSD Committee on Earthquakes

U.S. Society on Dams

Vision

To be the nation's leading organization of professionals dedicated to advancing the role of dams for the benefit of society.

Mission — USSD is dedicated to:

- Advancing the knowledge of dam engineering, construction, planning, operation, performance, rehabilitation, decommissioning, maintenance, security and safety;
- Fostering dam technology for socially, environmentally and financially sustainable water resources systems;
- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

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FOREWORD

In July, 1992, the U. S. Society on Dams (formerly the U.S. Committee on Large Dams) published a report title ***Observed Performance of Dams During Earthquakes.*** The report included general observations on the performance of embankment and concrete dams, Table 1 listing case histories and references on dams affected by earthquakes, and descriptions of observed performance for 11 selected dams. Volume II of the report was published in October 2000. It contained an updated general discussion, the table with additional dams listed, additional references, and 16 case histories of observed dam performance. This Volume III follows the same format, providing 12 more case histories of dam performance. Table 1 has been updated to include additional dams and to provide earthquake magnitudes currently listed by the USGS. The list of references has been expanded to include all dams in Table 1. Each case history contains additional references.

This report was prepared by a subcommittee of the Committee on Earthquakes, comprising: Donald Babbitt, Thomas Brown, Richard Kramer, Jack Montgomery, Michael Knarr, Larry Nuss and Andrew Sinnefield. Other members of the Committee and members of the ICOLD Committee of the Seismic Design of Dams provided reports on dam performance and review comments. Gilles Bureau coordinated the preparation of Volumes I and II. The USSD Publication Review Committee was Ross Boulanger, Chair; and Michael Forrest, David Kleiner and Eric Kollgaard. Their contributions are gratefully acknowledged.

USSD COMMITTEE ON EARTHQUAKES PUBLICATIONS

Bibliography on Performance of Dams During Earthquakes (1984)

Bibliography on Reservoir-Induced Seismicity (1986)

Compilation of U.S. Dams with Strong Motion Instruments and Reservoir Seismicity Networks (1985)

Dam Modifications to Improve Performance During Strong Earthquakes (2003)

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INTRODUCTION

In October 2000, the U.S. Society on Dams published Volume II of ***Observed Performance of Dams During Earthquakes***. Since then several earthquakes, including three events of magnitude 8 or greater, have affected an appreciable number of existing dams. The most significant of these recent earthquakes are the following:

- October 6, 2000 M_w 6.7 Western-Tottori Prefecture, Japan
- January 26, 2001 M_w 7.7 Bhuj, India
- October 23, 2004 M_w 6.6 Mid-Niigata, Japan
- March 25, 2007 M_J 6.9 Noto Hanto, Japan
- June 14, 2008 M_w 6.9 Iwate-Miyagi Nairku, Japan
- May 12, 2008 M_w 8 Wenchuan, China
- January 8, 2009 M_w 6.2 Costa Rica
- February7, 2010 M_w 8.8 Maule, Chile
- March 11, 2011 M_w 9 Tohoku, Japan

These events have provided significant additional information regarding the seismic performance of dams.

This report is a sequel to the 2000 publication and the first volume, which was published in 1992. It includes 12 case histories of dams that were exposed to moderate to strong earthquake shaking. The case history on Bajina Basta Dam does not discuss the performance of a dam during an earthquake, but rather a related phenomenon (neotectonic movement). The introduction of the 2000 publication has been essentially reproduced in the next paragraphs, expanded to include the performance of dams during the earthquakes listed in the previous paragraph and some earlier earthquakes.

Historically, few dams have been significantly damaged by earthquakes. On a worldwide basis, only about a dozen dams are known to have failed completely as the result of an earthquake. These dams were primarily tailings or hydraulic fill dams, or relatively old, small, earthfill embankments of perhaps inadequate design. About a dozen other embankment or concrete gravity dams of significant size have been severely damaged. Several of the embankment dams experienced near total failure, and were replaced. Yet, in the United States alone, more than 6,269 dams are higher than 50 feet; more than 1,666 exceed 100 feet; and more than 469 exceed 200 feet (U.S. Army Corps of Engineers, 2010 National Inventory of Dams). Hence, if one considers the total number of existing large dams, in the U.S. or on a worldwide basis, the current performance record appears outstanding, based on the limited number of failures.

Earlier volumes of this report have noted that this excellent record, however, may be largely related to the fact that few dams have been shaken by earthquakes of duration and intensity sufficient to jeopardize their structural integrity. They noted that except for several well-known examples, existing dams have not been tested by levels of ground motion equivalent to the applicable Design Basis Earthquake (see USCOLD, *Updated Guidelines for Selecting Seismic Parameters for Dam Projects,* 1999) and that a few dams have experienced significant damage under shaking substantially less demanding than what was, or should have been, considered in their design. Volume III contains case histories on Aratozawa, Convento Viejo, Ishibuchi and Shapai Dams that have withstood shaking that exceeded their design earthquake loadings and suffered no significant damage.

An updated inventory is presented of the principal dams that have experienced significant earthquake shaking in Table 1. It includes, where available, principal earthquake parameters, dimensions and types of dams, epicentral distances, and crude indicators of the severity of the damage incurred, if any has been reported.

The 27 case histories selected for detailed coverage in the 1992 and 2000 publications and the 12 presented in Volume III were chosen based on several factors, including: the importance of the dams involved; the severity of the ground motion to which they were subjected; the occurrence of, or the lack of observed damage; the availability of quality strong motion records near or on the dam; and the significance of these specific case histories to the dam engineering profession. The information provided is merely descriptive in nature. No attempt has been made to explain in detail why poor or satisfactory performance was observed.

The Committee intends to prepare additional reports as more information on performance of dams during earthquakes becomes available in the future.

PERFORMANCE OF EMBANKMENT DAMS

SINCE SEPTEMBER 2000

The Western-Tottori Prefecture, Japan Earthquake of M_w 6.7 occurred at 1:30 p.m. on October 6, 2000 at a focal depth of 11.3 km located at 35° 16.5' north latitude and 133° 21.0' east longitude. Fortunately no people lost their lives, but 132 people were injured, 28 homes were completely destroyed, 82 homes were half destroyed, and 5,050 homes were partially destroyed. Numerous aftershocks were recorded with one magnitude 5 and three greater than 4. After the earthquake, 180 dams were inspected. Eighteen small earth dams suffered minor damage. As discussed in Performance of Concrete Dams, three gravity dams performed very well.

On January 26, 2001 M_w 7.7 Bhuj, India Earthquake struck the Gujarat Province in the western part of India. The death toll from the earthquake was estimated at more than 20,000 with over three billion dollars in property damage. In this area of India it is common to find loose sand and silt deposits which may extend to great depths. Where saturated these soils appear to have liquefied during shaking leading to widespread damage such as sand boils, ground cracking and lateral spreading of ground and embankments. An important aspect of the Bhuj earthquake was the performance of small and moderate size embankment dams. This part of India is arid and receives the majority of its rainfall during the monsoon season (June to September). Out of necessity, many dams have been constructed in an effort to store water for irrigation and domestic water supply needs. More than 300 dams were inspected after the earthquake and the following damage classifications were developed to prioritize the repairs.

Damage Class	Description	Medium Dams	Minor Dams	Total Number
1	Significant damage. Dam is critical to region and repairs will be completed before monsoon.	4	7	11
2	Major damage. Not possible to repair before monsoon. Partial cut is made to allow water to pass, but allow for some storage.	2	0	2
3	Major damage. Damage is too extensive to store water. Full cut is made to pass flood waters.	1	3	4
4	Minor Damage. Repairs completed before monsoon.	80	165	245
Total		87	175	262

(After Patel and Brahmabhatt 2003)

As a result of the October 23, 2004 M_w 6.8 Mid-Niigata (Chuetsu), Japan Earthquake, 40 people were killed and numerous landslides of various types took place on natural slopes and man-made banks. (Ohmachi 2004) Strong motion accelerometers of K-net (the Japanese strong motion recording system) registered peak accelerations of 1.5 and 1.8g during the main shock. There were many strong aftershocks in the weeks after the main shock. Several embankment dams built for hydroelectric power generation or agricultural irrigation were damaged by the earthquake. (Yasuda et al., 2004) The irrigation reservoirs were empty at the time of the main shock, because it was the end of the growing season.

The power generation dams, Yamamoto, Shin-Yamamoto and Asagawara were located 6, 5, and 22 km from the epicenter respectively. They suffered moderate, but non-threatening damage. Eight irrigation dams were inspected and evaluated.

The epicenter of M_w 6.7 Noto Hanto, Japan Earthquake of March 25, 2007, was located offshore west of the Noto Peninsula in Ishikawa Prefecture. Immediately after the earthquake, emergency inspections were carried out on 108 dams. The inspection results showed that the earthquake did not adversely affect the safety of the dams (Iwashita et al., 2008). Oya Dam, a 186-foot high rockfill dam with a central core, completed in 1993, was located 48 km from the epicenter. Peak accelerations at the bottom of the inspection gallery were 0.08g in the stream direction, 0.16g in the dam axis direction and 0.08g in the vertical direction. At the crest, they were 0.32g in the stream direction, 0.48g in the dam axis direction and 0.21g in the vertical direction. The earthquake caused a maximum crest settlement of 1 inch, which brought the total settlement since construction of the dam to 3-1/2 inches. A temporary increase in seepage of 1.2 gallons/minute was recorded, but it may have been due to rainfall.

The June 14, 2008, Iwate-Miyagi-Nairiku Earthquake (M_w 6.9) struck the northern part of Japan. The earthquake produced many strong-motion records including one with peak vertical acceleration of 4g. Twenty-three people were killed or missing, 450 were injured, and about 2,000 houses were damaged. The earthquake occurred in a less-populated mountainous area, and the resulting building damage was rather small in comparison with observed strong-motion records with high accelerations. However, many landslides and slope failures caused extensive damage to roads and isolated people in the mountainous area. One hundred thirty-four dams were inspected and 12 were found to have been damaged, but not seriously enough to threaten uncontrolled reservoir releases. (Yamaguchi et al., 2008) Notably, 243-foot high Aratozawa Dam was shaken by a recorded peak ground acceleration of 1.03g and suffered only slight damage. A massive landside slid into the north arm of the reservoir caused a seiche that overtopped the spillway, but not the dam. Ishibuchi Dam, a concrete faced rockfill, was also severely shaken and suffered only minor damage.

Masazawa Dam in Iwate Prefecture is a 115-foot high embankment. It is 30 feet wide at the crest and 730 feet wide at the base. Based on vegetation growing low on the reservoir slopes, water appears to have been low at the time of the earthquake. The embankment experienced significant deformation of up to 5 feet of vertical of settlement on the upstream side of the crest. Steel pipe guard railings along the crest were bent and tilted outward (upstream and downstream). There were transverse cracks in the crest pavement. Settlement between the spillway and adjacent embankment was between 4 and 8 inches and irregular. (Kayen et al., 2008)

Koda Dam, located on the Nagasaki River in northwestern Miyagi Prefecture, is a 143-foot high zoned embankment with a clay core dam and earth and rockfill shells. Its construction was completed in 2005. The upstream slope of the dam is 2.8 to 1 and the downstream slope is 2.0 to 1. The clay core is 30 feet wide at the crest and broadens to 140 feet at the base. The dam was about 32 km from the epicenter. There was minor damage to the dam. The crest had a maximum settlement of 1-1/2 inches, 2 inches of downstream movement and some compression of curb stones along the crest, but no cracking of the pavement. Leakage doubled from 90 liters/minute before the earthquake to a maximum of 190 liters/minute after the earthquake and then slowly lessened. (Yamaguchi et al., 2008; Kayen et al., 2008)

The January 8, 2009 Costa Rica Earthquake was a M_w 6.1 event. The epicenter of the earthquake was in northern Costa Rica, about 30 kilometers north-northwest of San José. The damage from the earthquake to roads, bridges and houses resulted primarily from landslides. The earthquake resulted in at least 34 deaths and numerous injuries and displaced persons.

Two Instituto Costarricense de Electricidad (ICE) dams are located relatively close to the earthquake epicenter. Toro II is an embankment dam 66 feet high with a crest length of approximately 1700 feet, located approximately 11.5 Km from the epicenter. A peak ground acceleration of 0.65g was recorded on the crest of the dam. Damage resulting from the earthquake was limited to longitudinal cracks in the crest of the dam. Cipreses is an embankment dam approximately 100 feet high with a crest length of approximately 1100 feet. A peak acceleration of 0.45g was recorded at a surge tank approximately 10.3 Km from the earthquake epicenter. The dam, located approximately 1.7 Km from the surge tank, is closer to the epicenter. Damage to this embankment also consisted of longitudinal cracks in the crest of the dam. Damage resulting from the earthquake was limited to an extent that did not threaten the stability or safety of the structures, and repairs were reported to be underway shortly after the earthquake. The primary affects on the projects were mud and debris flow resulting from earthquake shaking. (Climent and Bolaños 2009)

The May 12, 2008 Wenchuan Earthquake was M_w 8 event. The epicenter was in Sichuan Province in southwest China at the foot of the Tibetan Plateau, where the plateau collides with the Sichuan basin. The earthquake started on a west dipping thrust fault and became a strike/slip event as it broke 270 km northeastward on the Longmenshan fault. The duration of strong ground motion was up to 120 seconds at sites underlain by deep alluvium. There were about 300 aftershocks, some over M_w 6. There were very few strong motion recorders operating in the earthquake area, so the estimates of peak ground accelerations are judgments considering observed damage and distances to faults. More than 80,000 people were killed by rockfalls and structure collapses. (Babbitt and Charlwood 2009)

Twenty- two hundred sixty-six of the 35,601 reservoirs in Sichuan and seven nearby provinces were damaged. There were 69 dam breaks and 331 other highly dangerous situations. Ninety-five of damaged reservoirs were formed by small earth dams. Well built dams performed as designed. Zipingpu Dam, a 512-foot high concrete faced rockfill dam was 7 km from the fault break, experiencing an estimated peak ground acceleration of 0.5 to 0.6g. The crest settled 3 feet, damaging small parts of the face slab. Bikou Dam is a 335-foot high central core embankment that experienced an estimate peak ground acceleration of 0.5 g. Its crest settled 9 inches as a result of the shaking. (Babbitt and Charlwood 2009)

On February 27, 2010 at 03:34 am local time, the M_w 8.8 Maule Earthquake struck central Chile. The epicenter of the earthquake was approximately 8 km off the central region of the Chilean coast. With an inclined rupture area of more than 80,000 square km that extends onshore, the region of Maule was subjected to a direct hit, with intense shaking of duration of at least 100 seconds, and peak horizontal and vertical ground acceleration of over 0.6 g. The earthquake caused 521 deaths, with almost half of the fatalities caused by the consequential tsunami. Over 800,000 individuals were directly affected through death, injury and displacement. More than a third of a million buildings were damaged to varying degrees, including several cases of total collapse of major structures. At least 16 dams were moderately to severely shaken, with no reported failures. Some non-threatening slope failures, longitudinal and minor transverse cracking occurred. (Noguera, 2010; Bray et al., 2010) Convento Viejo Dam, a 105-foot-high embankment, was not damaged even though it experienced a peak ground acceleration of 0.38g, which was higher that was expected when the dam was designed. Coihueco Dam suffered non-threatening sloughs on its upstream face and crest cracking apparently without liquefaction occurring. .

The M_w 9.0 Tohoku, Japan Earthquake occurred on March 11, 2011, near the northeast coast of Japan. The earthquake and tsunami caused enormous damage to Japan. As of April 8, 2011 12,731 people were found dead, 14,706 people were still missing and 216,818 houses or buildings were damaged or destroyed. Fukushima No.1 nuclear power station was severely damaged. Immediately after the earthquake, inspection of dams was started. As of March 31, more than 400 dams were inspected. Generally, dams performed well with minor or moderate cracking occurring at embankment dams. (Matsumoto et al., 2011, Yamaguchi et al., 2102) The crest of Surikawa Dam, a 172-foot high central core rock fill completed in 2006, settled a maximum of 7 inches, transverse cracking of the crest paving occurred near the abutments and the leakage temporarily increased from 18 to 25 gallons/minute. The measured peak horizontal accelerations were 0.11g at the foundation and .47g at the crest. (Yamaguchi et al., 2012) The cracks were trenched and found to be only one foot deep. Kejauma Dam is a 79-foot high central core rockill that was completed in 1995. (Yamaguchi et al., 2012) Peak horizontal accelerations of 0.27 and 0.5g were recorded at its foundation and crest, respectively. A maximum crest settlement of 6 inches occurred; leakage temporarily increased from 5 to 110 gallons/minute; and transverse cracking of the crest paving was up to one inch wide. Minamikawa Saddle Dam is a 64-foot high asphalt faced rockfill dam that was completed in 1987. (Yamaguchi et al., 2012) The earthquake caused a temporary increase in leakage from 5 to 23 gallons/minute, a crack in the asphalt face, and a maximum crest settlement of 4 inches. A peak horizontal acceleration of 1.3g was measured at the crest. And, 0.27g was measured on the foundation of the main dam, 1 km away. Yamaguchi et al., (2012) analyzed the accelerations, settlements and leakage increases at these and other dams and noted the apparent effects of the long duration of the M_w 9 earthquake shaking.

The exception to the good dam performance was that the Fujinuma ike (agricultural dam) failed, killing 8 people. The dam was completed in 1949 and was a 60-foot-high, 436-foot-long embankment, impounding a 12,000 acre-foot reservoir for recreation use as well as for agriculture. It was 80 km from the fault break. A preliminary report (Harder et al., 2011) mentioned flaws in the embankment, such as thick lifts, so the dam may not have been constructed to modern standards. The official investigation report (Panel 2012) confirms that observation and mentions the long duration of shaking.

PRIOR TO OCTOBER 2000

The M_w 7.9 1906 San Francisco Earthquake affected about 30 medium-sized earthfill dams located within 50 km of the fault rupture trace, 15 of these being at a distance of less than five km. The majority of these survived the shaking with minimum damage. Such satisfactory performance under extreme loading has been attributed more to the clayey nature of these embankments than to their degree of compaction. (Seed el al 1978)

The 1923 M_w 7.9 Kanto, Japan Earthquake represents perhaps the first documented case of occurrence of significant damage to an embankment dam. Ono Dam, a 122-foot-high earthfill dam, was fractured in many places including a fissure that extended down 70 feet along the puddled clay core wall. Ono Dam settled nearly one foot, with longitudinal cracking up to 200 feet long and 10 inches wide. Local slides about 60 feet long from scarp to toe developed on its downstream face. (Seed et al., 1978)

Some moderate damage was experienced by embankment dams during the M_w 7.4 Kern County, California Earthquake (Seed et al., 1978). The 20-foot Eklutna Dam suffered serious damage during the 1964 Alaska Earthquake (M_w 9.2), and was subsequently abandoned. (Seed et al., 1978) However, it was not until the 1971 San Fernando, California, Earthquake that engineers' concerns regarding the vulnerability of certain types of earth dams were confirmed. The 1971 event received considerable attention from both the media and the general public, as two of many dams that were affected, the Upper and Lower Van Norman dams, were located in a highly developed urban area. A major catastrophe was narrowly avoided. The Lower Van Norman Dam (sometimes referred to as Lower San Fernando Dam), a 140-foot-high hydraulic fill dam, experienced widespread liquefaction and major slope failures. Overtopping of the crest and flooding to an area involving more than70,000 downstream residents did not occur, but only because the reservoir water level was relatively low for the season when the earthquake occurred. The 80-foot-high Upper Van Norman Dam was also severely damaged (Seed et al., 1978).

The near-failure of the Lower Van Norman Dam became a true milestone in earthfill dam performance evaluation. It brought to the attention of engineers and public agencies involved in dam safety the potential vulnerability of embankments constructed of poorly compacted saturated fine sands and silts. It also triggered numerous, state-mandated re-assessments of dam safety, and led to significant advances in the numerical methods of dynamic analysis of dams.

Another event of interest was the 1985 Mexico Earthquake (M_w 8.0), that involved two large earth-rock and rockfill dams, La Villita (197 feet high) and EI Infiernillo (485 feet high). (Bureau and Campos-Pina 1986) While neither of these dams experienced significant damage during the 1985 earthquake, they were shaken from 1975 to 1985 by a unique sequence of closely spaced events, five of which were larger than magnitude 7.2. Cumulative earthquake-induced settlements of La Villita Dam, an earth-rockfill embankment with a wide, central, impervious clay core, approached one percent of its original height in 1985. Based on ten years of careful monitoring, La Villita Dam's settlements showed a tendency to increase in amplitude with the later recent events, perhaps due to progressive weakening of some of the embankment materials. Similar increases have not been observed at EI Infiernillo Dam, the deformations of which have remained small in amplitude, and consistent from one event to the next. Of interest is the fact that these two Mexican dams have actually experienced small, but measurable permanent deformations, at relatively low levels of ground shaking during several of these events.

Two events of moderate magnitude, the 1987 M_w 6.6 Edgecumbe, New Zealand Earthquake, which damaged the 259-foot-high Matahina Dam, and the 1987 M_w 6.0 Whittier Narrows, California Earthquake, which affected several embankment dams in the greater Los Angeles area, are considered to be significant from a dam engineering point of view because of the quality of performance data and strong motion records collected as a result of these events. (EQE 1987, Horowitz and Ehasz 1987)

The October 17, 1989, M_w 6.9 Loma Prieta, California Earthquake involved a wide region south of the San Francisco Bay Area and induced strong shaking to about a dozen embankment dams located within the epicentral area. Over 100 dams of various sizes, mostly embankment dams, were located within 100 km from the epicenter. All but one of the dams concerned performed well, as had been generally predicted in prior seismic evaluation studies. (Bureau et al., 1989) The event emphasized how rarely dams situated in areas of high seismic hazard are tested to the full strength of the ground motion that must be considered in their design. The dams affected by the Loma Prieta Earthquake need to be capable of withstanding earthquakes of higher intensities and longer duration than were experienced during the October 17, 1989 event. This is because of the proximity of nearby active faults and because the strong phase of shaking (accelerations greater than 0.05g) lasted less than eight seconds at rock and firm soil sites in the epicentral area, a relatively short duration for a M_w 6.9 earthquake, due to the bilateral fault rupture of this earthquake. Also, at the time of the earthquake, most of the reservoirs were at between 10 to 50 percent of their maximum capacity, due to several consecutive years of low rainfall. Hence, the drought may have been a beneficial factor for the seismic resistance of the affected earthfill dams, since phreatic surfaces within the embankments were probably below normal. Hydrodynamic loads, which affect concrete dams more than embankment dams, were also significantly reduced as a result of low reservoir levels.

The exception to the good performance was Austrian Dam, a 200-foot-high, 700-foot crest length embankment dam constructed in 1950-51, about 2,000 feet northeast of the San Andreas Fault. The epicenter was 12.5 km from the dam, but the fault broke northward past the dam, producing an estimated peak ground acceleration in excess of 0.7g. The reservoir was at mid height during the earthquake. The maximum crest settlement and horizontal movements were 2.8 and 1.1 feet, respectively. There was longitudinal cracking on the upstream face up to 1 foot wide and 14 feet deep and extensive cracking at both abutments. Transverse cracking and embankment separation from the spillway structure occurred to a depth of 23 feet and a maximum width of 10 inches, apparently due to poor compaction of the embankment, a very steep abutment, soil/structure interaction and deflection of the spillway wall. A transverse crack was traced 30 feet down the left abutment where the dam had been constructed on a thin, loose soil layer overlying a foundation of weathered, highly fractured rock. (Rodda et al., 1990) Both the poor compaction and loose soil layer were artifacts of the need to complete the dam in adverse conditions caused by early winter rains in 1951. The 2.8 feet of settlement was reasonably consistent with the 10 feet that had been predicted for the M 8.3 Design Basis Earthquake in a pre-earthquake seismic reevaluation of the dam, but the damage allowed by latent defects at the abutments was not predicted, neither was the longitudinal cracking on the upstream face.

The July 16, 1990, M_w 7.7 Luzon, Philippines, Earthquake produced estimated peak ground acceleration of 0.60 to 0.65g at Ambuklao Dam, a 426-foot high central core/dumped rockfill; Binga Dam, 335-foot high sloping core/partially compacted rockfill; Masiway, a 82-foot high central core embankment, with conglomeratic and alluvial shells and Pantabangan Dam, a 351-foot high sloping central core embankment with conglomeratic and alluvial shells. Overall performance of the dams was good, but there were two notable occurrences (USCOLD 2000). The left end of the Ambuklao embankment and a contiguous spillway wall moved 20 inches upstream apparently sliding on a compacted clay blanket that had placed to control leakage through the abutment. Cracks up to 12 inches wide and 300 feet long occurred on the crest of Binga Dam, near its maximum section. The cracks grew in size for a few days after the earthquake at a time that the reservoir was being drawn down. The shell zones of Masiway Dam were constructed on alluvium and the dam suffered liquefaction caused damage.

