United States Committee on Large Dams

USCOLD

Observed Performance of Dams During Earthquakes

Volume II

October 2000

Prepared by the USCOLD Committee on Earthquakes

USCOLD

The United States Committee on Large Dams (USCOLD), a Member of the International Commission on Large Dams, is a professional organization dedicated to:

- advancing the technology of dam engineering, construction, operation, maintenance and safety;
- fostering socially, environmentally and financially responsible water resources projects; and
- promoting public awareness of the role of dams in the beneficial and sustainable development of the nation's water resources.

Printed in the United States of America

Library of Congress Catalog Card Number: 00-107393

ISBN 1-884575-17-X

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FOREWORD

In July 1992, the U.S. Committee on Large Dams published a report titled "***Observed Performance of Dams During Earthquakes.***" The report included general observations on the performance of embankment and concrete dams, a table listing case histories and references on dams affected by earthquakes, and detailed descriptions of observed performance for 11 selected dams. This report is a sequel to the 1992 publication. It includes 16 additional case histories of dams that were historically exposed to moderate to strong earthquake shaking.

This publication was prepared by the USCOLD Committee on Earthquakes, chaired by Joseph L. Ehasz. Gilles J. Bureau, Vice-Chairperson, coordinated the Committee's efforts to prepare this report and wrote the introductory section.

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TABLE OF CONTENTS

OBSERVED PERFORMANCE OF DAMS DURING EARTHQUAKES
(Volume II)

In July 1992, the U.S. Committee on Large Dams published a report titled "***Observed Performance of Dams During Earthquakes.***" Since 1992, several earthquakes, including three major events, have affected an appreciable number of existing dams. The most significant of these recent earthquakes were the January 17, 1994, Northridge, California, Earthquake (moment magnitude 6.7), the January 17, 1995, Kobe, Japan, Earthquake (moment magnitude 6.9), and the Chi-Chi, Taiwan, Earthquake of September 21, 1999 (moment magnitude 7.6). These earthquakes have provided additional information regarding the seismic performance of dams.

The present report is a sequel to the 1992 publication. It includes 16 additional case histories of dams that were historically exposed to moderate to strong earthquake shaking. The original introduction of the previous report has been essentially reproduced in the next paragraphs, but was expanded to include the Northridge, Kobe, Chi-Chi and other recent experiences.

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 half 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, over 6,806 dams are higher than 50 feet; over 1,639 exceed 100 feet; and over 440 exceed 200 feet (U.S. Army Corps of Engineers, National Inventory of Dams, March 1999). Hence, if one considers the total number of existing large dams, in the U.S. or on a world-wide basis, the current performance record appears outstanding, based on the limited number of failures.

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. Except for several well-known examples, existing dams have not been tested by levels of ground motion equivalent to the applicable Design Basis Earthquake (DBE, see USCOLD, "***Guidelines to Select Seismic Criteria for Dams***," 1985, updated 1999). Conversely, a few dams have experienced significant damage under shaking substantially less demanding than what was, or should have been, considered in their design.

While much has been published on the performance of dams (see USCOLD, "***Bibliography on Performance of Dams During Earthquakes***," 1984), applicable literature is often scattered, very technical, and not easily accessible to dam owners or the general public. This condition has created a need for this and the previous USCOLD publication on the subject, which provide an overview of the seismic performance of dams of all types.

First, an updated inventory is presented of the principal dams that have experienced significant earthquake shaking. This information is summarized in Table 1. It includes, where available, principal earthquake parameters, dimensions and types of dam, epicentral distances, and crude indicators of the severity of the damage incurred, if any has been reported. The 11 case histories selected for detailed coverage in the 1992 report and the 16 presented in this report 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.

At this time, as in 1992, it is not possible to consider all of the dams that would justify being included among the selected case histories. The USCOLD Committee on Earthquakes anticipates that future publications on this subject will include other case histories of interest to the profession.

PERFORMANCE OF EMBANKMENT DAMS

The September 21, 1999, Chi-Chi, Taiwan, Earthquake (Mw 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 ruptured 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. 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. 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 foot. 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.

The January 17, 1995, Kobe, Japan, Earthquake (Mw = 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 of the Nojima Fault experienced during that event was very 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 (ECRD's).

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 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 was 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 level 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 Northridge, California, Earthquake (Mw = 6.7) occurred on January 17, 1994, and was centered about 32 km west-northwest of the 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. In 1971, the San Fernando Earthquake (M = 6.5) damaged several embankment (hydraulic fill) dams and caused near-total failure of the Lower Van Norman Dam (this dam is sometimes named Lower San Fernando Dam in the literature).

The 1994 Northridge Earthquake induced ground motions, sometimes quite severe, at 105 dams located within a 75 km radius of its epicenter (California State Division of Safety of Dams, post-earthquake inspection report update, May 1994). 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 an 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 empty reservoir for flood control. It experienced two- to three-and-a-half-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 toward upstream.

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.

The 130-foot-high Los Angeles Dam, which now replaces the two San Fernando dams, is located between these two flood-control, dry embankments. It experienced extensive, but not safety-threatening, cracking of its asphalt lining and settled 3.5 inches near its maximum section. Maximum horizontal crest movement was about 2.2 inches. The Los Angeles Dam experience is covered in more detail as one of the case histories presented in this report.

Lastly, 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).

The October 17, 1989, Loma Prieta, California, Earthquake ($M_s = 7.1$) 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. Like the Northridge Earthquake, the Loma Prieta Earthquake demonstrated the ability of well-designed embankment dams to withstand severe ground motion safely. It also 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 can reasonably be said to be capable of withstanding earthquakes of higher intensities and longer duration than were experienced during the October 17, 1989, event. This is because the strong phase of shaking (accelerations greater than 0.05g) during that earthquake lasted less than eight seconds at rock and firm soil sites in the epicentral area, a relatively short duration for a magnitude greater than 7. 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. All but one of the dams concerned performed well, as had been generally predicted in prior evaluation studies.

Austrian Dam, a 200-foot-high earthfill dam located about 12.5 km from the Loma Prieta epicenter, with a reservoir water level only at mid-height at the time of the earthquake, was the exception and experienced substantial abutment cracking and a maximum crest settlement of nearly three feet. While Austrian Dam remained safe, it reminded us of the limits of our present knowledge, and how we can learn from each particular experience. The non-recoverable earthquake-induced deformations of Austrian Dam remained well below the ten feet which the dam had been predicted to experience under the applicable DBE (a magnitude 8.3 event, centered along the San Andreas Fault at its closest distance to Austrian Dam). The 1989 earthquake was considerably less demanding than a DBE, in terms of overall duration and seismic energy content, but the dam was severely damaged. The observed settlements of this gravely clayey sand embankment might not have been predicted under loading conditions similar to those experienced, based on some frequently used numerical methods of dam safety evaluation. This experience reminded us of the constant need to learn from actual performance of dams, so that seismic safety evaluation procedures can be improved.

Prior to the Chi-Chi, Kobe, Northridge and Loma Prieta earthquakes, performance or damage reports for embankment dams had been obtained from approximately a dozen major earthquakes. The most significant of these included the San Francisco, California (1906); Kanto, Japan (1923); Kern County, California (1952); Hebgen Lake, Montana (1959); Tokachi-Oki, Japan (1968); San Fernando, California, (1971); Chile (1971, 1985); Mexico (1985); Whittier Narrows, California (1987); and Edgecumbe, New Zealand (1987) earthquakes.

The 1906 San Francisco Earthquake (M 8.3, estimated) 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.

The 1923 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.

Moderate damage was experienced by embankment dams in the Los Angeles area during the 1952 Kern County, California, Earthquake (M 7.7). The 20-foot-high Eklutna Dam suffered serious damage during the 1964 Alaska Earthquake (M 8.4), and was abandoned subsequently. 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, a 140-foot-high hydraulic fill dam, experienced widespread liquefaction and major slope failures. Overtopping of the crest and flooding to an area involving over 70,000 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.

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 (Ms 8.1), that involved two large earth-rock and rockfill dams, La Villita (197 feet high) and El Infiernillo (485 feet high). 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 have shown a tendency to increase in amplitude with more recent events, perhaps due to progressive weakening of some of the embankment materials. Similar increases have not been observed at El Infiernillo Dam, an earth core rockfill dam (ECRD), 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.

Lastly, two events of moderate magnitude, the 1987 Edgecumbe, New Zealand, Earthquake (M 6.2), which damaged the 259-foot-high Matahina Dam, and the 1987 Whittier Narrows, California, Earthquake (M 6.1), 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.

From a detailed review of past experience records, it has become 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 believed to be 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. In contrast, older embankments built 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 virtually 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.

PERFORMANCE OF CONCRETE DAMS

Several concrete gravity (Shih-Kang, Mingtan) or arch dams (Techi) were severely shaken during the Chi-Chi, Taiwan, Earthquake. These dams performed satisfactorily, with the exception of Shih-Kang Dam, which was destroyed by the fault rupture. Shih-Kang Dam is the first concrete dam known to have failed as a result of an earthquake.

Perhaps hundreds or more other concrete dams have been shaken by earthquakes felt at or near the dam site, but only about 20 have experienced recorded or estimated peak ground accelerations of 0.20g or higher. The most severely shaken dams include all principal types of concrete structures: arch, multiple arch, gravity and buttress. No significant damage has ever been suffered by an arch dam, although three such structures have historically experienced substantial ground motions.

Arch Dams

Techi Dam is a 600-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 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 0.30 and 0.50g. 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 whether damage occurred or not (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.

The 1994 Northridge earthquake severely shook Pacoima Dam, a 372-foot-high concrete arch dam, located a short distance away from the epicenter. As it did previously during the 1971 San Fernando Earthquake, this dam, a flood-control facility (which, therefore, had a low reservoir level at the time of occurrence of the earthquake), experienced nearby ground accelerations well above one g, at its left abutment. Indeed, horizontal and vertical peak ground accelerations (PGA) recorded in 1994 near the top of that abutment were 1.76g 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).

In 1994, the joint between the left abutment concrete thrust block and the left end of Pacoima Dam opened about two inches. The left abutment thrust block also moved 1/2 inch 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 vertical movement of the rock mass at another location. This experience also confirmed that some lift joints did open (CSMIP, 1994). 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. Overall, Pacoima Dam performed satisfactorily during the 1994 Earthquake, as it did before in 1971.

During the 1971 San Fernando, California, Earthquake (M 6.5), Pacoima Dam had been subjected to estimated base accelerations of perhaps 0.70g. A then unprecedented peak acceleration of 1.25g had been recorded on rock at the left abutment, slightly above the dam crest. However, as was concluded in 1994, 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. Pacoima 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.

Ambiesta Dam, Italy, a 194-foot-high arch dam, was shaken during the 1976 Friuli Earthquake (M 6.5) by ground motion recorded as 0.33g at the right abutment. It suffered no damage, confirming results of previous physical model studies, which had indicated that substantially larger accelerations (0.75g or greater) would be required to cause damage to the structure.

Other arch dams shaken by earthquakes include Honenike Dam, in Japan, a 98-foot-high multiple arch dam, which developed a crack in an arch near a buttress during the 1946 Nankai earthquake (M 7.2). The crack was repaired by grouting. Several other major concrete arch dams, such as Santa Anita and Big Tujunga, California; Barcis and Maina diSauris, in Italy; Kariba, in Zambia; Monteynard and Granval, in France; and Kurobe, in Japan, were located at 50 km or less from epicenters of various events of magnitude between 4.9 and 6.6, but were undamaged. However, the local intensities of shaking at those sites were probably moderate.

Gravity and Buttress Dams

Shih-Kang Dam, located about 50 km north of the Chi-Chi Earthquake epicenter, is a buttress gravity 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 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, 2000).

Fault crossing rupture across Shih-Kang Dam. (Photo courtesy of Robin G. Charlwood, Acres International; and Tim Little, BC Hydro.)

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. The owner plans to repair the dam.

The Mingtan hydroproject was also affected by the Chi-Chi Earthquake. It has a 269-foot-high concrete gravity dam, which was subjected to peak accelerations of 0.30 to 0.50g. The dam experienced no damage. Pressure relief wells in the foundation experienced an increase in head, but were redrilled and uplift pressures went back to normal.

Several gravity dams, including Aono, Gohonmatsu, and Sangari dams, located about 15, 19 and 30 km from the 1995 Kobe epicenter, respectively, were undamaged. Local shaking at these rock sites was probably moderate, as suggested by undisturbed tile roofs 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. As previously stated, this could be due to the modest ground motion experienced at rock sites. However, in other earthquakes, a few concrete gravity and buttress dams have been affected more severely by earthquakes than the above Japanese gravity dams and arch dams, in general. This experience is briefly described below.

Blackbrook Dam, in 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 magnitude 6.5 (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. Sefid-Rud, 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 7.3).

Lower Crystal Springs Dam, a 127-foot-high curved concrete gravity dam built of imbricated concrete blocks, withstood the 1906 San Francisco Earthquake (M 8.3, estimated) without a single crack. The rupture trace of the San Andreas Fault was less than 500 feet 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 as for 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, a 726-foot-high curved gravity dam, has been suspected of being the cause of moderate reservoir-triggered seismicity (M 5.0 or less) which did not affect the dam. Lastly, Poiana Usului Dam, in Romania, a buttress dam, was located about 60 km away from the epicenter of the 1977 Romanian earthquake (M 7.2), and performed satisfactorily.

Overall, the performance of concrete dams has been satisfactory, and such dams could be implied to be more earthquake-resistant than embankment dams. This is perhaps because concrete dams may have been built to design standards higher than used for some of the earlier embankment dams. Furthermore, concrete dams are probably less susceptible to aging, materials deterioration, seepage and poor maintenance than are older embankment dams. However, the true test of a major thin arch concrete dam in a highly seismic area and subjected to its DBE has yet to come. The Shih-Kang dam experience confirmed that concrete dams cannot be designed to accommodate fault rupture.

SELECTED CASE HISTORIES

The following case histories of dam performance during earthquakes were in the first USCOLD publication (1992):

- Lower Crystal Springs, California; San Francisco Earthquake (1906)
- Hebgen, Montana; Hebgen Lake Earthquake (1959)
- Koyna, India; Koyna Earthquake (1967)
- Lower Van Norman, California; San Fernando Earthquake (1971)
- Pacoima Dam, California; San Fernando Earthquake (1971)
- Rapel, Chile; Chilean Earthquake (1985)
- El Infiernillo, Mexico; Mexico Earthquake (1985)
- Long Valley, California; earthquake sequences (1978 to 1986)
- Matahina, New Zealand; Edgecumbe Earthquake (1987)
- Austrian; California; Loma Prieta Earthquake (1989)
- San Justo Dam; California; Loma Prieta Earthquake (1989)

The following new case histories are covered in detail in the present publication:

- Ambiesta, Italy; Friuli Earthquake (1976)
- Ambuklao, Philippines; Philippines Earthquake (1990)
- Bear Valley, California; Landers Earthquake (1992)
- Binga, Philippines; Philippines Earthquake (1990)
- Cerro Negro, Chile; Central Chile Earthquake (1985)
- Chabot, California; San Francisco Earthquake (1906)
- Cogoti, Chile; Illapel Earthquake (1943)
- La Villita, Mexico; Michoacan Earthquake (1985)
- Los Angeles, California; Northridge Earthquake (1994)
- Los Leones, Chile; Central Chile Earthquake (1985)
- Masiway, Philippines; Philippines Earthquake (1990)
- Mochikochi, Japan; Izu-Ohshima-Kinkai Earthquake (1978)
- 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)

ACKNOWLEDGMENTS

The USCOLD Committee on Earthquakes is grateful to URS Greiner Woodward Clyde, Oakland, California, for providing word processing and graphical support.

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TABLE 1: HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES

* See explanation for dam type at end of table [1] Treated in detail in Volume I [2] Treated in detail in Volume II

DAM NAME	Country	Type (*)	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
AUGUSTA	GA, USA	E	---	Charleston	31-Aug-1886	7.0	180.0	Collapse	Duke, C.M. (1960)
STEPHENSON CREEK	CA, USA	M	---	----	13-Jul-1894	---	64.0	Minor	CA DWR Report 116-3
O. SAN ANDREAS	CA, USA	E	28	San Francisco	19-Apr-1906	8.3	0.0	Minor	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
N. SAN ANDREAS	CA, USA	E	97	San Francisco	19-Apr-1906	8.3	0.0	Minor	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAKE RANCH	CA, USA	E	38	San Francisco	19-Apr-1906	8.3	0.1	Moderate	Ambraseys (1960), DWR (Pers. Com., 1999)
BEAR GULCH	CA, USA	E	45	San Francisco	19-Apr-1906	8.3	3.2	None	Ambraseys (1960)
PILARCITOS	CA, USA	E	103	San Francisco	19-Apr-1906	8.3	3.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SARATOGA	CA, USA	E	---	San Francisco	19-Apr-1906	8.3	0.1	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. HOWELL	CA, USA	E	38	San Francisco	19-Apr-1906	8.3	0.2	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
L. HOWELL	CA, USA	E	36	San Francisco	19-Apr-1906	8.3	0.2	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CROCKER	CA, USA	E	45	San Francisco	19-Apr-1906	8.3	2.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BURLINGAME	CA, USA	E	24	San Francisco	19-Apr-1906	8.3	1.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EMERALD LAKE No.1	CA, USA	E	57	San Francisco	19-Apr-1906	8.3	2.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CROCKER	CA, USA	E	45	San Francisco	19-Apr-1906	8.3	2.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
NOTRE DAME	CA, USA	E	50	San Francisco	19-Apr 1906	8.3	3.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. CRYSTAL SPRINGS	CA, USA	E	75	San Francisco	19-Apr-1906	8.3	0.0	Moderate	Ambraseys (1960)
L. CRYSTAL SPRINGS [1	CA, USA	GA	127	San Francisco	19-Apr-1906	8.3	0.4	None	ICOLD (1974)
SEARSVILLE	CA, USA	GA	64	San Francisco	19-Apr-1906	8.3	60.0	None	Confidential report, Dames & Moore (1998)
LAGUNITA	CA, USA	E	15	San Francisco	19-Apr-1906	8.3	6.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BELVEDERE	CA, USA	E	48	San Francisco	19-Apr-1906	8.3	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U.MOUNT N. BASIN	CA, USA	E	17	San Francisco	19-Apr 1906	8.3	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAGUNITAS	CA. USA	E	48	San Francisco	19-Apr-1906	8.3	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
COWELL	CA, USA	E	50	San Francisco	19-Apr-1906	8.3	19.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ESTATES	CA, USA	E	93	San Francisco	19-Apr-1906	8.3	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BERRYMAN	CA, USA	E	40	San Francisco	19-Apr-1906	8.3	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SUMMIT	CA, USA	E	21	San Francisco	19-Apr-1906	8.3	30.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CHABOT [2]	CA, USA	E	135	San Francisco	19-Apr-1906	8.3	40.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
PACIFIC GROVE	CA, USA	E	20	San Francisco	19-Apr-1906	8.3	41.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAKE RALPHINE	CA, USA	E	35	San Francisco	19-Apr-1906	8.3	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
TEMESCAL	CA, USA	E	105	San Francisco	19-Apr-1906	8.3	29.0	Minor	Ambraseys (1960)
U. SAN LEANDRO	CA, USA	E	125	San Francisco	19 Apr 1906	8.3	37.0	None	Ambraseys (1960)
PIEDMONT NO. 1	CA, USA	E	52	San Francisco	19-Apr-1906	8.3	30.0	Minor	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
PORT COSTA	CA, USA	E	45	San Francisco	19-Apr-1906	8.3	44.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
FORREST LAKE	CA, USA	E	60	San Francisco	19-Apr-1906	8.3	44.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAKE HERMAN	CA, USA	E	50	San Francisco	19 Apr-1906	8.3	51.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
L. ST HELENA	CA, USA	E	50	San Francisco	19-Apr-1906	8.3	51.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. ST HELENA	CA, USA	E	50	San Francisco	19-Apr-1906	8.3	51.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAKE CAMILLE	CA, USA	E	30	San Francisco	19-Apr-1906	8.3	52.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LAKE FREY	CA, USA	E	83	San Francisco	19-Apr-1906	8.3	59.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
VOLCANO LAKE	Mexico	E	12	Imperial V.	22-Jun 1915	5.3	0.0	Collapse	Ambraseys (1960)
FAIRMONT	CA, USA	E	---	Imperial V.	22-Oct-1916	6.0	22.0	None	Ambraseys (1960)
MURAYAMA SHIMO	Japan	E	52	Kanto	01 Sep-1923	8.2	18.0	Moderate	Nakayama, A. (1964)
U. MURAYAMA	Japan	E/HF	79	Kanto	01-Sep-1923	8.2	61.0	Moderate	Ambraseys, N.N. (1960)
L. MURAYAMA	Japan	E	101	Kanto	01-Sep-1923	8.2	61.0	Moderate	Ambraseys, N.N. (1960); Uesaka (1999)
ONO	Japan	E	161	Kanto	01 Sep-1923	8.2	51.0	Serious	ICOLD (1974); Uesaka (1999)
TOKYO W.S.	Japan	E	79	Kanto	01-Sep-1923	8.2	24.0	Minor	Duke, C.M. (1960)
SHEFFIELD [2]	CA, USA	E	25	Santa Barbara	29-Jun-1925	6.3	11.2	Collapse	Seed, H.B.; Lee, K.L.; Idriss, I.M. (1969)
BARAHONA	Chile	T	200	Talca	01-Oct-1928	8.4	160.0	Collapse	Smith, E.S. (1969)
CHATSWORTH NO. 2	CA, USA	HF	44	Santa Monica	30-Aug-1930	5.3	1.0	Moderate	Sherard, J.L.; et al (1963)
TUAI	New Zlnd	CG	14	New Zealand	02-Feb-1931	X	----	None	Robinson, Benjamin (1932)
TUAI DIVERSION	New Zlnd	E	17	New Zealand	02-Feb-1931	X	----	Minor	Robinson, Benjamin (1932)
HOOVER	NV, USA	GA	726	Induced eqk?	- 1936	5.0	8.0	None	Hansen, Roehm (1979)
MALPASO	Peru	ECRD	255	Peru	10-Oct-1938	VI	----	Minor	Ambraseys, N.N. (1960)
Misc. Embankmts	Japan	E	50/6	Ojika	- 1939	6.6	----	Severe	Akiba, M.; Semba, H. (1941)
VOLCANO LAKE	Mexico	E	12	El Centro	18-May 1940	7.1	0.0	Collapse	Ambraseys, N.N. (1960)
LAGUNA	CA, USA	E	50	El Centro	18-May-1940	7.1	67.0	Minor	Ambraseys, N.N. (1960)
COGOTI [2]	Chile	CFRD	280	Illapel	06-Apr-1943	7.9	89.0	Minor	Arrau, L.; Ibarra, I.; Noguera, G.(1985,1986)
OTANIIKE	Japan	E	88	Nankai	21-Dec-1946	7.2	80.0	Moderate	ICOLD (1974)
HONENIKE	Japan	CAB	98	Nankai	21-Dec-1946	7.2	50.0	Minor	ICOLD (1974)
HOSOROGI	Japan	E	28	Fukui	28-Jun-1948	7.3	4.8	Collapse	Ambraseys, N.N. (1960)

DAM NAME	Country	Type	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
NORTH END	CA, USA	E	---	Calipatria	20-Jul-1950	5.4	3.0	Minor	CA DWR Rpt 116-3
POGGIO CANCELLI	Italy	E	56	Gran Sasso	05-Sep-1950	5.5	6.4	None ?	Ambraseys, N.N. (1960)
BOUQUET CANYON	CA, USA	E	190	Kern County	21-Jul-1952	7.7	73.6	None	Ambraseys (1960)
ISABELLA	CA, USA	E	185	Kern County	21-Jul-1952	7.7	86.0	None	Ambraseys (1960)
DRY CANYON	CA, USA	HF	66	Kern County	21-Jul-1952	7.7	70.0	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BUENA VISTA	CA, USA	E?	20	Kern County	21-Jul-1952	7.7	28.0	Moderate	Sherard, J.L.; et al (1963)
SOUTH HAYWEE	CA, USA	HF	81	Kern County	21-Jul-1952	7.7	151.0	Minor	Sherard, J.L.; et al (1963)
FAIRMONT	CA, USA	HF	121	Kern County	21-Jul-1952	7.7	57.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
DRINKWATER	CA, USA	E	105	Kern County	21-Jul-1952	7.7	67.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
TEJON STORAGE	CA, USA	E	32	Kern County	21-Jul-1952	7.7	6.4	Minor	Ambraseys (1960)
LAHONTAN	NV, USA	E	125	Fallon	23-Aug-1954	6.7	48.0	None	Ambraseys (1960)
COLEMAN	NV, USA	Comp	---	Fallon	23-Aug-1954	6.7	24.0	Collapse	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SAGUSPE	NV, USA	E	---	Fallon	23-Aug-1954	6.7	24.0	Collapse	Ambraseys (1960)
ROGERS	NV, USA	M	---	Fallon	23-Aug-1954	6.7	80.0	Collapse	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
PONTEBA	Algeria	CG	59	Orleansville	09-Sep-1954	6.8	3.5	Major	Thevenin, J.; Le May, Y. (1964)
STEEG	Algeria	CGB	295	Orleansville	09-Sep-1954	6.8	70.0	None	Thevenin, J.; Le May, Y. (1964)
OUEDD FODDA	Algeria	CG	331	Orleansville	09-Sep-1954	6.8	5.0	None	Hansen, Roehm (1979)
ARCATA	CA, USA	E	55	Eureka	21-Dec-1954	6.6	8.0	Moderate	Ambraseys (1960)
ST MARY'S	CA, USA	M	50	Concord	23-Oct-1955	5.4	3.0	Minor	Sherard, J.L.; et al (1963)
BLACKBROOK	Gr. Britain	M	68	?	11-Feb-1957	5.6	6.4	Moderate	Walters (1964)
PINZANES	Mexico	CFRD	220	Mexico	28-Jul-1957	7.5	----	None	Ambraseys (1960)
HEBGEN [1]	MN, USA	E	90	Hebgen Lake	17-Aug-1959	7.1	16.0	Serious	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
KUROBE	Japan	CA	612	Induced eqk?	- -1960	5.0	9.0	None	Hansen, Roehm (1979)
XIGEER	China	E	21	Bachu	13-Apr-1961	6.8	35.0	Major	Wang, W.S. (1981), IWC & HPR, Beijing
MIBORO	Japan	ECRD	426	Kitamino	19-Aug-1961	7.0	17.0	Minor	Nakayama, A.et al. (1964)
HSINFENGKIANG	China	CGB	344	Hsinfengkiang	19-Mar-1962	6.1	1.1	Serious	Kollgaard, E.B.; Sharma, R.P. (1976)
MONTEYNARD	France	CA	508	Induced eqk?	23-Apr-1963	4.9	4.0	None	Hansen, Roehm (1979)
KARIBA	Rhodesia	CA	420	Induced eqk?	23-Sep-1963	6.1	----	None	Hansen, Roehm (1979)
EKLUTNA	AK, USA	Comp	20	Good Friday	27-Mar-1964	8.4	100.0	Serious	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
MINASE	Japan	CFRD	220	Oga	16-Jun-1964	6.9	198.0	None	Matsumoto,N; Takahashi, M.; Sato, F.(1985
MINASE	Japan	CFRD	220	Niigata	16-Jun-1964	7.5	147.0	Minor	Matsumoto,N; Takahashi, M.; Sato, F.(1985
LA CALERA	Mexico	ECRD	92	Mexico	1964	VIII	----	Serious	ICOLD (1974)
EL SOLDADO	Chile	T	---	Chile	28-Mar-1965	7.1	----	Collapse	Haws, E.T.; Reilly, N. (1981)
LLIU LLIU	Chile	E	66	Chile	28-Mar-1965	7.1	----	Moderate	Lastrico, Monge (1973)
EL CERRADO	Chile	T	---	Chile	28-Mar-1965	7.1	37.0	Moderate	Dobry, Alvarez (1967)
CATAPILCO	Chile	T	45	Chile	28-Mar-1965	7.1	---	Serious	Lastrico (1977), Lastrico, Monge (1973)
LA PATAGUA	Chile	T	---	Chile	28-Mar-1965	7.1	22.0	Collapse	Dobry, Alvarez (1967)
SAUCE	Chile	T	---	Chile	28-Mar-1965	7.1	66.0	Moderate	Dobry, Alvarez (1967)
RAMAYANA	Chile	T	---	Chile	28-Mar-1965	7.1	85.0	Serious	Dobry, Alvarez (1967)
CERRO BLANCO	Chile	T	---	Chile	28-Mar-1965	7.1	96.0	Minor	Dobry, Alvarez (1967)
BELLAVISTA	Chile	T	---	Chile	28-Mar-1965	7.1	55.0	Serious	Dobry, Alvarez (1967)
EL COBRE	Chile	T	---	Chile	28-Mar-1965	7.1	35.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
KREMASTA	Greece	CA	541	Induced eqk?	05-Feb 1966	6.2	11.0	None	National Academy of Science (1972)
FUYANG RIVER	China	E		Xingtai	22-Mar-1966	7.2	21.0	Major	Wang, W.S. (1981), IWC & HPR, Beijing
KOYNA [1]	India	CG	338	Koyna	11-Dec-1967	6.5	3.0	Serious	Chopra, A.K.; Chakrabarti, L. (1971)
VIR	India	E	79	Koyna	11-Dec-1967	6.5	8.0	Moderate	ICOLD (1974)
HAYAGAKENUMA	Japan	E	40	Tokachi-Oki	16-May-1968		----	Collapse	Shibata et al (1971)
ICHIRIGOYA	Japan	E	26	Tokachi-Oki	16-May-1968		----	Collapse	Shibata et al (1971)
NIMAI-BASHI	Japan	E	---	Tokachi-Oki	16-May-1968		----		Shibata et al (1971)
KATTAI	Japan	E	---	Tokachi-Oki	16-May-1968		----		Shibata et al (1971)
GAMANOSAWA	Japan	E	34	Tokachi-Oki	16-May-1968		----	Collapse	Shibata et al (1971)
SHOREY	Peru	T	---	Peru	1969		----	Collapse	M-K Engineers (pers. comm., 1977)
WANGWU	China	E	85	Bohai Gulf	18-Jul- 1969	7.2	?	Major	Wang, W.S. (1981), IWC & HPR, Beijing
YEYUAN	China	E	82	Bohai Gulf	18-Jul- 1969	7.2	?	Serious	Wang, W.S. (1981), IWC & HPR, Beijing
KUZURYU	Japan	ECRD	419	Gifu	09-Sep-1969	6.7	40.0	None	Nose, M.; Baba, K. (1981)
KISENYAMA	Japan	ECRD	312	Gifu	09-Sep-1969	6.7	----	None	Takahasi, T.; et al (1977)
HUACHOPOLCA	Peru	T	---	Peru	1970		---	Collapse	Smith, E.S. (1971)
LOPEZ	CA, USA	E	166	San Fernando	09-Feb-1971	6.5	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
YARNELL DEBRIS	CA, USA	E	49	San Fernando	09-Feb-1971	6.5	8.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
DRY CANYON	CA, USA	HF	66	San Fernando	09-Feb-1971	6.5	10.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)

DAM NAME	Country	Type	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
SAN FERNANDO DK. B	CA, USA	E	36	San Fernando	09-Feb-1971	6.5	10.4	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CHANNEL DIV. DIKE	CA, USA	E	43	San Fernando	09-Feb-1971	6.5	10.4	Minor	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
HANSEN	CA, USA	E	97	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
GREEN VERDUGO	CA, USA	E	117	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
DRINKWATER	CA, USA	E	105	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
PORTER ESTATE	CA, USA	E	41	San Fernando	09-Feb-1971	6.5	16.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BOUQUET CANYON	CA, USA	E	190	San Fernando	09-Feb-1971	6.5	17.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
RESERVOIR NO. 1	CA, USA	E	35	San Fernando	09-Feb-1971	6.5	22.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CHATSWORTH	CA, USA	HF	44	San Fernando	09-Feb-1971	6.5	23.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SEPULVEDA	CA, USA	E	57	San Fernando	09-Feb-1971	6.5	24.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
HAROLD	CA, USA	E	30	San Fernando	09-Feb-1971	6.5	26.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
DIEDERICH	CA, USA	E	60	San Fernando	09-Feb-1971	6.5	27.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ENCINO	CA, USA	E	168	San Fernando	09-Feb-1971	6.5	26.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SANTA FELICIA	CA, USA	E	200	San Fernando	09-Feb-1971	6.5	26.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
RUNKLE	CA, USA	E	41	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. HOLLYWOOD	CA, USA	E	87	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. FRANKLIN	CA, USA	E	55	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. STONE CANYON	CA, USA	E	110	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
GLENOAKS	CA, USA	E	62	San Fernando	09-Feb-1971	6.5	30.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
STONE CANYON	CA, USA	E	185	San Fernando	09-Feb-1971	6.5	30.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
FAIRMONT	CA, USA	HF	121	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
L. FRANKLIN	CA, USA	HF	103	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EAGLE ROCK	CA, USA	E	113	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ROWENA	CA, USA	E	29	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
RUBIO DEBRIS BASIN	CA, USA	E	55	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SAWTELL	CA, USA	E	34	San Fernando	09-Feb 1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
J.W.WISDA	CA, USA	E	50	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SILVER LAKE	CA, USA	HF	53	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EATON WASH	CA, USA	E	63	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ELYSIAN	CA, USA	E	72	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
WOOD RANCH	CA, USA	E	146	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ASCOT	CA, USA	E	73	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CHEVY CHASE	CA, USA	E	35	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
L. VAN NORMAN [1]	CA, USA	HF	140	San Fernando	09-Feb-1971	6.5	11.2	Major	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
U. VAN NORMAN	CA, USA	HF	80	San Fernando	09-Feb-1971	6.5	11.2	Serious	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
PACOIMA [1]	CA, USA	CA	372	San Fernando	09-Feb-1971	6.5	5.0	None	Hansen, K.D.; Roehm, L.H. (1979)
BIG TUJUNGA	CA, USA	CA	251	San Fernando	09-Feb-1971	6.5	32.0	None	Hansen, K.D.; Roehm, L.H. (1979)
SANTA ANITA	CA, USA	CA	251	San Fernando	09-Feb-1971	6.5	27.0	None	Hansen, K.D.; Roehm, L.H. (1979)
EL COBRE	Chile	T	—	Chile	08-Jul- 1971	7.5	80.0	Serious	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
CATAPILCO	Chile	E	46	Chile	08-Jul- 1971	7.5	110.0	Moderate	Lastrico, Monge (1973)
COLLAGUA	Chile	T	—	Chile	08-Jul- 1971	7.5	----	None	Lastrico, Monge (1973)
EL MELON	Chile	T	---	Chile	08-Jul- 1971	7.5	----	None	Lastrico, Monge (1973)
SALAMANCA	Chile	T	---	Chile	08-Jul- 1971	7.5	110.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
ILLAPEL	Chile	T	26	Chile	08-Jul- 1971	7.5	100.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
LIMAHUIDA	Chile	T, E	33	Chile	08-Jul- 1971	7.5	100.0	Moderate	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
LLIU LLIU	Chile	E	66	Chile	08-Jul- 1971	7.5	----	Serious	Eisenberg, Luco (1973)
CERRO NEGRO	Chile	T	---	Chile	08-Jul- 1971	7.5	----	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
LOS MAQUIS	Chile	T	---	Chile	08-Jul- 1971	7.5	----	Moderate	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
LAS PATAGUAS	Chile	T	---	Chile	08-Jul- 1971	7.5	----	Moderate	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
SHIMEN LING	China	E	147	Haicheng	04-Feb-1975	7.3	33.0	Serious	Wang, W.S. (1981), IWC & HPR, Beijing
OROVILLE	CA, USA	ECRD	770	Oroville	01-Aug-1975	5.7	6.9	None	Vrymoed (1981)
LA VILLITA	Mexico	ECRD	197	Mexico	11-Oct-1975	5.9	40.0	None	Comision Federal de Electricidad (1985)
EL INFIERNILLO	Mexico	ECRD	485	Mexico	11-Oct-1975	5.9	79.0	None	Comision Federal de Electricidad (1985)
LA VILLITA	Mexico	ECRD	197	Mexico	15-Nov-1975	7.2	27.0	None	Comision Federal de Electricidad (1985)
EL INFIERNILLO	Mexico	ECRD	485	Mexico	15-Nov-1975	7.2	23.0	None	Comision Federal de Electricidad (1985)
TSENGWEN	Taiwan	ECRD	430	Taiwan	14-Apr-1976	5.3	8.0	None	Ishihara, et al. (1990)
LUMIEI	Italy	CA	446	Friuli	06 May-1976	6.5	30.0	None	EDF (1987)
MAINA DI SAURIS	Italy	CA	446	Friuli	06-May-1976	6.5	43.0	None	Hansen, K.D.; Roehm, L.H. (1979)
BARCIS	Italy	CA	164	Friuli	06-May-1976	6.5	48.0	None	Hansen, K.D.; Roehm, L.H. (1979)
AMBIESTA [2]	Italy	CA	194	Friuli	06-May 1976	6.5	22.0	None	Hansen, K.D.; Roehm, L.H. (1979)

DAM NAME	Country	Type	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
PAIHO (BAIHE)	China	E	213	Tangshan	28-Jul-1976	7.8	150.0	Moderate	CSCPRC, Report 8 (1980); Wang (1981)
SIATHINZE	China	GA	66	Tangshan	28-Jul-1976	7.8	----	Moderate	Shen, C.;Chen, H. (1981)
TOUHO (DOUHE)	China	E	72	Tangshan	28-Jul-1976	7.8	----	Serious	CSCPRC, Report 8 (1980)
IZVORUL MONTELVI	Romania	CG	417	Vrancea	04-Mar-1977	7.2	100.0	None	EDF (1987)
POIANA USULUI	Romania	CGB	262	Vrancea	04-Mar-1977	7.2	60.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & D
PEREZ CALDERA	Chile	T	---	Argentina	24-Nov-1977	7.4	350.0	Minor	Smith, E.S. (pers. comm.)
MOCHIKOSHI No. 1 [2]	Japan	T	98	Nr Izu-Oshima	14-Jan-1978	7.0	35.0	Collapse	Marcuson, W.F. et al (1979)
MOCHIKOSHI No. 2 [2]	Japan	T	98	Nr I-O Aftshk	15-Jan-1978	5.8	----	Serious	Okusa, S.; Anma, S.; Maikuma, H (1980)
TARUMIZU	Japan	ECRD	141	Miyagiken Oki	06-Dec-1978	7.4	100.0	None	Yanagisawa, E.; Fukui, T. (1980)
EL INFIERNILLO	Mexico	ECRD	485	Guerrero	14-Mar-1979	7.6	110.0	Minor	Romo, M.P.; Resendiz, D. (1981)
LA VILLITA	Mexico	ECRD	197	Guerrero	14-Mar-1979	7.6	110.0	Minor	Romo, M.P.; Resendiz, D. (1981)
VERMILION [2]	CA, USA	E	150	Mammoth Lake	27-May-1980	6.3	21.0	None	Leps, T.M. (Pers. Comm., 1987)
LONG VALLEY [1]	CA, USA	E	126	Mammoth Lake	27-May-1980	6.3	5.0	None	Seed, H.B. (1985) - EERI Newsletter, Vol.9
EL INFIERNILLO	Mexico	ECRD	485	Mexico	25-Oct-1981	7.3	30.6	Minor	Romo, M.P.; Resendiz, D. (1981)
LA VILLITA	Mexico	ECRD	197	Mexico	25-Oct-1981	7.3	54.7	Minor	Romo, M.P.; Resendiz, D. (1981)
NAMIOKA	Japan	ECRD	171	Japan	26-May-1983	7.7	145.0	None	Ishihara, et al. (1990)
LEROY ANDERSON	CA, USA	ECRD	235	Morgan Hill	24-Apr-1984	6.2	16.0	Minor	Bureau, G; Tepel, R.E.; Volpe, R.L. (1984)
COYOTE	CA, USA	E	140	Morgan Hill	24-Apr-1984	6.2	24.0	None	Bureau, G; Tepel, R.E.; Volpe, R.L. (1984)
MAKIO	Japan	ECRD	262	Naganoken	14-Sep 1984	6.8	----	Minor	EERI Nesletter (1985)
CERRO NEGRO [2]	Chile	T	105	Chile	03-Mar-1985	7.7	----	Collapse	Castro, G. (pers. comm., 1986)
LOS LEONES [2]	Chile	ECRD	354	Chile	03-Mar-1985	7.7	----	None	Edwards, G.R. (1990)
LA PALMA	Chile	T	26	Chile	03-Mar-1985	7.7	----	Serious	De Alba, et al. (1987)
LA MARQUESA	Chile	T	33	Chile	03-Mar-1985	7.7	----	Serious	De Alba, et al. (1987)
VETA DE AGUA	Chile	T	---	Chile	03 Mar-1985	7.7	----	Collapse	Castro, G. (pers. comm., 1986)
RAPEL [1]	Chile	CA	361	Chile	03-Mar-1985	7.7	----	Moderate	Coyne et Bellier (1987)
LA VILLITA	Mexico	ECRD	197	Michoacan	19-Sep-1985	8.1	44.0	Minor	Bureau, G.; Campos-Pina, M. (1986)
EL INFIERNILLO [1]	Mexico	ECRD	485	Michoacan	19-Sep-1985	8.1	75.0	Minor	Bureau, G.; Campos-Pina, M. (1986)
LA VILLITA [2]	Mexico	ECRD	197	Michoacan	21-Sep-1985	7.5	61.0	None	Bureau, G.; Campos-Pina, M. (1986)
EL INFIERNILLO	Mexico	ECRD	485	Michoacan	21-Sep-1985	7.5	80.0	None	Bureau, G.; Campos-Pina, M. (1986)
TUAI DIV.	New Zlnd	CG	17	Bay of Plenty	02-Mar-1987	6.2	11.0	None	Robinson, R.; Benjamin, H.L. (1987)
MATAHINA [1]	New Zlnd	ECRD	259	Bay of Plenty	02-Mar-1987	6.2	23.0	Moderate	EQE (1987), Gillon (1988)
GARVEY RESERVOIR	CA, USA	E	160	Whittier	01-Oct-1987	6.1	3.0	None	Horowitz, Ehasz (USCOLD Newletter, 1987)
ORANGE COUNTY RES.	CA, USA	E	114	Whittier	01-Oct-1987	6.1	23.0	None	Horowitz, Ehasz (USCOLD Newletter, 1987)
PUDDINGSTONE	CA, USA	E	148	Whittier	01-Oct-1987	6.1	57.6	None	Bray, J.d; Seed, R.B, Boulanger, R.W.(1993)
WHITTIER-NARROWS	CA, USA	E	94	Whittier	01-Oct-1987	6.1	4.0	None	Horowitz, Ehasz (USCOLD Newletter, 1987)
NAGARA	Japan	ECRD	171	Japan	17-Dec-1987	6.9	28.8	None	Tani, S. (1991)
AUSTRIAN [1]	CA, USA	E	185	Loma Prieta	17-Oct-1989	7.1	11.5	Serious	Bureau et al (USCOLD Newsletter, 1989)
NEWELL	CA, USA	E	182	Loma Prieta	17 Oct-1989	7.1	18.4	Moderate	Bureau et al (USCOLD Newsletter, 1989)
LEXINGTON	CA, USA	E	205	Loma Prieta	17-Oct-1989	7.1	20.6	Minor	Bureau et al (USCOLD Newsletter, 1989)
VASONA Percolation	CA, USA	E	34	Loma Prieta	17-Oct-1989	7.1	24.5	Minor	Bureau et al (USCOLD Newsletter, 1989)
LEROY ANDERSON	CA, USA	E	235	Loma Prieta	17-Oct-1989	7.1	26.9	Minor	Bureau et al (USCOLD Newsletter, 1989)
ELMER J. CHESBRO	CA, USA	E	95	Loma Prieta	17-Oct-1989	7.1	19.0	Moderate	Bureau et al (USCOLD Newsletter, 1989)
GUADALUPE	CA, USA	E	142	Loma Prieta	17-Oct-1989	7.1	18.1	Minor	Bureau et al (USCOLD Newsletter, 1989)
RINCONADA	CA, USA	E	40	Loma Prieta	17-Oct-1989	7.1	26.2	Minor	Bureau et al (USCOLD Newsletter, 1989)
ALMADEN	CA, USA	E	110	Loma Prieta	17-Oct-1989	7.1	15.5	Minor	Bureau et al (USCOLD Newsletter, 1989)
COASTWAYS	CA, USA	E	46	Loma Prieta	17-Oct-1989	7.1	37.9	Minor	Bureau et al (USCOLD Newsletter, 1989)
SODA LAKE	CA, USA	E	35	Loma Prieta	17-Oct-1989	7.1	28.2	Moderate	Bureau et al (USCOLD Newsletter, 1989)
MILL CREEK	CA, USA	E	76	Loma Prieta	17-Oct-1989	7.1	30.2	None	Bureau et al (USCOLD Newsletter, 1989)
SAN JUSTO [1]	CA, USA	E	147	Loma Prieta	17-Oct-1989	7.1	24.0	None	United States Geological Survey (1989)
L. CRYSTAL SPRINGS	CA, USA	CG	127	Loma Prieta	17 Oct-1989	7.1	69.0	None	Bureau et al (USCOLD Newsletter, 1989)
SAN ANTONIO	CA, USA	E	160	Pomona Valley	28-Feb-1990	5.5	3.0	Minor	Jephcott, D.K. (EERI Newsletter, 1990)
SEFID-RUD [2]	Iran	CGB	348	Manjil	21-Jun-1990	7.3	32.0	Moderate	Indermaur et al. (1991); IrCLD (1992)
AMBUKLAO [2]	Luzon	ECRD	426	Philippines	16-Jul-1990	7.7	80.0	Moderate	Swaisgood, Au-Yeung Y. (ASDSO, 1991)
AYA	Luzon	E	35	Philippines	16-Jul-1990	7.7	46.4	Minor	Swaisgood, Au-Yeung Y. (ASDSO, 1991)
BINGA [2]	Luzon	E	335	Philippines	16-Jul-1990	7.7	48.0	Moderate	Swaisgood, Au-Yeung Y. (ASDSO, 1991)
CANILI	Luzon	ECRD	230	Philippines	16-Jul-1990	7.7	16.0	Minor	Swaisgood, Au Yeung Y. (ASDSO, 1991)
MAGAT	Luzon	ECRD	328	Philippines	16-Jul-1990	7.7	80.5	None	Swaisgood (1998)
DIAYO	Luzon	E	197	Philippines	16-Jul-1990	7.7	32.0	Moderate	Swaisgood, Au Yeung Y. (ASDSO, 1991)
MASIWAY [2]	Luzon	E	82	Philippines	16 Jul-1990	7.7	19.2	Serious	Swaisgood, Au-Yeung Y. (ASDSO, 1991)
PANTABANGAN [2]	Luzon	E	351	Philippines	16-Jul-1990	7.7	48.0	Minor	Swaisgood, Au-Yeung Y. (ASDSO, 1991)