The Northridge, California Earthquake (M_w 6.7) occurred on January 17, 1994, and was centered in the north-central San Fernando Valley, on a blind thrust fault dipping south-southwest below the valley. In addition to considerable damage being inflicted to buildings, lifelines and highway bridges, the Northridge Earthquake was significant to the dam engineering profession for two reasons. First, it reemphasized the seismic hazard associated with concealed faults in California, a region where engineers and geologists thought the distribution of tectonic features to be reasonably well understood. Secondly, it was the second significant event in less than 25 years to affect the San Fernando Valley. As previously noted, in 1971, the M_w 6.6 San Fernando Earthquake damaged several embankment (hydraulic fill) dams and caused near-total failure of the Lower Van Norman Dam.

The earthquake induced ground motions, sometimes quite severe, at 105 dams located within a 75 km radius of its epicenter (CA DSOD 1994a). These dams included most of those shaken in 1971. Eleven earthfill and rockfill dams experienced some cracking and slope movements as a result of the Northridge Earthquake. Yet, none of these presented an immediate threat to life and property. This satisfactory performance may result, to a significant extent, from the fact that, in California, most significant dams have been reevaluated for the Maximum Credible Earthquake (MCE), during investigations initiated after the San Fernando Earthquake. Questionable or unsafe embankments have been upgraded or decommissioned, or the owners have been required to operate the reservoirs with increased freeboard.

One of the few embankment dams that suffered noticeable damage from the Northridge Earthquake was, again, the 125-foot high Lower Van Norman Dam, a hydraulic fill dam. The dam has been abandoned as a water storage facility since 1971, but is still used with an empty reservoir for flood control. It experienced 2- to 3-1/2-inch wide cracks, several hundred feet long. Some of these cracks were at least five feet deep. Sand boils and a sinkhole were also observed along the upstream face. Maximum crest settlement was eight inches, and maximum horizontal crest movement was about four inches upstream. (Bardet and Davis 1996)

The 82-foot-high Upper Van Norman Dam, which was also left with an empty reservoir since it was severely damaged in 1971, experienced transverse cracks near its right abutment, on the downstream slope, and near its left abutment, up to 60 feet long and two to three inches wide. Maximum non-recoverable crest displacements were about 2.4 feet of settlement, and over six inches of horizontal upstream movement. (Bardet and Davis 1996)

The 130-foot-high Los Angeles Dam, which replaced the Van Norman dams, is located between the two empty reservoirs. It experienced extensive, but not safety-threatening, cracking of its asphalt lining and settled 5.4 inches near its maximum section. Maximum horizontal crest movement was about 3.5 inches downstream. (Bureau et al., 1996)

The Northridge Earthquake caused minor damage in the form of transverse cracks and settlement to Lower Franklin Dam (103 feet high); Santa Felicia Dam (213 feet high); Sycamore Canyon Dam (40 feet high); Schoolhouse Debris Basin Dam (38 feet high); Cogswell Dam (266 feet high); Porter Estate Dam (41 feet high); and Rubio Basin Dam (64 feet high).

Like the Loma Prieta Earthquake, the Northridge Earthquake demonstrated the ability of well-designed embankment dams to withstand severe ground motion safely.

The January 17, 1995, Kobe, Japan, Earthquake (M_w 6.9), also named the HyogoKen Nanbu Earthquake, occurred 20 km southwest of Kobe, a densely populated city with a population of approximately 1.5 million people. The bilateral mode of movement along the Nojima Fault experienced during that event was similar to the fault rupture mechanism of the 1989 Loma Prieta, California Earthquake. It involved a rupture length estimated at between 30 and 50 km. Over 5,300 people were killed and nearly 27,000 injured. Extensive structural damage occurred to buildings, highway and railroad bridges, the port facilities at Kobe, and water, waste water, and natural gas facilities in the area. No large embankment dams were affected by the Kobe Earthquake, but about 50 dams higher than 40 feet were located within 50 km of the epicenter. Including small earth dams, about 266 embankment dams were within that range of distance and a seismic intensity rating of 5 on the Japanese [JMA] scale. (Tamura et al., 1997; Yoshida et al., 1999). About half of the dams higher than 40 feet were earth or earth core rockfill dams. (ECRDs)

Three small earth dams, belonging to the Koyoen Reservoir system, were located within the epicentral area, a few kilometers away from where extensive damage occurred to older homes. Another small earth dam, Niketo Dam, also near the zone of large seismic intensities, collapsed completely. The Koyoen Reservoir pools were quite low when the earthquake occurred, due to a prolonged dry period. A post-earthquake reconnaissance report prepared by the U.S. Army Corps of Engineers Waterways Experiment Station indicated that the Koyoen embankments were each about 230 feet long, 25 to 32 feet high, with slopes of about 2:1(horizontal to vertical). They were built of a well graded, slightly cohesive mixture of materials ranging in size from gravel, sand and silt, with some clay. The slopes were faced with concrete.

The upper and middle embankments of the Koyoen complex experienced destructive, massive sliding failures toward the downstream; this was in the absence of reservoir loading. No evidence of water having flowed through the slide debris was found after these failures. The lower embankment suffered extensive loss of strength and severe downstream slope movements, but without being breached. Relatively frail structures adjacent to the site, and a cemetery located about 300 feet away from the upper pool, did not suffer much damage, in contrast to other locations only a few kilometers away. Only about 10 percent of the tombstones were toppled. The intake structure at the Koyoen Reservoir, a relatively small, cylindrical, reinforced concrete tower, experienced small foundation movements and slight tilting. Its access footbridge was shoved through the door of the control chamber, at the top of the tower. Yet, the tower appeared to have remained functional. Overall, these three embankments provide a rare example of earthquake damage to earthfill dams at low reservoir levels and under probably modest intensity of shaking.

Damage to other embankment dams from the Kobe Earthquake was limited. Tokiwa Dam, a zoned earthfill dam with a height of 110 feet, about 10 km from the epicenter, experienced moderate cracking in the crest pavement, near both of the abutments. One of these cracks extended to the core, but remained confined within the freeboard zone. Kitamaya Dam, an 80-foot high embankment, built of decomposed granite with a vertical chimney drain, was about 31 km away from the epicenter. It experienced shallow surficial sliding of its upstream slope. No other damage was observed in earthfill dams higher than 40 feet. Smaller embankment dams, however, suffered various forms of damage such as longitudinal cracking, transverse cracking, settlement, deformation of the dam body, and up to complete failure. The limited damage to embankment dams could be partially explained by the overall assessment of peak acceleration levels at dam locations, which was estimated to be approximately 0.22g at rock sites.

The September 21, 1999, Chi-Chi, Taiwan, Earthquake (M_w 7.6) affected a mountainous area of east central Taiwan and the counties of Taichung and Nantu. It was caused by the rupture of the Chelungpu Fault, a north-northwest/south-southeast striking thrust fault that dips at about 30 degrees to the east. About 2,400 lives were lost, 10,000 people injured, 10,000 buildings destroyed and another 7,500 seriously damaged. Fault rupture was about 80 km long, and was accompanied with spectacular offsets and fault scarps (6.5 to 9.5 feet high along the southern end of the rupture zone, and 13 to 29 feet high in the northern end). The largest scarps included the effects of folding in the hanging wall.

The area affected by the Chi-Chi Earthquake included several dam projects, including the Tachia River project, the Mingtan pumped storage project, and Sun-Moon-Lake-

Reservoir. (Charlwood 1999) Several medium-size embankment dams were affected and experienced some settlement and surficial cracking. However, they did not leak, and otherwise performed satisfactorily. Shui-Chih Dam is an earthfill dam with a clay core and central concrete core wall. It was built in 1934 by the Japanese and has a height of about 98 feet and a crest length of about 1,200 feet. The estimated peak ground acceleration at the dam site was about 0.30g. The crest and upper part of the dam experienced longitudinal cracks, one-half to two inches wide and 300 to 1,000 feet long. The downstream slope settled 0.4 feet. The owner, the Taiwan Power Corporation, immediately filled the cracks with asphalt to prevent rainfall infiltration and lowered the reservoir 13 feet as a precautionary measure. Tou-Shih Dam has a design similar to Shui-Chih Dam, with a height of 62 feet and a crest length of 540 feet. It was built at about the same time as Shui-Chih Dam. Small cracks in the embankment, 5 to 20 inches long and a crest settlement of about 9 inches were reported at this second site.

SUMMARY OF PERFORMANCE

From a detailed review of past experience records, it is apparent that embankment dams have fared both satisfactorily and poorly when subjected to strong earthquake motion. Their performance has generally been closely related to the nature of the materials used for construction. Most well-built earthfill dams are capable of withstanding substantial earthquake shaking with no detrimental effects. Dams built of compacted clayey materials on clay or bedrock foundations have historically withstood extremely strong levels of ground motion, even when obsolete or inefficient compaction procedures were used (Coiheuco Dam may be an exception to this trend.). In contrast, older embankments built on sandy materials or of insufficiently compacted sands and silts and tailings dams represent nearly all the known cases of failures, primarily as a result of the liquefiability of these materials. Therefore, hydraulic fill dams, a type of construction now abandoned, and tailings dams represent the most hazardous types of embankment dams. Conversely, rockfill dams or concrete face rockfill dams (CFRD's) are generally considered to be inherently stable under extreme earthquake loading, and represent desirable types of dams in highly seismic areas. However, the Zipingpu Dam experience shows that concrete face slabs can crack.

PERFORMANCE OF CONCRETE DAMS

Prior to October 2000 perhaps hundreds or more concrete dams had been shaken by earthquakes felt at or near the dam site, but only about 20 had experienced recorded or estimated peak ground accelerations (PGA)s of 0.2g or higher. Now in 2013, there are about 20 dams that have experienced PGAs over 0.3g. The duration of motion of the M_w 9.0 Tohoku Earthquake was extraordinary long from 150 to 300 seconds. Following the earthquake, about 240 concrete dams were inspected. Reports indicate that the concrete dams appear to have performed very well during the main earthquake and numerous large aftershocks. Dam performance during earthquakes since September 2000 is discussed first in the following sections.

ARCH DAMS

Since September 2000

Shapai Dam, China, the highest RCC arch dam in the world at 132 meters, was moderately to severely shaken by the May 12, 2008 M_w 8 Wenchaun, China, Earthquake. There was no damage to the dam body.

Rapel Dam, Chile, is a double curvature arch dam built in 1968. The dam has large spillways on both abutments. The dam has a structural height of 364 feet, a crest length of 886 feet, a crest width of 18 feet, and a base width of 62.3 feet. On March 3, 1985, the magnitude M_w 7.8 Santiago Earthquake occurred offshore 92 km at a depth of 33 km on the Benioff Subduction Zone. Prior to this main event, a swarm of 300 earthquakes of lesser magnitude occurred. Measured peak free-field accelerations near the dam were 0.31g in the cross-canyon direction, 0.14g in the upstream to downstream direction, and 0.11g vertical. The arch dam did not experience any damage, but the appurtenant structures did have damage. The spillway walls were cracked and there was leakage at the wall of the right spillway. The upper part of one intake tower cracked and separated from the dam.

Rapel Dam was also shaken by the M_w 8.8 Maule (also called Cauguenes) Earthquake on February 27, 2010, with its reservoir full. With the epicenter of the quake not precisely determined, the dam was about 232 km away. Although Chilean attenuation relationships indicated a PHGA of 0.30g at the site, the maximum PHGA from a single accelerogram located on the dam was 0.21g. A block of the concrete arch on its left abutment is next to a fault named "Nido de Aquila" (Eagle's Nest). The joint between this block with its adjacent concrete block showed a rise of 0.02 inches (0.5 mm). Seepage again increased along the right abutment; this time from a normal 3.4 gal/sec to10.6 gal/sec and some concrete pavement at the dam crest cracked. It is not known if the right abutment seepage reduced with time.

Prior to October 2000

Gibralter Dam, USA, a 169-foot high, 500-foot long concrete arch dam located near Santa Barbara, California was severely shaken by the M_w 6.3 Santa Barbara Earthquake of June 29, 1925. There was no damage to the dam from the PHGA estimated at greater than 0.3g. In 1990, the dam was strengthened by addition of a 92,500 cu yd (70,700 m^3) buttress of roller-compacted concrete (RCC) on the downstream face which in effect changed the dam from a concrete arch to a curved concrete gravity dam.

Ambiesta Dam, Italy, is 194-foot high symmetrical double curvature arch, designed between 1949 and 1954 to be earthquake resistant. It has a crest length of 475 feet and is in the eastern Alps of Italy. The dam was undamaged by the May 6, 1976, Gemona-Friuli M_w 6.5 Earthquake. The peak ground acceleration was 0.33g. (USCOLD 2000)

Pacoima Dam, USA, a 372-foot-high concrete arch dam, was subjected to estimated base accelerations of perhaps 0.7g during the 1971 San Fernando, California Earthquake (M_w 6.6). A then unprecedented peak horizontal acceleration of 1.25g was recorded on rock at the left abutment, slightly above the dam crest. However, it was concluded that this large acceleration was presumed to have been related to the local narrow ridge topography and possible shattered condition of the bedrock in the area of the strong motion instrument. The primary purpose of the dam is flood control facility so that the reservoir depth at the time of occurrence of the earthquake was about 60% of its impounding depth. The dam did not develop structural cracks or experience relative movements between adjacent blocks as a result of the 1971 earthquake. Yet, the left abutment had to be strengthened through installation of post-tensioned tendons to stabilize two large rock wedges that moved several inches as a result of the earthquake.

The 1994 M_w 6.7 Northridge Earthquake also severely shook Pacoima Dam, which located 11 miles from the epicenter. As it did during the 1971 earthquake, the dam experienced ground accelerations well above one g, at its left abutment. Indeed, horizontal and vertical peak ground accelerations recorded in 1994 near the top of that abutment were l.76g and over 1.60g, respectively. Downstream records, near the toe of the dam, were only 0.44g (horizontal) and 0.22g (vertical), emphasizing the significance of ridge effects upon amplifying ground motion, and perhaps the influence of the distress previously experienced in 1971 within the left abutment rock mass. (USCOLD, 1992)

At the time of the 1994 earthquake, the reservoir was at about one-third of its impounding depth. The joint between the left abutment concrete thrust block and the left end of the dam opened about two inches. The left abutment thrust block also moved 0.5 inches downstream, relative to the crest. The protective gunite cover was severely cracked at both abutments. Post-earthquake surveys indicated a maximum horizontal displacement of about 19 inches at one location on the left abutment, and 14 inches of downward movement of the rock mass at another location. This experience also confirmed that some lift joints did open. Post-tensioned tendons, installed in 1971 to hold down potentially unstable rock wedges in the upper left abutment, became inoperable for post-tensioning adjustments, due to failed O-rings. They were subsequently repaired and re-stressed.

Techi Dam, Taiwan, is a 607-foot-high double curvature concrete arch dam with a crest length of 950 feet. The dam is founded on a "pulvino" (large foundation footing) and was designed using a pseudo-static coefficient of 0.15g. It was reevaluated in 1992 using an evaluation earthquake with a peak acceleration of 0.35g. It is located about 85 km from the epicenter of the M_w 7.6 Chi-Chi Earthquake. A peak acceleration of 0.86g was recorded near the crest, at the end of the spillway crest, but the base acceleration was not recorded. Peak ground acceleration at the site was estimated at between 0.3 and 0.5g. No damage to the dam concrete was observed. There were no signs of vertical joint movements. Minor curb cracking was observed at the access roadway. It was reported that four out of five pumps at the power plant went out of service, but no details are known where damage occurred (AFPS, 2000). Collected seepage increased in the days following the earthquake but returned to normal. The reservoir was lowered 10 feet as a precautionary measure.

GRAVITY AND BUTTRESS DAMS

Since September 2000

Kasho, Sugesawa, and Uh Dams, Japan, experienced peak ground acceleration of approximately 0.5g, 0.16g and 1.0g, respectively during the Western-Tottori Prefecture M_w 6.7 Earthquake of October 6, 2000. These gravity dams, which are 152, 241, and 46-feet high, suffered no significant damage.

Hakkagawa Dam, Japan, a 171-foot high concrete gravity dam completed in 1995, was 14 km from the epicenter of the M_w 6.7 Noto Hanto, Japan Earthquake of March 25, 2007 (Iwashita et al., 2008). The rock foundation of the dam is mainly andesite and volcanic conglomerate with a P-wave velocity of 3.6–4.5 km/s. Seismometers are installed at the bottom of the inspection gallery and at the crest. The seismometer at the inspection gallery (at the foundation) recorded peak accelerations of 0.17 g in the stream direction, 0.21 g in the dam axis direction and 0.17 in the vertical direction. The peak accelerations at the crest (installed in the pier top of the crest bridge) were 0.87g in the stream direction, 0.72g in the dam axis direction, and 0.27g in the vertical direction. The only damage to the dam was minor spalling at joints. Total drainage, which is the total amount from drilled foundation drain holes and from contraction joints, was 0.5 gallon/minute prior to the earthquake. It increased to one gallon/minute immediately after the earthquake, but fell to 0.7 gallons/minute within an hour back to 0.5 gallons/min in a day and a half. A plumb line installed in Block 7 of the dam body revealed a permanent displacement of 0.02 inches in the downstream direction and 0.008 inches toward the left-bank side immediately after the earthquake. Uplift pressure data showed no significant change due to the earthquake.

Baozhusi, Yingxiuwan, Taipinyi and Futang Dams, China, were shaken by the May 12, 2008 M_w 8 Wenchaun, China Earthquake. (Babbitt and Charlwood 2009) Baozhusi Dam is a 426-foot high gravity structure. The estimated peak ground acceleration of 0.2g caused only surficial damage. Yingxiuwan, Taipinyi and Futang are modern 69 to 102-foot high gravity diversion dams, with radial gates, located on the Minjiang River upstream of Zipingpu Reservoir. They were from less than 1 to 9 km from the fault break.

Their dam bodies and reinforced concrete appurtenances were not damaged by the shaking, but massive walls were broken and a radial gate was dislodged by a rock falls. Takou Dam in Iwate Prefecture Japan is a 252-foot-high concrete gravity dam with a crest length of 1056 feet. When the dam was subjected to the March 11, 2011 Tohoku Earthquake M_w 9 main shock, a power failure occurred and the accelerograph did not function. But when an aftershock of magnitude M 7.1 occurred on March 17, accelerograms were recorded. The PGA at the foundation was 0.38, 0.29 and 0.27 g in the upstream to downstream direction, the cross-canyon direction, and the vertical direction, respectively. Peak accelerations at the crest in the same directions were 1.79, 2.04 and 1.33 g, respectively. Notice the significant amplitude at the crest of the dam in the cross-canyon direction. PHGA during the main shock is estimated 0.4 g from analysis of the accelerograms recorded at surrounding instrumentation stations of KiK-net. The dam is a multi-purpose dam including flood control and the water level during main shock was 108 feet below the crest elevation. The wall of the gate house was cracked and there was an offset in the dam's parapet wall. However, the dam showed no damage.

Miyatoko Dam, Japan, a 157-foot-high RCC dam located north of Sendai in Miyagi Prefecture, is the second RCC dam severely shaken by an earthquake. A strong motion instrument located in the gallery recorded a PHGA of 0.32 g during the Tohoku Earthquake. No damage was reported.

Prior to 2000

Lower Crystal Springs Dam, USA, a 127-foot-high curved concrete gravity dam built of imbricated concrete blocks, withstood the 1906 San Francisco Earthquake (M_w 7.9) without a single crack (USCOLD 1992). The rupture trace of the San Andreas Fault was less than ¼ mile from the dam, and a right-lateral slip of about ten feet was measured nearby. Searsville Dam, another 64-foot-high gravity arch constructed of imbricated concrete blocks near the San Andreas Fault, also performed satisfactorily in 1906. Searsville Dam was designed by Herman Schussler, the same engineer who designed Lower Crystal Springs Dam. Both Lower Crystal Springs and Searsville dams were moderately shaken by the 1989 Loma Prieta Earthquake and were unaffected.

Hoover Dam, USA, a 726-foot-high curved gravity dam, has been suspected of being the cause of moderate reservoir-triggered seismicity (M_w 5.0 or less) after initial reservoir filling in the 1930s, which did not affect the dam.

Blackbrook Dam, Great Britain, a 100-foot-high concrete gravity dam with an upstream brick facing and a downstream stone facing, is the only dam in Great Britain to have been damaged by an earthquake (1957). The event, rated at VIII on the British Intensity scale with a maximum of X, was estimated to be centered about 6.4 km from the dam site. It cracked the mortar of the downstream stone facing. All of the large coping stones which topped the parapet walls on both sides of the crest of Blackbrook Dam were lifted from their mortar bed and dropped back, crushing the mortar in the process.

Koyna Dam, India, a 338-foot-high straight gravity dam, and Hsinfengkiang Dam, China, a 344-foot-high buttress dam, were shaken as the result of nearby earthquakes of M_w 6.3 (1967) and 6.1 (1962), respectively. Both dams developed substantial longitudinal cracking near the top. Damage was attributed to design or construction details that would be avoided in modern structures. The two dams were repaired, and are still in service.

Poiana Usului Dam, Romania, a buttress dam, was located about 60 km away from the epicenter of the 1977 Romanian earthquake (M_w 7.5), and performed satisfactorily.

Sefid-Rud Dam, Iran, a 348-foot-high buttress dam, suffered severe cracking in the upper part of some buttresses and other forms of damage during the 1990 Manjil Earthquake (M_w 7.4). The dam was rehabilitated and remains in service (USCOLD 2000). Some 20 years later, a blister on the steel lining in the elbow of the one of the two glory hole spillways in the left abutment and increased leakage into the spillway lead to discovery of previously undiscovered damaged caused by the earthquake (Faghihimohaddess et al., 2011). Exploration holes disclosed poor quality rock, backfill concrete and cracking in the concrete around the steel liner. The rock and concrete were repaired by grouting. The grouting significantly decreased the leakage. The second spillway apparently was not damaged. It is located in better rock closer to the dam.

Bear Valley Dam, USA, located in southern California, is significant in that it was shaken by two distinct earthquakes one day apart. Bear Valley Dam is a 92-foot- high concrete multiple arch dam with a crest length of 360 feet that was completed in 1912 and modified in 1988. (USSD 2003) The modification was due to the concern with adequacy of the dam when subjected to the design earthquake or overtopping by large floods. The structural upgrade consisted of converting the multiple arch to basically a gravity structure by partially infilling the arch bays with conventional concrete.

On June 28, 1992, the fault rupture on the Landers earthquake (M_w 7.3) located 45 km away shook the dam. Then on June 29, 1992, the more critical M_w 6.5 Big Bear Earthquake occurred about 14.5 km from the dam site on an unnamed fault in response to the rupture on the Landers Fault. At the Big Bear Lake Civic Center, located about 4 km from the dam, PGA of 0.18g horizontal and 0.08g vertical was recorded during the Landers Earthquake and 0.57g horizontal and 0.21g vertical due to the closer Big Bear Earthquake.

Thorough post-earthquake investigations indicated Bear Valley Dam was damaged. The only indication of shaking at the site was a slight displacement of girders on the highway bridge located at the dam crest.

Gohonmatsu, Sangari and Aono Dams, Japan, are gravity dams located about 1, 12 and 22 km from fault that ruptured in the 1995 M_w 6.9 Kobe Earthquake, were undamaged. Shaking at these dams was probably moderate, because undisturbed tile roofs were observed at nearby houses. Aono and Sangari dams are concrete dams, while Gohonmatsu (109 feet high) is the first Japanese dam (built in 1900) constructed of concrete rubble masonry. At Gohonmatsu Dam, hairline cracks were observed in the capping concrete on the crest wall, but no cracks were observed in the dam body. Two other gravity dams, Nunobiki (109 feet high) and Karasubara (105 feet high) survived the earthquake with no apparent damage. Hence, medium-size concrete gravity dams performed very well during the Kobe earthquake.

Shih-Kang Dam, Taiwan, located about 50 km north of the M_w 7.6 Chi-Chi Earthquake epicenter, is a gravity/spillway dam which regulates the Tachia River in its lower course. Shih-Kang Dam is about 82 feet high, and has about 18 gated bays that serve as the spillway. The dam was directly intersected by the Chelungpu fault rupture, with a differential movement of about 29 feet vertical and 6.5 feet horizontal under bays 16 to 18. The fault had not been mapped at the site prior to the earthquake. Bays not affected by the fault rupture survived essentially undamaged. Peak ground acceleration was reported at 0.56g in a town nearby. (Charlwood 1999) The failure of Shih-Kang Dam did not result in catastrophic release of the reservoir water. Due to upstream changes in topography and the failed gates and piers obstructing passage of the water, uncontrolled release was limited to between 3,500 and 7,000 cfs and the reservoir drained overnight without flooding downstream.

Mingtan Dam, Taiwan, is a concrete gravity dam with a structural height of 269 feet. Being 7.5 miles away, the Chi Chi Earthquake induced PHGA at the dam estimated between 0.4 to 0.5g. There was no reported damage to the dam. Hydrostatic pressures increased in relief wells in the foundation after the earthquake, but returned to normal after redrilling.

Bajina Basta Dam

Bajina Basta (one of the case histories in this volume) has not been damaged by earthquake shaking, rather is believed to have experienced enough neotectonic movement to cause significant cracking in the last hollow gravity section at its left abutment and the dam has moved as a whole towards the left bank.

PERFORMANCE SUMMARY

Overall, the performance of concrete dams has been satisfactory. (Nuss et al., 2012) However, the true test of a major thin arch concrete dam, with a full reservoir, subjected to a peak ground acceleration exceeding 0.5g has yet to come. The Shih-Kang Dam experience confirmed that concrete dams are vulnerable to a major fault rupture.

Concrete buttress dams when subjected to severe shaking have developed horizontal cracks at the elevation high in the dams where the downstream buttresses intersect the vertical "chimney" section. This is an area where the stiffness of the concrete structures significantly changes.

Some other specific conclusions are:

- While a fault located directly below Shih Kang Dam (Taiwan) caused a rupture and relative vertical displacement of 29 feet, the remaining damaged concrete limited an immediate total and sudden release of the reservoir.
- PHGAs are amplified from the base of the dam to the crest. In three cases, this amplification produced measured peak accelerations at the crest in excess of 2.0g: 1) Pacoima arch dam in the USA (2.3g), 2) Kasho gravity dam in Japan (2.05g), and 3) Takou gravity dam in Japan (2.04g).
- Peak accelerations at the crest are greater with full reservoirs, as expected.

- Several dams have been severely shaken on two occasions by separate major earthquakes with only minor damage: Bear Valley Dam, USA (one day apart), Pacoima Dam, USA (23 years apart), and Rapel Dam, Chile (25 years apart). Many concrete dams have also been shaken by high intensity aftershocks that occurred after the main earthquake without suffering additional damage.
- Shapai Dam, China, an RCC arch dam, and Miyatoko Dam, Japan, an RCC gravity dam, performed no differently than a dam built of conventionally placed concrete despite concern by some engineers of possible less strength at the many lift joints.
- Where damage has been identified, it has been cracking high in the dam and where additional features such as curbs, railings, gates, or guard or control houses are located. Cracking in buttress dams appeared to be due to upstream to downstream motions and not cross-canyon motions.
- Very little in the way of increased leakage has occurred in concrete dams subjected to major earthquakes. This can be attributed, in part, to the fact that any cracking caused by the earthquake has mainly been horizontal and located high in the dam together with the reservoir not being full in many cases. Some rock foundations have experienced a temporary increase in seepage following an earthquake.

There may be a number of reasons why concrete dams have performed well and invariably better than that predicted by design or analysis when shaken by an earthquake. The main reasons may be:

- Concrete dams are redundant structures that provide considerable capacity to redistribute load once damage occurs in the structure. Being so massive, typically there is plenty of concrete volume around damaged areas of the dam to carry loads around damaged sections of the dam.
- The duration of strong shaking may be too short to cause failure. Shake table tests at the Bureau of Reclamation showed that it takes considerable time at high levels of shaking to fail a medium-thick arch dam compared to a thin arch dam.
- The dynamic tensile strength of concrete many times is taken as 50 percent higher than the static tensile strength of the concrete. This increase in strength makes dams stronger during seismic shaking and increases resiliency.
- Damping mechanisms can increase in the dam during the earthquake and reduce the seismic impact on the dam. Damping increases as the concrete cracks and contraction joints open and close.
- The seismic impact of the earthquake on the dam may be reduced because the natural frequency of the dam does not match the postulated frequency content of the earthquake during design. For example, a gravity dam with a natural frequency of 7 Hz would not align with a ground motion with a peak spectral acceleration at 3 Hz.
- The three-dimensional effects of the dam help prevent failure. The curvature in plan view of the dam or the narrowness of the canyon greatly increases the seismic stability of a dam. The potential for sliding of a gravity dam wedged in a narrow canyon is remote.

A generally accepted potential failure mode for concrete dams during an earthquake is cracking of the concrete, cracking through the dam that forms removable blocks, sliding of the blocks during or after the earthquake to cause failure. Severely shaken concrete dams to date have cracked at locations of change in geometry (reentrant corners), but have not formed removable concrete blocks. Thus the entire potential seismic failure mode has not been fully achieved or experienced for concrete dams.

While concrete dams are designed to withstand a higher degree of seismic shaking than buildings and have performed well in the past, we should not become overconfident of their performance in the future. Great care should be taken in the design details and quality of construction. Particular attention should be given to possible faults located under the dam.

Table 1. Historic Performance of Dams During Earthquakes

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- Ambiesta, Italy; Friuli Earthquake (1976)
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- Los Angeles, California; Northridge Earthquake (1994)
- Los Leones, Chile; Central Chile Earthquake (1985)
- Masiway, Philippines; Philippines Earthquake (1990)
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- Pantabangan, Philippines; Philippines Earthquake (1990)
- Sefid-Rud, Iran; Manjil Earthquake (1990)
- Sheffield, California; Santa Barbara Earthquake (1925)
- Vermilion, California; Eastern Sierra Nevada earthquake sequence (1980)

The following case histories are included in this volume:

- Aratozoawa, Japan; Iwate-Miyagai-Nairku (2008)
- Bajina Basha, Serbia
- Bhuj, India Earthquake (2001)
- Coihueco, Chile; Maule (2010)
- Colbun, Chile; Maule (2010)
- Convento Viejo, Chile; Maule (2010)
- Ishibuchi, Japan; Iwate-Miyagai-Nairku (2008)
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ARATOZAWA DAM, JAPAN

Aratozawa Dam was shaken by a recorded peak ground acceleration of 1.03g during the Iwate-Miyagi-Nairiku, Japan Earthquake of June 14, 2008. The dam suffered only slight damage. A massive landside slid into the north arm of the reservoir causing a seiche that overtopped the spillway, but not the dam. (Kayen et al., 2008)

ARATOZAWA DAM

The dam is a 74-meter-high zoned earth and rockfill embankment, with an upstream slope of 2.7:1 and downstream slope of 2:1 (Figures 1, 2 & 3). (Omachi and Tahara 2001) The crest is 10 meters wide and 414 meters long. The embankment volume is 3.05 million m^3. The dam is founded on rock. The embankment was virtually completed in 1991. Following appurtenant structure construction the dam was place in service in 1998. It is on the Nihazama River in Miyagi Prefecture and impounds a 14,139 million m^3 reservoir. The purposes of the reservoir are flood control, hydroelectric power generation and irrigation. It is owned by the Ministry of Agriculture, Forestry and Fisheries.

EARTHQUAKE

The 2008 Iwate-Miyagi-Nairiku earthquake (M_w 6.9) struck the northern part of Japan. (Midorikawa et al., 2008) The earthquake produced many strong-motion records including one with peak vertical acceleration of 4 g. Twenty-three people were killed or missing, 450 were injured, and about 2,000 houses were damaged. The earthquake occurred in a less-populated mountainous area, and the resulting building damage was rather small in comparison with observed strong-motion records with high accelerations. However, many landslides and slope failures caused extensive damage to roads and isolated people in the mountainous area.

The earthquake occurred in a region of convergence between the Pacific Plate and the Okhotsk section of the North American Plate in northern Japan, where the Pacific plate is moving west-northwest with respect to the North American plate at a rate of approximately 8.3 cm/yr. The hypocenter of the earthquake indicates shallow thrusting motion in the upper (Okhotsk) plate, above the subducting Pacific plate, which lies at approximately 80 km depth at this location. The fault rupture plane dips at a shallow angle to the west. The focal depth was about 8 km.

EMBANKMENT EARTHQUAKE PERFORMANCE

The reservoir was 6 meters below the spillway when the earthquake occurred. The dam was 15 km from the epicenter. Its three 3 component strong motion installation recorded the following peak accelerations during the main shock (Omachi 2011):

Location	Elevation (m)	Transverse	Longitudinal	Vertical
Crest	278.5	0.54g	0.46g	0.63g
Mid core	250.0	0.55	0.49	0.48
Bottom gallery	203.7	1.03	0.92	0.70

The clay core settled along the axis of the dam up to 400 mm and the rockfill shells settled a maximum of 200 mm. (Tani et al., 2008) The crest road is a bordered by granite columns that are held in place at their base by a small rebar peg and are connected to one another by a chain. The effect of the differential settlements of the rockfill shells and core was to cause a general rotation of columns inward toward the center line of the dam (Figure 4). Some of columns collapsed in irregular directions. A 200 mm step formed at the embankment/spillway contact (Figure 5).

Another indication of core settlement is that the top pipe of a cross-arm embankment settlement measuring device emerged from a manhole near the center of the dam crest (Figure 6). (Yamaguchi et al., 2008) The pipe appeared to have moved upwards relative to the dam crest approximately 27 cm. The pipe may have been partially dragged down with the dynamically settling crest approximately 13 cm and otherwise decoupled from the remaining settlement of the crest. This allowed it to emerge from a manhole cover. The pipe segments are 5 meters long and the devise extends the full 74 meters height of the dam.

The upstream shell deformed laterally towards the reservoir between 24 and 43 mm. Despite the deformations, there was no apparent damage to the hand placed rock facing on the downstream or upstream sides of the reservoir (Figure 1).

In the inspection gallery beneath the impervious core zone, approximately 1 liter per minute of leakage was discovered through fine cracks in its ceiling and from the boundary between the upstream side wall and invert and small openings caused by the earthquake were observed at the joints of blocks of the gallery. The gallery was dry before the earthquake. The measured leakage declined steadily after the earthquake, as it usually does at severely shaken dams.

Ohmachi (Ohmachi 2011) has analyzed the strong motion data from the main shock and 185 aftershocks and demonstrated that the large strains induced by the strong motion decreased the shear wave velocity and shear modulus of the core material and that it recovered with time. Similar computations have been made using data from Lexington Dam. (Makdisi et al., 1991, Mejia et al., 1992) The strains also significantly increased pore pressures in the core. At a piezometer in the lower part of the core, about 42% of the increased pore pressure dissipated in one day and the remainder dissipated in 3 months (Ohmachi and Tahara 2011).

APPURTENANT STRUCTURE PERFORMANCE

The minor cracking of a retaining wall that is part of the spillway structure shown in Figure 7 may be due to offset on a short fault that moved as a result of the earthquake shaking. Figure 8 indentifies locations of road damage that is in line with the cracking. This cracking is the only reported damage to appurtenant structures. (Kayen et al., 2008)

LANDSLIDE INTO RESERVOIR

The total volume of the massive landslide was estimated to be 50 million cubic meters, of which approximately 1.5 million cubic meters slid into the reservoir (Figure 9). The landslide was approximately 1.3 km long and 0.8 km wide. Even though the base slope was very gentle, about 3-4 degrees, the soil masses moved some 200 to 500 meters. The lake elevation increased 2.4 m (Elev. 268.5 to 270.9) during the earthquake, due to the landslide and possibly also due to some tectonic deformation. Three and a half kilometers of an access road to the dam were obliterated by the landslide. (Kayen et al., 2008)

CONCLUSIONS

The embankment and appurtenant structures Aratozawa Dam withstood severe shaking from a M_w 6.9 earthquake, including a peak ground acceleration of 1.03g, with minimal damage.

The accelerations recorded have been used to quantify the core's shear modulus reduction and recovery with time.

A 50 million cubic meter landslide upstream of the reservoir serves as a reminder of the need to assess the effects of potential large earthquakes on reservoirs.

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Figure 1. Embankment from left abutment (from Kayen et al., 2008).

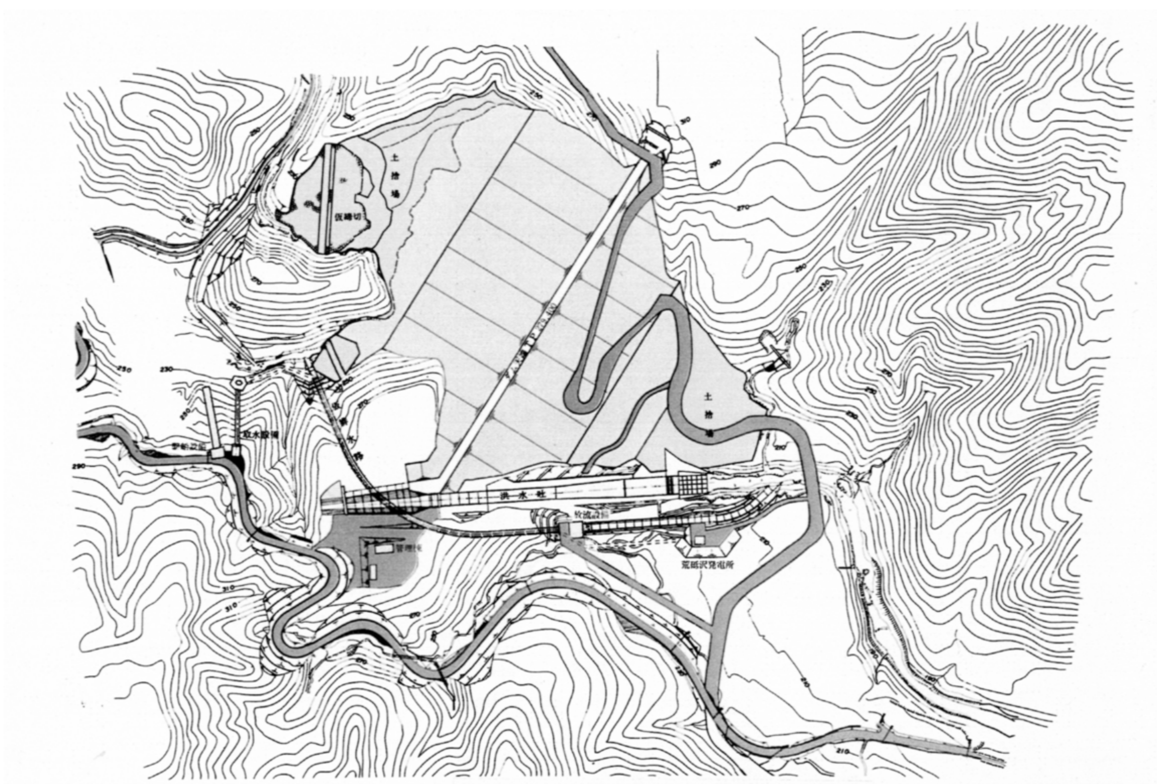


Figure 2. Plan view (from PWRI, 2008).

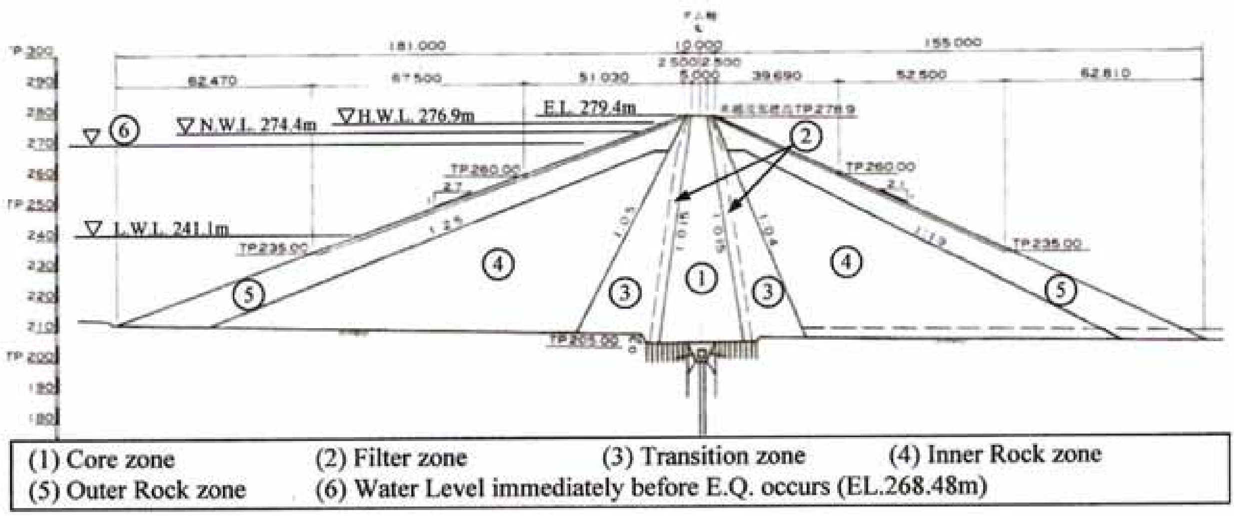


Figure 3. Section (from PWRI 2008).

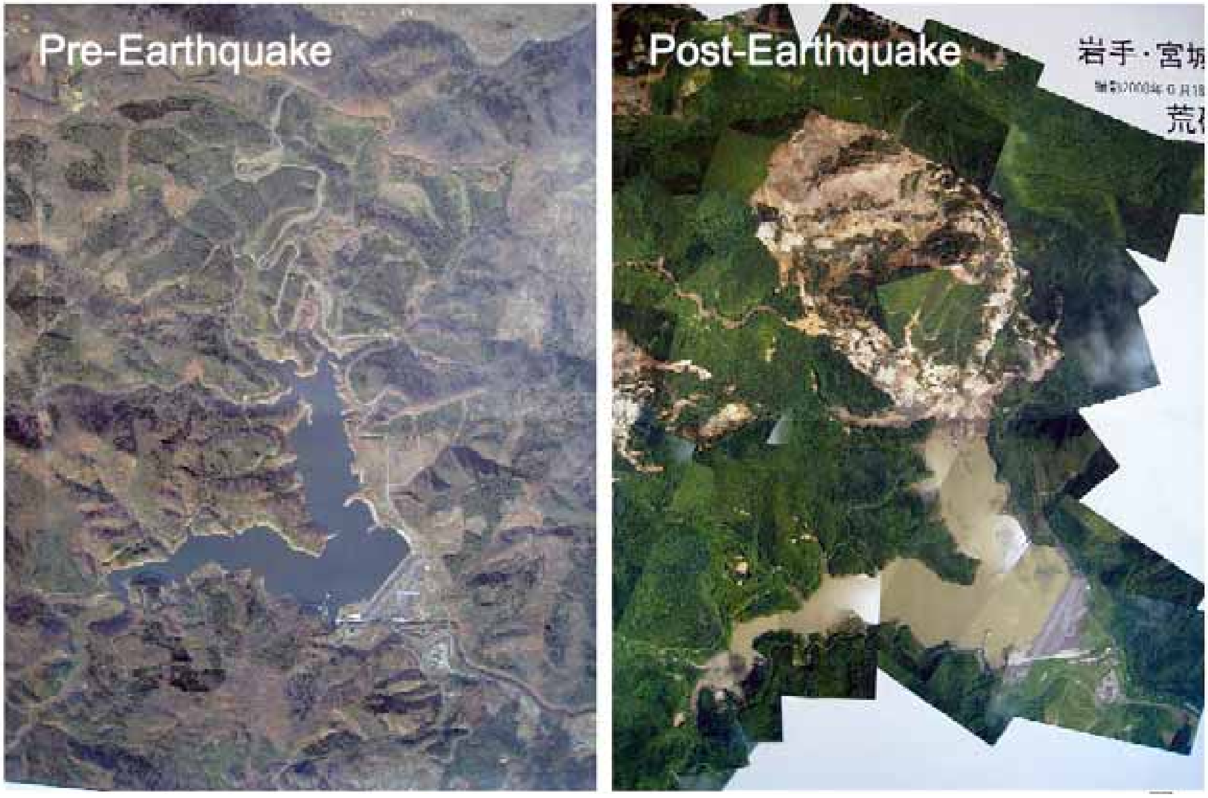
Figure 4. Dam crest showing tilting and fallen monuments (from Yamaguchi et al., 2008).

Figure 5. 10 cm of embankment settlement at the spillway (from Tani, et al., 2008).

Figure 6. Settlement device (from Yamaguchi et al., 2008)

Figure 7. Cracking of spillway retaining wall structure (from Kayen et al., 2008).

Figure 8. Aerial photo denoting three areas of roadway cracking. The cracking of the spillway wall shown in Figure 7 occurred below the right most arrow (from Kayen et al., 2008).



Before earthquake After earthquake

Figure 9. Landslide into reservoir (from Kayen et al., 2008).

BAJINA BASTA DAM, SERBIA

This dam is believed to have experienced enough neotectonic movement to cause significant cracking in the last hollow gravity section at its left abutment. Observation of the dam has shown that it has moved as a whole towards the left bank. This case history is based on reference ICOLD 1998.

During the first five years after impounding water, geodetical observations indicated that dam movements were satisfactorily similar. However, after prolonged evaluation, the movements were recognized and accepted as a case of persistent neotectonic movements.

BAJINA BASTA DAM

The Bajina Basta Dam is a 90-meter-high dam on the Drina River in Serbia. It is a hollow gravity concrete dam, which began impounding water in 1966. It is formed by hollow gravity blocks with twin buttresses. A photo of the dam is shown in Figure 1.

Figure 1. Photo of Bajina Basta Dam (from ICOLD 1998).

NEOTECTONIC MOVEMENTS

Neotectonics comprises all kinds of tectonic movements active in the earth's crust in recent geologic times and is inclusive of seismic and aseismic movements along distinct faults and movements of larger tectonic units. Tectonic movements include large tectonic plate movements (such as subduction zone movements) and also smaller tectonic units within the planet's crustal envelope.

Normally, when a dam is built on an active tectonic unit, it will experience uniform movements without affecting the dam's integrity. However, if the dam body is crossing a zone where differential movements take place, the dam's structural integrity and safety may become endangered as a consequence of induced differential movements.

In the case of Bajina Basta Dam, the situation was understood as a combination of expected normal dam movements under design loads and neotectonic influence, producing the observed unexpected total movements, which gradually approached the final position of crest target vectors as shown in Figure 2.

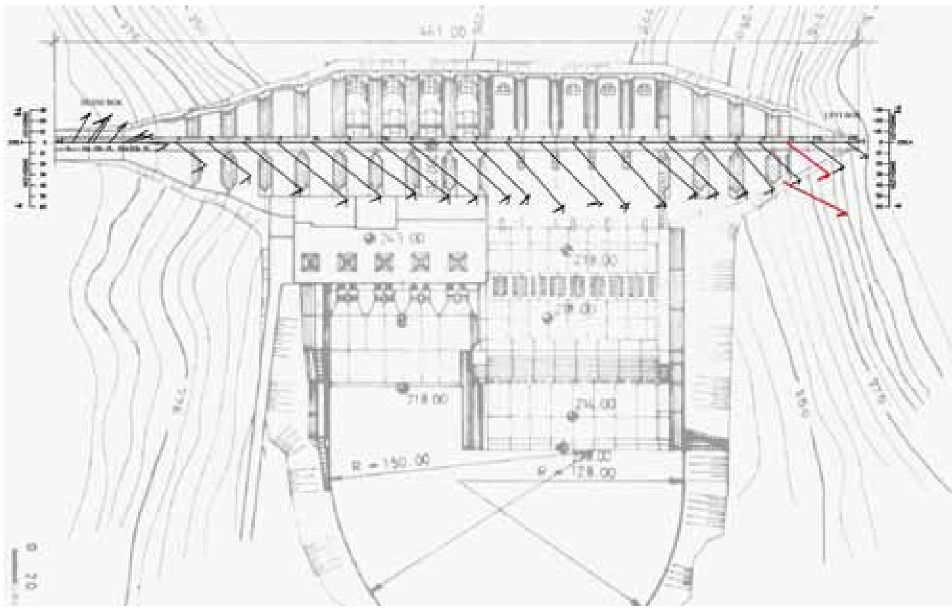


Figure 2. Movement along the dam crest observed in 2009 (from ICOLD 1998).

DAM PERFORMANCE

Movement of the dam has been rigorously monitored. Apart from general movements, no significant damage was recognized during dam inspections until about 1990, when significant cracking was observed in the hollow gravity Block No. 21 (the last buttress element at the left abutment).

A large vertical crack had formed through the whole thickness along the central part of Block No. 21. It was rehabilitated by placing heavily reinforced concrete plates, which were connected by pre-stressed anchors along each side of the cracked web of the block as shown in Figure 3. In the following 19 years (1992-2011), no further damage has occurred at the rehabilitated Block No. 21.

Figure 3. Rehabilitation of Block 21 (from ICOLD 1998).

The explanation for development of the crack in Bajina Basta Dam was the existence of persistent neotectonic movements. This statement was introduced in ICOLD Bulletin No. 112 (ICOLD 1998).

GPS observation of the Bajina Basta site and wider surroundings were undertaken starting in 1994 to better understand this situation. This observation included the dam and a part of left-bank background, as well as a much larger background on the right bank, covering the neighboring Tara Mountain. A vector plot of movements obtained by GPS monitoring confirm such development of crustal mobility influencing the Bajina Basta Dam.

The general conclusion which can be formulated after 44 years of Bajina Basta service is that neotectonic movements of a creeping type, influencing much larger areas than the dam site itself, constitute a tectonic hazard of lower intensity than seismicity. If this kind of tectonic movements is slowly translating the dam without inducing differential deformations in the dam body, the dam can follow such movements without significant damages. But if differential movements are induced, adverse stress concentrations are the consequence and might require significant rehabilitation works.

The Bajina Basta Dam did experience differential movement in a way as to be seemingly harmless, except at the contact with the stiffened Melaphyres formation at the far left bank. The local differential movements required significant rehabilitation work to the dam.