TABLE 1 (continued): HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES

DAM NAME	Country	Type	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
COGSWELL	CA, USA	ECRD	266	Sierra Madre	28-Jun-1991	5.8	3.8	None	CA Dpt of Water Resources (memo 9/9/91)
ROBERT MATTHEWS	CA, USA	E	151	California	25-Apr-1992	6.9	64.4	None	Vrymoed (1993)
BEAR VALLEY	CA, USA	MA	80	Bear Mountain	28-Jun-1992	6.6	14.4	None	CA Dpt of Water Resources (1992)
WIDE CANYON	CA, USA	E	84	Landers	28-Jun-1992	7.4	29.8	Minor	Fong, Bennett (1995)
YUCAIPA No. 1	CA, USA	E	43	Landers	28-Jun-1992	7.4	28.3	Minor	EERI (1992), Vrymoed (1993)
YUCAIPA No. 1	CA, USA	E	49	Landers	28-Jun-1992	7.4	28.3	Minor	EERI (1992), Vrymoed (1993)
BEAR VALLEY [2]	CA, USA	MA	80	Landers	28-Jun-1992	7.4	44.8	None	CA Department of Water Resources (1992)
UPPER LAKE MARY	AZ, USA	E	43	Arizona	29-Apr-1993	5.5	77.3	None	City of Flagstaff (1993)
NIWAIKUMINE	Japan	E	?	Hokkaido Nans	12-Jul-1993	7.8	74.0	Serious	Uesaka (1999)
FENA	Guam	E	135	Guam	08-Aug-1993	8.1	60.0	Moderate	Woodward-Clyde (1997)
L. SAN FERNANDO	CA, USA	HF	125	Northridge	17-Jan-1994	6.7	9.4	Serious	CA DSOD (1994); Persson, V.H. (1994)
U.SAN FERNANDO	CA, USA	HF	82	Northridge	17-Jan-1994	6.7	10.2	Moderate	CA DSOD (1994); Persson, V.H. (1994)
LOS ANGELES [2]	CA, USA	E	130	Northridge	17 Jan-1994	6.7	9.8	Minor	Bardet & Davis (1996); Bureau, et al. (1996)
LAS LLAJAS	CA, USA	E	96	Northridge	17-Jan-1994	6.7	----	Minor	Fong, Bennett (1995)
LOWER FRANKLIN	CA, USA	E	103	Northridge	17-Jan-1994	6.7	18.1	Minor	CA DSOD (1994); Persson, V.H. (1994)
SANTA FELICIA	CA, USA	E	213	Northridge	17-Jan-1994	6.7	33.1	Minor	CA DSOD (1994); Persson, V.H. (1994)
PACOIMA	CA, USA	CA	365	Northridge	17-Jan-1994	6.7	18.2	Minor	CA DSOD (1994); Persson, V.H. (1994)
SYCAMORE CANYON	CA, USA	E	40	Northridge	17-Jan-1994	6.7	24.0	Minor	CA DSOD (1994); Persson, V.H. (1994)
SCHOOLHOUSE D. B.	CA, USA	E	38	Northridge	17-Jan-1994	6.7	13.8	Minor	CA DSOD (1994); Persson, V.H. (1994)
MORRIS S. JONES	CA, USA	E	49	Northridge	17-Jan-1994	6.7	42.7	Minor	CA DSOD (1994); Persson, V.H. (1994)
RESERVOIR 5, LA Cty	CA, USA	E	36	Northridge	17-Jan-1994	6.7	19.0	Minor	CA DSOD (1994); Persson, V.H. (1994)
COGSWELL	CA, USA	E	266	Northridge	17-Jan-1994	6.7	52.3	Minor	CA DSOD (1994); Persson, V.H. (1994)
BRAND Diversion Basin	CA, USA	E	45	Northridge	17 Jan-1994	6.7	----	Minor	Fong, Bennett (1995)
PORTER ESTATES	CA, USA	E	41	Northridge	17-Jan-1994	6.7	5.3	Moderate	CA DSOD (1994); Persson, V.H. (1994)
RUBIO Debris Basin	CA, USA	E	64	Northridge	17-Jan-1994	6.7	38.7	Minor	CA DSOD (1994); Persson, V.H. (1994)
INDIAN CREEK	CA, USA	E	71	S. Lake Tahoe	12-Sep-1994	6.0	12.5	None	CA DSOD (1994)
HARVEY PLACE	CA, USA	E	72	S. Lake Tahoe	12-Sep-1994	6.0	13.0	Minor	CA DSOD (1994)
HEENAN LAKE No. 1	CA, USA	E	35	S. Lake Tahoe	12-Sep-1994	6.0	14.9	Minor	CA DSOD (1994)
MEADOW LAKE	CA, USA	ECRD	77	S. Lake Tahoe	12-Sep-1994	6.0	35.7	None	CA DSOD (1994)
CAPLES LAKE	CA, USA	ECRD	71	S. Lake Tahoe	12-Sep-1994	6.0	29.6	None	CA DSOD (1994)
UPPER KOYOEN	Japan	E	30	Kobe	17-Jan-1995	6.9	----	Collapse	U.S.A.E. Waterways, CEWES-GV-Z (1995)
CENTRAL KOYOEN	Japan	E	30	Kobe	17-Jan-1995	6.9	----	Collapse	U.S.A.E. Waterways, CEWES-GV-Z (1995)
LOWER KOYOEN	Japan	E	30	Kobe	17-Jan-1995	6.9	----	Serious	U.S.A.E. Waterways, CEWES-GV-Z (1995)
HITOKURA	Japan	CG	246	Kobe	17-Jan-1995	6.9	47.0	None	Tamura, et al. (1997)
MINOOGAWA	Japan	ECRD	154	Kobe	17-Jan-1995	6.9	49.0	None	Tamura, et al. (1997)
DAINICHIGAWA	Japan	CG	143	Kobe	17-Jan-1995	6.9	48.0	None	Tamura, et al. (1997)
TENNO	Japan	CG	111	Kobe	17-Jan-1995	6.9	16.0	None	Tamura, et al. (1997)
TACHIGAHATA	Japan	M	109	Kobe	17-Jan-1995	6.9	15.0	None	Tamura, et al. (1997)
SENGARI	Japan	M	139	Kobe	17-Jan-1995	6.9	39.0	None	Tamura, et al. (1997)
MARUYAMA	Japan	CG	102	Kobe	17-Jan-1995	6.9	31.0	None	Tamura, et al. (1997)
NARINI-IKE	Japan	M	108	Kobe	17-Jan-1995	6.9	42.0	None	Tamura, et al. (1997)
TOKIWA	Japan	E	110	Kobe	17-Jan-1995	6.9	10.0	Moderate	Tamura, et al. (1997); Uesaka (1999)
TANIYAMA	Japan	E	92	Kobe	17-Jan-1995	6.9	7.0	Minor	Tamura, et al. (1997)
DONDO	Japan	CG	235	Kobe	17-Jan-1995	6.9	19.0	None	Tamura, et al. (1997)
KOJIYA	Japan	ECRD	145	Kobe	17-Jan-1995	6.9	48.0	None	Tamura, et al. (1997)
KAMOGAWA	Japan	CG	143	Kobe	17-Jan-1995	6.9	37.0	None	Tamura, et al. (1997)
OKAWASE	Japan	CG	167	Kobe	17-Jan-1995	6.9	40.0	None	Tamura, et al. (1997)
OTANI	Japan	E	54	Kobe	17-Jan-1995	6.9	7.0	Minor	Tamura, et al. (1997)
SHOWA-IKE	Japan	E	52	Kobe	17-Jan-1995	6.9	4.0	Minor	Tamura, et al. (1997)
GOHONMATSU	Japan	M	109	Kobe	17-Jan-1995	6.9	19.0	None	Tamura, et al. (1997)
FUKATANI	Japan	ECRD	134	Kobe	17-Jan-1995	6.9	33.0	Moderate	Tamura, et al. (1997)
KITAYAMA	Japan	E	80	Kobe	17-Jan-1995	6.9	31.0	Moderate	Tamura, et al. (1997) ; Uesaka (1999)
NITEKO	Japan	E	?	Kobe	17-Jan-1995	6.9	<10.0	Collapse	Yoshida, et al. (1999)
NUNOBIKI	Japan	CG	109	Kobe	17-Jan-1995	6.9	17.0	None	Ohmachi, et al. (1999)
KARASUBARA	Japan	CG	105	Kobe	17-Jan-1995	6.9	14.0	None	Ohmachi, et al. (1999)
YUZURUHA	Japan	CG	138	Kobe	17-Jan-1995	6.9	43.0	Minor	Tamura, et al. (1997)
AONO	Japan	CG	---	Kobe	17-Jan-1995	6.9	30.0	None	U.S.A.E. Waterways, CEWES-GV-Z (1995)
SENGARI	Japan	CG	---	Kobe	17-Jan-1995	6.9	15.0	None	U.S.A.E. Waterways, CEWES-GV-Z (1995)
ZHONG HAI	China	CG	---	Lijiang	03-Feb-1996	7.0	4.0	Serious	ASCE TCLEE (1997)
SHIH-KANG	Taiwan	CG	82	Chi-Chi	21 Sep-1999	7.6	0.0	Collapse	Charlwood (1999)

TABLE 1 (continued): HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES

DAM NAME	Country	Type	H [ft]	Earthquake Name	Earthquake Date	M or MMI	Dist. [km]	Damage Rating	Principal References Consulted
MINGTAN	Taiwan	CG	269	Chi-Chi	21-Sep-1999	7.6	----	None	Charlwood (1999)
TECHI	Taiwan	A	600	Chi-Chi	21-Sep-1999	7.6	85.0	None	Charlwood (1999)
TOU-SHIH	Taiwan	E	62	Chi-Chi	21-Sep-1999	7.6	----	Minor	AFPS (2000)
SHUI-CHIH	Taiwan	E	98	Chi-Chi	21-Sep-1999	7.6	----	Minor	AFPS (2000)

LEGEND

CA	**Concrete Arch**
GA	**Concrete Gravity Arch**
MA	**Multiple Concrete Arch**
CAB	**Concrete Arch Buttress**
CG	**Concrete Gravity**
CGB	**Concrete Gravity Buttress**
M	**Masonry**
E	**Earthfill**
Comp	**Composite (fill / concrete)**
ECRD	**Earth Core Rockfill**
CFRD	**Concrete Face Rockfill**
HF	**Hydraulic Fill**
T	**Tailings**

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AMBIESTA DAM, ITALY

Ambiesta Dam is a 194-foot high concrete arch dam located in Northern Italy. On May 6, 1976, the dam was subjected to the Gemona-Friuli (Friuli) Earthquake, an earthquake of magnitude 6.5 that resulted in hundreds of deaths and extensive property damage. A peak ground acceleration of 0.33g was recorded at the site. The dam did not suffer any damage from the main shock, nor from any of its foreshocks and aftershocks.

Ambiesta Dam

Ambiesta Dam is located near Tolmezzo, in the Eastern Alps, Italy, approximately northwest of the City of Udine (Figure 1). The 194-foot high dam is built across the Ambiesta River, a tributary of the Tagliamento River. It has a crest length of 475 feet, a crest thickness of 6.7 feet, a bottom thickness of 25.5 feet, and a reservoir storage of 2,919 acre-feet. The dam was designed between 1949 and 1954. Construction of the dam began in 1955, and was completed in 1956. The dam was constructed to impound a supply reservoir for the Medio Tagliamento-Somplago hydroelectric plant.

Located in an area of recognized high seismicity, Ambiesta Dam was designed to be earthquake-resistant. The dam was constructed as a symmetrical, double curvature arch with a marked downstream overhang, referred to as a “cupola” arch (Figure 2). The designers felt that this type of construction would offer the best capacity to withstand severe overloads. The double curvature arch abuts on a “pulvino,” which is essentially a thickened perimeter concrete joint, poured along the dam footprint.

Ambiesta Dam was built across an erosion valley, carved in dolomite of the Upper Triassic. The site is intensely fractured by faults that strike across the valley. The fracturing of the rock mass is thought to be largely the result from intense orogenic movements of the Alpine Belt. The fault zones are often filled with mylonite. However, on the valley floor, the rock is sound and shows no longitudinal faulting. The rock formations dip in the upstream direction.

In anticipation of potential earthquake effects on the structure, seismic analyses were performed during the design phase, using horizontal earthquake load coefficients. Experimental tests were also conducted on four 1:50 and 1:75 scale models of the structure (Semenza et al., 1958). Tests were first conducted by regularly increasing horizontal loads simulating hydrostatic pressure on the 1:50 scale model, until its complete failure. Failure occurred for loads about twelve times the magnitude of normal hydrostatic load. Two of the 1:75 scale models were tested for horizontal seismic forces, using a specially constructed frame and cyclic loading of the chord of the arch. Failure of the upper part of the model, at full reservoir condition, corresponded to an equivalent applied acceleration of 0.75g. Tests were also performed to simulate vertical earthquake loading on another 1:75 scale model. Collapse of the upper part of the arch occurred under repetitive vertical loads equivalent to 0.76g acceleration. It was felt at the time by the designers that the applied horizontal and vertical oscillatory “earthquake” forces would largely exceed those expected at the Ambiesta site, a recognized highly seismic area. Based on the results of these model studies, the sill of the overflow spillway structure (Figure 3) was stiffened to increase the load-carrying capacity of the crest of the arch.

The May 6, 1976, Gemona-Fruili Earthquake

The May 6, 1976, earthquake, with a magnitude of 6.5, caused 965 deaths, injured 2,286 people, and inflicted extensive property damage, estimated at $2.8 billion. The dam was located 14 miles from the epicenter. A maximum acceleration of 0.33g was recorded at the right abutment of the dam. The May 6 earthquake was preceded by a foreshock of magnitude 4.5, about one minute before the main shock. Major aftershocks of magnitude 5.1, 5.5, 5.9 and 6.0, respectively, occurred in the area over a period of approximately four months following the main shock.

Earthquake Effects and Observed Performance

Ambiesta Dam, as well as 13 other concrete arch dams in the affected region, did not suffer damage from the 1976 Gemona-Friuli earthquake sequence. Two of the other dams within the epicentral area were also thin arch dams, Maina di Sauris Dam (446 feet high), located 27 miles (43 km) from the epicenter, and Barcis Dam (164 feet high). According to the references consulted for the preparation of this case history, no differential movements within Ambiesta Dam body, and especially at the “pulvino,” were reported by the Italian engineers who inspected the dam after the earthquake.

Instrumentation and Strong Motion Records

Ambiesta Dam was well instrumented at the time of construction. Original instruments included 20 temperature gauges, 64 extensometers, 14 dilatometers and 3 inclinometers, as well as survey monuments. Several strong motion accelerographs were installed subsequently, and were functional at the time of the Friuli Earthquake. One of those accelerographs recorded a peak ground acceleration of 0.33g at one of the abutments.

Following the largest aftershock (September 15, 1976) of the Friuli Earthquake, the Instituto Sperimentale Modelli E Strutture (ISMES) installed an automatic recording system on Ambiesta Dam, including 30 seismometers, to record horizontal motions of the aftershocks. Figure 3 shows the layout of these instruments on the dam. There were five foundation locations; 20 locations along the downstream face, with two sensors mounted transversely and parallel to the valley; and 20 additional locations along the downstream face, with one sensor mounted radially. From October 8 to October 27, 1976, many smaller aftershocks were recorded, the largest with measured peak velocities of 0.10 in/s at the base of the dam and 0.41 in/s at the right abutment. Analysis of the aftershocks records indicated a 5.8:1 amplification factor between crest center and base records, in the stream (radial) direction, and a 10.6:1 amplification factor at the left abutment quarter point. Largest spectral amplifications of the recorded motions occurred at frequencies between 8 and 10 Hertz. The recorded responses of the structure to several of the aftershocks of the earthquake were compared with the corresponding theoretical responses obtained from a dynamic finite element analysis of the dam, using the processed acceleration histories of those aftershocks as input excitations. The mathematical model of the dam had been calibrated through the use of forced vibration testing with a 10-ton mechanical actuator, delivering sinusoidal oscillations at frequencies ranging from 2 to 20 Hertz. The dam analyses assumed an infinitely rigid foundation. Figure 4 shows a comparison between recorded and computed crest responses to some aftershocks of the earthquake.

Conclusions

The satisfactory observed performance of Ambiesta Dam during the 1976 Gemona-Friuli earthquake sequence is another example which confirms that arch dams have, to date, performed extremely well when subjected to strong ground shaking from nearby earthquakes of moderate size.

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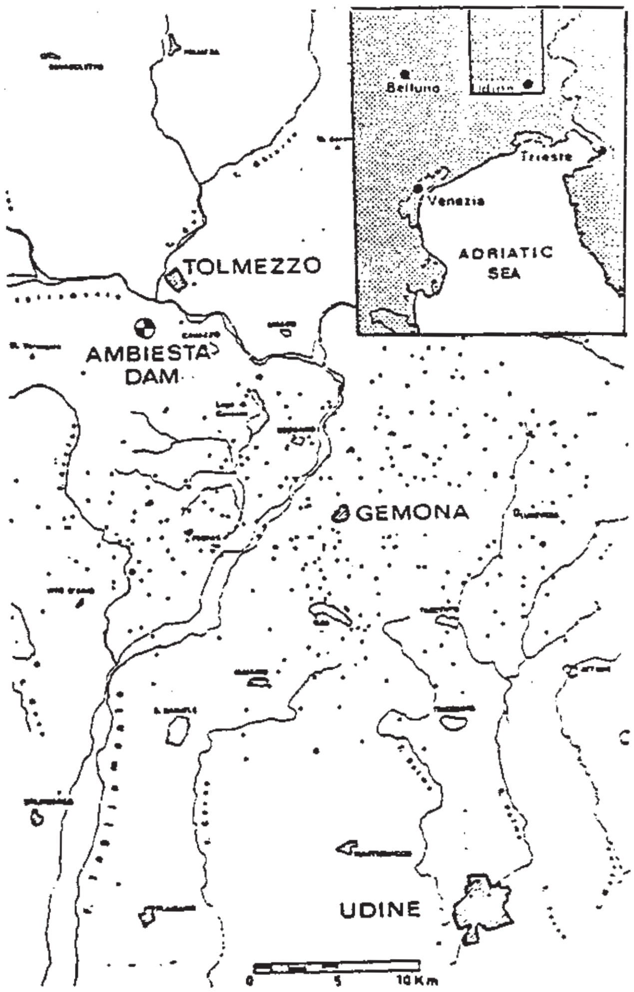

Figure 1
EPICENTERS OF SHOCKS
Ambiesta Dam
From Castoldi, 1978

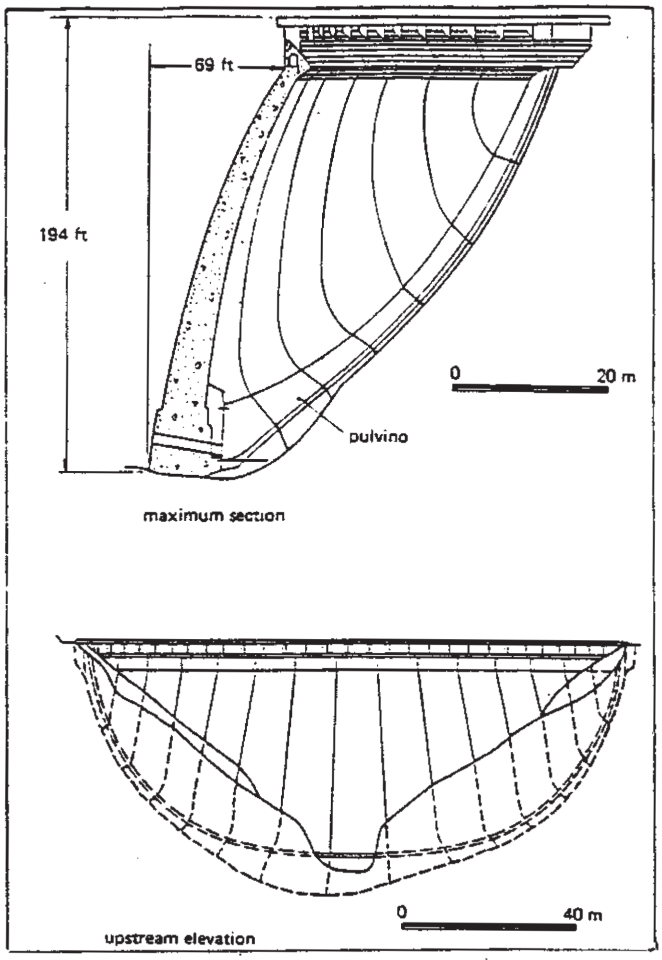

Figure 2
SECTION & ELEVATION
Ambiesta Dam
From Hansen, 1979

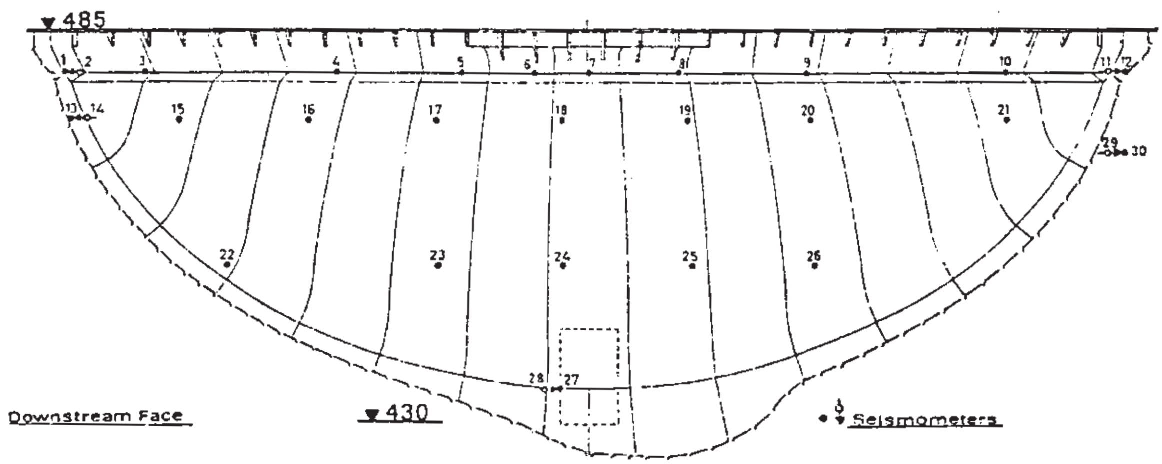

Figure 3
LAYOUT OF MEASURING POINTS
Ambiesta Dam
From Castoldi, 1978

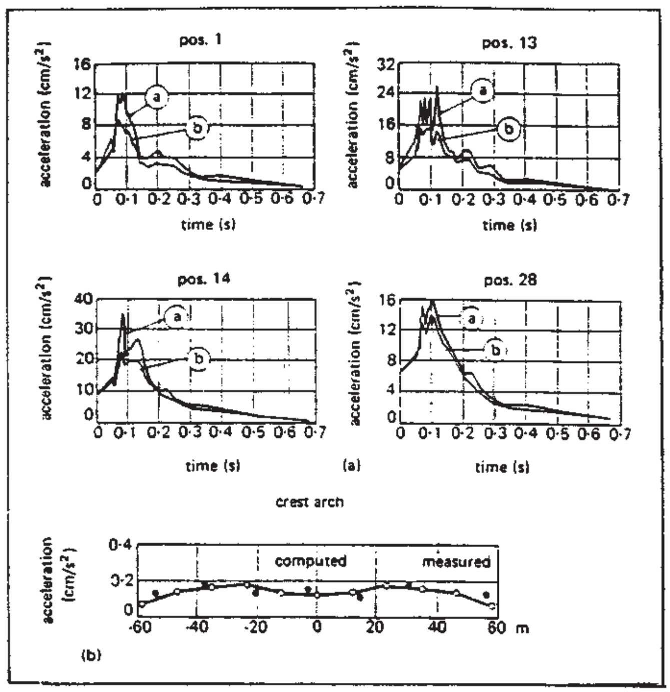

Figure 4
(a) RESPONSE SPECTRA TO AFTERSHOCKS
(b) COMPARISON BETWEEN COMPUTED & RECORDED ACCELERATIONS
Ambiesta Dam
From Fanelli, 1985

AMBUKLAO DAM, PHILIPPINES

On July 16, 1990, a large earthquake (M 7.7) struck the Philippines. Ambuklao Dam, owned by the country's irrigation and power administration, the Philippines National Power Corporation, is one of six dams that were located within a short distance from the epicenter. The dam experienced non-recoverable earthquake induced deformations of about one meter horizontally in the upstream direction and a maximum crest settlement of 1.1 meter. The spillway also experienced permanent movements and opening of a contraction joint about 50 cm wide. At the powerhouse, the turbine scroll cases became jammed and the entrance to the power intake conduit was buried under an underwater slide of the reservoir sediments. Estimated ground motion at the dam site was of the order of 0.60 to 0.65g.

Ambuklao Dam

The Ambuklao Project was placed in service in 1956. Ambuklao Dam, Luzon, Philippines is a 130-meter-high vertical core dumped rockfill dam (Figures 1 and 2.) The layout of the dam is shown in Figure 3. Crest width is 12.17 m. The upper part of the upstream and downstream slopes were built at 1.75:1 (horizontal to vertical) and the lower part of both slopes at 2:1 (h to v). The upstream and downstream slopes of the central clayey core slope at 1:4 (h to v). Both sides of the core are protected by thin filter zones. Other project features include a concrete chute spillway, an intake and power-tunnel and an underground powerhouse (Figure 3). On July 16, 1990, the date of the earthquake, the reservoir level was at El. 752 m. In the 19 days following the earthquake, the reservoir was lowered and reached a restricted elevation of 742.5 m.

The July 16, 1990, Earthquake

On July 16, 1990, the heavily populated Island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 20,000 square miles. At least 1,700 people were killed and perhaps 1,000 were missing. At least 3,500 persons were severely injured. Over 4,000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake; even many of the better hotels were damaged.

Seismologically, the July 16 earthquake is particularly difficult to characterize since it appears to have had two centers of energy release that were apparently triggered within a few seconds of each other (Figure 4). The first one was located on the Philippine Fault near the city of Cabanatuan; the second center of energy release was on the Digdig Fault, which belongs to the same system as the Philippine Fault and branches off northeast from that feature. The two faults broke along a combined length of about 75 km. The fault displacements were left-lateral strike slip. The maximum mapped displacement was on the order of 6 meters.