The risk related to neotectonic movements of a creeping type is considerably lower than those connected with seismic movements. However, if the possibility of such creep movements is realized, an investigation should be directed towards ascertaining whether differential movements capable of harming the dam body could occur. The existence of such condition might require corresponding treatment. The case history of Bajina Basta dam shows how such a risk can be recognized and treated.

A final note is that the application of GPS monitoring was of particular significance for understanding and clarifying the observed phenomena.

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ICOLD Bulletin 112, "Neotectonics and Dams," 1998.

PERFORMANCE OF DAMS DURING THE BHUJ EARTHQUAKE, INDIA

On January 26, 2001 an earthquake (M_w 7.7) struck the Gujarat Province in the western part of India. The death toll from the earthquake was estimated at more than 20,000 with over three billion dollars in property damage. (GEER 2001) The nearest accelerogram was located in the city of Ahmedabad which is approximately 200 km from the epicenter. Figure 1 shows a map of the region with contours of Modified Mercalli intensity as compiled and presented by Krinitzsky and Hynes, 2002. In this area of India it is common to find loose sand and silt deposits which may extend to large depths. Where saturated these soils appear to have liquefied during shaking leading to widespread damage such as sand boils, ground cracking and lateral spreading of ground and embankments. (Sitharam and Govindaraju, 2004) Liquefaction was attributed as the cause of damage to many embankment dams in this region.

PERFORMANCE OF EMBANKMENT DAMS

One important aspect of the Bhuj earthquake was the performance of small and moderate size embankment dams. This area of India is arid and receives the majority of its rainfall during the monsoon season (June to September). Many dams have been constructed in this region in an effort to store water for irrigation and domestic water supply needs. Following the earthquake the Water Resources and Water Supply Department of Gujarat inspected more than 300 of these dams following ICOLD guidelines (Patel and Brahmabhatt, 2003). Each dam was categorized based on the damage, downstream consequences and importance to water supply. These classifications were used to determine the appropriate course of action for restoration. Each of the damage classes and the total number of dams placed into each class are summarized in Table A.

Multiple post-earthquake investigators noted liquefaction related damage to embankments throughout the affected area, as evidenced by sand boils, ground cracking and lateral spreading (e.g., Krinitzsky and Hynes, 2002; Towhata et al., 2002; Patel and Brahmabhatt, 2003; Singh et al., 2005). Many of these dams were constructed directly on loose alluvial deposits and investigators attributed the damage to the liquefaction of this material. At the time of the earthquake there was very little water in the reservoirs and the majority of damage to the embankments occurred in the valley section where the low pool kept the alluvium saturated. Table B provides details about some of the dams investigated by researchers following the earthquake, as well as references for additional information. As was discussed earlier, no strong motion data was available at or near any of the dams. However Krinitzsky and Hynes (2002) estimated the Modified Mercalli intensity based on visits to the region and Singh et al. 2005, estimated the maximum acceleration using the epicentral distance and a region-specific attenuation relationship.

Following the earthquake, repairs to the dams were conducted in two stages. The first stage of repairs focused on emergency work needed to ensure that the maximum number of dams could be operational during the monsoon. Dams that could not be immediately repaired were partially or completely breached to ensure safe passage of the flood waters.

Stage one repairs were completed within 60 days after the earthquake. (Patel and Brahmabhatt 2003) The second stage of repairs would focus on upgrading the dams to ensure they will be safe in future earthquakes. A seismic coefficient has been adopted for these future repairs and the selection of this factor and guidelines for designs is discussed in Patel and Brahmabhatt (2003).

SELECTED CASE HISTORIES

Two of the dams damaged during the Bhuj earthquake were the Chang and Tapar dams. Observations at both of these dams suggest that liquefaction of the upstream alluvium took place leading to deformations and cracking. The two dams responded very differently and their individual responses will be examined in more detail in the following sections.

CHANG DAM

Chang Dam was constructed in 1959 as a zoned earth fill dam with a masonry core wall (Figure 2). Different references give different pre-earthquake heights and lengths for the dam, but the dam was between 15.5 and 17 meters tall. The foundation material consisted of loose alluvium on top of sandstone bedrock. The GEER investigation team reported that the dam was located approximately 2 km from observed tectonic deformations and there was an upstream slope failure at the maximum section that dropped the crest an estimated 10 meters (GEER 2001). The worst damage was concentrated on the upstream side in the valley section, however bulging was observed at both the upstream and downstream toes. (Patel and Brahmabhatt, 2003) Several large longitudinal cracks were observed on the crest and upstream slopes (Figure 3) and the core wall failed due to embankment deformations. The reservoir was nearly empty at the time of the earthquake, but multiple investigators noted sand boils near the upstream toe which suggests the alluvium was saturated and liquefied during shaking (Figure 4). Towhata et al., 2002 performed Swedish weight soundings after the earthquake near the upstream toe and reported that the top 2.5 meters of the foundation was soft and loose. The damage to the dam was considered to be too large to repair before the monsoon season, so a full-height cut was made in the valley section to allow the flood waters to safely pass. (Patel and Brahmabhatt, 2003)

TAPAR DAM

Tapar Dam was constructed in 1975 and is considered to be a very important dam in the region due to the agricultural and drinking water the reservoir provides. At the time of the earthquake the reservoir was very low, but not dry. Tapar is a 15.5-meter-tall zoned earth fill dam (Figure 5) and a seismic coefficient was used in the design which led to the construction of berms at both the upstream and downstream toes. (Brahmabhatt and Judav 2003) The dam was founded on deep alluvium which likely liquefied during the earthquake as evidenced by sand boils and lateral spreading near the upstream toe (Figure 6). Damage at this dam was significantly less than noted at Chang Dam. Crest settlements were estimated at approximately 0.5 meters and movements were primarily in the upstream direction. (Singh et al., 2005) Longitudinal cracks occurred along the crest to a depth of 3 meters (Figure 7) and a nearly continuous crack occurred at the junction of the berm and the upstream slope. (Brahmabhatt and Jadav, 2003) As with most of the dams, damage was concentrated in the valley section of the dam and there was no evidence of liquefaction near the abutments where the embankment was virtually undamaged. (Krinitzsky and Hynes, 2002). The spillway wall was constructed of unreinforced masonry and was also undamaged. The top of the control tower collapsed however the remaining section appeared to be intact (Figure 8). The tower also appeared to have tilted indicating the foundation material had lost strength. (Krinitzsky and Hynes, 2002) Following the earthquake, the dam was repaired by excavating cracks along the crest and upstream slope and filling the excavated area with compacted impervious material. The damaged portion of the upstream slope was removed completely and recompacted in lifts. A 1-meter-thick compacted stone layer was placed on the upper and lower berms. Details on the repairs can be found in Brahmabhatt and Jadav, 2003.

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Table A. Damage classifications used to prioritize repairs
(after Patel and Brahmabhatt 2003).

Damage Class	Description	Medium Dams	Minor Dams	Total Number
1	Significant damage. Dam is critical to region and repairs will be completed before monsoon.	4	7	11
2	Major damage. Not possible to repair before monsoon. Partial cut is made to allow water to pass, but allow for some storage.	2	0	2
3	Major damage. Damage is to extensive to store water. Full cut is made to pass flood waters.	1	3	4
4	Minor Damage. Repairs completed before monsoon.	80	165	245
Total		87	175	262

Table B. Performance of selected dams.

Dam Name	Dam Height[1,2]	Crest Length[1,2]	Epicentral Distance[1]	Distance to Source[2]	Max Acceleration[1]	Modified Mercalli Intesity[2]	Damage Description[1,2]	Additional References
	M	m	Km	km	g			
Bhukhi	23.4	1550	-	80	-	IX	Minor cracking on dam crest	Krinitzsky and Hynes 2002
Chang	15.5 (17[3])	370 (1227[3])	13	-	0.5	-	Failure of downstream and upstream slopes, slumping, cracking.	GEER 2001, Patel and Brahmabhatt 2003, Singh et al. 2005, Towhata et al. 2002
Demi-1	17	4268	-	90	-	VII	Superficial disturbances	Krinitzsky and Hynes 2002
Fatehgadh	11.6	4049	80	32	0.3	X	Shallow upstream failure, cracking	GEER 2001, Krinitzsky and Hynes 2002, Singh et al. 2005
Gajod	14	403	-	75	-	VIII	Superficial disturbances	Krinitzsky and Hynes 2002
Kalaghogha	14.9	1346	-	65	-	VIII	Minor cracking on dam crest	Krinitzsky and Hynes 2002
Kaswati	12.9	1455	110	25	0.28	X	Shallow upstream failure, major longitudinal cracking, leakage	GEER 2001, Krinitzsky and Hynes 2002, Singh et al. 2005
Mathal	21	1603	-	100	-	VIII	Minor cracking on dam crest	Krinitzsky and Hynes 2002
Niruna	34.4	2034	-	65	-	IX	Minor cracking on dam crest, major displacements on dam shoulders	Krinitzsky and Hynes 2002
Puna	18.3	1432	-	115	-	VII	Superficial disturbances	Krinitzsky and Hynes 2002
Rangmati	15	2765	-	110	-	VII	Superficial disturbances	Krinitzsky and Hynes 2002
Rudramata	27.4	875	-	40	-	X	Shallow upstream failure, cracking on crest, leakage	Brahmabhatt and Jadav 2003, GEER 2001, Krinitzsky and Hynes 2002, Patel and Brahmabhatt 2003, Sitharam and Govindaraju 2004, Towhata et al. 2002
Sapada	20.5	344	-	105	-	VII	Superficial disturbances	Krinitzsky and Hynes 2002
Sasoi	20	3767	-	120	-	VII	Superficial disturbances	Krinitzsky and Hynes 2002
Shivlakha	18	300	28	-	0.45	-	Failure of downstream and upstream slopes, cracking.	Brahmabhatt and Jadav 2003, GEER 2001, Patel and Brahmabhatt 2003, Singh et al. 2005
Suvi	15	2097	37	10	0.42	X	Shallow upstream failure, cracking	GEER 2001, Krinitzsky and Hynes 2002, Patel and Brahmabhatt 2003, Singh et al. 2005
Tapar	15.5	1350 (5395[3])	43	10	0.41	X	Shallow upstream failure, cracking	Brahmabhatt and Jadav 2003, GEER 2001, Krinitzsky and Hynes 2002, Patel and Brahmabhatt 2003, Singh et al. 2005, Sitharam and Govindaraju 2004

Notes: 1. Reference: Singh et al., 2005
2. Reference: Krinitzsky and Hynes, 2002
3. Reference: Patel and Brahmabhatt, 2003

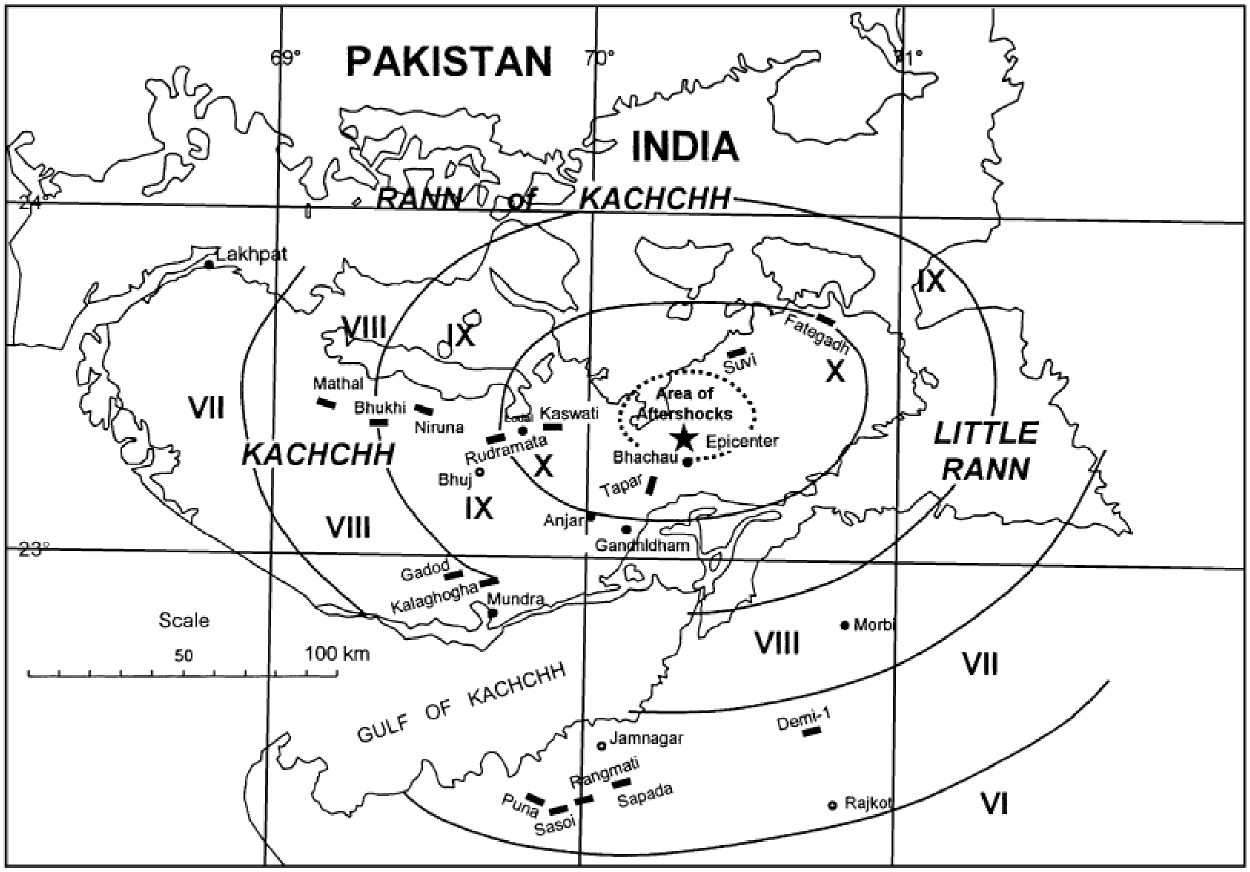

Figure 1. Map of the Kachchh Region of India with modified Mercalli contours shown (from Krinitzsky and Hynes, 2002).

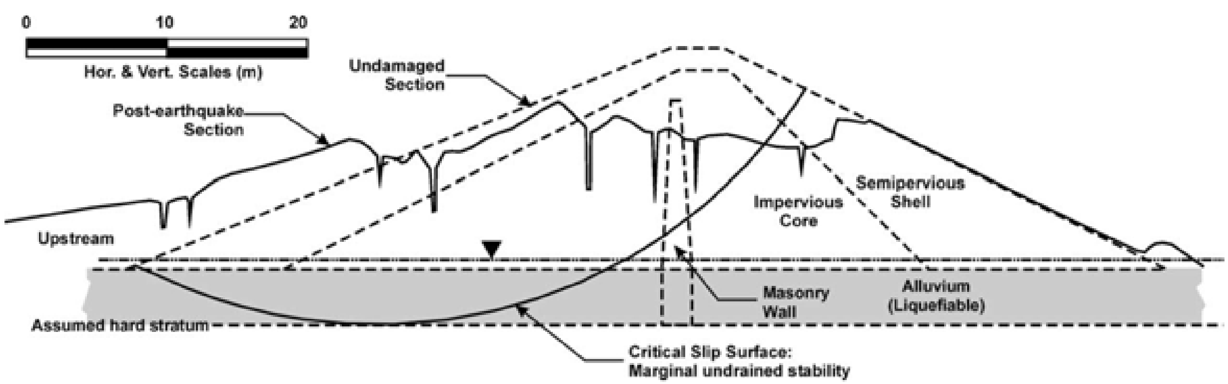

Figure 2. Cross-section of Chang Dam showing undamaged and damaged Embankment. The critical slip surface was calculated based on a post-earthquake stability analysis performed by the original author (from Singh et al., 2005).

Figure 3. A large longitudinal crack, approximately 10 meters deep, was observed near crest of Chang Dam (from GEER 2001).

Figure 4. Sand boil observed in the reservoir near the upstream toe of Chang Dam (from GEER, 2001)

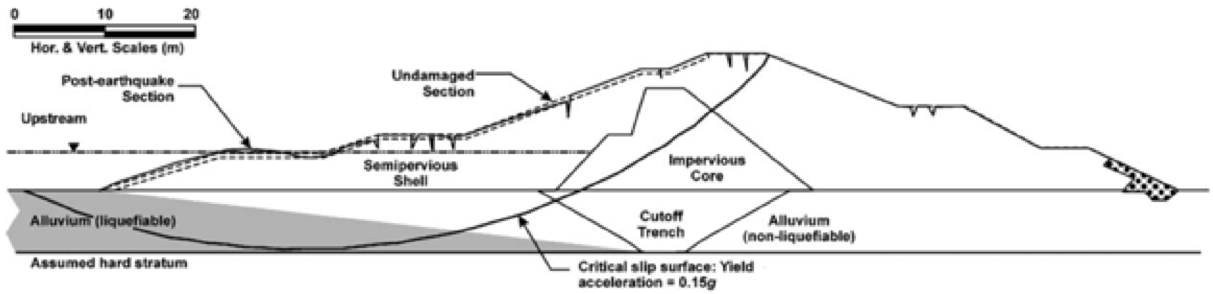

Figure 5. Cross-section of Tapar Dam showing undamaged and damaged embankment. The critical slipsurface was calculated based on a post-earthquake stability analysis performed by the original author (from Singh et al., 2005).

Figure 6. Lateral spreading occurred near the upstream Toe of Tapar Dam (from GEER, 2001).

Figure 7. Longitudinal cracks occurred along crest of Tapar Dam. Excavation showed the cracks extended to a depth of 3 meters (from GEER, 2001).

Figure 8. Damaged intake tower at Tapar Dam (from Krinitzsky and Hynes 2002).

COIHUECO DAM, CHILE

Coihueco Dam suffered non threatening sloughs on its upstream face and crest cracking during the Maule, Chile, Earthquake of February 27, 2010.

COIHUECO DAM

Coihueco Dam is a 31-meter-high zoned embankment. It has central silt core, transition zones and sand and gravel shells (Figure 1). (Yasuda et al., 2010; Bray et al., 2010) The crest is 5 meters wide and upstream and downstream slopes are 3:1 and 2.5:1. Except for a wide cutoff trench, the 1040-meter-long embankment is founded on silt. There is a saddle dam south of the main embankment. The dam was built between 1964 and 1970 and impounds a 2.26 million m^2 acre reservoir (Figure 2). It is located 35 km east of Chillan in central Chile.

MAULE EARTHQUAKE

On February 27, 2010 at 03:34 am local time, a powerful earthquake of M_w 8.8 struck central Chile (Elnashai et al., 2010). The epicenter of the earthquake was approximately 8 km off the central region of the Chilean coast. With an inclined rupture area of more than 80,000 square km that extends onshore, the region of Maule was subjected to a direct hit, with intense shaking of duration of at least 100 seconds, and peak horizontal and vertical ground acceleration of over 0.6 g. The earthquake caused the death of 521 persons, with almost half of the fatalities caused by the consequential tsunami. Over 800,000 individuals were directly affected through death, injury and displacement. More than a third of a million buildings were damaged to varying degrees, including several cases of total collapse of major structures.

EMBANKMENT EARTHQUAKE PERFORMANCE

Coihueco Dam was 138 km from the epicenter of the earthquake. The peak ground acceleration at the dam was estimated to be 0.57g by using the Martin y Saragoni 2005 attenuation relationship. (Noguera, 2010)

As shown on the figures, three sloughs occurred on the upstream face and there was some cracking at the crest. (Bray et al., 2010; Noguera 2010, Yasuda et al., 2010) None of the sloughs or cracks occurred at the highest part of the dam.

Section 15 of Bray et al., 2010, relates that Coiheuco Dam was visited by three GEER teams. They observed slumps on the upstream face, looked for, but not finding potential evidence of liquefaction. On the third trip they made observations using a LIDAR – Reflector survey, SASW to obtain shallow shear wave velocities and Dynamic (hand held) CPTs and hand excavated a shallow trench at Slump A, the most prominent one (Figures 3 – 7). The slump is on the left abutment. The LIDAR survey results were used to show there was a volumetric balance between material from the head scarp and in the toe bulge. The hinge point of the material and accretion appeared to be approximately half way down the slope. The DCPT sounding was made near the waterline (after the reservoir was drawn down). It indicate that the soils were determined to be fine-grained – primarily volcanic silty clays derived from the Chillan volcano located approximately 30 km to the west of the dam. Dynamic penetration in this material ranged from about 3 to 12 blows in the upper 3 meters, then increased approximately linearly with depth. The shallow trench was dug in the upstream slope after removing the riprap from the slope surface. The near-surface soil in the upstream shell consisted of gravelly sand and sandy gravel.

Figures 8 through 10 show the other bulges and Figures 11 through 14 the crest cracking. The maximum depth of cracking was reported as 1.9 meters. (Yasuda et al., 2010)

CONCLUSION

Loss of strength in the fine grained materials underlying the embankments due to the earthquake shaking seems to be the mostly likely cause of the sloughing and possibly the crest cracking. It would be interesting to know if the material was clayey.

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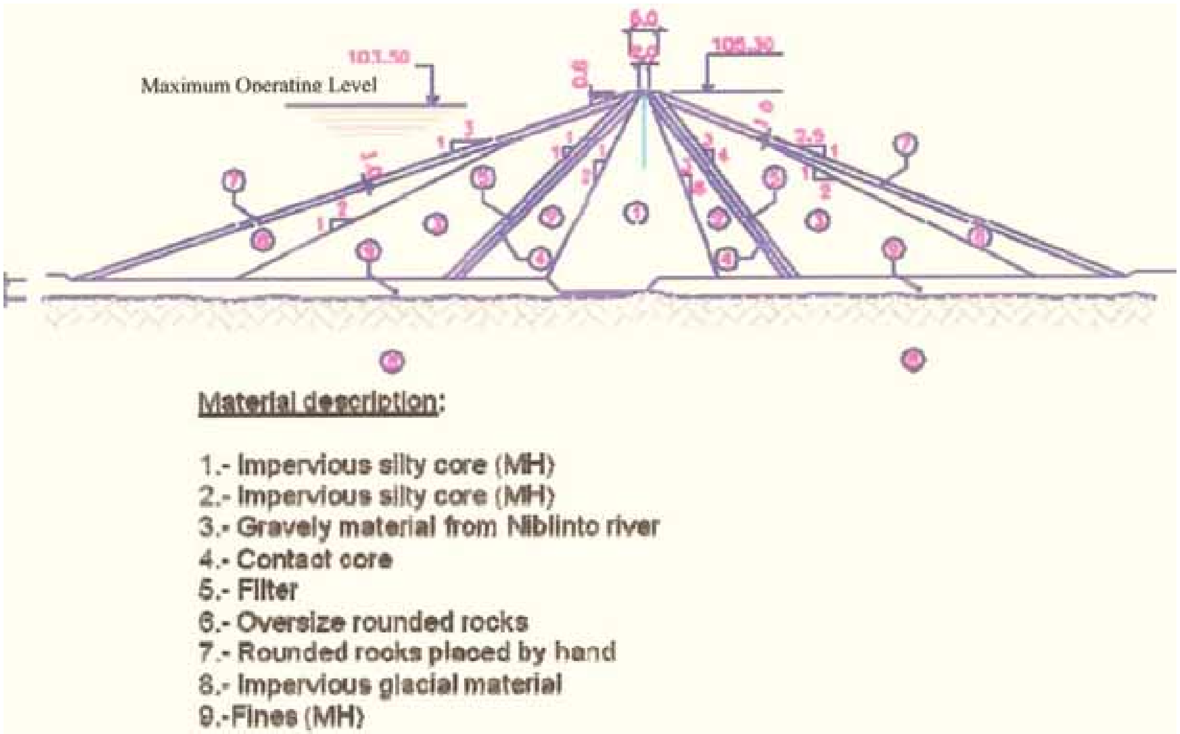

Figure 1. Embankment maximum section (from Bray et al., 2010).

Figure 2. Pre-earthquake aerial photo of reservoir (from Bray et al., 2010).

Figure 3. Slough A — near left abutment scarp at dam crest and bulge in lower right of photo (from Bray et al., 2010).

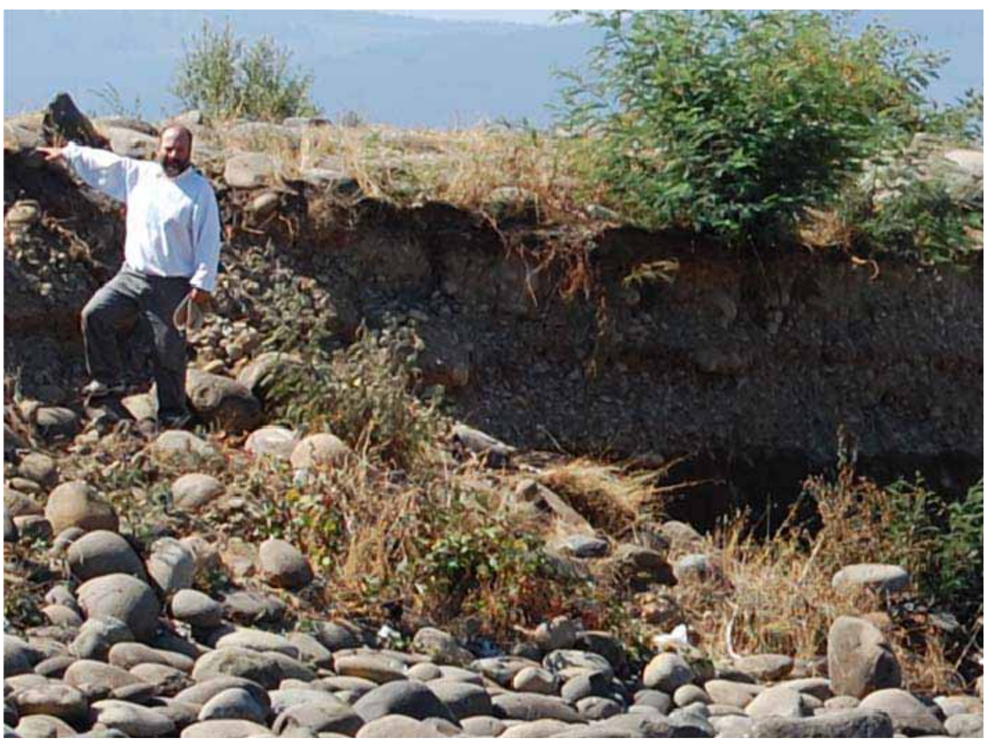

Figure 4. Close up of scarp (from Bray et al., 2010).

Figure 5. Slough A – bulge (from Bray et al., 2010).

Figure 6. Scarp and bulge (from Bray et al., 2010).

Figure 7. Slough A — side view of bulge (from Bray et al., 2010).

Figure 8. Slough B located right of the highest part of the embankment (from Bray et al., 2010).

Figure 9. Slough B from its toe (from Bray et al., 2010).

Figure 10. Slough C on right side of embankment (from Bray et al., 2010).

.

Figure 11. Crest cracking (from Bray et al., 2010).

Figure 12. Crest cracking near right end of embankment (from Bray et al., 2010).

Figure 13. Saddle Dam (from Bray et al., 2010).

Figure 14. Cracking in crest of Saddle Dam (from Bray et al., 2010).

COLBUN DAM, CHILE

Colbun Dam suffered only minor damage during the February 27, 2010 Maule Earthquake in Chile. The measured peak ground acceleration at the dam was 0.37g.

COLBUN DAM

Colbun Dam is a 116 meter-high, 550 meter-long, 13,870,000 m^3 embankment dam (Figures 1 and 2). It has an inclined sandy clay core and compacted gravel shells. The foundation is fluvial, 68 meters deep. A concrete cut off was constructed to control seepage through the pervious foundation. The dam impounds a 1500 million m^3 reservoir on the Maule River, 30 km northeast of Linares, in central Chile. It is part of Colbun-Machicura Hydroelectric Central and also supplies irrigation water. The dam was completed in 1985 by Guy F. Atkinson Construction Company. There are three small dikes on the reservoir perimeter.

MAULE EARTHQUAKE

On February 27, 2010, at 03:34 a.m. local time, a powerful earthquake of M_w 8.8 struck central Chile (Elnashai et al., 2010). The epicenter of the earthquake was approximately 8 km off the central region of the Chilean coast. With an inclined rupture area of more than 80,000 square km that extends onshore, the region of Maule was subjected to a direct hit, with intense shaking of duration of at least 100 seconds, and peak horizontal and vertical ground acceleration of over 0.6g. The earthquake caused the death of 521 persons, with almost half of the fatalities caused by the consequential tsunami. Over 800,000 individuals were directly affected through death, injury and displacement. More than a third of a million buildings were damaged to varying degrees, including several cases of total collapse of major structures.

EARTHQUAKE PERFORMANCE

All of the information on the earthquake performance is from Noguera, 2010.

The dam was 183 km from the epicenter and 100 km from the fault break. An accelerograph in a rock tunnel registered a peak horizontal acceleration 0.37g. The peak vertical acceleration was 50% over the maximum horizontal acceleration.

An electrical conduit next to the downstream edge of the crest was displaced a maximum of more than 2 meters horizontally and almost 1 meters vertically (Figure 3), but inclinometers along the downstream shoulder did not show deformation due to the earthquake.

The rest of the embankment suffered minor settlements and displacement (not more than 10 cm).

Transverse cracks across the crest next to both abutment had a thickness of 1 cm, and when excavated disappeared at a depth of 3 meters.

There were no changes in Casagrande piezometers in the abutments of the dam.

Pneumatic piezometers in the embankment did not operate.

The displacement of the cable duct disturbed three monuments along the dam's crest.

No other problems were observed.

CONCLUSIONS

Colbun Dam performed adequately during a very large earthquake. The transverse cracking at the abutments, a common occurrence in embankment dams with steep abutments, was within the freeboard.

The movement of the electrical conduit appears to have been caused by high accelerations at the dam crest. Unfortunately no measurements of the accelerations are available.

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Google Maps

Noguera, Guillermo (2010), "Dam Performance during the Cauquenes (Chile) Earthquake 27/02/2010," presentation to ICOLD Committee on Seismic Aspects of Dam Design, Hanoi, Vietnam, May 24, 2010

Figure 1. Dam from downstream right (from Bray el al. 2010).

Figure 2. Dam from downstream (from Chile.com).

Figure 3. Displaced electrical conduit on downstream edge of crest (from Noguera 2010).

CONVENTO VIEJO DAM

Convento Viejo Dam suffered only very minor damage as a result of the February 27, 2010, Maule Earthquake. A peak ground acceleration of 0.38g was measured at the toe of the dam. The intensity of shaking was greater than anticipated during design of the dam.

CONVENTO VIEJO DAM

Convento Viejo Dam is a 32 meter-high central clay core embankment with compacted gravel shells and intermediate transition zones of selected granular materials (Figures1 and 3). It is 728 meters long and has a volume of 3 million m^3. A spillway/outlet structure is located north of the hill that forms the right abutment of the embankment (Figure 2). The dam was constructed in stages between 1973 and 2008. It is located on Chimbarango Creek, 25 km north of Curico in central Chile and impounds a 237 million m^3 reservoir, with a surface of 4,500 ha, for irrigation, flood control and hydroelectric power generation. (Alverez et al., 1982; Noguera, 2010)

The embankment is founded on fluvial materials, mainly gravels and sands. The deposit has a maximum depth of 55 meters beneath the central part of the dam. Near the surface it is sandy and clayey gravels (GP-GC), with a maximum thickness of 10 meters in the central part of the dam and a relatively high degree of compactness. Beneath this stratum are silty sands with few fines (SP-SM or SW-SM), also showing relatively high compactness. This layer is 15 meters thick in the center part of the dam. Beneath these materials and resting on bedrock there are sandy gravels (GW or GP) with relatively high densities.

A 7- to 16- meter-deep cut-off trench was excavated in the foundation and filled with compacted clay. Due to the onset of winter and consequent flood risk, an emergency action was taken, filling the right portion of trench partially beneath water with sandy gravel, most probably without compaction control. The depth of this fill is about 10 meters, and it was about 180 meters long (Figure 4).

A 500-meter-long, 55-meter maximum depth plastic concrete wall was constructed through the cutoff trench and fluvial materials in 7.2-meter panels. (Alverez et al., 1982)

During, design, the tension-deformation behavior and the dynamic response of the embankment were analyzed using the finite differences method. Two sections were analyzed, one with clay in the cutoff trench, the other with the relatively loose granular materials.

MAULE EARTHQUAKE

On February 27, 2010, at 03:34 a.m. local time, a powerful earthquake of M_w 8.8 struck central Chile. (Elnashai et al., 2010) The epicenter of the earthquake was approximately 8 km off the central region of the Chilean coast. With an inclined rupture area of more than 80,000 square km that extends onshore, the region of Maule was subjected to a direct hit, with intense shaking of duration of at least 100 seconds, and peak horizontal and vertical ground acceleration of over 0.6g. The earthquake caused the death of 521 persons, with almost half of the fatalities caused by the consequential tsunami. Over 800,000 individuals were directly affected through death, injury and displacement. More than a third of a million buildings were damaged to varying degrees, including several cases of total collapse of major structures.

EARTHQUAKE PERFORMANCE

Both the embankment and spillway/outlet structure performed very well during and after the earthquake. There were no signs of significant problems or troubling instrument readings. Minor longitudinal cracks were observed on the crest in the highest part of the embankment. The maximum settlement was 279 mm. The seismic analysis had predicted 230 mm. There was a noticeable depression in the crest approximately over where sandy gravel was placed in the cutoff trench (Figure 5). (Campana et al., 2011, Noguera, 2010)

The dam was 251 meters from the epicenter and 90 km from fault break. The three accelerographs registered the following:

NAME	LOCATION	Amax E-W	Amax N-S	Amax VERT
A1	Dam Crest	0.49g	0.50g	0.44g
A2	Dam Toe	0.30	0.38	0.27
A3	Tunnel in Rock	0.19	0.15	0.20

A2 is on the fluvial deposit. Large aftershocks were also recorded. (Noguera, 2010)

Campana et al., 2011, state: “During the February 27th earthquake, at the foot of the dam the free field acceleration recorded was 0.49g (resultant from the E-W and N-S components). This value is 11% higher than the maximum credible Earthquake adopted in the seismic risk study. On the other hand, the maximum destructive potential, which is defined as the Arias intensity (IA) divided by the square of the number of crossings by zero, gave a result equal to 47x10-4 (g·s3) for the design earthquakes. The February 27th earthquake presented a destructive potential equal to 63x10-4 (g·s3), which means a value 34% higher. In relation to the seismic amplification between the foot and the crest of the dam, the dynamic analyses estimated a value equal to 25%, whereas for the actual earthquake, the corresponding amplification was 46%.”

There are 14 Casagrande piezometers, 7 in the core and 7 in the foundation below the cutoff trench and downstream of the slurry wall and 15 electric piezometers distributed in the core downstream filter and downstream foundation. The water levels in all the Casagrande peizometers increased during the earthquake. All but two returned to their pre-earthquake levels in three days. Those two were in the area where the sandy gravel was placed in the cutoff trench. The electric piezometers also responded to the earthquake, but their readings decreased the day of the earthquake. Piezometer readings were also affected by the several strong aftershocks. (Noguera, 2010)

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Figure 1. Dam from right abutment (from Noguera).

Figure 2. Spillway/outlet from downstream (from Google Maps, R. Castillo).

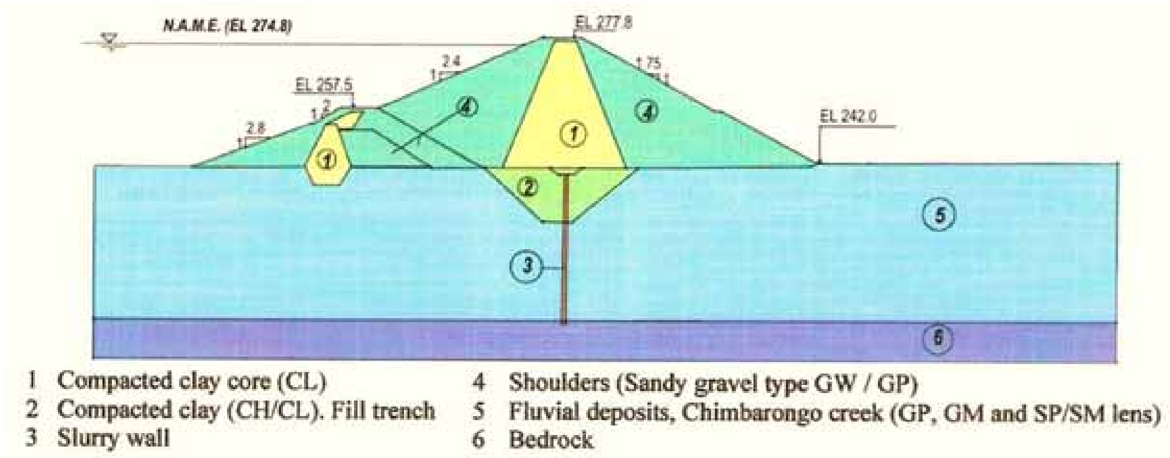

Figure 3. Embankment cross section (from Campana et al., 2011)

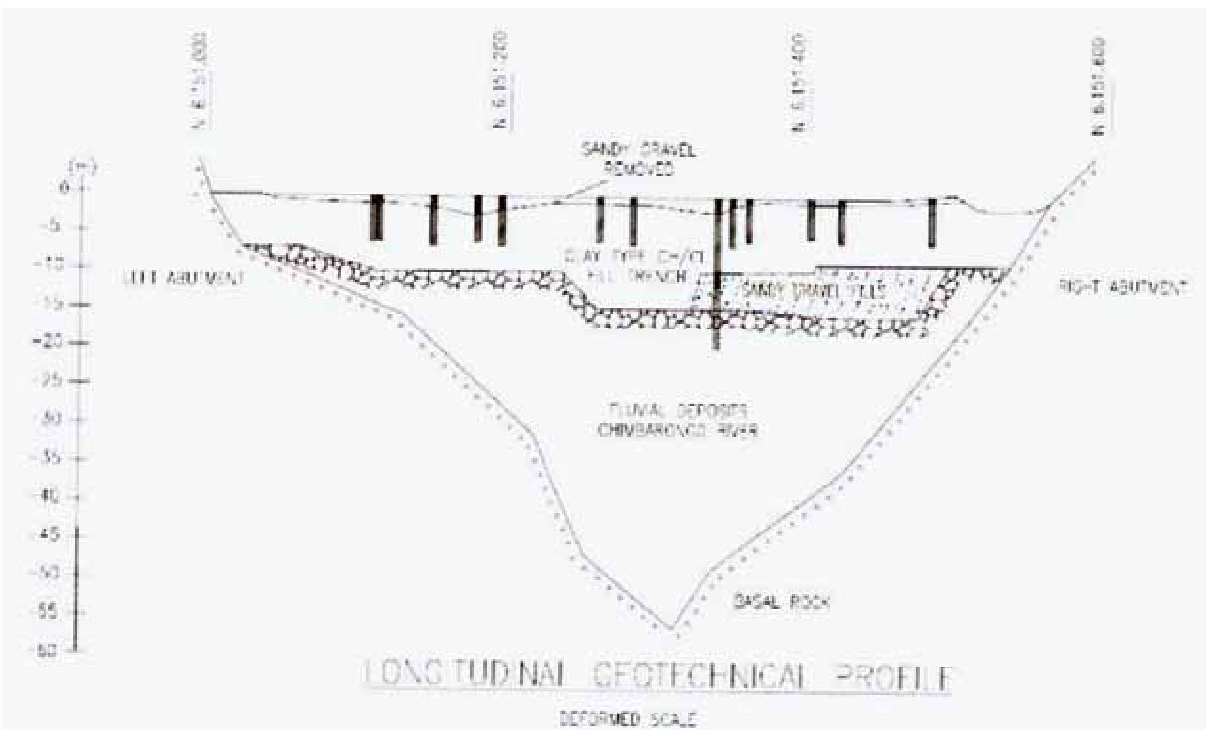

Figure 4. Foundation profile (from Campana et al., 2011).

Figure 5. Depression in crest (from Noguera, 2010).

ISHIBUCHI DAM, JAPAN

Ishibuchi Dam was 11 km from the epicenter of the M_w 6.9 Iwate-Miyagi Nairiku Earthquake in 2008 and suffered only minor damage. Accelerometers on dam recorded the strong motions and detailed studies have used the accelerations and deformation measurements to explain the response of the dam to the earthquake. The dam was also shaken by the March 11, 2011, M_w 9.0 Tohoku Earthquake.

ISHIBUCHI DAM

Ishibuchi Dam, a 53-meter-high concrete faced rockfill dam (CFRD) (Figures 1 and 2), has a crest elevation of 323 meters and crest length of 345 meters. It impounds a 16.2 million m^3 reservoir on the Kitakami River for flood control, power generation and water supply. The watershed above the reservoir is 154 km^2. The spillway is located at the left abutment. It has radial gates that are about 6 meters high. When completed in 1953, Ishibuchi was the first CFRD constructed in Japan (JCOLD 2009).

The 13.5 m^3 embankment (Figure 3) is primarily dumped rockfill. However, the outer 5 meters of the upstream slope and 1.5 meters of the downstream slope are masonry rock. The average size of the masonry rock is about 1 meter. The upstream slope varies from 1.2:1 to 1.4:1 and the downstream slope from 1.4:1 to 1.5:1. (Matsumoto et al., 2011 B) The rockfill was dumped from a railroad trestle (Figure 4). The concrete piers that supported the trestle remain in the dam. The face slab consists of 10 meters by 10 meters reinforced concrete panels, 40, 50 or 60 cm thick depending on the reservoir depth.

EARTHQUAKES

The 2008 Iwate-Miyagi-Nairiku Earthquake (M_w 6.9) struck the northern part of Japan. (Midorikawa et al., 2008) The earthquake produced many strong-motion records including one with peak vertical acceleration of 4 g. Twenty-three people were killed or missing, 450 were injured, and about 2,000 houses were damaged. The earthquake occurred in a less-populated mountainous area, and the resulting building damage was rather small in comparison with observed strong-motion records with high accelerations. However, many landslides and slope failures cause extensive damage to roads and isolated people in the mountainous area.

The earthquake occurred in a region of convergence between the Pacific Plate and the Okhotsk section of the North American Plate in northern Japan, where the Pacific plate is moving west-northwest with respect to the North American plate at a rate of approximately 8.3 cm/yr. The hypocenter of the earthquake indicates shallow thrusting motion in the upper (Okhotsk) plate, above the subducting Pacific plate, which lies at approximately 80 km depth at this location. The fault rupture plane dips a shallow angle to the west. The focal depth was about 8 km. Some studies suggest that the dam is on the hanging wall of the fault.

The M_w 9.0 Tohoku, Japan Earthquake occurred on March 11, 2011, at the plate boundary, near the northeast coast of Japan. The earthquake and tsunami caused enormous damage to Japan. As of April 8, 2011, 12,731 people were found dead, 14;706 people were still missing and 216,818 houses or buildings were damaged or destroyed. Fukushima No.1 nuclear power station was severely damaged (Matsumoto et al., 2011A).

EMBANKMENT PERFORMANCE DURING THE IWATE-MIYAGI-NAIRIKU EARTHQUAKE

The reservoir was about 4 meters below normal pool during the earthquake.

The accelerometers are located in a drainage gallery under the dam, on the downstream face and on the downstream edge of the crest. The accelerometer under the dam malfunctioned during the main shock, but recorded aftershocks. The foundation acceleration time history was calculated by using the spectral ratios of the foundation and downstream face records of the aftershocks. The peak accelerations measured or computed were:

	Up/downstream	Cross channel	Vertical
Downstream face	1.41g	2.14g	1.78g
Crest	1.49	0.94	2.11
Foundation	0.47	0.67	0.63

There is a concern that movement of large rocks on the face (See next paragraph.) may have affected the crest acceleration recording (Matsumoto et al., 2011 B).

As shown in Figure 5, the earthquake motions formed 50 cm high waves in the dam crest pavement (Yamaguchi and Iwashita 2008). The phenomenon was caused by the embankment settling and the concrete piers from the construction trestle not yielding. Figures 6 and 7 show damage caused by movement of some large rocks in the upper part of the downstream face, as would be expected from the high accelerations. The maxim earthquake caused crest settlement was about 55 cm. The embankment had settled a maximum of about 43 cm by 2007. Figure 8 shows the seismically induced movements at five sections along the dam.

Fifteen GPS sensing points were installed on the embankment after the earthquake and settlement measurements were taken for several months (Yamaguchi et al., 2010). They show the dam continued to settle a small amount. When the GPS measurements were compared with pre-earthquake survey results, they indicated the embankment deflected under the load of the reservoir as it was filled more that it did before the earthquake. This condition continued for about a year.

The leakage measured in the riverbed increased from 800 to about 1500 gallons/minute as a result of the earthquake and some turbidity was noticed. The leakage stabilized in a few days and tracked the reservoir level. The increased leakage was about one-third less than the leakage measured for the same reservoir levels after initial filling in 1954. A diver inspected the upstream face after the earthquake and did not find any damage to the slabs or the joints (Matsumoto et al., 2011 B).

EMBANKMENT PERFORMANCE DURING THE TOHOKU EARTHQUAKE

The earthquake epicenter was 204 km from the dam. The peak horizontal accelerations measured on a downstream terrace and at the embankment crest were 0.18 g and 0.61 g, respectively. The leakage increased from 500 to about 750 gallons per minute. The maximum crest settlement was 0.5 inches (Yamaguchi et al., 2012).

SPILLWAY EARTHQUAKE PERFORMANCE

The radial gates were not damaged by and functioned after the Iwate-Miyagi-Nairiku Earthquake. There was about 2.4 meters of water against them. There was minor cracking on the piers. (Matsumoto 2011 B)

No spillway damage was reported after the Tohoku Earthquake.

POST EARTHQUAKE STUDIES

During the Iwate-Miyagi-Nairiku Earthquake, the dumped rockfill dam only settled about 1% of its height, even though it was severely shaken, and Figure 9 shows that deformations of the downstream face were significantly larger that of the upstream face. Dynamic analyses of the residual deformation of the dam were performed using program FLAC in order to understand these occurrences (Yoshida el al., 2011). Analysis runs with and without the masonry rock demonstrated that those zones were very important in the earthquake performance of the dam.

CONCLUSIONS

This is a very interesting case history, but extrapolating the earthquake performance of Ishibuchi Dam to other rockfill dams must be done with caution. As shown in reference Yoshida et al., the hand placed rock on the outer slopes had a significant effect on the performance. The concrete piers that supported the trestle remain in the dam and probably reduced the settlement that would have otherwise occurred. The face slab's horizontal joints likely helped keep it from cracking. The dam is not nearly as high as concrete faced rockfill dams that are being constructed today.

The excellent performance of the appurtenant structures should not be over looked.

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Figure 1. Ishibuchi Dam (from JCOLD 2009).

Figure 2. Spillway – upstream face – outlet tower (from Yamaguchi, 2010)

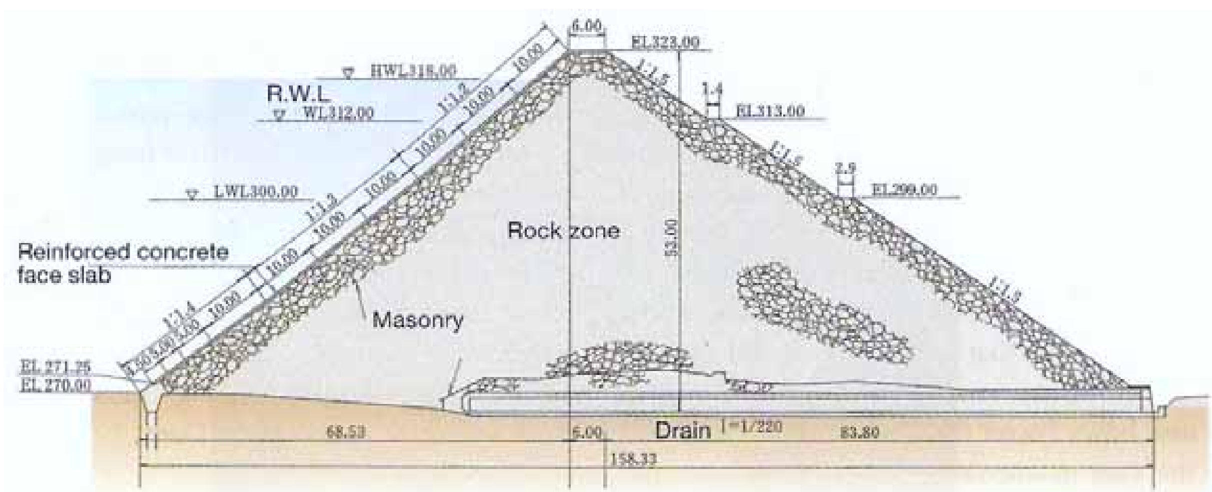

Figure 3. Embankment – maximum section (from JCOLD 2009).

Figure 4. Rockfill construction – note pier and spillway wall (from JCOLD 2009).

Figure 5. Dam crest – note waves in pavement (from Yamaguchi 2010).

Figure 6. Deformation at downstream edge of crest. (from Yamaguchi, 2010).

Figure 7. Displaced rock on downstream slope (from Yamaguchi, 2010).