The energy released in the combination of the two events has been reported to correspond to a Richter magnitude of 7.7. In the years that followed the earthquake, seismologists have been continuing studies related to defining better the magnitude level, because of the difficulties resulting from the superimposition of two distinct events.

Ambuklao Dam was about 10 km from the segment of the Digdig Fault that broke on July 16, 1990. That distance is very approximate and is based on discussions with staff members from the Philippines National Power Corporation, PHILVOCS, the dam owner.

Earthquake Effects and Observed Performance

Reservoir level. On July 16, 1990, the reservoir elevation was El. 752 m. The reservoir was lowered to El. 742.5 m immediately following the earthquake.

Dam. Both the upstream shell of the dam in the vicinity of the spillway and the right training wall of the spillway experienced severe displacements. The maximum embankment damage occurred at the dam's smallest section, 20 to 30 m high, built on the ridge extension of the left abutment where the spillway is located. In order to reduce seepage and provide a better cutoff at the left abutment, where highly weathered materials were encountered during construction, an impervious clay blanket had been placed over the weathered foundation materials. Dumped rock fill was placed over the blanket and, in turn, formed the foundation for part of the spillway right approach wall.

Observed deformations of the upstream parapet wall indicate that the upstream shell of the embankment rotated in the upstream direction around a vertical axis located some 50 to 70 m from the spillway contact. The maximum horizontal movement was about one meter and occurred near the spillway wall. The two furthest upstream sections of the wall moved horizontally upstream by about 50 cm.

Adjacent to the spillway wall, the embankment appeared to have caved into a hole several meters deep. The likely cause seemed to be the opening of the spillway wall through which embankment material may have washed out during reservoir drawdown. It was postulated that the horizontal rotation of the upstream shell and section of the spillway wall was related to the presence of the clay blanket placed during construction on the left abutment ridge to improve its water tightness. The blanket terminates at El. 725 m where it forms a horizontal triangular platform, about 25 m wide at the spillway.

The upstream sections of the spillway wall were founded on a 10 m thick layer of rockfill overlying the clay blanket. Stability calculations predicted that sliding would occur on the plane at El. 725 m, for accelerations exceeding about 0.3 to 0.4g. The deformations that did occur did not present any immediate danger to the reservoir impounding capability of the dam, but it was determined after the earthquake that the reservoir should not be brought back to maximum operating pool elevation before remedial measures were taken. Other deformations of the embankment were as could be normally expected. The embankment settled 20 cm at the spillway contact, an amount that represents less than one percent of the embankment height over the left abutment ridge. Longitudinal cracks were observed near the top of the upstream shell along most of the embankment crest. These can be attributed to the settlement of the upstream shell during the earthquake. Some similar cracks were probably present on the downstream shell near the crest. A survey conducted by the owner in the months following the earthquake indicated that the dam crest settled as much as 1.1 m at the maximum section and moved upstream by about the same amount.

Spillway. The two sections of the right spillway training wall located further upstream moved in the upstream direction and rotated counter-clockwise, resulting in an opening at the contraction joint of approximately 50 cm and severe damage to a double waterstop seal installed on the spillway side of the wall. There was probably some movement (opening of the joint) at the contraction joint, where a second double waterstop seal was installed. There was no obvious damage to that other seal.

There was some concrete spalling at the spillway bridge girders and piers, which was a result of the pounding of different structural elements against each other. Also, there was some concrete spalling at the transverse joint between the spillway ogee crest and chute slab.

Powerhouse. The plant manager reported that there were no structural failures in the powerhouse. However, the turbine scroll cases became jammed with logs and debris. This was attributed to a “stirring-up” of such materials in the reservoir during the earthquake with the materials subsequently being drawn into the water intakes and scroll cases. During the process of removing the logs and debris from the scroll cases, the powerhouse was flooded. The flooding was attributed to a loosening of the draft tube bulkhead seal at Unit 3.

Power Intake. After the earthquake, the water conduit was in service until the units’ scroll cases became jammed with logs and debris. There was no indication that the intake structure had been damaged by the earthquake. The intake ports are at elevation 695 m, or approximately 47 meters below the reservoir surface elevation at the time of the inspection and, therefore, could not be observed. The reservoir bottom was surveyed by the owner following the earthquake. It appears that a massive underwater flow slide of sediments was triggered by the earthquake, raising the sediment level by some 20 m near the intake, and thus burying the sill of the power intake under about six meters of sediments.

Instrumentation and Strong Motion Records

Three weeks after the earthquake, the office of PHILVOCS indicated that no strong motion records of the event of July 16, 1990, had yet been recovered. The status of the accelerograph on Ambuklao Dam was unknown to PHILVOCS a short time after the earthquake, and no further information has been obtained.

Conclusions

Both the upstream shell of the dam in the vicinity of the spillway and the right training wall of the spillway experienced substantial deformations. These deformations, however, did not present any immediate danger to the reservoir impounding capability of the dam. Post-earthquake safety measures were taken by lowering the reservoir to a couple of meters below the spillway ogee crest.

The likely cause of the damage to the dam was sliding of the upstream rockfill shell on the clay blanket that covers the left abutment ridge and was placed to control underseepage. In sliding, the rockfill dragged along the section of the spillway training wall that is founded upon it. Some embankment materials were lost through the opening in the wall between the section that remained in place and the section that moved upstream, thereby creating the depression in the embankment surface that was visible along the wall following the earthquake.

The power intake was buried under several meters of sediments and the intake conduit was choked with silt and debris. Since the low level outlet had not been operated since 1969, and the low level intake is now under some 60 meters of sediments, there will be no emergency release of the reservoir possible at the project until the sediments are removed and the functionality of the gate is verified, a condition that could become critical after another earthquake.

References

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April 1992

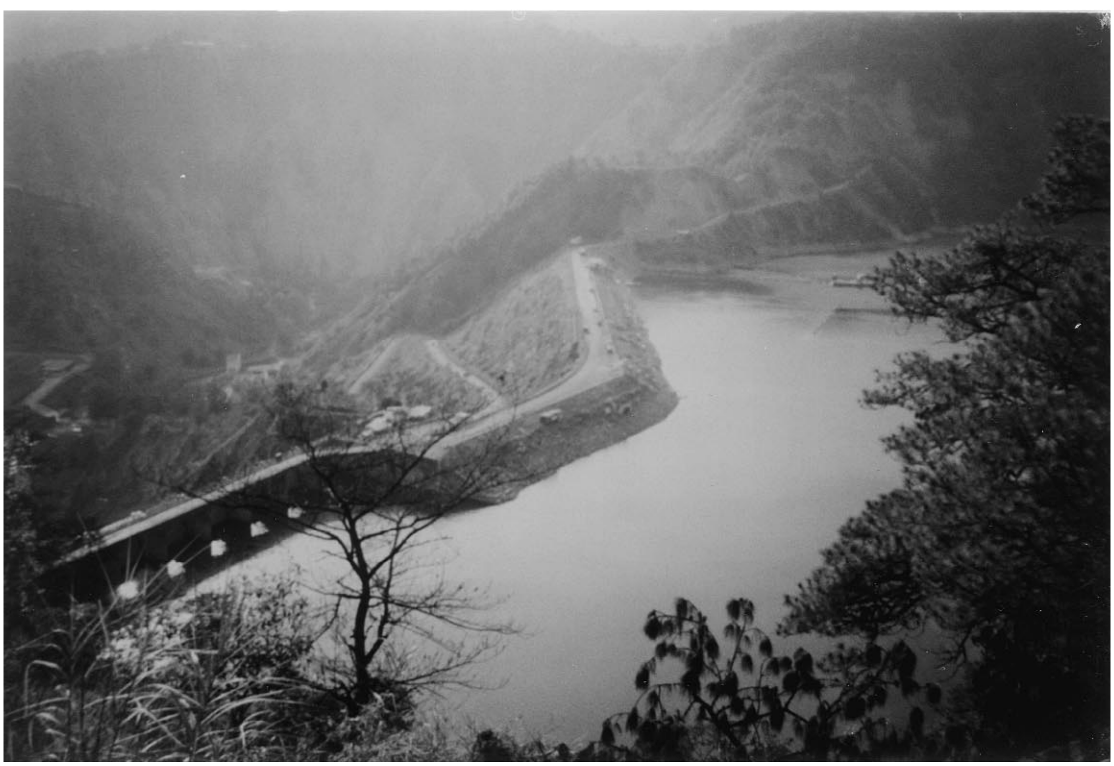
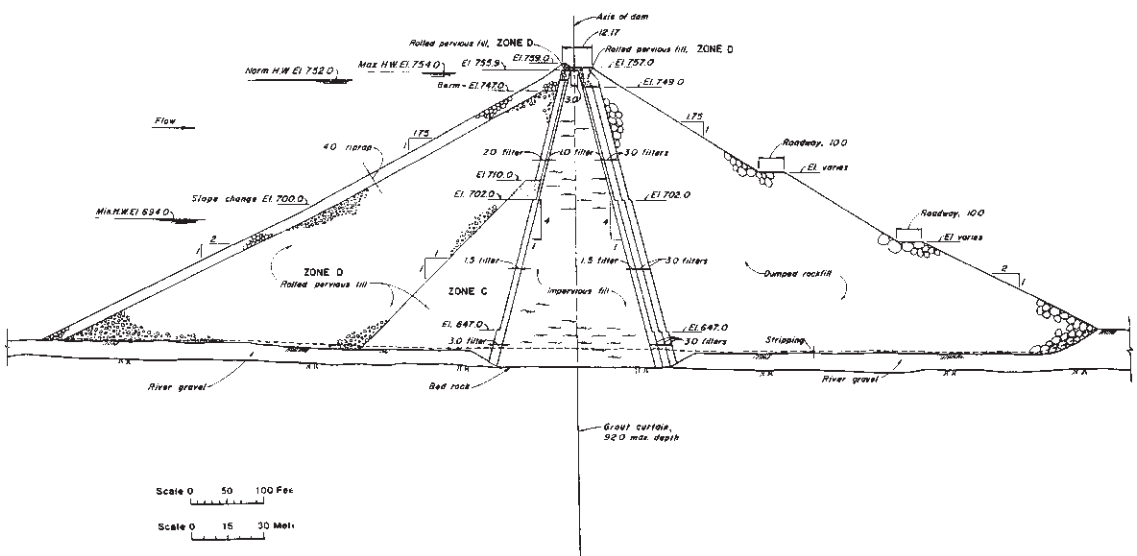


Figure 1
GENERAL VIEWS OF AMBUKLAO DAM
Ambuklao Dam

AMBUKLAO DAM											
MAXIMUM HEIGHT: 131 M (430 FEET)					FILL VOLUME: 7,674,000 M³ (10,050,000 CU. YDS.)						
MATERIAL		γ sat		γ moist		C		C'		φ	φ'
USE	DESCRIPTION	T/m^3	pcf	T/m^3	pcf	T/m^2	psf	T/m^2	psf	deg.	deg.
FILL:											
CORE	CLAY	-	-	2.08	130	2.5	500	-	-	15	
UPSTREAM SHELL, ZONE C	SAND AND GRAVEL SIZE PARTICLES FROM QUARRY	-	-	-	-	-	-	0	0	-	35
UPSTREAM SHELL, ZONE D	PREDOMINATELY GRAVEL SIZE PARTICLES FROM QUARRY	-	-	-	-	-	-	0	0	-	40
DOWNSTREAM SHELL	PREDOMINATELY COBBLE AND BOULDER SIZE QUARRY	-	-	-	-	-	-	0	0	-	40
FOUNDATION:											
OVERBURDEN	RIVER GRAVELS	-	-	-	-	-	-	0	0	-	30
FOUNDATION DESCRIPTION: METAMORPHOSED ANDESITES (FRACTURED AND WEATHERED), DIORITE.											



AMBUKLAO DAM, PHILLIPINES (1956)
(Metric)

Figure 2
EMBANKMENT, MAXIMUM CROSS SECTION
Ambuklao Dam

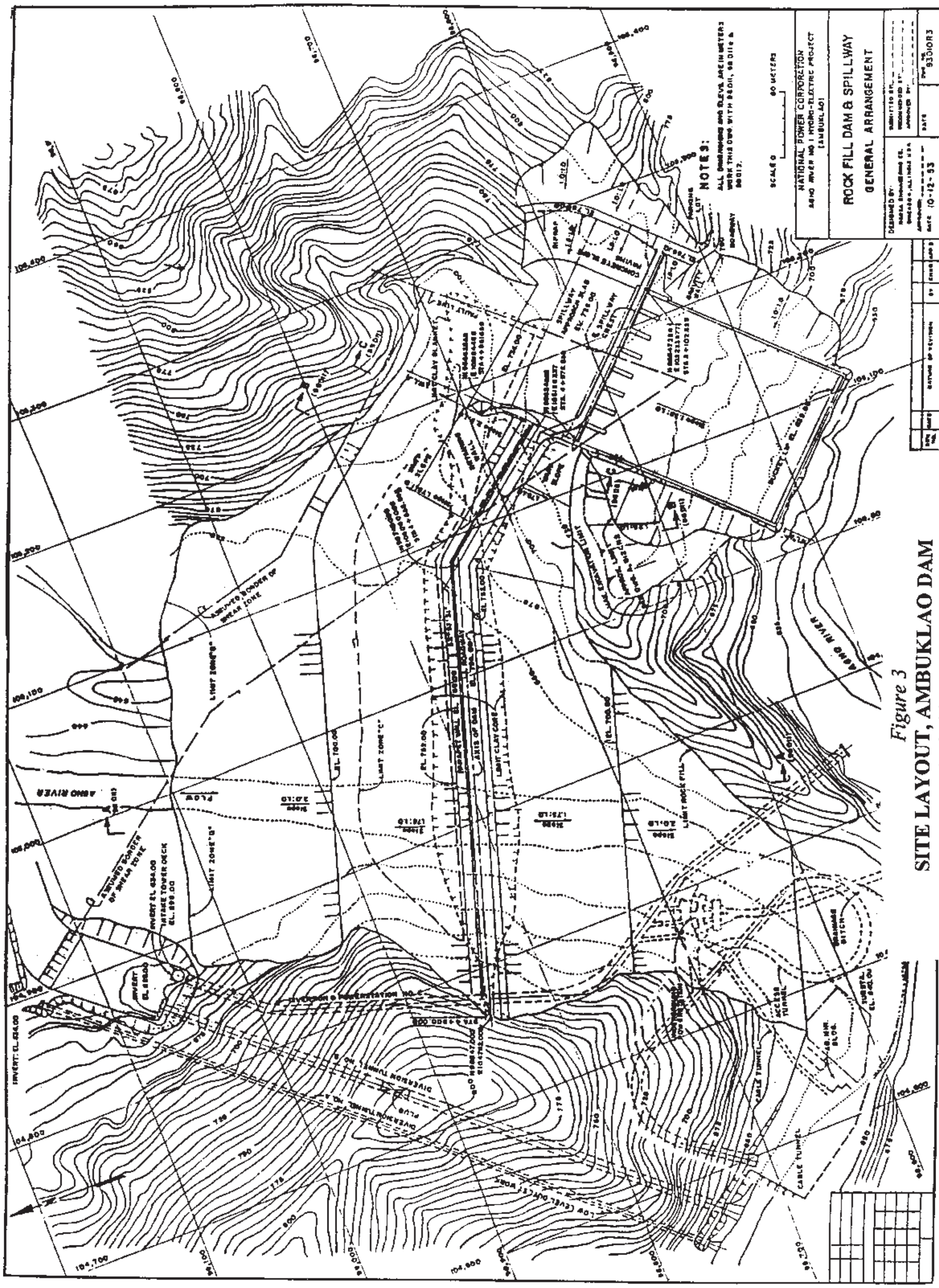


Figure 3
SITE LAYOUT, AMBUKLAO DAM
Ambuklao Dam

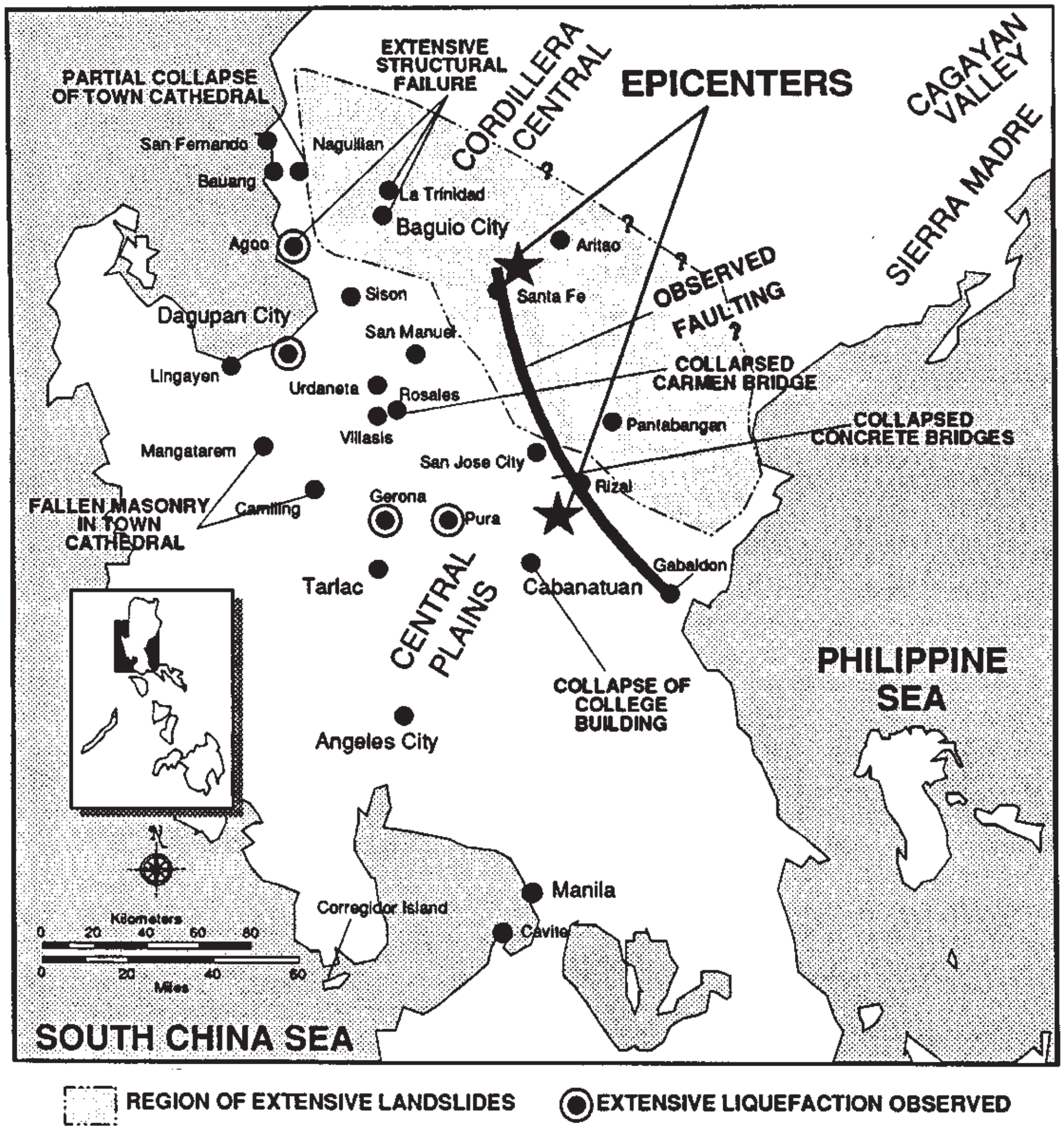


Figure 4
AMBUKLAO EPICENTER LOCATIONS
Courtesy of EQE Engineering

BEAR VALLEY DAM, CALIFORNIA, USA

For the second time in two years, Southern California was jolted awake on June 28, 1992, by the M 7.4 Landers and the Bear Mountain M 6.6 earthquakes, on the anniversary of the 1991 magnitude 5.8 Sierra Madre event. Bear Valley Dam, a rehabilitated 80-foot-high concrete dam, was strongly shaken by these two events. The closest distances between the dam and the fault ruptures were 28 miles (Landers) and 9 miles (Big Bear). Thorough inspections after the earthquakes disclosed that the dam was not damaged. The only indication of the shaking was possible slight displacement of girders on the highway bridge located on the dam crest. Estimated peak ground accelerations at the dam site were between 0.40g and 0.50g during the second event.

Bear Valley Dam

Bear Valley Dam is located on Bear Creek in the San Bernardino Mountains, 80 miles east of Los Angeles. It impounds 2,600 acre- foot Big Bear Lake, a year-round recreation facility in Southern California.

Bear Valley Dam was constructed in 1911-1912 as a 80-foot-high, 360-foot-long multiple arch structure. There are nine 17-foot radius (extrados) arches, with a crest elevation of El. 6743.2 feet. The thicknesses of the arches vary from 12 inches at the top, to a maximum of 17.5 inches. A two-lane concrete girder-type highway bridge is supported by the dam buttresses. Several years prior to the earthquake, concerns over the structural adequacy of the dam during possible severe earthquake shaking or overtopping by large floods had led to reanalysis and rehabilitation of the dam.

The structural upgrade method was conversion of the multiple arch to a gravity dam by infilling the arch bays with conventional mass concrete (Figure 1). The existing arches and buttresses functioned as the upstream and side forms for the mass concrete. The downstream slope was formed at 0.25:1 (horizontal to vertical), except for the top 47 feet, which are vertical. Approximately 15,000 cubic yards of concrete were placed. The original dam and mass concrete were made monolithic by providing a gap at their interfaces and contact grouting later. The rehabilitation was accomplished in 1988 and 1989.

The strengthening of the dam included seismic considerations. Two Maximum Credible Earthquakes (MCE) were considered, an M 8.3 earthquake centered along the San Andreas Fault (10 miles away), with a peak ground acceleration (PGA) of 0.45g and 35 seconds of bracketed duration (duration between the first and last peak of 0.05g or greater). The other was a M 6.0 event, centered on the Helendale Fault, also 10 miles away, with a 0.22g PGA and 10 seconds bracketed duration.

June 28, 1992, Earthquakes

At 4:58 a.m. on June 28, 1992, the M 7.4 Landers Earthquake occurred on the Johnson Valley-Homestead Valley-Emerson-Camp Rock faults, near the juncture of the Mojave Desert and the San Bernardino Mountains (Figure 2). The rupture zone stretched north-northwest from Sky Valley for more than 70 km, cutting across several of these known fault traces, rather than following a single previously recognized fault trace. Dramatic fault scarps and up to 20-feet of lateral offsets in the Johnson Valley have resulted from this event. Stress changes in the earth's crust resulting from this earthquake caused the M 6.6 Big Bear Earthquake on an unnamed fault to occur at 8:05 a.m. in response to the first rupture sequence. One death, due to falling masonry from a fireplace, and 400 injuries were attributed to the earthquakes. The sparse population on the desert and in the mountains is the reason for these relatively low casualty figures.

Both earthquakes occurred near the "Big Bend" of the San Andreas fault, causing scientists to speculate about a larger earthquake on this conspicuously quiet stretch of the longest fault in California.

Severe damage occurred to many structures around Big Bear Lake. The most common residential damage was broken chimneys and unreinforced masonry infill facades. Pipelines and water storage reservoirs were broken and left some desert communities without water for many days. Numerous rockfalls throughout the San Bernardino Mountains, several of them massive, blocked highways and added to the damage caused directly by the earthquake shaking. Media attention was drawn to the Yucca Bowl, a bowling alley that suffered collapse of a large wall.

Earthquake Effects and Observed Performance

The closest distances between the dam and the fault ruptures were 28 miles (Landers) and 9 miles (Big Bear). Thorough inspections after the earthquakes disclosed that the Bear Valley Dam had not been damaged. No indication of cracks or distress was visible for both the old and newer parts of the structure. The only indication of the shaking sustained by the dam was possible evidence of slight displacement of girders on the highway bridge located on the dam crest.

Instrumentation and Strong Motion Records

Bear Valley Dam was not instrumented to record earthquake motions. Accelerations of as much as 1g were recorded in Lucerne Valley. Two instruments located in Big Bear Lake City (4 miles away from the dam) and at the Forest Fall Post Office (18 miles away) provide indications of the shaking that may have been experienced at the site. At Big Bear Lake City, 0.18g (horizontal) and 0.08g (vertical) were recorded during the M 7.4 Landers Earthquake; PGAs of 0.57g (h) and 0.21g (v) were recorded during the Bear Valley Earthquake. At Forest Falls P.O., PGAs of 0.12g (h) and 0.09g (v) were measured during the first event, and 0.26g (h) and 0.30g (v) during the second event.

The Big Bear Lake City station where the 0.57g peak acceleration was recorded is on shallow alluvium over bedrock; and it was five miles closer to the causative fault break than was the dam. It is estimated that Bear Valley Dam may have experienced up to 0.40 to 0.50g at its base during the Bear Valley Earthquake. The shaking was likely less severe during the Landers Earthquake, but of longer duration.

Conclusions

The severe damage to the structures around Big Bear Lake, massive rockfalls in the vicinity and the 0.57g peak ground acceleration, measured four miles away, indicate that Bear Valley Dam was severely shaken by the June 28, 1992, earthquakes.

The dam might have been severely damaged, had it not been rehabilitated only three years before the earthquakes. Even if the unreinforced dam had not breached, the reservoir would have had to be lowered, causing impact to the local economy which is heavily dependent on the recreation lake. In this particular instance, insight of the dam owner and of the California State Division of Safety of Dams to proceed with such upgrade proved to be timely and probably avoided substantial damage during the June 1992 earthquakes.

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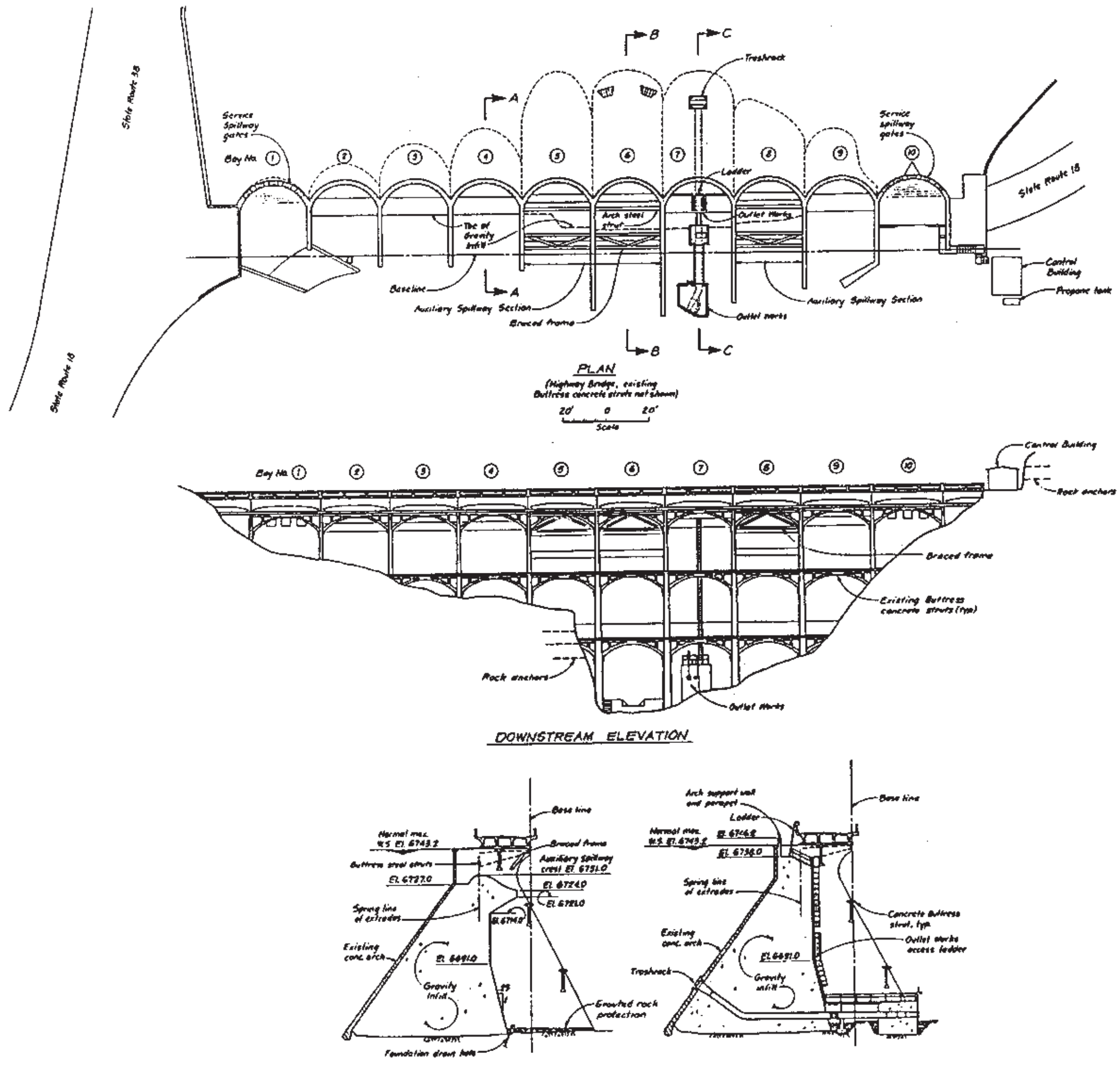

Figure 1

STRENGTHENING SCHEME

Bear Valley Dam

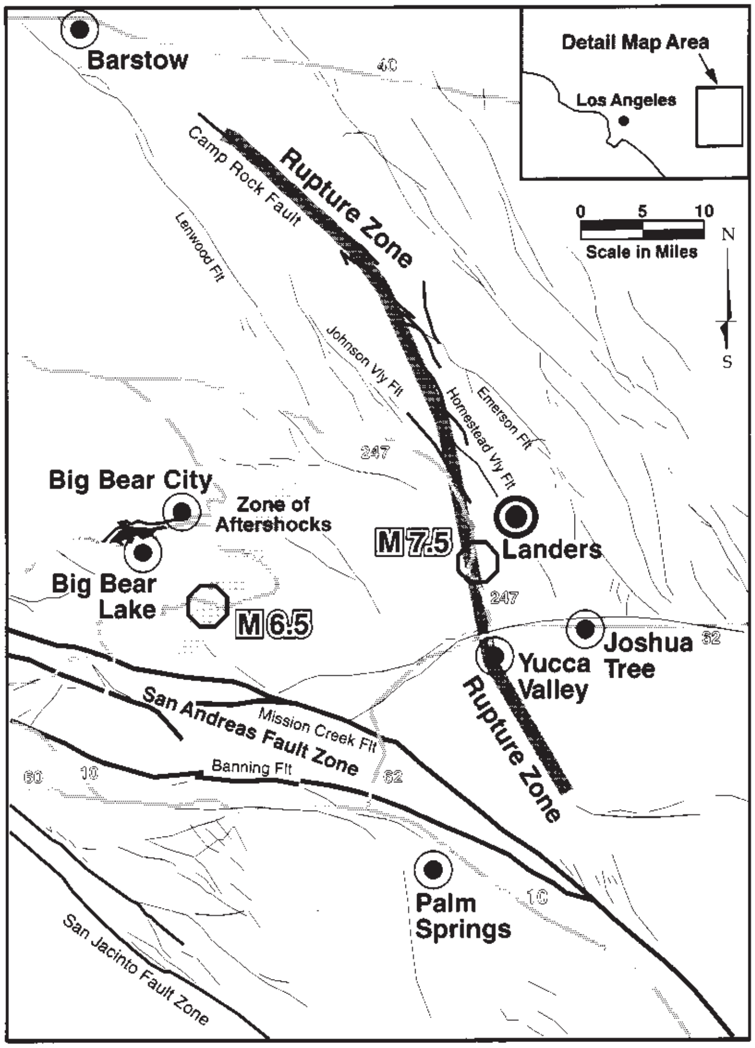

Figure 2

EPICENTRAL AND FAULT LOCATIONS

Bear Valley Dam

BINGA DAM, PHILIPPINES

On July 16, 1990, a large earthquake (M 7.7) struck the Philippines. Binga Dam, owned by the country's irrigation and power administration, the Philippines National Power Corporation, is one of six dams that were located within about 15 km from the causative fault and a short distance from the epicenter.

The greatest evidence of distress was found in the presence of about 100-meter-long longitudinal cracks along the upstream side of the dam crest. Diagonal and transverse cracks across the crest were also observed. Spalling of concrete at the extremities of the spillway bridge girders and piers was observed, and one of the spillway gates became inoperable. Binga Dam is about 15 km from the Digdig Fault, one of the two faults that ruptured during this event. Estimated peak ground acceleration at the site was about 0.60g.

Binga Dam

The Binga Dam Project (Figure 1) includes a 102-meter-high inclined core rockfill dam. Portions of the rockfill on both sides of the inclined core were rolled (compacted). The outer shells consist of dumped rockfill. Aerial photographs of the dam are shown in Figure 2. The dam layout and cross-section are shown in Figures 3 and 4. Other project features include a concrete chute spillway, an intake and power tunnel and an underground powerhouse. The Binga Project was placed in service in 1960. On July 16, 1990, the date of the earthquake, the reservoir was at El. 575 m. By August 4, 1990, the reservoir had been drawn down to El. 555 m.

The July 16, 1990, Earthquake

On July 16, 1990, the heavily populated Island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 20,000 square miles. At least 1,700 people were killed and perhaps 1,000 were missing. At least 3,500 persons were severely injured. Over 4,000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake; even many of the better hotels were damaged.

Seismologically, the July 16 Earthquake is particularly difficult to characterize since it appears to have had two centers of energy release that were apparently triggered within a few seconds of each other. The first one was located on the Philippine Fault near the city of Cabanatuan; the second center of energy release was on the Digdig Fault, which belongs to the same system as the Philippine Fault and branches off northeast from that feature. The two faults broke along a combined length of about 75 km. The fault displacements were left-lateral strike-slip. The maximum mapped displacement was on the order of 6 meters.

The energy released by the combination of the two events has been reported to correspond to a Richter magnitude of 7.7. In the years that followed the earthquake, seismologists have been continuing studies related to defining better the magnitude level, because of the difficulties resulting from the superimposition of two distinct events.

Binga Dam was about 15 km from the segment of the Digdig Fault that broke on July 16, 1990. That distance is very approximate and based on discussions with staff from the Philippines National Power Corporation, PHILVOCS.

Earthquake Effects and Observed Performance

Reservoir level. The reservoir was at its normal maximum operating pool El. 575 m at the time of the earthquake. Following the earthquake, the reservoir was quickly drawn down at a rate of about several meters per day, based on its observed level at El. 555 m at the time of a post-earthquake inspection. Such a high rate of drawdown is likely to have contributed to some of the damage observed along the upstream side of the dam crest.

Dam. The dam was severely shaken by the earthquake. The greatest evidence of distress was found in the presence of longitudinal cracks along the upstream side of the dam crest. The length of the cracks, which were located over the maximum section of the embankment, was on the order of 100 m. The crack widths varied up to 30 cm. The cause of the cracks could have been attributed to sliding of the upstream rockfill shell along the sloping core possibly as a result of the inertia forces induced by main shock and aftershock motions, but also likely was the result of the high rate of drawdown of the reservoir following the earthquake. Such interpretation was supported by a report from the powerplant manager, who stated that the cracks apparently opened to their maximum width a few days after the main shock.

Other less severe features of damage on the dam crest were suspected to be due to a combination of several possible causes:

Settlement of the dumped rockfill shells, causing longitudinal cracks on the crest both upstream and downstream;

- Tensile stresses caused by differential settlements induced by changes of geometry in the foundation of the dam’s right abutment, producing diagonal cracks across the crest; and
- Embankment settlement causing tensile stresses at the contact with the spillway structure and producing a transverse crack across the crest.

Spillway. There was some concrete spalling at the ends of the spillway bridge girders and supporting piers. The spalling was attributed to the occurrence of pounding between the girders and piers as a result from the earthquake shaking of these structures. The plant operator reported that spillway gate No. 2 was inoperable following the earthquake. The gate hoist tripped off before the gate could be moved.

Powerhouse. The Binga Powerhouse is underground and the plant manager reported that there was no damage and that the turbine/generator units were believed to be fully operational. The powerhouse was not inspected.

Weir. A weir installed at the toe of the Binga Dam measures embankment seepage collected. It was reported that there was no change in the quantity of seepage measured before and after the earthquake. The water remained clear at all times, indicating no evidence of piping of core materials.

Instrumentation and Strong Motion Records

Three weeks after the earthquake, the office of PHILVOCS indicated that no strong motion records of the event of July 16, 1990, had been recovered. The status of one accelerograph that was located on Binga Dam was unknown to PHILVOCS a short time after the earthquake. No further information has been obtained.

Conclusions

The cracks observed on the dam crest were regarded as not serious with respect to the immediate safety of the dam. Repairs were recommended, however, following the post-earthquake inspections of the embankment. The formation of these cracks was attributed to either of several factors or their possible combination, including: settlement of the rockfill shells; sliding of the upstream shell along the sloping core or as a result of the high rate of drawdown of the reservoir following the earthquake; differential settlement near the right abutment due to variations of the foundation geometry; and embankment settlement causing tensile stresses at the contact with the spillway structure and producing transverse cracking across the crest.

Based on this example, the sloping core design could be considered to have been somewhat detrimental to the stability of the upstream shell during an earthquake. It is unlikely, however, that a slide of the upstream shell would progressively lead to breaching of a dam such as Binga, since it would have to propagate through the core and the unsaturated downstream shell.

References

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Swaisgood, J.R.; Au-Yeung, Y. (1991), “Behavior of Dams During the 1990 Philippines Earthquake,” Association of State Dam Safety Officials, San Diego, California, pp. 296-314.