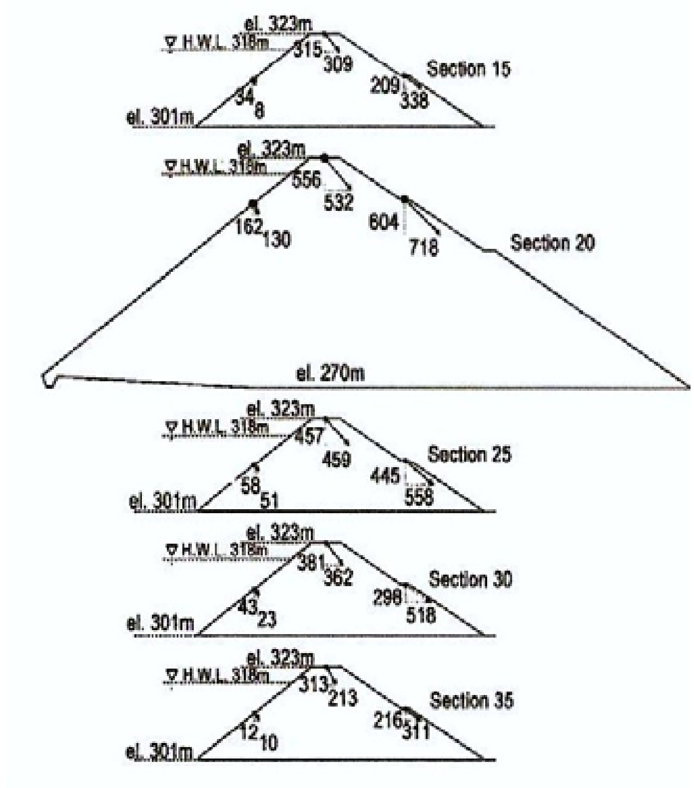

Figure 8. Movements in millimeters at five sections (from Matsumoto et al., 2011).

KASHO, SUGESAWA AND UH DAMS, JAPAN

On October 6, 2000 at 1:30 p.m., the Western Tottori Prefecture in Japan experienced a magnitude M_w 6.7 earthquake. This section discusses three concrete gravity dams shaken by this earthquake: Kasho, Sugesawa, and Uh Dams. This case history is based on references Omachi 2000, Takasu et al., 2000 and Yamaguchi et al., 2002.

THE EARTHQUAKE

The Western-Tottori Prefecture M_w 6.7 Earthquake occurred at 1:30 p.m. on October 6, 2000, at a focal depth of 7 miles located at 35° 16.5' north latitude and 133° 21.0' east longitude. Fortunately no people lost their lives, but 132 people were injured, 28 homes completely destroyed, 82 homes half destroyed, and 5,050 homes partially destroyed. After the earthquake, 180 dams were inspected. Numerous aftershocks were recorded with one magnitude 5 and three greater than 4 (see Figure 1). Figure 2 is a close up map showing the epicenter and the locations of Kasho, Sugesawa and Uh Dams. Measurements indicated that the upper end of Kasho reservoir dropped about 7.9 inches (20 cm) and the reservoir at the dam dropped 2.4 inches (6 cm). Figure 3 shows how the ratio of vertical to horizontal peak acceleration varies with horizontal distance from the earthquake fault, the ratio getting smaller with distance. The epicenter of the earthquake was about 1.9 miles (3 km) from the dam.

KASHO DAM AND OBSERVED PERFORMANCE

Kasho Dam is a concrete gravity dam built in 1989 with a structural height of 152 feet (46.4 meters), a crest width of 25 feet (7.5 meters) at elevation 408 feet (124.4 meters), a base width of 117 feet (35.6 meters), and a crest length of 571 feet (174 meters) (see Figures 4 to 7). The reservoir has a normal elevation of 387 feet (118 meters) and was at elevation 368.1 feet (112.2 meters) during the main earthquake.

Peak accelerations measured in the lower inspection gallery of the dam at elevation 285.4 (87 meters) were 0.54g (N-S), 0.54g (E) and 0.49g (V) and at an elevator shaft at the crest elevation 408 (124.4 meters) were 2.09g (N-S), 1.43g (E), and 0.90g (V) (see Figure 8). Figure 7 shows the locations in the dam where ground motions were recorded. The acceleration response spectra at 5 percent damping of the motions measured in the inspection gallery are shown in Figure 9.

Using acceleration measurements from the main shock and aftershocks, the vibration period of the dam showed a noticeable transient increase first and a decrease next with periods of 0.84, 0.96, 0.92, and 0.87 seconds in the upstream to downstream direction. During the main shock, the peak period initially lengthened then shortened, the change exceeding 10% and was observable in the upstream-downstream direction only. It is believed there is high probability that the change in the period resulted from non-linearity of the hydrodynamic pressure acting on the upstream face of the dam. From plumb line readings in the dam, the main shock induced a relative displacement of -0.1 inches (–2.8 mm) toward the right abutment and 0.03 inches (0.7 mm) in the upstream direction, with a resultant displacement of 0.11 inch (2.9 mm).

There was essentially no damage to the dam body of Kasho Dam, but there was minor damage to other structures. The walls and base of the control house, a cantilever structure projecting upstream from the dam crest, were cracked. The mountain side slope of the road on the right bank slid. Pavement settled around the management office building.

Linear elastic 2-dimensional dynamic finite element analyses were used to back calculate stresses in the dam. The reservoir water was assumed to be incompressible and at the same level as during the earthquake. Measured ground motion time histories were converted from N-S and E-W into the stream N20E direction for the analyses. Material properties used were a modulus of elasticity of 4,350,000 lb/in^2 (30,000 MPa), a Poisson's ratio of 0.2, and a density of 150 lb/ft^3 (2.3 t/m^3). A damping ratio of 10 percent was determined by comparing the measured accelerations at the top of the dam to the calculated accelerations at the same location. Maximum static plus dynamic tensile stresses of 216 lb/in^2 (1489 kPa) were computed at the heel of the dam.

SUGESAWA DAM AND OBSERVED PERFORMANCE

Sugesawa Dam is a concrete gravity dam built in 1968 with a structural height of 241 feet (73.5 meters) (see Figure 10).

Peak accelerations measured in the lower inspection gallery of the dam were 0.16 g (upstream/downstream), 0.13g (cross-canyon), and 0.11g (vertical).

There was insignificant damage to Sugesawa Dam and minor damage to the surrounding structures. A concrete spall of about 3.3 feet (1 meter) long and 1-foot (0.3-meter) wide chipped from the downstream face of the dam (see figure 11). Drainage from the dam foundation slightly increased immediately after the earthquake, but later stabilized. The walls and floor of the guard house cracked. A garage on the right bank downstream from the dam had a V-shaped 330-foot (100-meter-) long crack around the garage and cracking of a wall. The left bank slope upstream from the dam had a V-shaped crack. The management office building on the right bank had cracking on the surface of a wall.

UH DAM AND OBSERVED PERFORMANCE

Uh Dam is a concrete gravity dam with a height of 46 feet (14 meters) and a crest length of 112 feet (34 meters) (see Figure 12). Figure 13 shows its standard cross section. It is located about 0.6 miles (1 km) from the fault. During the earthquake, the reservoir water level was about 6.6 feet (2 meters) below the full water level. The Hino Observation Station of the Digital Strong-Motion Seismograph Network is located 66 feet (20 meters) from Uh Dam. At this location, seismographs installed at the surface (S wave velocity of 690 ft/sec (210 m/s)) and the underground (328 feet (100 meters)) below the surface (S-wave velocity of 2590 ft/sec (790 m/s)) recorded earthquake motion with a peak accelerations of 1.16g and 0.62g , respectively. Although the dam was subjected to this strong earthquake motion, the only damage to the dam was cracking with a width of 0.4 to 0.8 inches (1 to 2 cm) on the spillway channel near the foot of the downstream slope (see Figure 14).

CONCLUSIONS

Kasho Dam, a 152-foot- (46.4 meter-) high concrete gravity dam, was in the near field of the 2000 Western Tottori earthquake (M_J 7.3). Strong-motion acceleration at the dam during the main shock exceeded 2.04g at the top and 0.51g in the lower inspection gallery. Despite these large accelerations, the dam survived the earthquake without serious damage. The vibration period of the dam, however, showed a noticeable transient increase first and a decrease next. The change in the period was more than 10% during the main shock, and was observable in the upstream-downstream direction only. For these reasons, there is high probability that the change in the period resulted from non-linearity of the hydrodynamic pressure acting on the upstream face of the dam.

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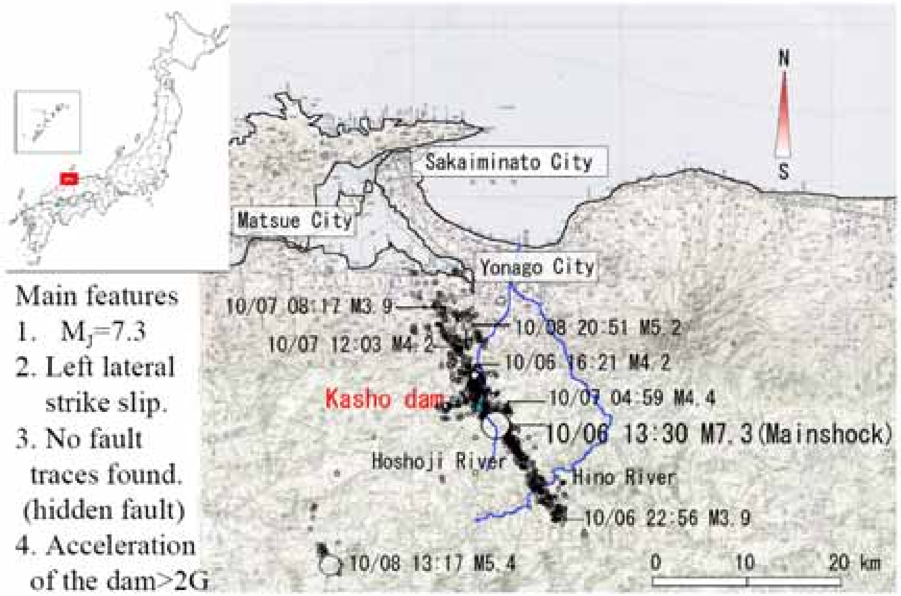

Figure 1. Kasho Dam in relation to main shock and aftershocks (from Ohmachi, 2007).

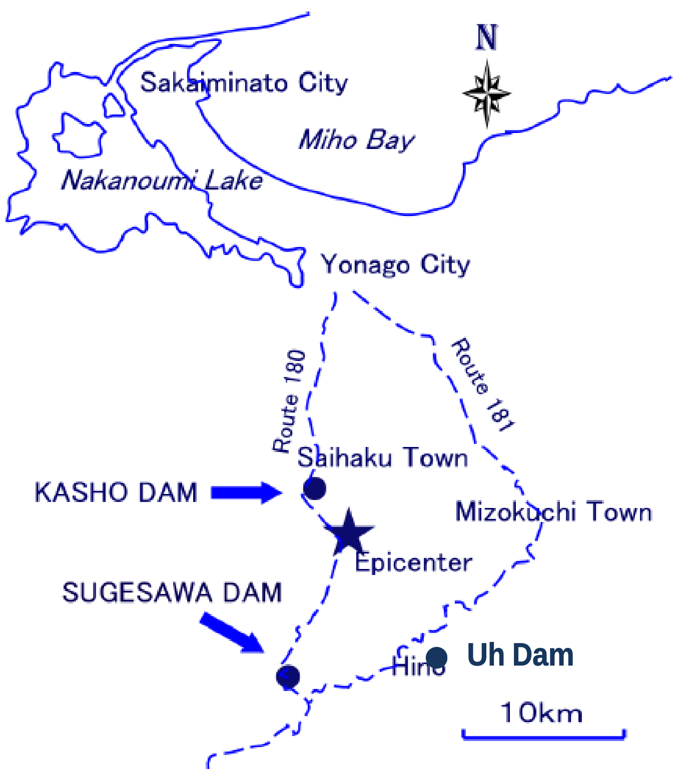

Figure 2. Close up of Kasho Dam, Sugesawa Dam and Uh Dam
and seismic activity (from Yamaguchi et al., 2002).

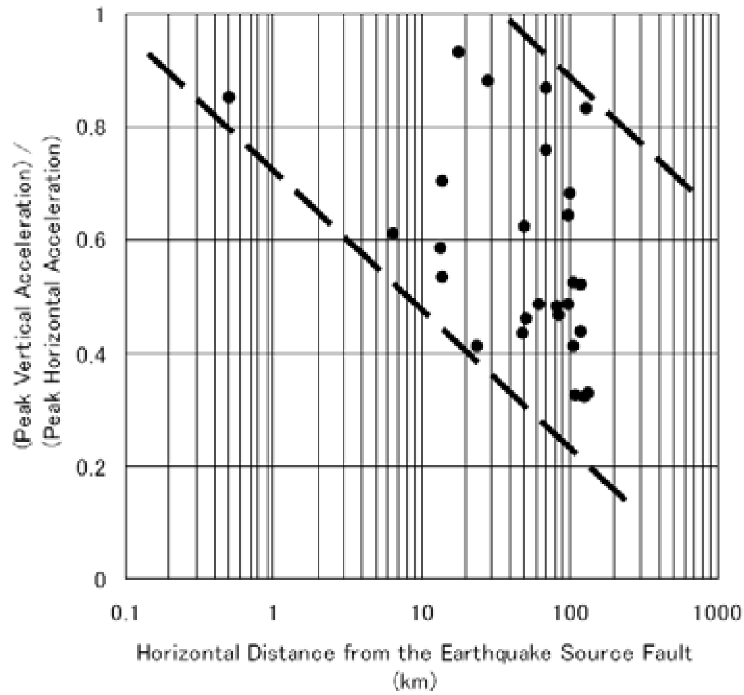

Figure 3. Ratio of vertical to horizontal peak accelerations vs. horizontal distance from the earthquake fault (from Yamaguchi et al., 2002).

Figure 4. Aerial view of Kasho Dam (from Yamaguchi et al., 2002).

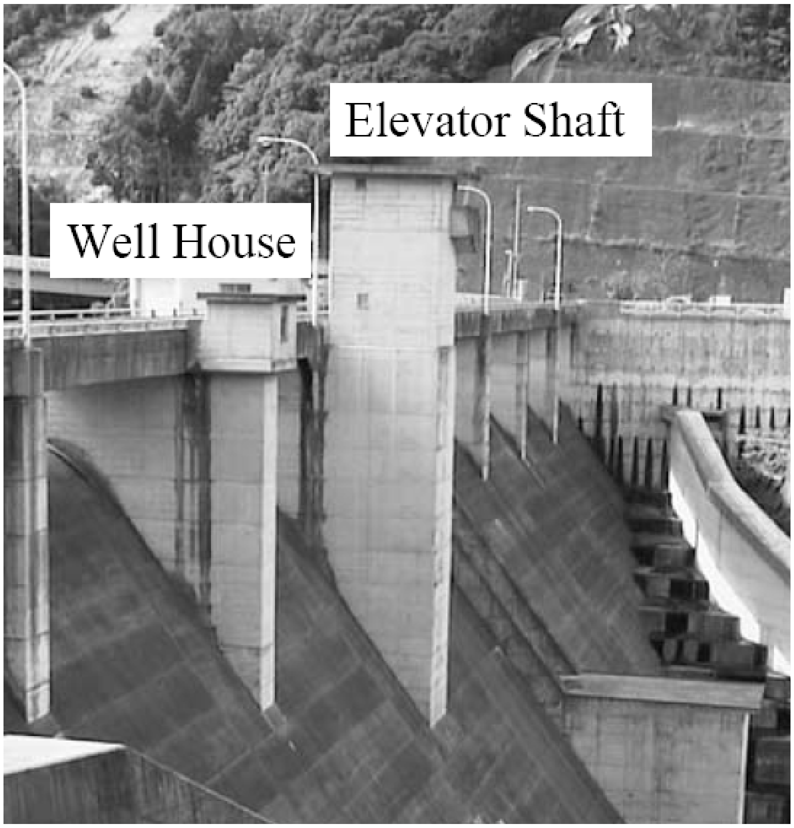

Figure 5. Downstream view of Kasho Dam (from Ohmachi, 2007).

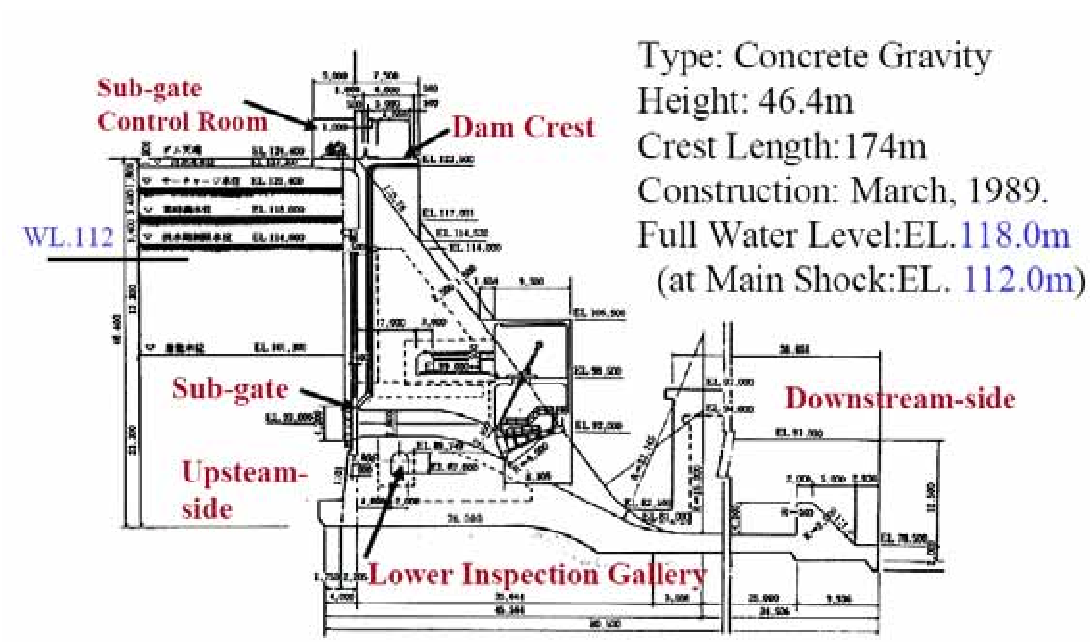

Figure 6. Section view of Kasho Dam (from Ohmachi, 2007).

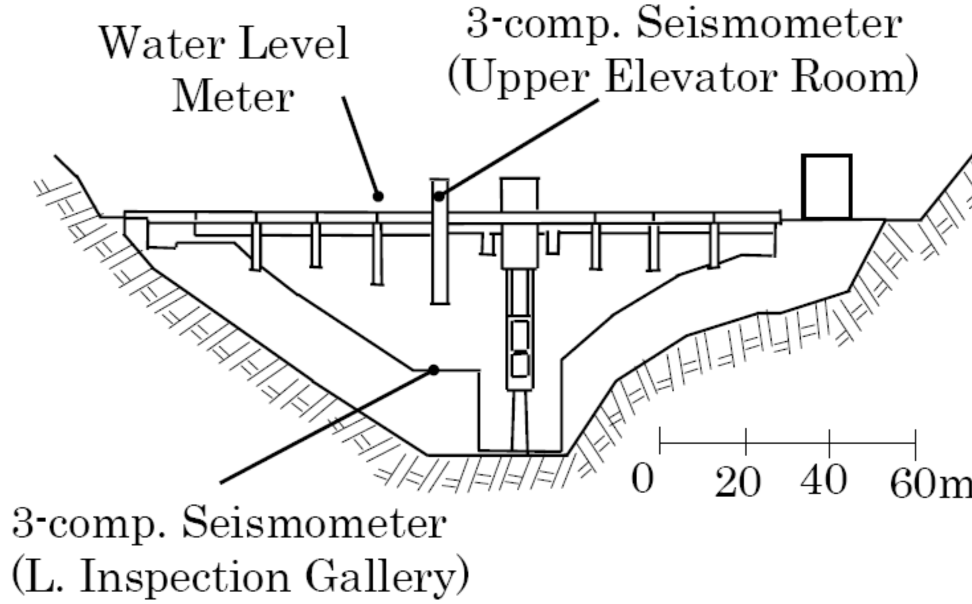

Figure 7. Profile of Kasho Dam (from Ohmachi, 2007).

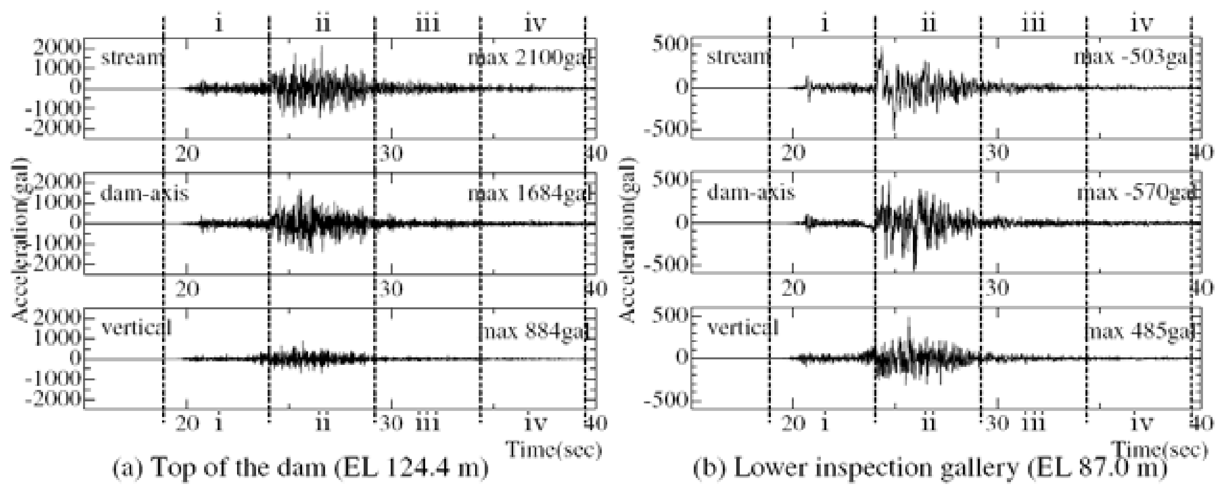

Figure 8. Acceleration time histories measured at Kasho Dam
(from Yamaguchi et al., 2002)

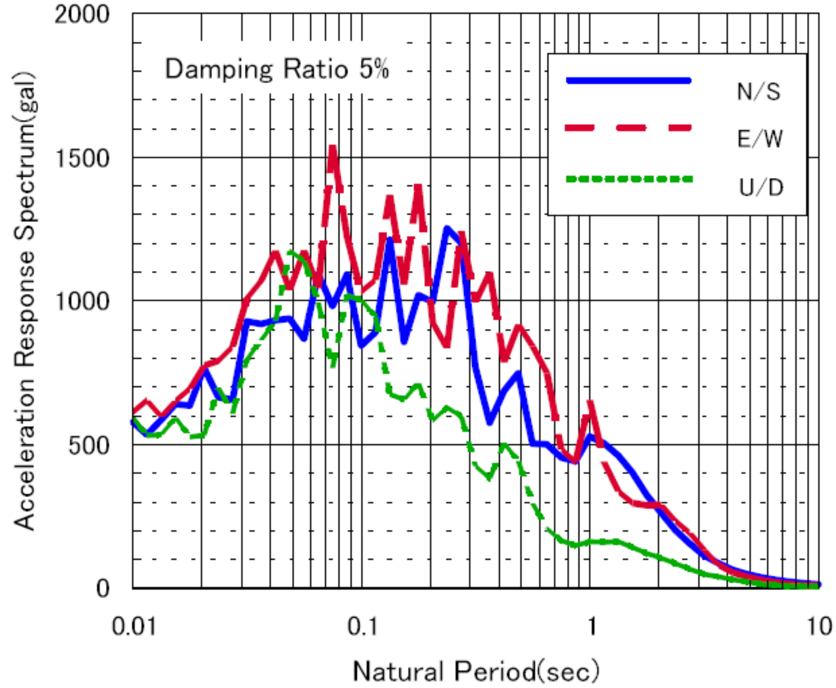

Figure 9. Acceleration response spectra at 5% damping computed from measured accelerations in the lower gallery (from Yamaguchi et al., 2002).

Figure 10. Aerial view of Sugesawa Dam (from Yamaguchi et al., 2002)

Figure 11. Concrete spall on the downstream face of Sugesawa Dam (from Yamaguchi et al., 2002).

Figure 12. Uh Dam (from Takasu et al., 2000).

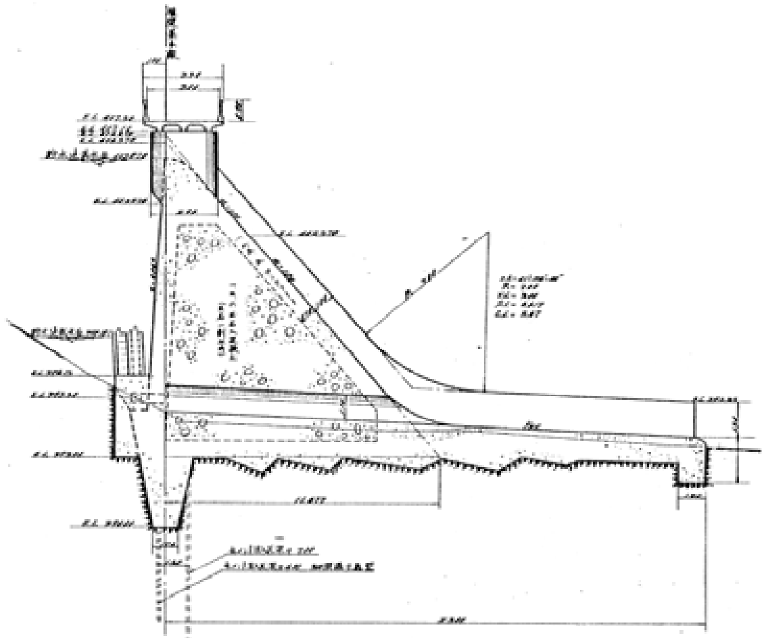

Figure 13. Cross-section of Uh Dam (from Takasu et al., 2000).