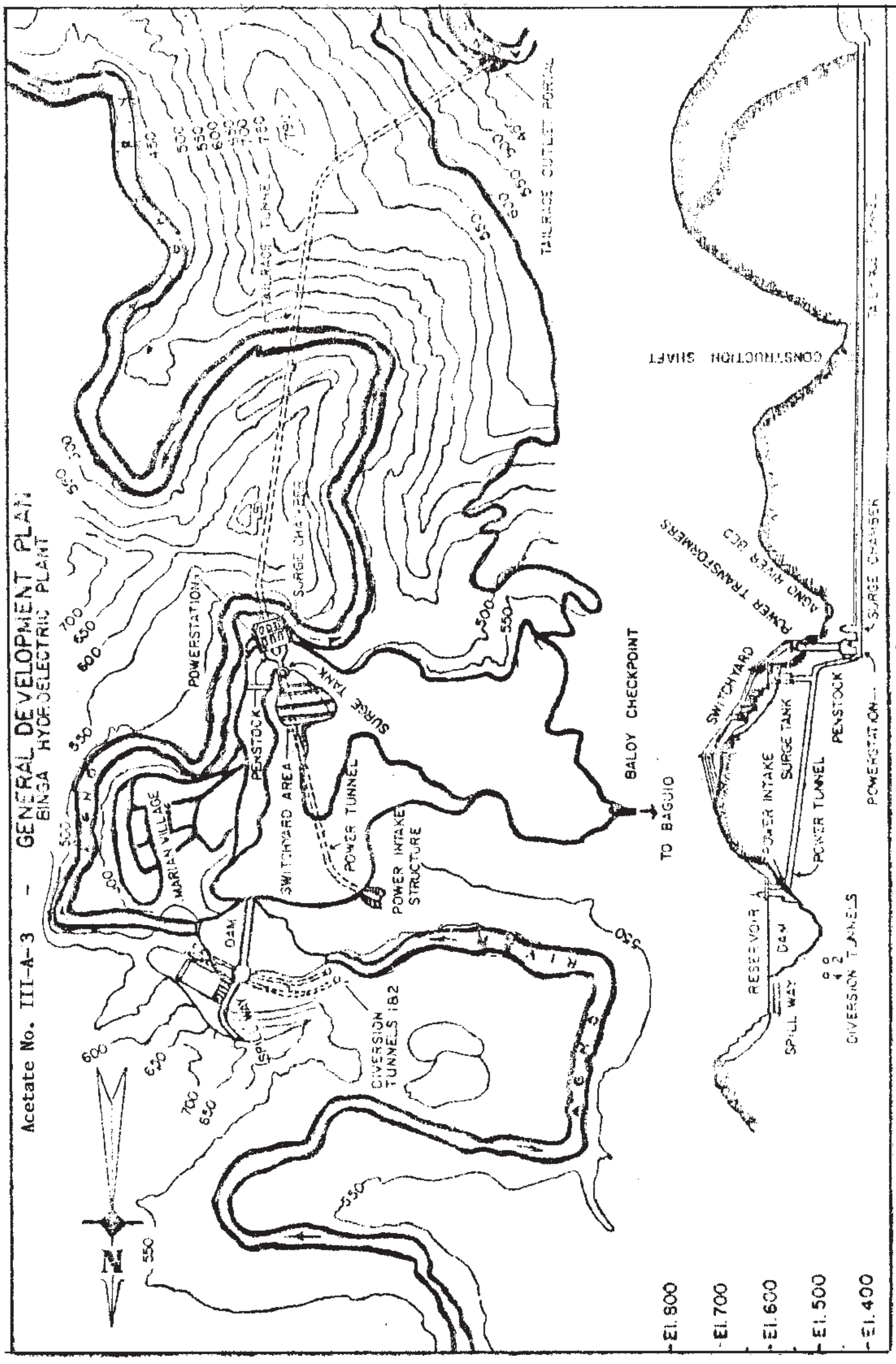

Figure 1
BINGA HYDROELECTRIC DEVELOPMENT
Binga Dam

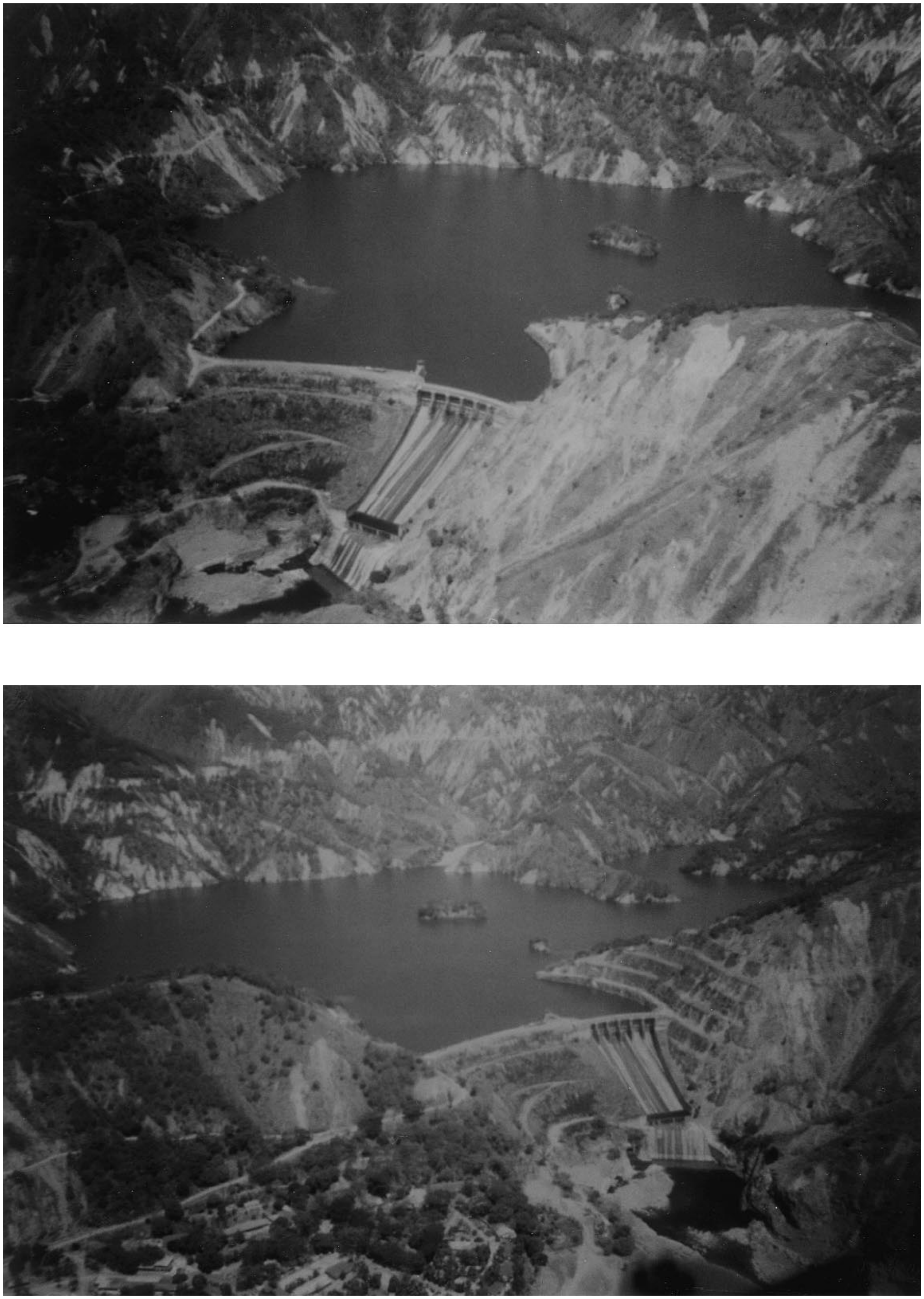

Figure 2
AERIAL VIEWS
Binga Dam

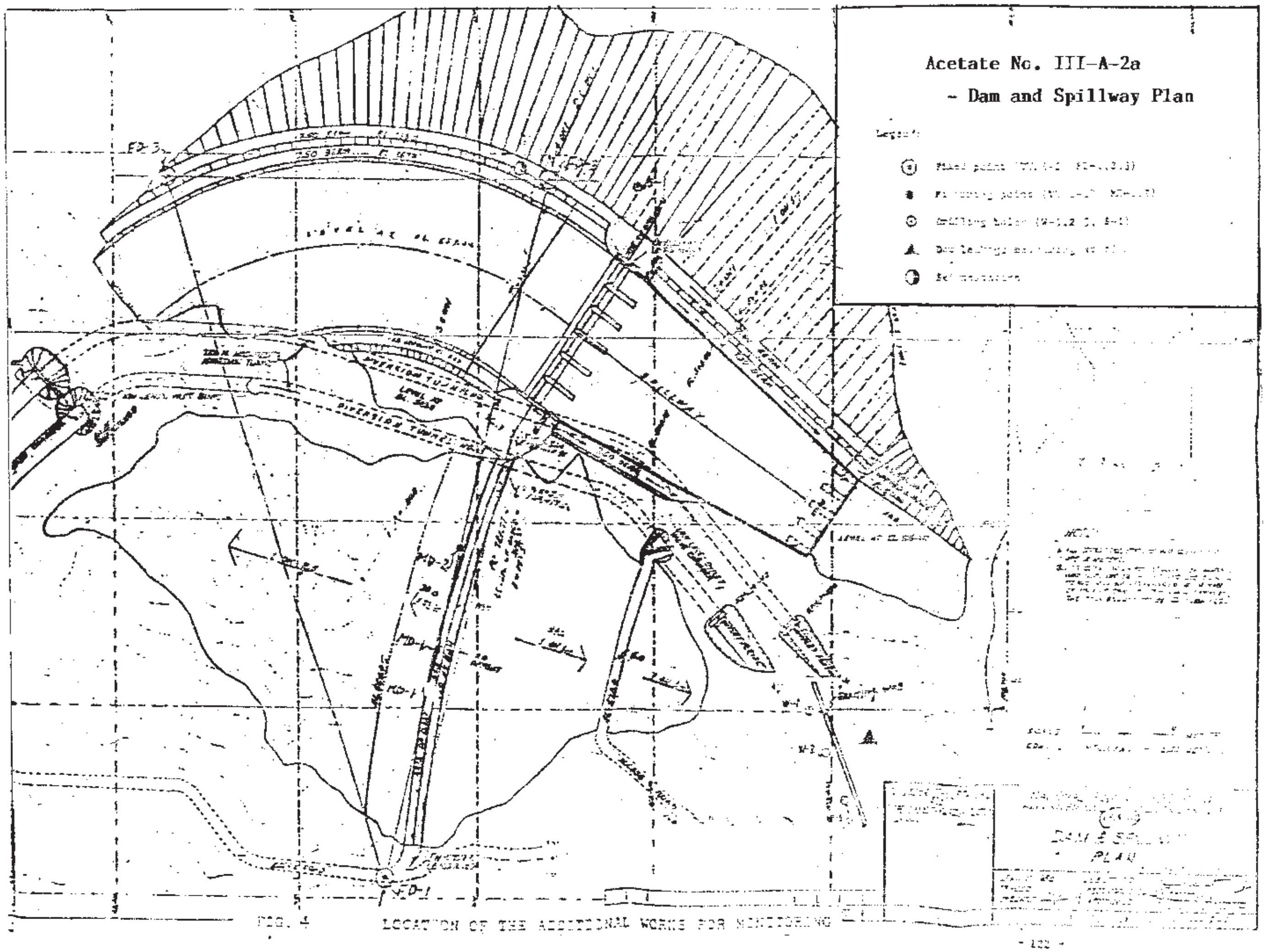

Figure 3
BINGA PROJECT LAYOUT
Binga Dam

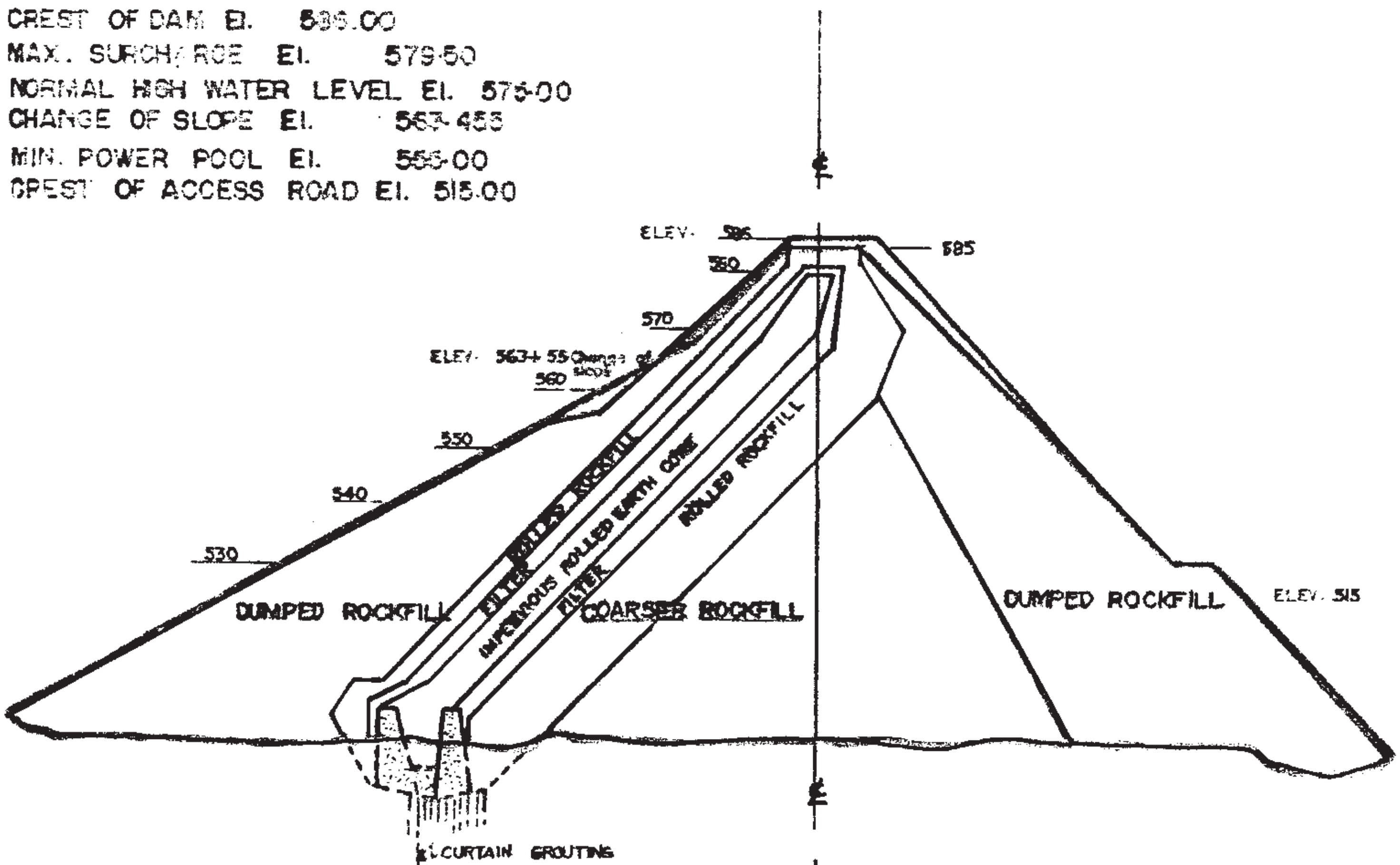

Figure 4
MAXIMUM SECTION
Binga Dam

CERRO NEGRO TAILINGS DAM, CHILE

Cerro Negro Tailings Dam Number 4 is one of two tailings impoundment facilities that failed during the March 3, 1985, central Chile Earthquake (M 7.8). The dam was built by mixed techniques which ranged from the upstream to the centerline methods of tailings dams construction. Failure was concluded to have been caused by progressive loss of strength in the liquefaction-susceptible tailings slimes. Due to the absence of downstream population, no injuries were reported.

Cerro Negro Dam Number 4

The Cerro Negro tailings impoundment Number 4 was built outward from a valley side slope. The dam consisted of three sections, a central section roughly parallel to the valley floor, and two transverse sections linking the central section to the valley slope (Figure 1).

The dam was constructed by separating tailings into sand and slime fractions by means of a small cyclone. The sand fraction was hydraulically placed along the perimeter of the impoundment to gradually build the outer dam, while the slimes were discharged into the reservoir. Tailings impoundment started in 1972, but was interrupted from 1980 to 1984. From 1984 to the time of occurrence of the 1985 earthquake, tailings were deposited at a rate of about 600 tons per day.

Reportedly, the outer dam was erected by a combination of the centerline and upstream methods of construction, depending on the availability of sand. In the centerline method, the location of the crest of the dam remains the same as successive lifts of the dam are built; the downstream toe of the dam, therefore, moves progressively toward downstream. Conversely, in the upstream method, the outer slope is kept fixed as the dam is being raised, while the crest location is displaced toward upstream; in that second method, the downstream toe remains in its original position. At the time of the 1985 earthquake, the central section of the dam had a maximum height of about 98 feet (30 m) and an average outer slope of about 1.7:1 (horizontal to vertical).

Three borings were drilled in 1987 at the locations shown in Figure 1 (Castro and Troncoso, 1989). Three zones were encountered: an outer zone consisting of the sand fraction; an intermediate zone of stratified sands and slimes; and an interior zone of slime. The boundaries shown in Figure 1 are based on boring logs data and observations of the walls of a large crevasse formed during the failure. This zoning confirmed the information regarding construction, which indicated a procedure intermediate between upstream and centerline construction methods. All three borings encountered a natural foundation material consisting of a dense gravelly sand.

The zone forming the sand fraction of the tailings (Figure 1) actually ranged from a silty fine sand, with about 20 percent of silt, to a non-plastic sandy silt. The percentage of fines increased with distance from the outer slope, as shown in Figure 2. Standard penetration testing (SPT) corrected blowcounts within the sand zone increased gradually with depth from about 10 blows/foot near the surface to about 30 blows/foot at a depth of about 66 feet (20 m).

The slimes consisted of a slightly plastic clayey silt, with a plasticity index typically in the range of 5 to 20. Blowcounts in the slimes are believed not to be representative of the conditions that prevailed at the time of the failure, since surficial drainage and dessication between 1985 (when the failure occurred) and 1987 (when the borings were made) probably caused a substantial increase of their measured strength.

The March 3, 1985, Chile, Earthquake

On March 3, 1985, at 19:47 local time, a strong earthquake shook central Chile, causing widespread destruction and resulting in 180 deaths, over 2,500 persons injured and about $2.6 billion in property damage. The earthquake had a magnitude of 7.8 (Ms). Its focus was located off the coast of Chile, at a depth of 16 km, and within a recognized subduction zone where the Nazca tectonic plate underrides the South American tectonic plate. The epicenter location and peak accelerations instrumentally recorded at various sites within the mesoseismal area are shown in Figure 3. The primary earthquake damage involved both old and modern buildings, industrial facilities, bridges, road embankments, and small earth and tailings dams.

Within 180 km of the epicenter, there were 16 active tailings impoundments in which about 140,000 cubic meters of tailings per day were being impounded. Two of these impoundments developed dam slope failures caused by liquefaction, leading to large releases of tailings with resulting negative environmental impacts. However, largely due to the absence of downstream population, no injuries were reported.

Central Chile is known to be seismically active, and previous earthquake-related failures of tailings dams were reported in 1928, 1965, 1971 and 1981. Two of these past failures, Barahona Dam in 1928, and El Cobre Dam in 1965 (Dobry, 1967) caused many deaths and led to restricting mining regulatory requirements regarding where tailings impoundments can be sited relative to populated areas. These requirements were further tightened after the 1985 earthquake: nowadays, large tailing impoundments must be contained behind a conventionally-designed, well-compacted embankment dam.

Earthquake Effects and Observed Performance

A detailed description of the earthquake effects on the Cerro Negro Dam Number 4 can be found in Castro and Troncoso (1989). As a result of the earthquake, a portion of the central section of Dam Number 4 dam failed, and about 130,000 metric tons of slimes and sands were released, forming a large crevasse and breaching the impoundment (Figure 1). Piles of sand, up to ten feet (a few meters) in height, were found within about 330 feet (100 m) of the dam, while some slimes flowed into the Pitipeumo Creek and downstream along the valley for distances of about five miles (eight kilometers). A witness indicated that the failure had been preceded by noticeable sloshing of the slimes during the earthquake, and it occurred rather suddenly.

As a result of the failure, a shallow layer of slimes flowed out into the crevasse and through the breach in the dam. Along the upstream edge of the outer dam, a series of shallow slides through the impoundment were also observed. Numerous craters and small sand and silt boils were found throughout the slimes area of the impoundment.

The Cerro Negro Dam Number 4 had an intake structure located near the valley slope. The intake structure was used to recover excess water. The tower was displaced upwards and tilted about 10 degrees as a result of the earthquake.

The 1987 investigation (Castro and Troncoso, 1989) revealed that the outer core of sands was medium dense, with a friction angle of about 36 degrees and undrained steady-state strength (S_{us}) values ranging from about 1.3 to 2.9 kg/cm^2, with a median of 2.0 kg/cm^2. The undrained strengths of the slimes were estimated based on laboratory vane shear test results, and were the following:

Undrained Peak Strength, S_{up}: $S_{up}/v_c = 0.27$

Undrained Steady-State Strength, S_{us}: $S_{us}/v_c = 0.07$

where vc represents the vertical effective overburden pressure.

Stability analyses based on the above strength estimates indicated that failure had been caused by a reduction of the effective strength of the slimes due to the earthquake shaking. The slimes were weakened to the point of reaching their residual strength, S_{us}.

Instrumentation and Strong Motion Records

No strong motion records were obtained at or near the dam, but peak ground accelerations were recorded in the general area surrounding the site (Figure 3). From an examination of Figure 3, it appears that the horizontal peak ground acceleration (PGA) at the dam site probably ranged from 0.30 to 0.40g. Acceleration time histories with similar PGAs recorded during this event generally show at least ten pulses with accelerations half of the PGA or greater, i.e., between 0.15 to 0.20g.

Conclusions

The failure of Cerro Negro Tailings Dam No. 4 during the 1985 earthquake represents one of many instances in which tailings dams built by hydraulic fill procedures have failed during earthquakes. The width of the available sand zone becomes a crucial factor in maintaining stability of such dams during earthquake shaking. The slimes have a very low undrained strength, while sands are generally better drained and can achieve a reasonable in-situ density (medium dense in the case of the Cerro Negro Dam). Availability of a wider sand zone, which would have been expected to remain reasonably well drained, should have improved the stability of the embankment.

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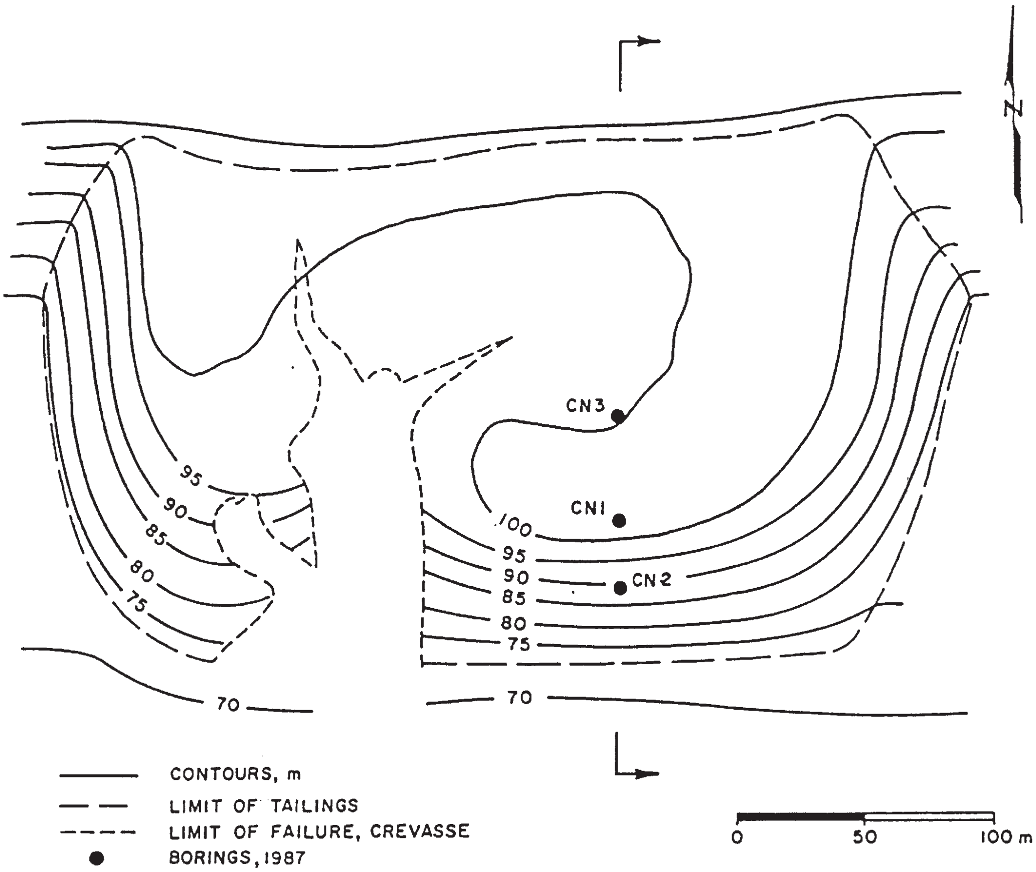
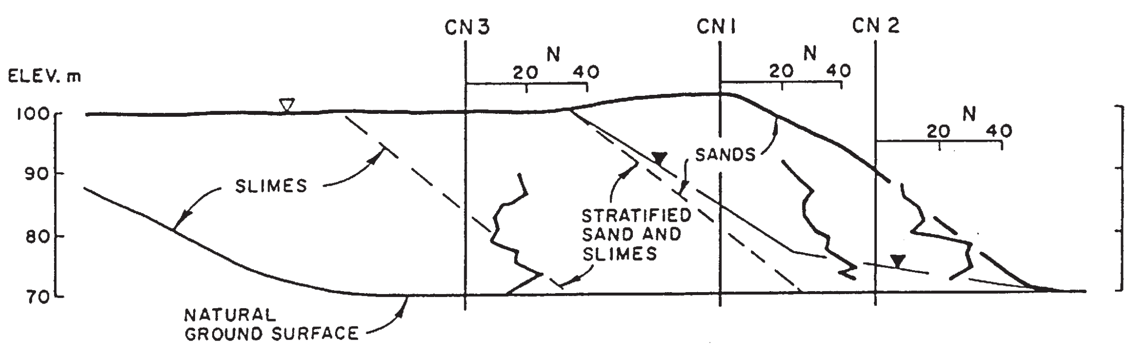


PLAN

After Castro and Troncoso (1989)

Survey performed in 1987



CROSS SECTION

Figure 1

PLAN & SECTION

Cerro Negro Tailings Dam

From Castro and Troncoso, 1989

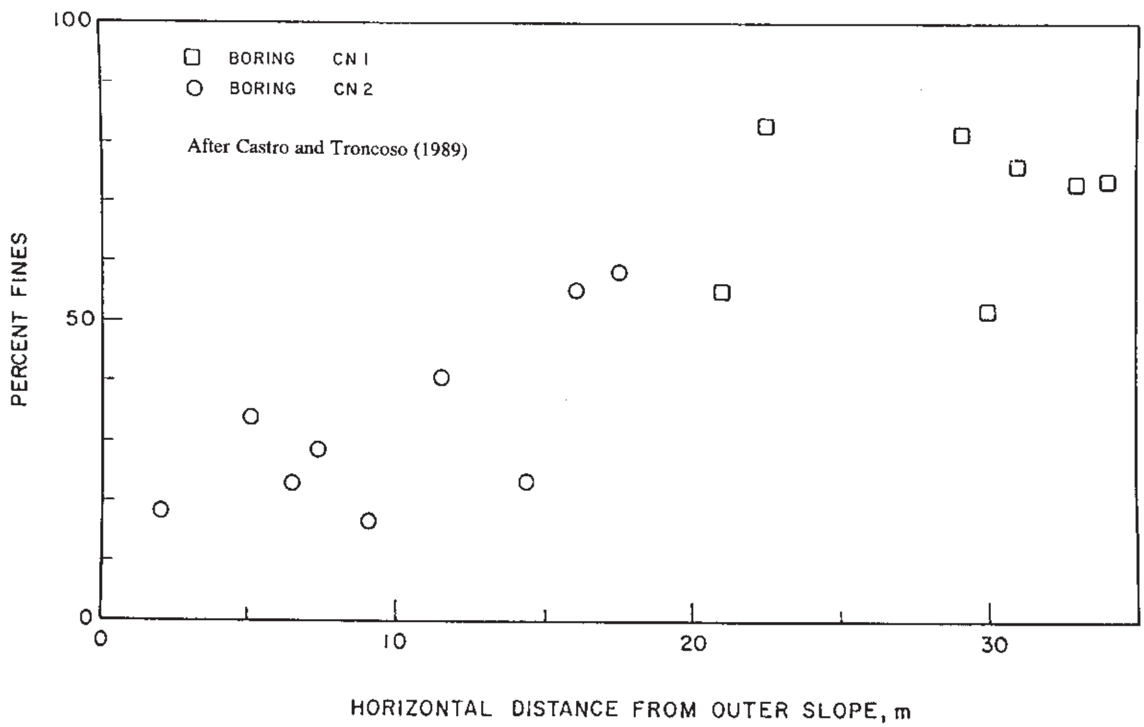
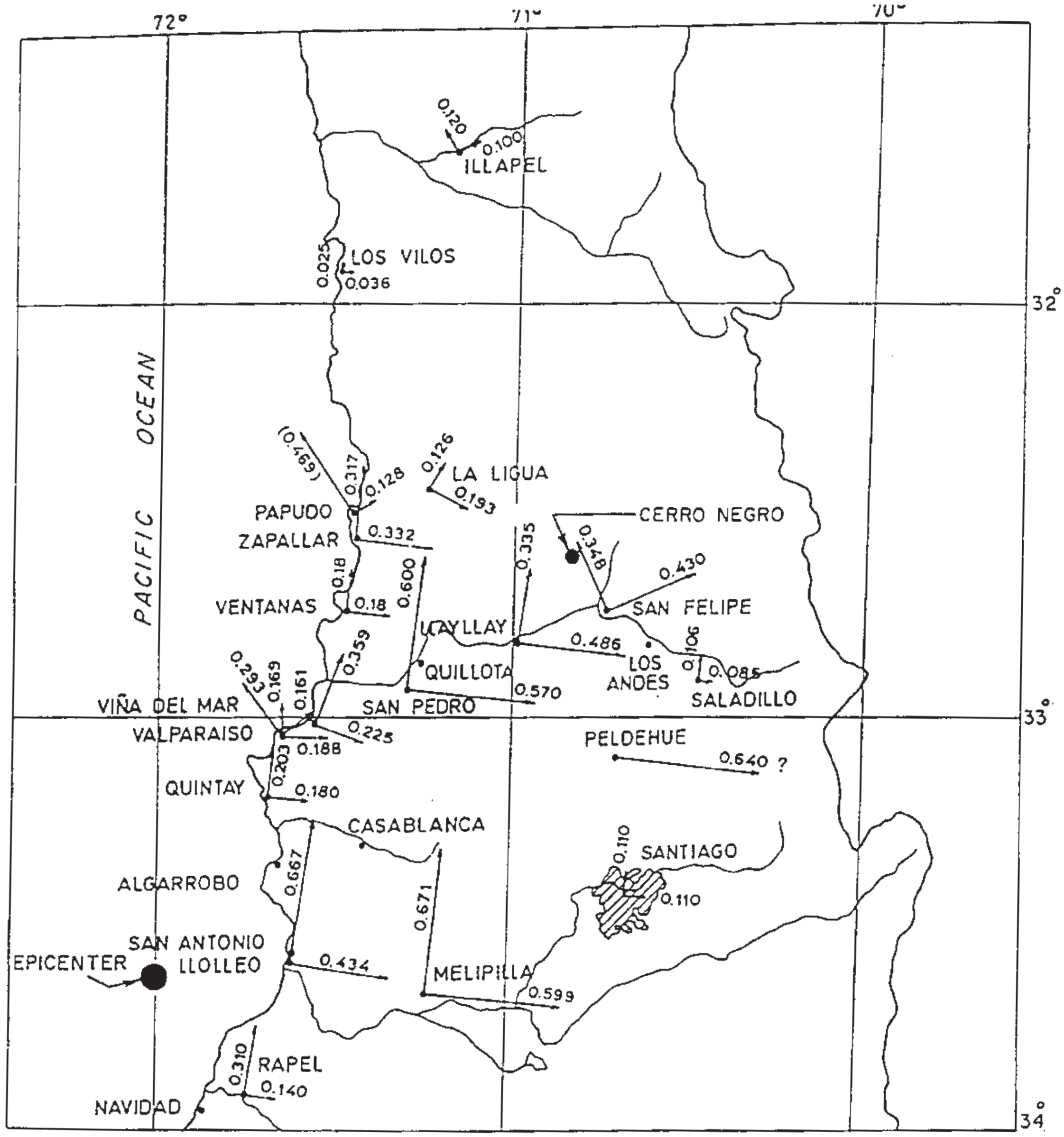


Figure 2
PERCENT FINES CONTENT
Cerro Negro Tailings Dam

After Saragoni et al. (1986)



NOTES: ARROWS INDICATE DIRECTION OF RECORDED PEAK HORIZONTAL ACCELERATION, NOTED AS A FRACTION OF GRAVITY

Figure 3
EPICENTER AND RECORDED ACCELERATIONS
Cerro Negro Tailings Dam

CHABOT DAM, CALIFORNIA

On April 18, 1906, the San Francisco Bay area was shaken by a magnitude 8+ earthquake that ruptured about 270 miles of the San Andreas Fault. This earthquake, known as the “Great San Francisco Earthquake,” resulted in substantial damage and destruction. Chabot Dam, a compacted clayey fill embankment about 130 feet high, is located about 19 miles from the San Andreas Fault and was strongly shaken by the earthquake. Peak ground acceleration at the dam site was estimated to be about 0.40g. The dam suffered no significant or observable damage.

Chabot Dam

Construction of Chabot Dam, formerly known as Lower San Leandro Dam, was begun in 1874 and completed in 1892 (McLean, 1937). Chabot Dam is situated on San Leandro Creek near the eastern boundary of San Leandro and the southern boundary of Oakland, California (Figure 1). The reservoir impounded by the dam has a storage capacity of approximately 12,000 acre-feet at the lower spillway crest elevation of 227.2 feet. There is a higher spillway crest at El. 233. The crest of the dam is at El. 243 feet (with a 2-foot-high concrete parapet wall extending to El. 245) and is about 400 feet long. At its maximum section, the dam rises about 165 feet above bedrock and 130 feet above the original streambed. The embankment contains about 622,000 cubic yards of material. A view of the dam is shown in Figure 2 and a cross-section through the embankment is shown in Figure 3. These figures apply to the original construction of Chabot Dam. In 1984, the embankment was raised when a new spillway was built.

Construction began in 1874 with stripping of the proposed dam “footprint” area to a depth of up to 3 feet to remove vegetation, roots, and loose topsoil. A core trench ranging from 10 to 30 feet in depth was excavated to bedrock. The main body of the dam (referred to as “wagon fill” in Figure 3) was constructed during 1874 and 1875 as a rolled-fill structure, employing teams of horses and horse-drawn equipment for transporting and compacting fill material. Although mention was made that selected material was placed in the “core,” subsequent exploration programs showed that the wagon fill can be characterized as a homogeneous mass of predominantly silty sandy clay with clayey sand and gravel.