Figure 14. Cracking of the spillway channel wall of Uh Dam
(from Takasu et al., 2000).

KITAYAMA DAM, JAPAN

INTRODUCTION

Kitayama Dam is one of five dams forming the Kitayama Reservoir. The reservoir is situated at the top of a hill to the northeast of the city of Kobe, Japan. The region was shaken by the January 17, 1995, M_w 6.9 Kobe Earthquake. The dam sustained large shallow residual deformation of the upstream slope below the water level at the time of the earthquake. Extensive post-earthquake field investigations, laboratory testing of embankment materials and numerical analyses have been performed. The results of the study were able to qualitatively simulate the observed behavior and damage pattern of the dam. This case history is based on references EERC, 1995 and Sakamoto et al., 2002.

KITAYAMA DAM

Kitayama Dam is the largest of the dams forming the reservoir. The dam is a rolled earthfill with a height of 25 meters and was completed in 1968. The dam zones are of primarily decomposed granite of the Rokko formation, Figure 1. It was well compacted in 30 cm lifts with control tests of the degree of compaction, D-value, of over 100%. The dam is founded on the Rokko granite formation.

Following the earthquake, a ground inspection of the dam revealed no observed damage and the embankment appeared to have generally performed well. However, an aerial reconnaissance discovered what appeared to be significant longitudinal cracking along approximately 100 meters of the upstream slope of the embankment. The cracking was the top of a shallow sliding block slope failure. At the time of the earthquake, the reservoir level was at the top of the failure. Reservoir drawdown revealed a 1- to 1.5-meter scarp and bulging at the toe, Figure 2. Test pits were excavated into the slope to determine the condition of the embankment and to obtain samples for testing. Beneath the riprap and gravel bedding was a loose layer of rolled embankment, which was followed by an extremely loose layer, the sliding layer. Unaffected embankment was found below the sliding layer, Figure 3. The sliding failure zone had a depth of 1.5 to 2 meters, Figure 4. Undisturbed samples of the embankment above the sliding block failure were not particularly loose.

EARTHQUAKE

The Kobe (Hyogoken-Nanbu) Earthquake was assigned a M_W of 6.9. Its focal depth was estimated at 14 km, a shallow earthquake. The epicenter is located just off of the northeast tip of Awaji Island, on the Nojima Fault, about 20 km southwest of downtown Kobe and 33 km southwest of Kitayama Dam, Figure 5. The earthquake was estimated to have induced peak ground accelerations between 0.2g and 0.4g in the vicinity of the Kityama Reservoir. No seismograph was located at the dam.

INVESTIGATIONS

After the earthquake, in-situ tests were performed including PS logging, density logging and micro-tremor observations. Undisturbed samples from shallow, mid and deep zones of the dam were recovered. Tests performed in the laboratory included core density, permeability, drained monotonic loading triaxial, undrained cyclic loading triaxial, drained cyclic torsonial simple shear deformation and others. Extensive numerical analyses were performed. Two input motions were used in the analyses. One simulated by the statistical Green's function approach and the other the record observed at Kobe University during the earthquake. The peak acceleration of the simulation input motion is 0.22g and has a duration of 14 seconds. The Kobe University motion is 0.28g and 20 seconds duration. The results of both input motions showed similar deformations, with the longer duration Kobe University motion showing larger deformations. Horizontal deformations were the greatest at the toe and vertical deformations were the greatest at the top of the sliding block.

CONCLUSIONS

The Kobe Earthquake caused shallow sliding of the upstream slope below the water level. The numerical analysis was able to qualitatively simulate the behavior and damage pattern of the dam.

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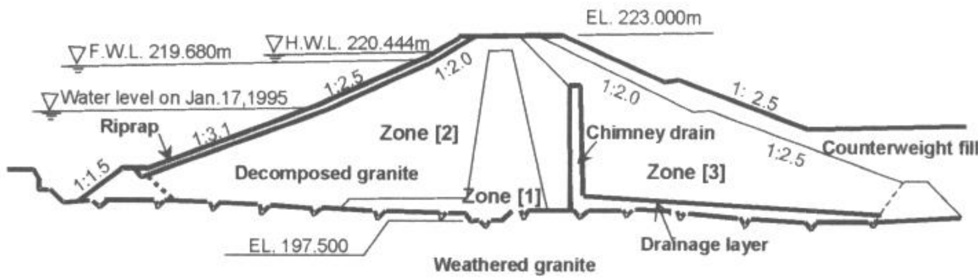

Fig.1 Cross section of the Kitayama Dam

Fig.2 Over view of damage to the Kitayama Dam due to the Kobe Earthquake

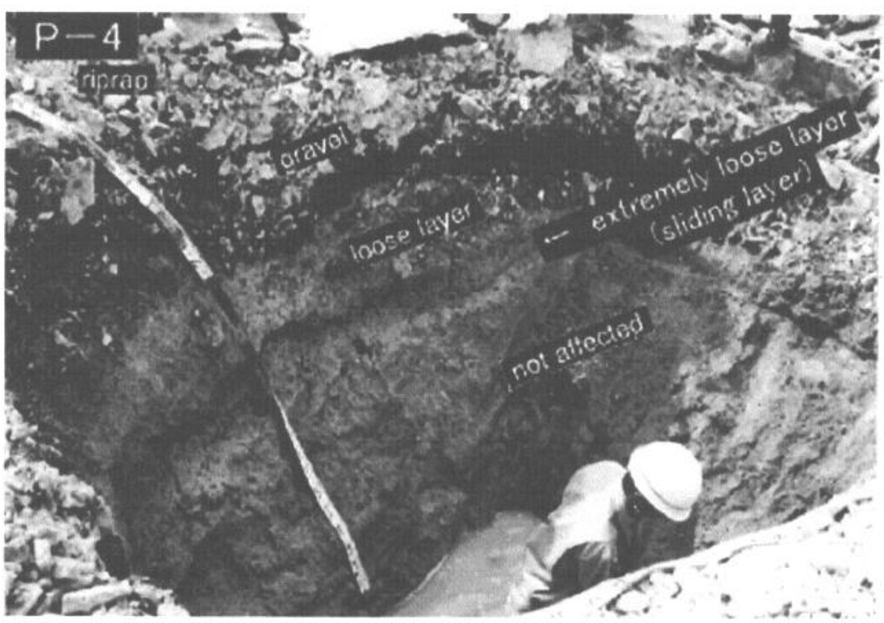

Fig.3 View of pit (P-4) excavated at the failure zone of the dam

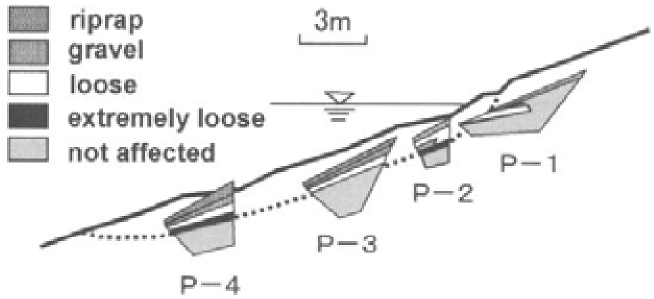

Fig.4 Cross section of the sliding failure by pit excavation investigation

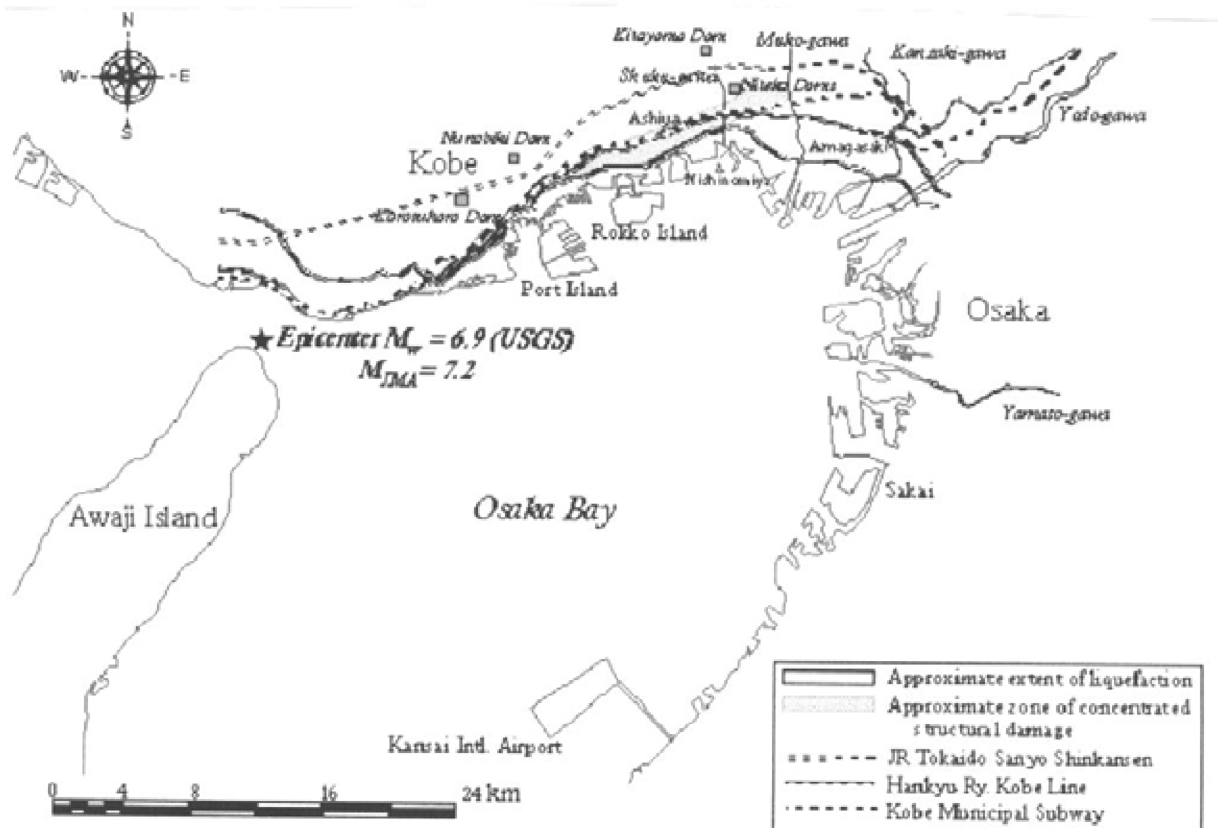

Fig. 5 Location map

(Figures 1-4 from Sakamoto et al., 2002. Figure 5 from EERC, 1995.)

MELADO DAM, CHILE

Minor transverse and longitudinal cracking was found in the crest of embankment of Melado Dam after the February 27, 2010, Maule, Chile Earthquake.

MELADO DAM

Melado Dam is a 90-meter-high, 310-meter-long central core embankment dam, with gravel shells (Figure 1). The core is clayey gravel. The shells were compacted to 85% relative density and the core was compacted to 95% of Standard Proctor. The 3.65 million m^3 embankment is founded on a 50 to 75meter (or more) thick coarse gravel fluvial deposit. There is a cutoff wall through the fluvial deposit. The upper part of the wall is reinforced. The canyon walls are steep and the embankment is contiguous to the reinforced concrete spillway on the right abutment. (Nogurea; 2010, Rodriguez-Roa, 1992)

The dam is on the Melado River 40 km upstream of Cobun Dam in central Chile. It was completed in 1991, is owned by Empresa Electrica Pehuenche S.A. and is used for generation of 500 MW of hydro-electric power. (Vinci)

MAULE EARTHQUAKE

On February 27, 2010 at 03:34 a.m. local time, a powerful earthquake of M_w 8.8 struck central Chile (Elnashai et al., 2010) The epicenter of the earthquake was approximately 8 km off the central region of the Chilean coast. With an inclined rupture area of more than 80,000 square km that extends onshore, the region of Maule was subjected to a direct hit, with intense shaking of duration of at least 100 seconds, and peak horizontal and vertical ground acceleration of over 0.6 g. The earthquake caused the death of 521 persons, with almost half of the fatalities caused by the consequential tsunami. Over 800,000 individuals were directly affected through death, injury and displacement. More than a third of a million buildings were damaged to varying degrees, including several cases of total collapse of major structures.

EARTHQUAKE PERFORMANCE

These comments on dam performance are based on reference Nogeura, 2010.

The dam is 201 km from the epicenter of the earthquake. The peak ground acceleration at the dam was estimated to be 0.15g by using the Martin y Saragoni 2005 relationship.

Transverse cracks "4 to 5 cm thick" were reported on the crest at the abutments. Transverse cracking in asphalt pavement at abutments is common in embankment dams with steep abutments (Figure 2) and a small crest length to dam height ratio, 3.4:1.

There was longitudinal cracking on upstream side of the crest in the highest part of the dam (Figure 3). By comparing Figures 1 and 3, it appears that the crack is along the upstream edge of the core and was most likely caused by the clayey gravel core settling less than the cohesionless embankment materials upstream of it.

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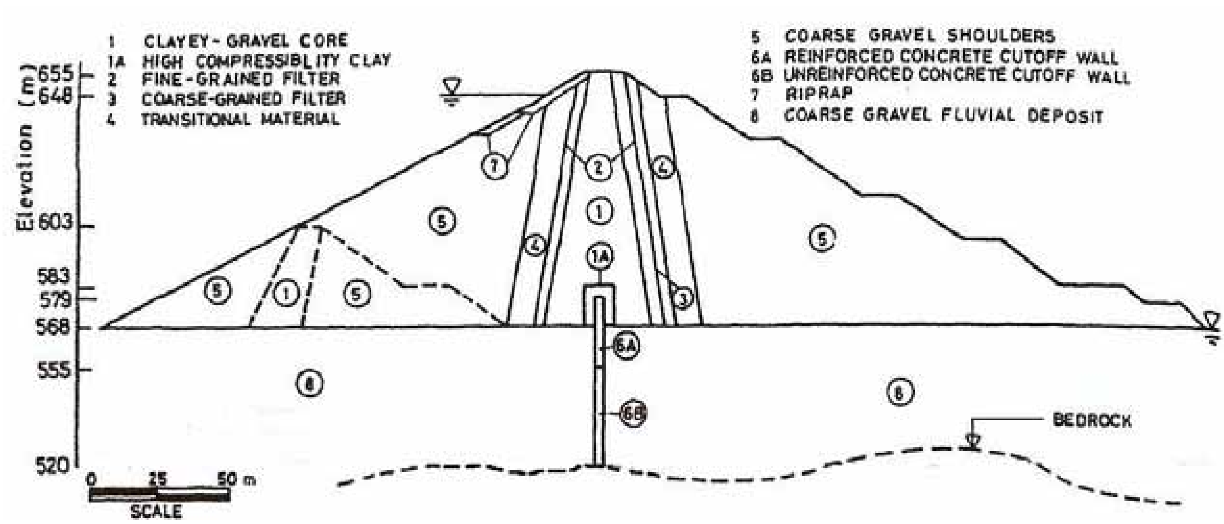

Figure 1. Typical section of embankment (from Rodriguez-Roa, 1992)

Figure 2. Photograph from downstream (from Vicini).

Figure 3. Longitudinal cracking on upstream side of crest (from Noguera, 2010)

SHAPAI DAM, CHINA

Shapai Dam was moderately to severely shaken by the May 12, 2008, Wenchuan Earthquake. There was no damage to the dam body and only minor damage to its appurtenant structures, except for the power plant. This cased history is primarily based on reference Babbitt and Charlwood 2009

SHAPAI DAM

Shapai Dam is the highest roller compacted concrete arch dam in the world at 132 meters and was completed in 2003 (Figures 1 and 2). It forms a 18 million m^3 hydroelectric reservoir on the Chaopo River, a tributary of the Minijiang River, in Sichuan Province.

The dam is a three centered arch and has a canyon width to height ratio of 1.7 to 1. The design of the dam was cautious. It has a thick section, 9.5 meters wide at the crest and 28 meters wide at the base, producing low static stress and only one small area of computed tension. There are 4 inclined transverse joints, 2 contraction and 2 induced. The joints were formed by placement of precast concrete elements and were grouted. The 383,000 m^3 of RCC in the dam was placed in one year and attained a 90-day strength of 20 MPa.

The foundation is granodiorite. Schistose rock was removed to provide an unyielding foundation, which was blanket and curtained grouted. Some conventional mass concrete was placed in the streambed area. Tensioned cable anchors were installed to add stability to some areas of the abutments.

A power tunnel starts in the right abutment. Three outlet gate towers for the tunnel are located on the upstream right abutment (Figure 3).

WENCHUAN EARTHQUAKE

The May 12, 2008 Wenchuan Earthquake was M_w 8 event. The epicenter was in Sichuan Province in southwest China at the foot of the Tibetan Plateau, where the plateau collides with the Sichuan basin. The earthquake started on a west dipping thrust fault and became a strike/slip event as it broke 270 km northeastward on the Longmenshan fault.

The duration of strong ground motion was up to 120 seconds at sites underlain by deep alluvium. There were about 300 aftershocks, some over M 6. There were very few strong motion recorders operating in the earthquake area, so the estimates of peak ground accelerations are judgments considering observed damage and distances to faults. More than 80,000 people were killed by rockfalls and structure collapses. 2266 of the 35,601 reservoirs in Sichuan and 7 nearby provinces were damaged. There were 69 dam breaks and 331 other highly dangerous situations. Ninety-five percent of the damaged reservoirs were formed by small earth dams.

EARTHQUAKE PERFORMANCE

The dam was 29 km from the fault surface break, on the hanging wall of the fault. The estimated peak ground acceleration (pga) is 0.25 to 0.5g. The reservoir was near normal pool level at the time of the earthquake and was drained following the earthquake. Damage was confined to the gate hoist structure and elevator shaft building (Figures 4 and 5). The gates remained operable. No cracking or damage to the main dam body was observed. The operator's home was severely damaged and the access road was blocked by rockfalls.

The pga for the original design was 0.13g. Subsequent detailed numerical modeling indicated that a pga of 0.27g would cause the joints to open and higher ground motions could cause failure in the dam body. The actual performance did not produce either result. (Wang and Shoa 2009)

SHAPAI POWER PLANT

Shapai Power Plant is a two unit, 36 MW power plant located in the steep, narrow Caopo River canyon, 5 km downstream of the dam.

The power house, control center buildings and operators homes were bombarded by rockfalls. The plant roof and walls were damaged and the plant filled with rocks, damaging the equipment (Figures 6 and 7). A rockfall ruptured a penstock coupling. The resulting spray was 80 meters high and caused more rocks to fall. The powerhouse was flooded. Falling rock heavily damage a tailrace flume. Little repair had been completed on these remote facilities when the photographs were taken 10 months after the earthquake.

In such terrain it is clearly necessary to consider to the potential for large rock falls and landslides. These can be very dangerous to personnel and can cause extensive damage to spillways, hoist structures, powerhouses and control buildings. They can make personnel access impossible for a period of time while aftershocks continue.

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Figure 1. Shapai Dam from upstream after the earthquake (Babbitt).

Figure 2. Downstream face (Babbitt).

Figure 3. Outlet tower (Babbitt).

Figure 4. Cracking on gate hoist structure (Babbitt).

Figure 5. Damaged elevator shaft building (Babbitt).

Figure 6. Inside Shapai Powerhouse (Babbitt).

Figure 7. Repaired penstock (Babbitt).

SHIN-YAMAMOTO, YAMAMOTO, ASAGAWARA DAMS, JAPAN

Shin-Yamamoto, Yamamoto, and Asagawara Reservoirs and Dams store and regulate water for the Shinanogawa Power Station and are supervised by the East Japan Railway Company (Figure 1). These three dams impound water from the Shinano River, the longest river in Japan. Shin-Yamamoto Dam and Yamamoto Dam are located in the City of Ojiya, and the Asagawara Dam is located in the City of Tokamachi, in the Niigata Prefecture, Japan. The power station is used to generate power for the Shinkansen super-express trains. During the conveyance of water to the reservoirs for energy generation, a M_w 6.6 magnitude earthquake forced the power station to immediately suspend operation and each reservoir to perform an emergency discharge of stored water. Due to the stoppage of power supply, the gate for emergency discharge for drawdown of each reservoir could not be opened immediately. As a result, the remaining water in the tunnels flowed into the reservoirs thus water levels became temporarily high before generators could be brought in to open the gate for discharge of water from reservoirs. All three-earth fill dams and the power station were seismically damaged by the Niigata-ken Chuetsu earthquake (also known as the Mid Niigata Prefecture Earthquake) on October 23, 2004. Restoration work on the dams lasted about 18 months and was completed in March 2006. These three dams are further described below in order of distance from the Niigata-ken Chuetsu earthquake epicenter. This case history is based on references JCOLD, 2009; Matsumoto, 2004, Omachi (undated); and Yasuda et al., 2004.

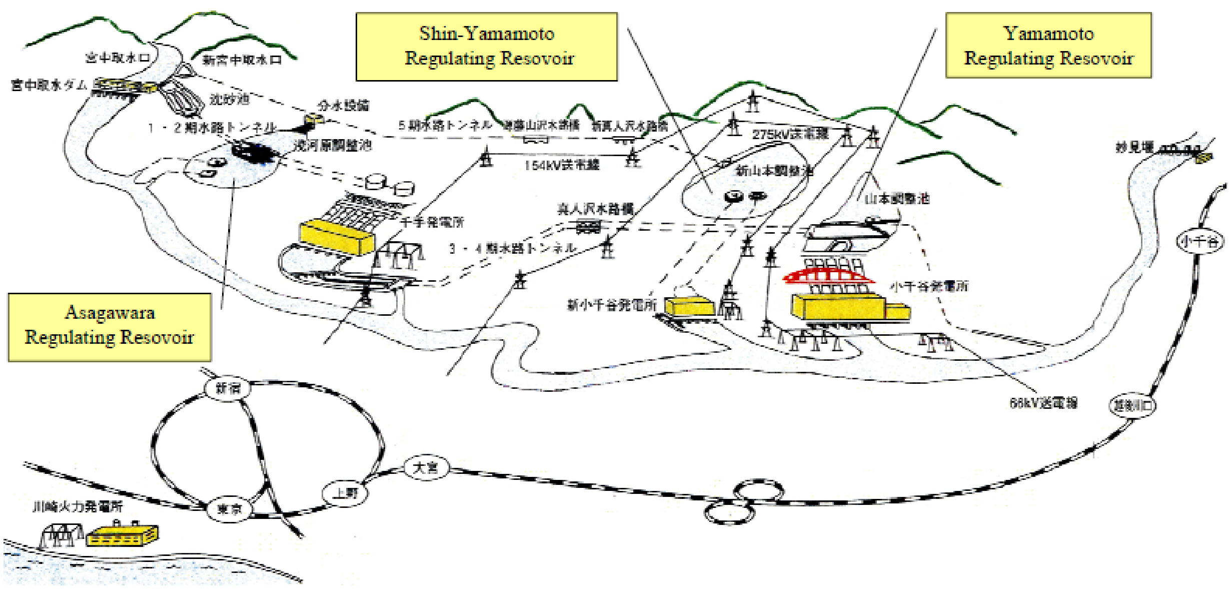
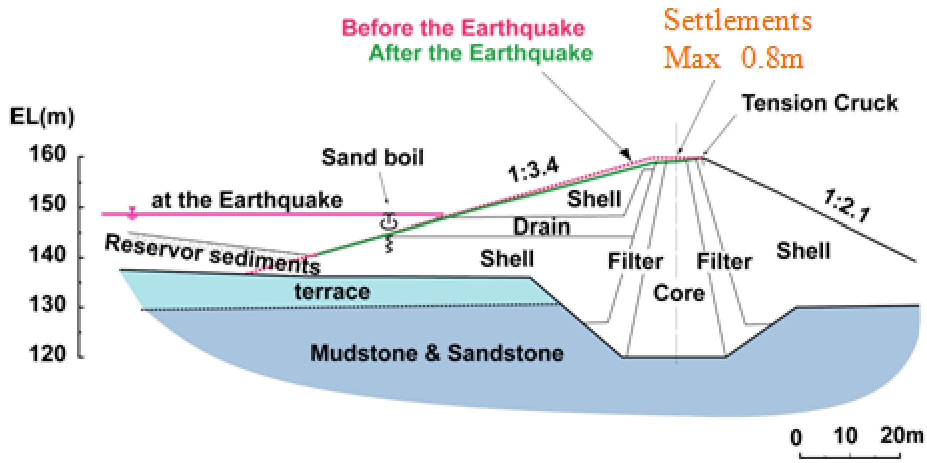


Figure 1. Location of reservoirs in relation to Shinnogawa Power Station
(from Yasuda et al.)

SHIN-YAMAMOTO DAM (ALSO KNOWN AS NEW YAMAMOTO DAM)

The Shin-Yamamoto Dam is located approximately 3.1 miles from the Niigata-ken Chuetsu earthquake epicenter. It is a zoned embankment with shells constructed of terrace materials, an upstream drain system, filters and center core consisting of a mixture of clay, sand, and gravel. The foundation is Plio-Pleistocene sedimentary soft rock. The dam is used for peaking generation, which results in a rapid draw down event every day, which is why this relatively modern dam was designed with an upstream drain system.

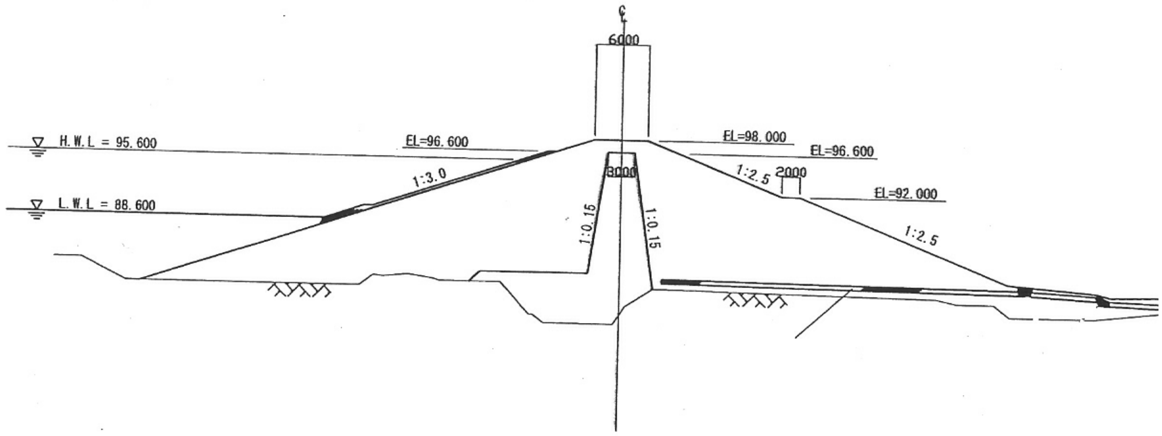


Cross-section of Shin-Yamamoto Dam (from Matsumoto).

The dam axis is of a semi-circular shape. The dam was built in 1990 and has a height of 139 feet with a crest length of 4,567 feet. The dam has instrumentation with various sensors including peizometers, earth pressure cells and differential settlement gauges, and a total of six seismographs with four at the crest and two at the toe of the downstream slope.

YAMAMOTO DAM

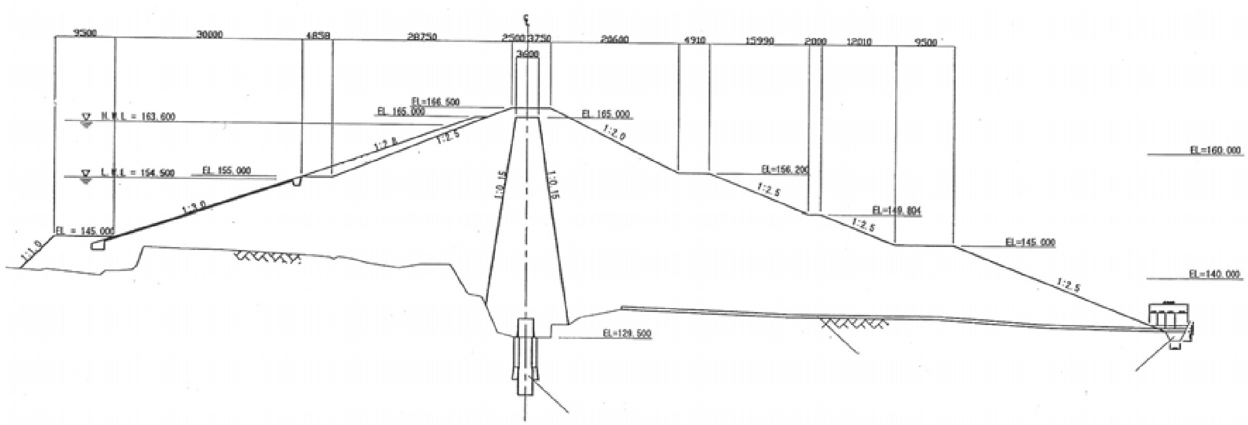
The Yamamoto Dam is located approximately 3.7 miles from the Niigata-ken Chuetsu earthquake epicenter. It is a zoned earth dam built in 1954 and has a height of 92 feet with a crest length of 3,041 feet. The central core zone is filled with mixtures of clay and riverbed sand gravel, and the upstream and downstream shells are filled with terrace deposits and riverbed sand gravel taken from the Shinano River. A concrete cutoff wall with a minimum width of 2.6 feet was embedded to a length of 164 feet beneath a crack in the dam crest on the right bank side.



Cross Section of Yamamoto Dam (from Omachi)

ASAGAWARA DAM

The Asagawara Dam is located approximately 13.7 miles from the Niigata-ken Chuetsu earthquake epicenter. It is a zone type earthfill dam built in 1945 and has a height of 121 feet with a crest length of 958 feet. The core and upstream shell were constructed using materials containing more than 40% of clay material while the downstream shell was constructed with coarse-grained soil with a maximum grain diameter of 100 mm. A concrete cutoff wall had been embedded along the centerline of the dam.



Cross-section of Asagawara Dam (from Omachi).

THE OCTOBER 23, 2004, EARTHQUAKE

The Niigata-ken Chuetsu Earthquake (M_w 6.6) October 23, 2004 (also known as Mid Niigata Prefecture Earthquake), was part of a sequence of powerful earthquakes that struck Japan on the nation's main island of Honshu, in the Chūbu region, Niigata Prefecture, near the city of Ojiya. The hypocenter of the main shock was located at 37.3N, 138.8E at a depth of 15.8 km. The intensity registered 6+ on the 7-grade Japanese intensity scale, with numerous aftershocks including some M 6 events following the main shock. Locations of the dams and reservoirs in relation to the earthquake epicenters are shown on Figure 2.

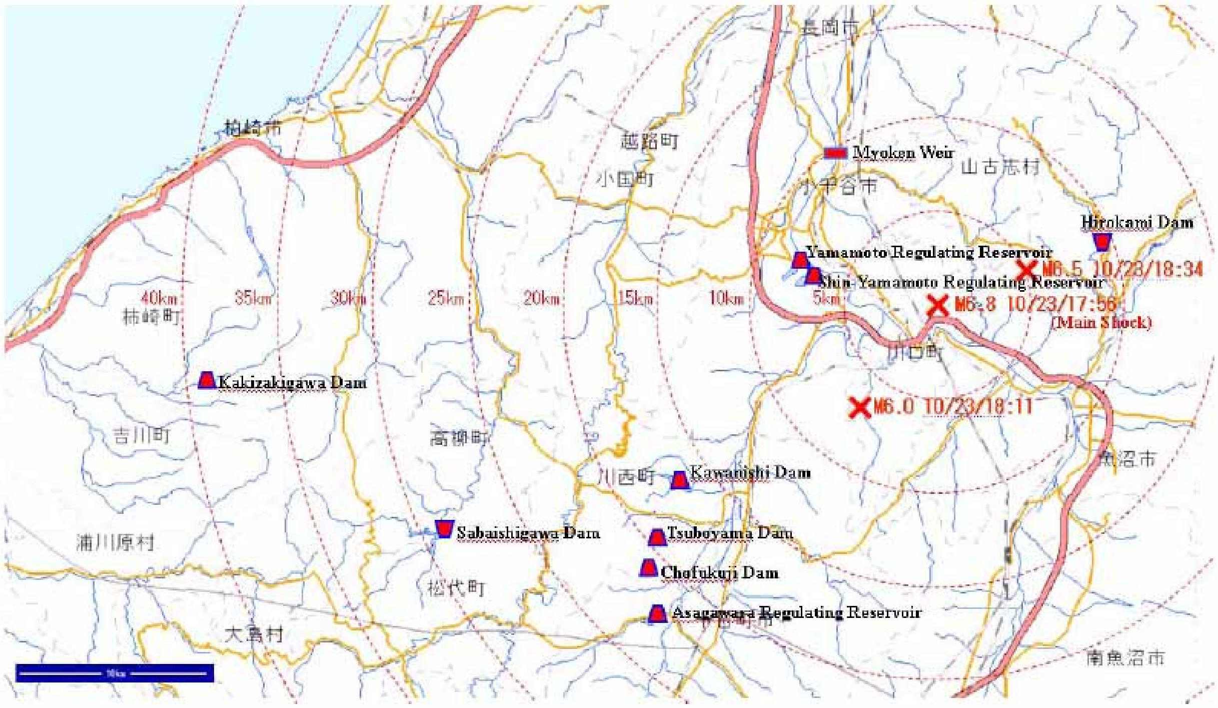


Figure 2. Locations of dams and epicenters (from Yasuda et al.)

This reverse fault type with the compression axis oriented to the NW/SE is consistent with historical earthquakes in this area. The maximum acceleration of 1.5 g was recorded at station Ojiya, nearest K-NET site to hypocenter. The earthquake caused substantial damage to many buildings, roads, railroads, and killed 67 people while injuring 4,805.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Shin-Yamamoto Dam

The post earthquake investigation revealed noticeable settlement of the dam body, cracks parallel to the dam axis, sand boils, and shallow slip deformation. The investigation also revealed some amounts of fines in the upstream drain, liquefaction of the upstream shell, and settlement. The amount of fines in the upstream drain suggested that the drain was contaminated with fines during the seismic event, because there had been strict quality control during construction. Excavation of sand boils showed the liquefaction was limited to the shallow surface materials, within three meters from the upstream surface. It is believed that the bottom portion of this layer was not adequately compacted allowing sand lenses to migrate to the surface during the intense seismic shaking. The maximum settlement was within the thin central clay core near the left bank and measured to be approximately 2.8 feet, which is nearly 2% of the dam height. The cause of the settlement has been attributed to the central clay core having been held up by the arching stresses from the shoulders and the seismic shaking releasing these stresses.

Crack in the asphalt pavement (from Omachi)

H-beam's top exposed showing settlement (from Omachi)

Yamamoto Dam

The seismic damage to the Yamamoto Dam included differential, shallow slip deformation, sand boils on the upstream slope, and transverse cracks along the right bank dam crest. Settlement of the dam body was measured to be approximately 0.3-0.4 feet throughout the structure.

The post earthquake investigation trenching showed that the sand boil source was shallow at a maximum depth of 3.3 feet. The slip deformation was found to be shallow and was approximately 1.6 feet at maximum. The sand boiling and the shallow slip deformation were found to be at the same elevation as the reservoir water level at the time of the earthquake.

Slip of reservoir side slope (from Yasuda et al.).

Asagawara Dam

The seismic damage to the Asagawara Dam included the formation of cracks at the dam crest and stepped settlement on the upstream slope. Along the crest, several cracks ran transverse to the dam axis and settled stepwise with a maximum difference at the centerline of 1.7 feet. The settlement of the dam body at maximum was 2.5 feet. No seismic deformation was observed on either the upstream or downstream slopes of the dam.

Based on the post earthquake investigation, the cracks and steps along the dam crest extended deeper in the upstream direction and were found to be no deeper than 10.8 feet. The crest deformation has been attributed to the insufficient compaction of the upper 3 meters of the dam body.

Downstream view
(from Omachi)

Cracks on the crest of dam
(from Omachi)

INSTRUMENTATION AND STRONG MOTION RECORDS

The exact maximum accelerations of the earthquake at the foundations of these three dams are unknown, it has been estimated that the seismic motion must have been greater than 0.5 g, a Level 2 Earthquake Motion. Available data indicate that higher accelerations were recorded at the dams along the hanging-wall side of the reverse-fault. Therefore, the ground motions at a given dam site was dependent on the intensity of the earthquake and distance from the focus of the earthquake but also the focal fault plane.

The results from the instrumentation cited earlier for Shin-Yamamoto dam were not obtained for this summary report.

CONCLUSIONS

The damages of the Shin-Yamamoto, Yamamoto, Asagawara Dams during the October 23, 2004 Mj6.8 Niigata-ken Chuetsu earthquake were relatively minor. Even though dam body settlement and shallow slip deformation occurred in all three dams, the post earthquake field investigation confirmed that the ability to store water was not compromised and that no leakage or seepage had occurred at the three dams.

These three dams show that earthen dams built at different times, with equipment and standards that have significantly improved over time, handled strong earthquake motions similarly.

Due to the earthquake, the hydroelectric generators had to be immediately shut down, however there was no backup power or generator system to operate the gate structures after the earthquake. This situation could have been worse if some other unforeseen incident would have occurred at the same time water was impounded and they were unable to release it.

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ZIPINGPU DAM, CHINA

Zipingpu Dam suffered relatively minor damage as a result of the devastating M_R 8 Wenchuan Earthquake in 2008. This case history is based on references Babbitt and Charlwood, 2009; Bureau, 2009; Chen (not dated); Li et al.,2009; Wen et al., 2011; and Xu 2009.

ZIPINGPU DAM

Zipingpu Dam, a 156 meter-high concrete faced rockfill dam (CFRD) (Figure 1), is located on the Minjiang River upstream of the City of Chengdu and was completed in 2005. It impounds a 1.1×10^9 m^3 reservoir and has a 760 MW power plant at the base of the right abutment. The penstocks and outlets are in tunnels. A reinforced concrete spillway is located on the right abutment (Figure 2). The quarried limestone embankment has a typical modern CFRD section, with 1.4:1 slopes, except that the upper 1/3 of the downstream slope is 1.5:1 and the crest is unusually wide at 12 meters (Figure 3). The crest is 664 meters long and the left abutment is steep, possibly 1:1. The plinth was constructed on bedrock, but there is some alluvium under the embankment.

WENCHUAN EARTHQUAKE

The May 12, 2008 Wenchuan Earthquake was M_w 8 event. The epicenter was in Sichuan Province in southwest China at the foot of the Tibetan Plateau, where the plateau collides with the Sichuan basin. The earthquake started on a west dipping thrust fault and became a strike/slip event as it broke 270 km northeastward on the Longmenshan fault.

The duration of strong ground motion was up to 120 seconds at sites underlain by deep alluvium. There were about 300 aftershocks, some over M_w 6. There were very few strong motion recorders operating in the earthquake area, so the estimates of peak ground accelerations are judgments considering observed damage and distances to faults. More than 80,000 people were killed by rockfalls and structure collapses. Approximately 2270 of the 35,600 reservoirs in Sichuan and seven nearby provinces were damaged. There were 69 dam breaks and 331 other highly dangerous situations. Ninety-five percent of the damaged reservoirs were formed by small earth dams.

EARTHQUAKE PERFORMANCE

The dam is 7 km from the fault and the estimated peak ground acceleration at the base of the dam was 0.5 to 0.6g. The reservoir water level was about 50 meters below normal maximum water level at the time of the earthquake. The embankment settled a maximum of about 1 meter and the crest moved downstream a maximum of about 0.5 meter. (Li, 2009) There was no indication that the alluvium under the embankment liquefied. The face slab and the parapet wall were damaged. A precast concrete pedestrian barrier along the downstream side of the crest was nearly all destroyed (Figure 4). An acceleration record from the crest has spikes up to 2g, but they were probably caused by the barrier falling or other secondary effects. The remainder of the record is below 1.0g. Some hand placed rock facing on the upper part of the downstream face was displaced. An increase in total leakage as measured by a weir at the collection point downstream was minor.

The upper part of the embankment settled 24 cm and moved downstream about 25 cm more than the parapet/retaining wall leaving a void under the upstream portion of the crest road and the slab (Figures 5 and 6). The reinforced concrete parapet/retaining wall apparently acted as beam or strut.

Overall, the face slab and water stops performed as designed. The damage was mainly above the reservoir water level. A vertical contraction joint near the maximum section and another one near the base of the steep left abutment (Figures 8 and 9), the highest horizontal construction joint (Figures 10 and 11), and the slab-parapet/retaining wall joint near the left abutment suffered enough compression damage that they were rebuilt. The parapet/retaining wall suffered compression damage near the maximum section and extension damage near the abutments, requiring repair. Campos Novos, Barra Grande and Mohale Dams, high CFRD dams with steep abutments, have suffered similar slab and wall damage due to post construction settlement from embankment volume loss, with no earthquake shaking.

There was no damage to the spillway or the anchored rock slopes. There was some damage to operating equipment, but it was repaired in a few days, however buildings constructed of reinforced concrete frames and masonry infill that housed operating equipment and controls were severely damaged.

POST EARTHQUAKE OPERATIONS

The post earthquake operations at the dam were very impressive. All four generation units shut down; the 500 KV transmission line was down; the gates for flood discharge tunnels #1 and #2 would not open; intake tower hoists and control rooms damaged, so there was no discharge capability. There was no communication offsite. Heavy rain started May 12; reservoir inflow was up to 600 m^3/s on the next day. Seven minutes after earthquake the first unit was back running at no load. Two more units were running at no load and the reservoir release was more than 100 m^3/s later in the day. Tunnels were inspected by hand in the dark. By May 13 outflow was up to 280 m^3/s. By May 28, the Tunnel #2 gate was working. Outflow was up to 850 m^3/s, which exceeded the inflow. Downstream water supply needs were met, vital power was generated and the reservoir level was stabilized.

Zipingpu Dam and Reservoir played major roles in the post earthquake disaster relief efforts. Access to Yingxiu Township, the town nearest the epicenter, was completely cut off by landslides and bridge collapses. All level areas around the dam, including the crest, became relief staging areas and the reservoir was held at a near constant level to provide barge access to and from the nearly destroyed town. Twenty thousand emergency workers went to the town and 20,000 survivors were evacuated on barges as shown in Figure 12.

CONCLUSIONS

The overall performance of embankment and appurtenant structures of Zipingpu Dam during the M_w 8 earthquake was excellent. The damage to the reinforced concrete face and parapet was probably unavoidable. Intense shaking causes high rockfills to become denser, hence settle enough to cause compression cracks in their slabs.

Buildings housing gate and valve controls need to be designed to withstand the same intensity of shaking as dams and the equipment in them needs to be securely anchored.

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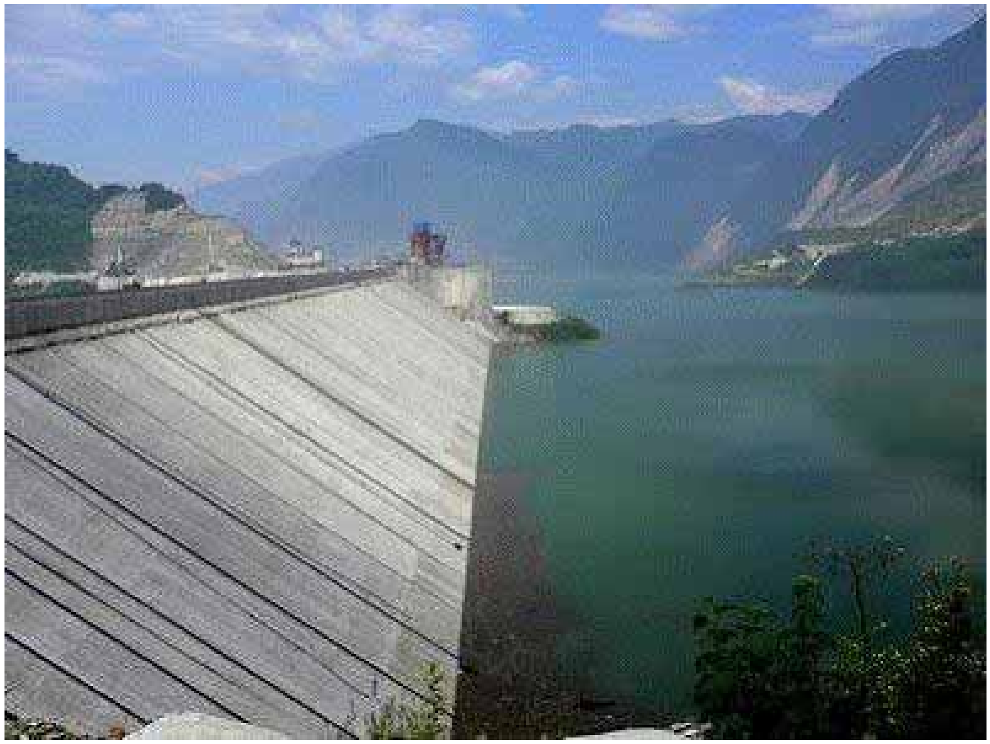

Figure 1. Upstream face (from Bureau).

Figure 2. Dam from downstream (Babbitt).

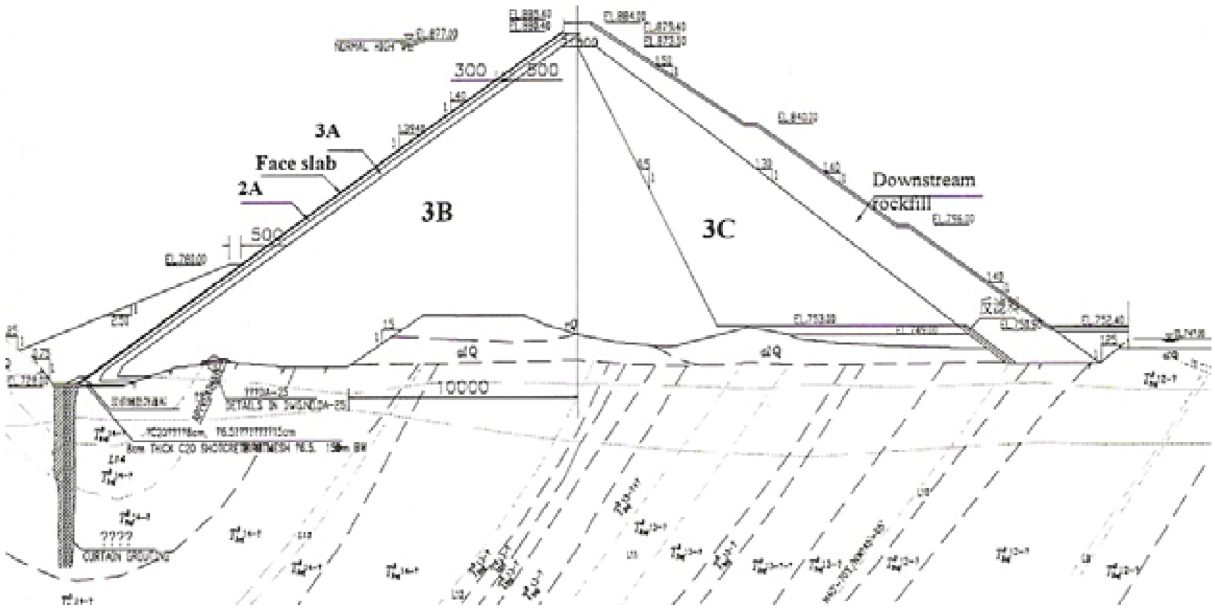

Figure 3. Embankment — maximum section (from Xu).

Figure 4. Precast concrete pedestrian barrier (Babbitt).

Figure 5. Gap at downstream edge of crest (Babbitt).

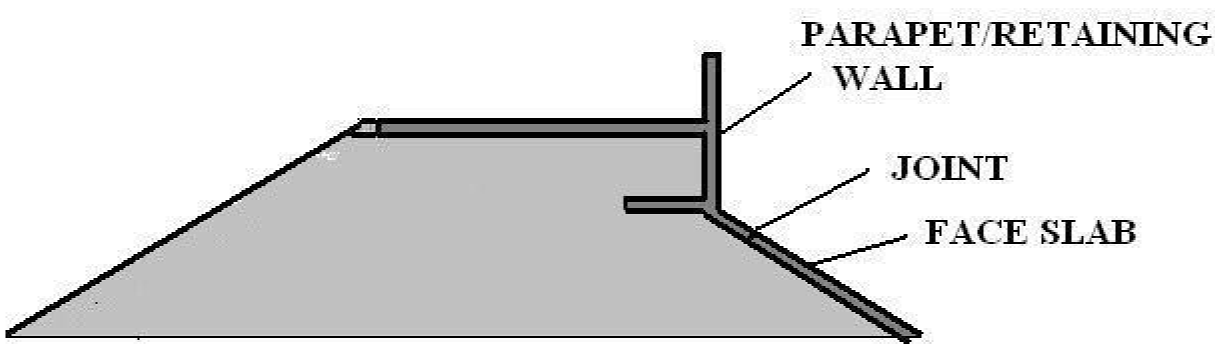

Figure 6. Schematic of pre earthquake crest detail (Babbitt).

Figure 7. Embankment settlement at spillway wall (Babbitt).

Figure 8. Face slab joint 5/6 near left abutment (from Wen).

Figure 9. Face slab joint 23/24 near maximum section (from Chen).

Figure 10. Highest horizontal construction joint (from Chen).

Figure 11. Highest horizontal construction joint detail (from Chen).

Figure 12. Rescue operation near left abutment.

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