In 1875, the dam crest was at El. 233. During the period from 1875 to 1888, the wagon fill was reinforced on the downstream slope by a sluiced fill buttress (referred to as “Hydraulic Fill” in Figure 3), which was initially constructed to El. 185. Between 1890 to 1891, this hydraulic fill buttress was raised to El. 222. During the period of 1891 to 1892, the wagon fill was raised to the present day crest elevation of 243 feet. Also, at that time, a berm was placed on the upstream face of the dam where an apparent slide had occurred during construction. During 1892 to 1895, local sandstone riprap was added to the upstream face from El. 200 to 240. This riprap was grouted in 1912, at which time the concrete parapet wall was constructed along the upstream edge of the dam crest to Elevation 245.

The San Francisco 1906 Earthquake

This probably was the greatest shock felt historically in California. It originated on the San Andreas Fault, north of San Francisco, and had a surface fault rupture of about 270 miles. Maximum horizontal surface displacements of 21 feet were observed near Tomales Bay. Ground fissuring along the San Andreas Fault was observed from Upper Mattole in Humbolt County to San Juan Bautista in San Benito County. Damage in the filled areas of the cities of San Francisco, Santa Rosa and San Jose was extensive. The earthquake had an estimated magnitude of about 8.3, caused at least 700 deaths and about $400 million (1906 dollars) in damage.

Earthquake Effects and Observed Performance

During the 1906 San Francisco Earthquake, the estimated intensities in the vicinity of the dam site were of the order of VII to VIII on the Modified Mercalli Intensity (MMI) Scale. The dam was in normal operation at the time of the earthquake, with the reservoir level at El. 232 (Figure 4). Older files of the Contra Costa Water Company show no records of any reported damage and, likewise, a review of the "Report of the State Investigation Commission" (Lawson, 1908) indicates that neither the dam nor the reservoir experienced any problem as a result of the earthquake (Woodward-Lundgren & Associates, 1974). It should be noted, however, that subsequent studies by Makdisi et al. (1978) suggested that Chabot Dam may have settled between 0.3 and 0.4 feet as a result of that event.

Chabot Dam provided the opportunity for one of the first witness descriptions of reservoir seiching. In an inspection report retrieved from the files of the California Division of Safety of Dams (dated May 27, 1930), Mr. G.F. Engle included a testimony from the dam resident caretaker, Mr. Tierney, as follows: *"Mr. Tierney also interestingly relates that a few minutes after the earthquake of April 18, 1906, he arrived at the dam and was surprised to find the water about 3 feet lower than it had been the night before. Thinking it had escaped through a rupture in the dam he commenced an investigation. In a few minutes, however, a wave traveled down the reservoir to slap up against the dam and return the water to its normal level of the night before. Apparently during the quake a tidal effect occurred in which the water was piled up in the upper reaches of the reservoir and soon returned in a prominent wave. Mr. Tierney says that no damage to the dam or appurtenant structures was evident as a result of the shock....at the time the reservoir was full."*

Instrumentation and Strong Motion Records

Chabot Dam was not instrumented at the time of the 1906 San Francisco Earthquake. The intensity of ground shaking (based on observed damage) in the San Francisco area was estimated at between VII and XI on the Rossi-

Forell scale (Lawson, 1908). Chabot Dam is located approximately 19 miles east of the San Andreas Fault, and peak bedrock acceleration at the dam site due to the 1906 San Francisco Earthquake was estimated at about 0.40g (Woodward-Lundgren & Associates, 1974; Makdisi et al., 1978).

Conclusions

Chabot Dam, a 130-foot-high compacted embankment, was strongly shaken during the 1906 San Francisco earthquake. The dam suffered no significant or observable damage. The embankment consisted of predominantly sandy clays with clayey sands and gravel and this experience confirms that, generally, well-compacted clayey dams can withstand severe ground motion shaking without experiencing significant damage. Estimated peak ground acceleration at the dam site were about 0.40g with a duration of significant shaking of about 50 seconds. Detailed dynamic finite element analyses were performed (Makdisi et al., 1978) to estimate embankment deformation due to ground motions similar to those experienced during the 1906 earthquake. The estimated settlement was found to be in reasonable agreement with the observed performance of the embankment and the lack of observed damage.

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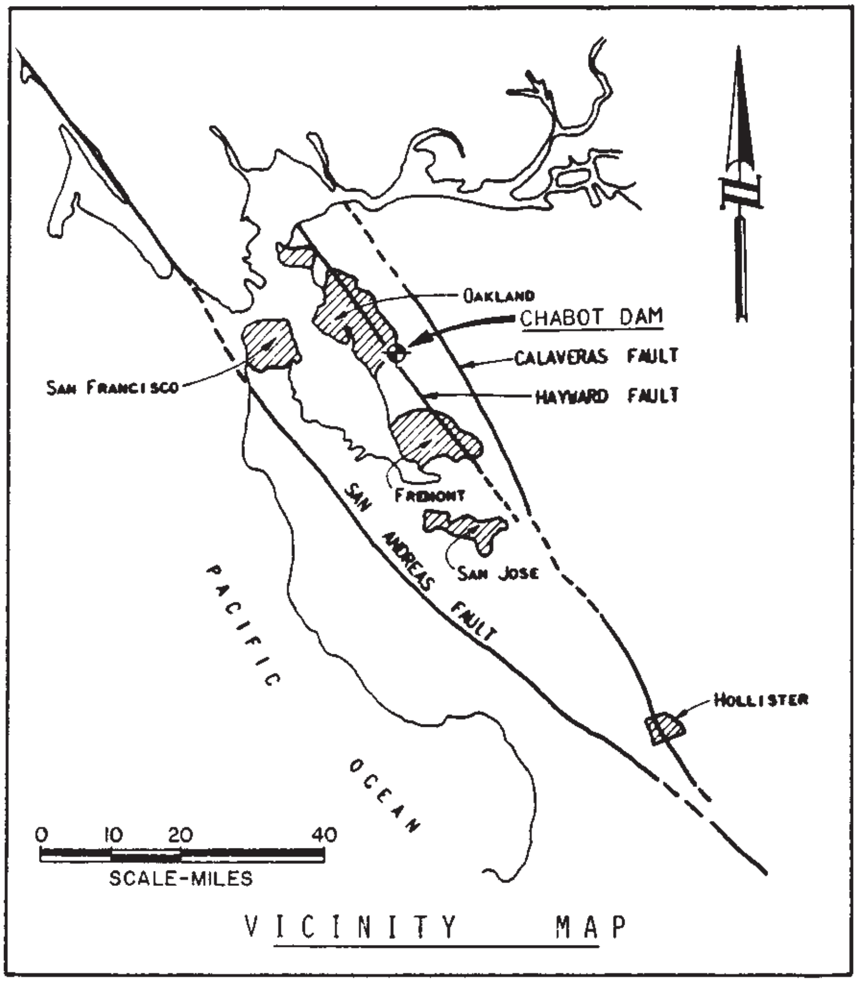


Figure 1
LOCATIONS OF CHABOT DAM RELATIVE TO SAN ANDREAS FAULT
Chabot Dam

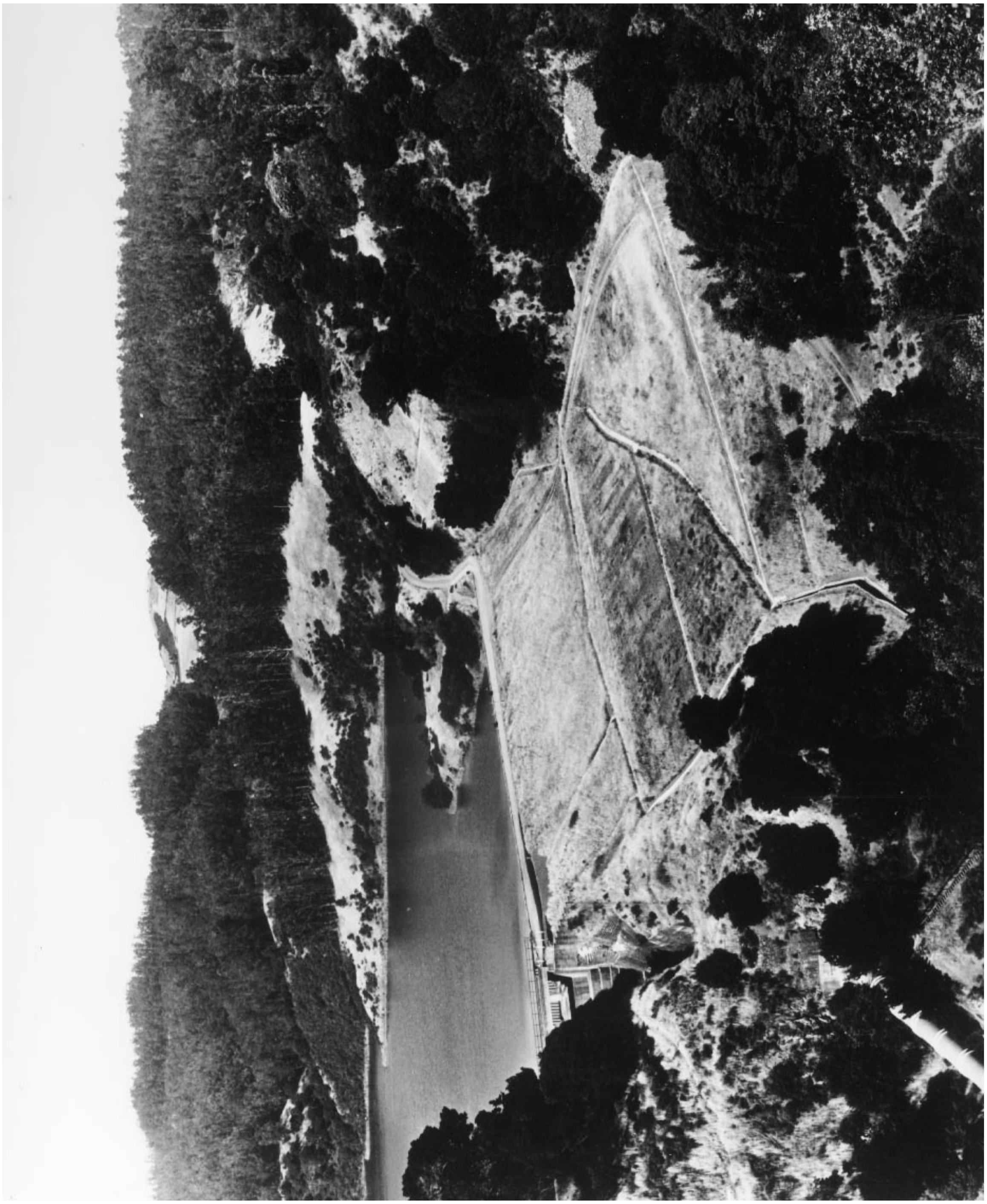
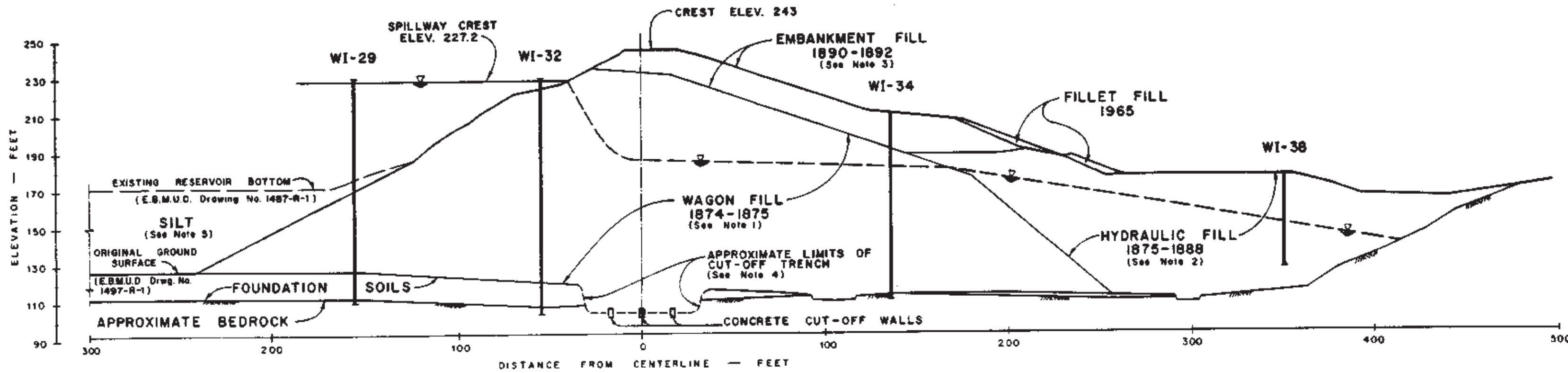


Figure 2
VIEW OF CHABOT DAM
Chabot Dam



SECTION A-A

NOTES:

1) WAGON FILL
CONSISTS MOSTLY OF CLAYEY SAND WITH FINE GRAVEL SIZE ROCK FRAGMENTS AND SOME POCKETS OF SILTY SANDY CLAY WITH SHALE FRAGMENTS.

2) HYDRAULIC FILL
CONSISTS MOSTLY OF VERY LEAN SANDY SILTY CLAY WITH GRAVEL AND ZONES OF GRAVELLY SILTY SANDS AND GRAVELLY CLAYEY SANDS.

3) EMBANKMENT FILL
CONSISTS OF HYDRAULIC FILL BELOW ELEVATION 222± AND OF WAGON FILL ABOVE.

4) INFORMATION ON CUTOFF TRENCH AND WALLS OBTAINED FROM EAST BAY WATER CO., DRAWING NO. 41-D, DATED 4-26-37.

5) THE SLOPE OF THE WAGON FILL AT THE UPSTREAM TOE WAS ASSUMED AS SHOWN TO EXTEND FROM THE EXISTING RESERVOIR BOTTOM TO THE ORIGINAL GROUND SUFRACE. IN THE DYNAMIC ANALYSES SILT, PRESENT AT THE UPSTREAM TOE OF THE DAM, WAS DETERMINED TO BE OF LITTLE CONSEQUENCE.

6) PHREATIC SURFACE FROM E.B.M.U.D. RECORDS 1964 TO 1972.

Figure 3
CROSS-SECTION THROUGH CHABOT DAM IN 1906
Chabot Dam

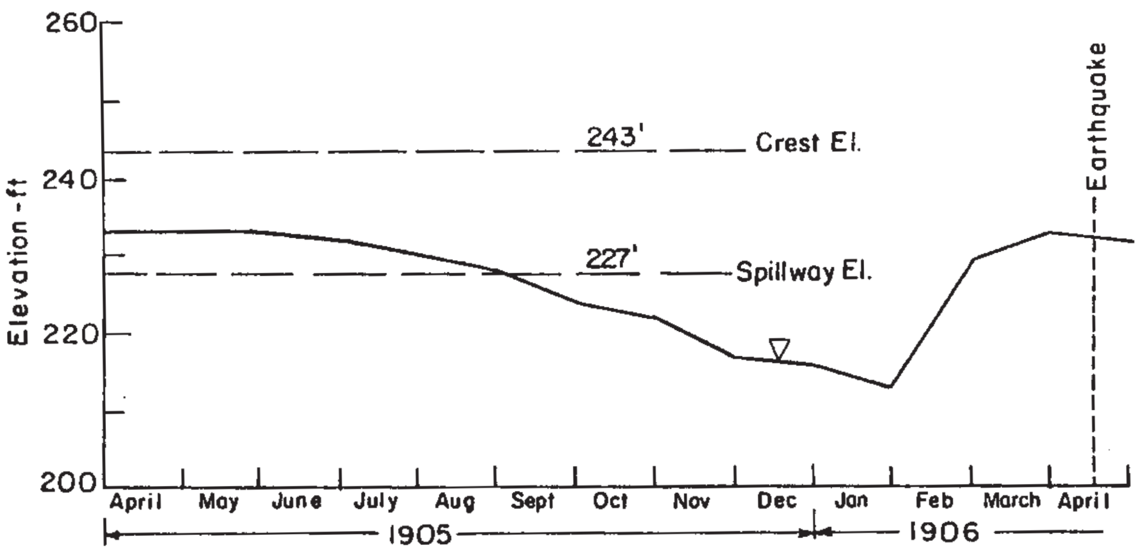


Figure 4

WATER LEVELS IN RESERVOIR OF CHABOT DAM
BEFORE AND DURING 1906 EARTHQUAKE

Chabot Dam

COGOTI DAM, CHILE

On April 6, 1943, a large earthquake (M 7.9) occurred approximately 125 miles (200 km) north of the City of Santiago, Chile. This earthquake, centered about 59 miles (95 km) from the Cogoti Dam site, affected this 280-foot (85-m) high rockfill dam, built in 1938. Peak ground acceleration at the site was estimated to be about 0.19g. Substantial settlement of Cogoti Dam was observed as a result of this earthquake.

Cogoti Dam

Cogoti Dam, a concrete face rockfill dam, is located in the Province of Coquimbo, Chile, about 47 miles (75 km) from the City of Ovalle. The dam site is situated within the foothills of the Andes Mountains, downstream from the confluence of the Pama and Cogoti Rivers, and in a deep gorge naturally carved by the Cogoti River. Cogoti Dam, shown in plan and cross section in Figure 1, has a maximum height of 280 feet, a crest length of 525 feet and a total rockfill volume of about 915,000 cubic yards. The upstream slope averages 1.4:1 (horizontal to vertical) and the downstream slope is about 1.5:1 (horizontal to vertical). The dam is primarily used for irrigation purposes and impounds a reservoir of 120,000 acre-feet capacity.

Local rock, which consists primarily of andesitic breccia, was used for construction. According to available construction reports, the main rockfill zone was started by blasting some of the abutment rock and allowing the blasted rock fragments to fall freely on the foundation. Following completion of the required abutment excavation, rockfill was dumped in lifts as thick as could be practical, and without mechanical compaction or sluicing.

The flexible, impervious, segmented reinforced concrete face was placed on a 6.6 foot-thick bedding zone of hand-placed, small-size, rock. It was designed as individually formed slabs, of 32.8 x 32.8 feet (10 x 10 m) average size, with a thickness tapering from 31.5 inches at the upstream toe to 8 inches at the crest of the dam. Horizontal and vertical joints with 24-inch-wide copper waterstops and rivets were provided. The spacing and bar sizes of the steel reinforcement vary as a function of elevation along the dam face, starting with a double curtain of one-inch bars at 12-inch spacing near the toe and ending with a single curtain of 3/4-inch bars at 8-inch spacing at the crest.

The spillway is an ungated channel with a reinforced concrete side-channel having broad crested weir control, and was excavated in the left abutment rock. It has a design capacity of 176,000 ft3/s.

The April 6, 1943, Earthquake

The April 6, 1943, Illapel Earthquake destroyed most of the towns of Combarbala, Ovalle and Illapel, about 125 miles (200 km) north of the City of Santiago. Damage was reported in a wide area, some including the City of Santiago. However, few references, and none of these technical, describe this earthquake. Presumably, this is because the affected onshore area is mountainous, was sparsely populated and was probably considered of minor economic importance in 1943. The shock was, however, felt as far away as Buenos Aires, Argentina, where dishes were broken and ink spilled from ink wells. Damage extended throughout the province of Coquimbo. A copper mine tailings dam collapsed near the City of Ovalle, killing five persons. Total reported lives lost were eleven. The epicenter was determined to be offshore, directly across the mouth of the Limari River. Earlier magnitude estimates were as high as 8.3, but were subsequently lowered to a maximum of 7.9. Many aftershocks were felt during the week that followed the earthquake.

Earthquake Effects and Observed Performance

The Illapel earthquake was centered about 59 miles (95 km) from Cogoti Dam. An intensity IX on the Rossi-Forel scale was reported at the dam site. The reservoir is believed to have been at its normal operating level at the time of occurrence of the earthquake. The principal observed effect of the 1943 earthquake on Cogoti Dam was to produce an instantaneous settlement of up to 1.35 feet. Settlement occurred throughout the length of the crest and the extreme upper part of the concrete face slab was exposed from the downstream side, as quoted in an internal report by ENDESA S.A., Santiago, Chile (1972). It is of interest to note that the maximum earthquake-induced settlement was about equal to that observed in the 4.5 years since the end of construction. The point where this settlement was measured was near the center of the crest, where the dam height is about 207 feet. This was not the highest dam section, which is located close to the right abutment. The settlement at the maximum dam height was less, presumably because of a restraining effect due to the nearby presence of the very steep abutment. Minor rockslides also occurred along the downstream slope of the dam.

Leakage had been observed at Cogoti Dam since the reservoir’s first filling in 1939. Intermittent records have been kept over the years, which indicate leakage to be directly related to the elevation of the reservoir and probably coming through the abutment or foundation, rather than the dam itself. No significant increase in dam leakage was observed as a result of the 1943 earthquake. No face cracks were caused by the earthquake. Yearly settlement and leakage data at Cogoti Dam are presented in Figure 2.

The dam has continued to settle after the 1943 earthquake. Interestingly, it was shaken again by three significant, although considerably more distant earthquakes: in 1965 (La Ligua Earthquake, M 7.1); in 1971 (Papudo-Zapallar Earthquake, M 7.5); and in 1985 (Llolleo-Algarrobo Earthquake, M 7.7). These more recent events, however, were centered at distances of more than 100 miles (165 km) from the dam and did not induce any noticeable settlement. Yet, in 1971, even though the reservoir was empty at the time of occurrence of that earthquake, the Papudo-Zapallar earthquake caused longitudinal cracking at the dam crest and dislodged some rocks along the downstream slope.

Instrumentation and Strong Motion Records

Cogoti Dam was not instrumented at the time of the 1943 earthquake, nor were accelerometers installed that could have recorded the subsequent earthquakes. Using an attenuation equation primarily developed from Chilean earthquake data (Saragoni, Labbe and Goldsack, 1976), the peak ground acceleration (PGA) induced at the Cogoti Dam site by the Illapel Earthquake was estimated to be 0.19g (Arrau et al., 1985). Peak ground accelerations generated by the subsequent earthquakes were probably less than 0.05g, therefore confirming that noticeable settlements were unlikely to occur under such moderate shaking conditions.

Conclusions

Cogoti Dam, a 280-foot-high concrete face rockfill dam, was constructed in 1938 and subjected in 1943 to ground motion of probably significant amplitudes and duration. Although significant settlement occurred, the dam performed extremely well and no seismic damage was observed to the concrete face. Although a rockfill construction method now obsolete had been used (which explains the observed settlement), Cogoti Dam's performance substantiates the generally accepted belief that concrete face rockfill dams have an excellent inherent capacity to withstand substantial earthquake motion without experiencing significant damage. Although Cogoti Dam's leakage has increased over the years, this has been related to aging and spalling of the concrete and joint squeezing, not to the 1943 Illapel nor to any of the subsequent earthquakes to which the dam was exposed.

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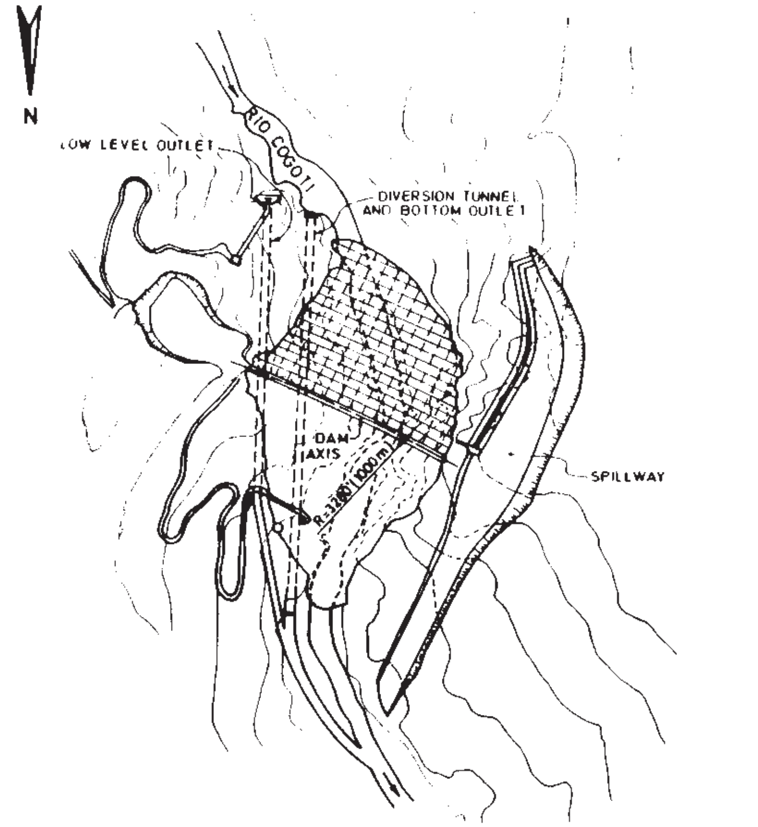
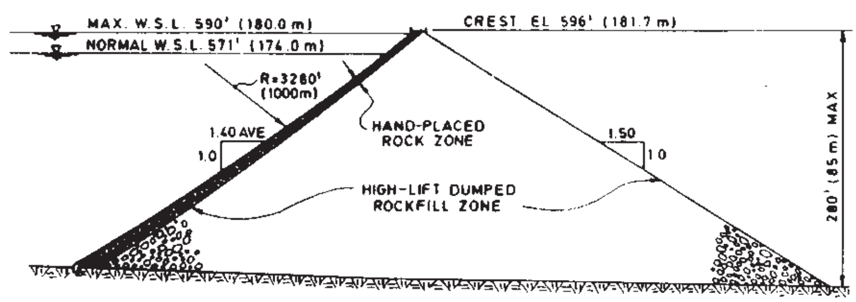


COGOTI DAM – SITE PLAN



COGOTI DAM – TYPICAL SECTION

Figure 1
CROSS-SECTION AND LAYOUT
Congoti Dam

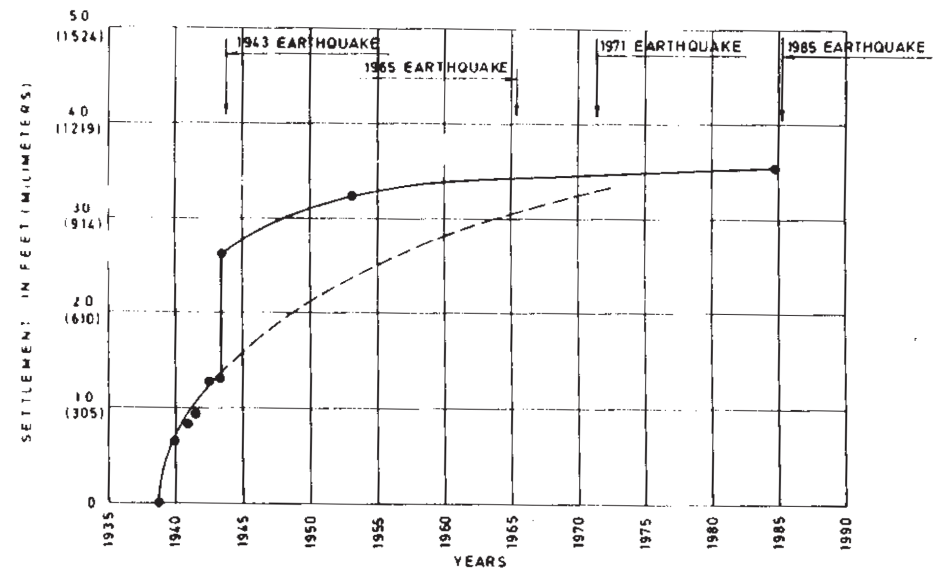


FIGURE 6 - CREST SETTLEMENT CURVE

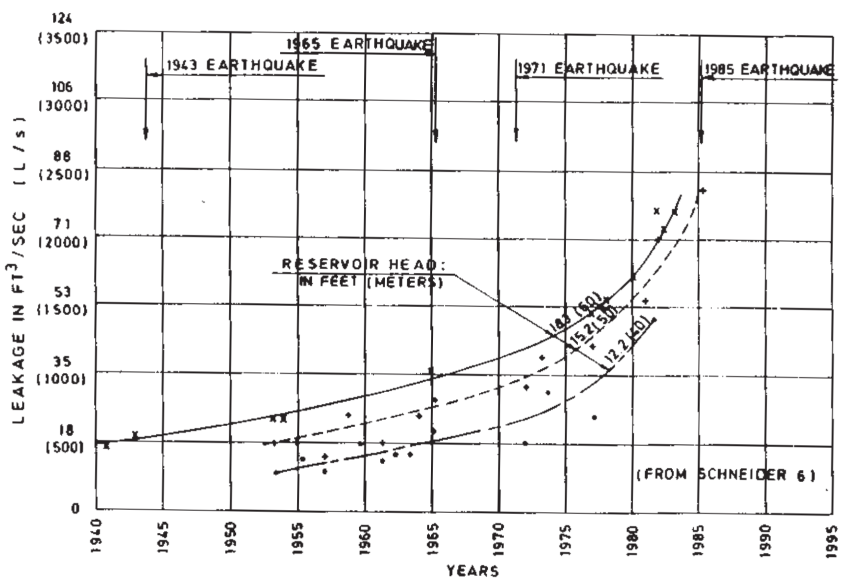


FIGURE 7 - LEAKAGE AT COGOTI DAM

Figure 2

SETTLEMENT AND LEAKAGE RECORDS

Congoti Dam

LA VILLITA DAM, MEXICO

On September 19, 1985, a large earthquake (Ms 8.1) struck the southwestern coast of Mexico. This event caused unprecedented damage in Mexico City, more than 250 miles (400 km) from the epicenter. It caused about 20,000 deaths in the Mexico City and left an estimated 250,000 homeless. La Villita Dam is one of two large embankment dams within 47 miles (75 km) of the epicenter that were affected by the earthquake. Peak bedrock acceleration at the site was measured at 0.13g, with dam crest acceleration at 0.45g.

La Villita Dam

Jose Maria Morelos (La Villita) Dam, an earth-and-rockfill embankment, was constructed from 1965 to 1967, about 8 miles (13 km) inland from the mouth of the Balsas River. The principal component of a 304 MW multi-purpose hydroelectric, irrigation and flood-control development, the embankment stands 197 feet high and has a crest length of about 1,400 feet. It was designed with a symmetrical cross-section with a central impervious clay core, well-graded filter and transition zones and compacted rockfill shells (Figure 1). The dam layout is shown in Figure 2. Upstream and downstream faces slope at 2.5:1, horizontal to vertical. The dam crest is slightly concave toward downstream.

La Villita Dam is founded on up to 250-foot-thick, well-graded alluvial deposits from the Balsas River. The alluvium is composed of boulders, gravels, sands and silts which taper toward the abutments. The abutments consist of stratified layers of andesite and andesitic breccias. A two-foot wide central concrete cutoff wall extends to bedrock across the entire dam foundation. The alluvium below the core was grouted on both sides of the cutoff wall to a depth of 85 feet.

The September 19, 1985, Earthquake

The September 19, 1985, Michoacan, Mexico, Earthquake (Ms 8.1, USGS) is the most serious natural disaster in Mexico’s recent history. The event occurred along a segment of the boundary between the Cocos and North American tectonic plates, previously identified as the Michoacan seismic gap. In this area, subduction is the main tectonic process, the plate contact being delineated by the Mid-American Trench (7.5 miles, or 12 km offshore from the Pacific Coast). The Cocos Plate underthrusts the North American Plate at an average angle between 10 and 20 degrees down to the east. The September 19 rupture occurred in two distinct events separated by about 25 seconds: slippage started in the northern portion of the seismic gap and then propagated to the southeast. A major aftershock (Ms 7.5, USGS) further extended the ruptured zone to the southeast on September 21, 1985. The epicenter of the principal shock was located near the mouth of the Balsas River, some 6.3 miles (10 km) offshore from the Michoacan coastline and about 15.6 miles (25 km) from La Villita Dam. The earthquakes of September 19 and 21 produced the most extensive strong motion data sets yet obtained in Mexico.

Earthquake Effects and Observed Performance

La Villita Dam was subjected to about 60 seconds of strong ground motion during the September 19 earthquake, which was recorded at the site and on the dam. Damage to the dam was noticeable as cracking, settlement and spreading. The overall safety of the embankment, however, was not threatened as a result of this event.

Two main systems of longitudinal cracks developed at the crest of La Villita Dam, parallel to its axis, some 16 feet away from the crest edges. These cracks formed along the buried shoulders of the central core and most likely resulted from differential settlement between the core and adjacent filter zones. A 260-foot-long crack, 1/4 to 2 inches wide at the surface, formed along the upstream side of the dam crest. Vertical offsets of 2 to 4 inches occurred between the lips of the crack, the upstream side settling the most. On the downstream side, another major crack system appeared, about 1,000 feet long, 0.4 to 0.6 inch wide, with vertical offsets ranging from 0.5 to 0.8 inch, the side toward the face of the dam being downthrown. Several other longitudinal cracks, up to two inches wide, but less extensive than the two principal crack systems, were also found. The location of the cracks is shown in Figure 3 and a photograph of the cracking, taken in the days that followed the event, is shown in Figure 4, Photographs (a) and (b). Note the toppling of sections of the parapet wall in Figure 4 (a).

The most significant cracks were investigated by trenching immediately after the earthquake and were confirmed to be only about 5-feet deep. Along the trench walls, several cracks were open between 2 and 4 inches and extended in depth for about two feet through the aggregate base layer of the paved road at the crest of the dam. They faded to hairline when reaching the sands of the filter zone. The cracks were concluded not to reach the impervious core zone and were found to disappear below two-feet depth, except near the right abutment, where one of the cracks was delineated as a closed fissure through a clay lens embedded at about three-foot depth within the filter sands.

La Villita Dam settled and spread laterally during the 1985 earthquake. Post-earthquake surveys showed that, in its central part, the dam settled between 7.9 and 12.6 inches on the upstream side and between 3.6 and 8.7 inches on the downstream side. Based on inclinometer readings, settlements decreased in magnitude to near zero toward the abutments and seemed to be evenly distributed within the dam cross-section, rather than associated with distinct surfaces. The downstream half of the dam moved horizontally up to 4 inches in the downstream direction and the upstream half up to 6.5 inches in the upstream direction. Downstream horizontal displacements were somewhat irregular, although generally more symmetrical with respect to the center of the dam than the upstream displacements. Settlements were particularly noticeable at several piezometer locations, where the piezometer tubes which extend down to deep within the embankment remained in place, while their protective concrete boxes settled along with the face of the dam (Figure 5).

The powerhouse and other appurtenant facilities were essentially unaffected by the earthquake. Mechanical and electrical equipment remained fully operational and no damage occurred at the spillway, spillway gates, power plant, substation and switchyard. Two 130-ton transformers (13.8 kV/230 kV), adjacent to the power plant building, showed evidence of about 0.4 inch of horizontal sliding on their pedestals, but were otherwise unaffected.

Instrumentation and Strong Motion Records

La Villita Dam is well-instrumented. Five strong motion accelerometers, which include AR-240 and SMA-1 instruments, are installed at various locations within the dam and abutments. The dam is also equipped with 21 vertical and horizontal extensometers, 20 inclinometers, three horizontal rows of hydraulic levels and five lines of survey monuments, two on either side of and parallel to the crest, two near the upstream and downstream toes and one at about mid-height of the downstream face. Forty-five piezometers, upstream and downstream from the concrete cutoff, monitor the effectiveness of this cutoff.

On September 19, 1985, the accelerometer at the center of the crest of the dam recorded a peak horizontal acceleration of 0.45g and, on the following day, a peak acceleration of 0.16g was measured during the strongest aftershock. Peak horizontal bedrock acceleration was recorded at 0.13g for the main event and 0.04g for the aforementioned aftershock. Bedrock records for the main event are shown in Figure 6. As can be seen in Figure 6 and as confirmed from post-earthquake seismological research studies, the September 19 earthquake resulted from two distinct bursts of energy lasting about 16 seconds each and separated by about 25 seconds. This dual rupture mechanism was more conspicuous on records from other strong motion stations closer to the epicenter than from the La Villita instruments.

Survey monuments, inclinometers and extensometers were essential to provide detailed information on the earthquake-induced deformations of La Villita Dam. Of particular interest was the fact that the dam had previously been shaken by several significant earthquakes in the 12 years that preceded the 1985 event. Figure 7 shows a record of crest settlements from 1968 to 1985. Earthquake-induced settlements have been found to exceed static post-construction settlements and appear to increase in magnitude from one earthquake to the other, perhaps indicating a change in stiffness of the dam materials or a slow, cumulative, deterioration of part of the embankment. Inclinometer records confirmed that permanent deformations decreased in magnitude from crest to bottom of the embankment and did not involve the foundation materials.

Conclusions

The 1985 Michoacan earthquakes induced significant shaking at La Villita Dam. Despite minor damage and occurrence of noticeable cracking and earthquake-induced permanent deformations, the dam owner, the Mexican Comision Federal de Electricidad, concluded that La Villita Dam and its appurtenant structures performed well and without evident impairment of its overall safety. Because the dam has been successively shaken by several large earthquakes of appreciable intensity and duration of shaking at the site, this example provides a somewhat unique case history of repetitive shaking of the same dam by different earthquakes. The fact that the most recent measured deformations seem to increase in magnitude has not been explained to date. Future earthquakes along the Michoacan subduction zone, which most likely will shake La Villita Dam again, may provide further insight to understand this phenomenon and explain if such observed increase of the dam deformations is fortuitous or could be typical of a dam aging process and progressive weakening of the dam materials as a result of repetitive cyclic shaking.

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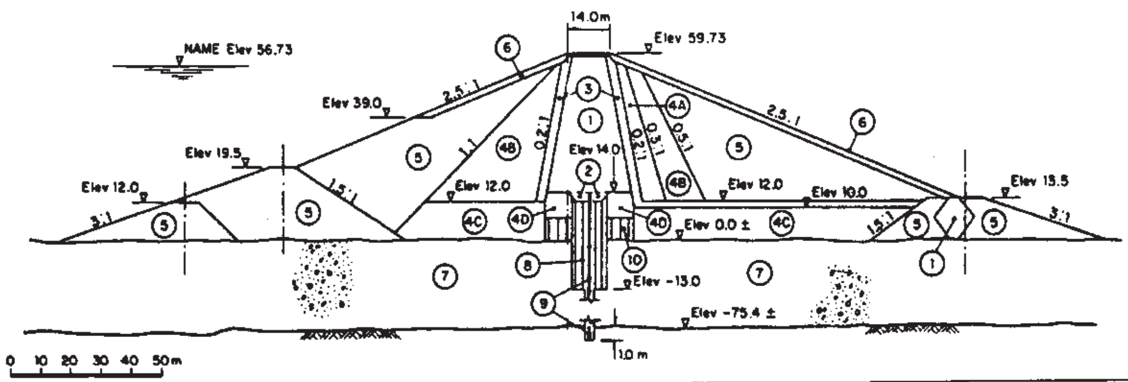
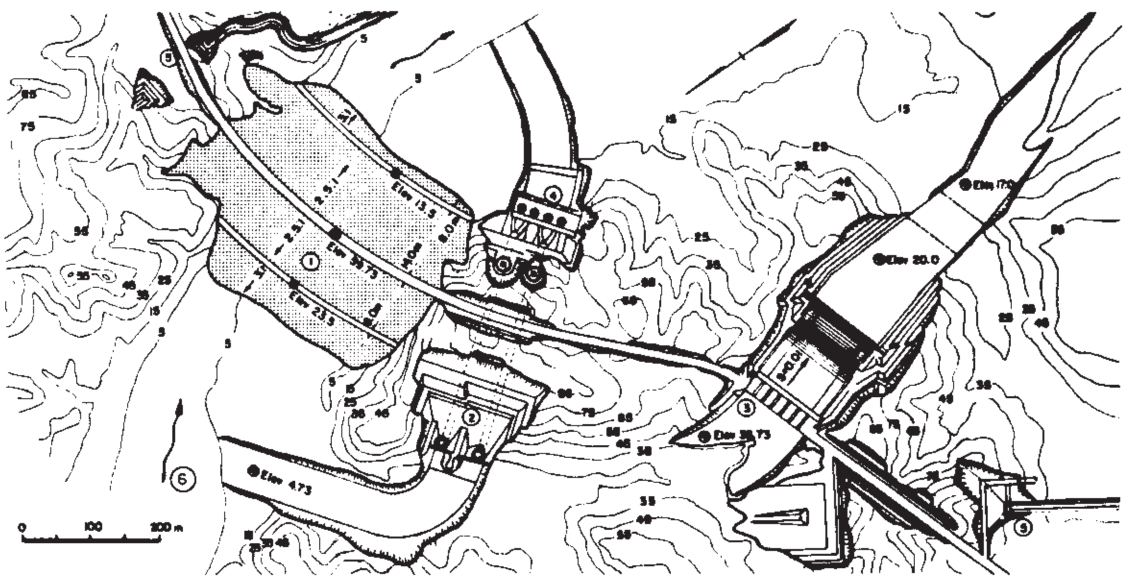


1 Compacted Impervious Material
2 Highly Plastic Clay
3 Sand Filters
4A Well Graded Gravel and Sand
4B Gravel and Sand
4C Dumped Gravel and Sand
4D Compacted Gravel, Sand and Muck
5 Rockfill
6 Selected Rockfill
7 Alluvium (Gravel & Sand)
8 Grout Curtain
9 ICOS-Type Concrete Cut-off Wall
10 Consolidation Grouting

Figure 1
CROSS-SECTION
La Villita Dam



(1) Dam
(2) Intake Works
(3) Spillway
(4) Powerhouse
(5) Irrigation Outlet

Figure 2
DAM LAYOUT
La Villita Dam

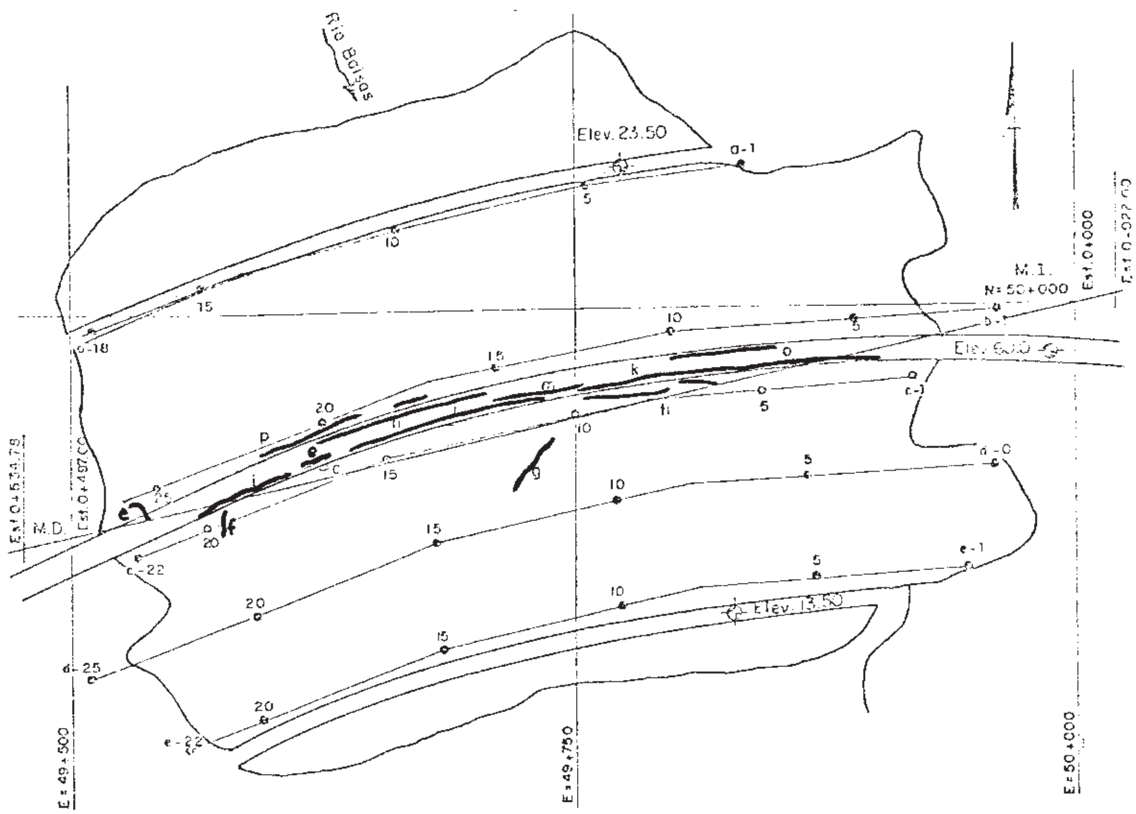


Figure 3
CRACK LOCATIONS
La Villita Dam

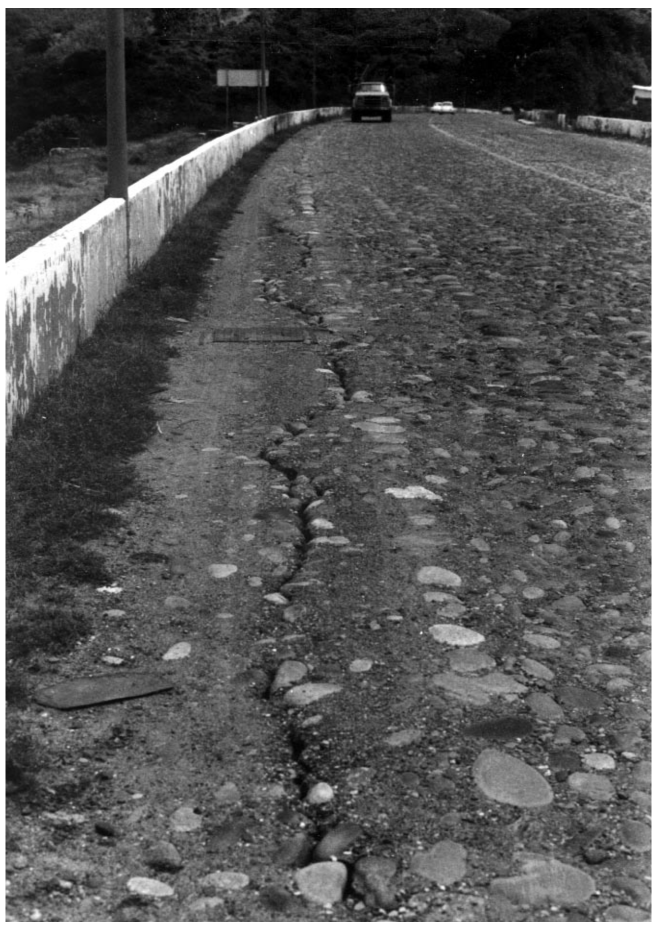


Figure 4
CRACK PHOTOGRAPHS
La Villita Dam

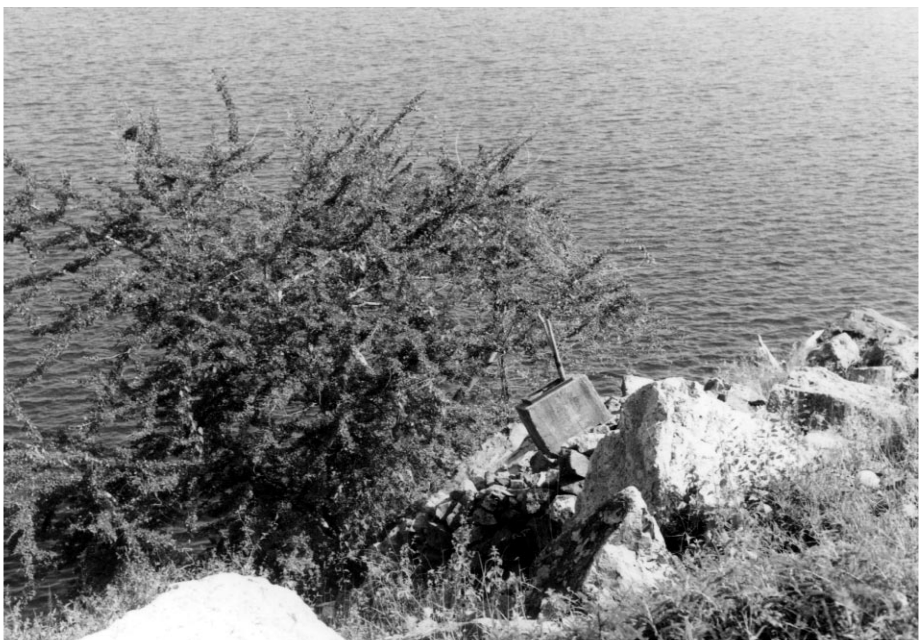


Figure 5
SETTLEMENT AT PIEZOMETER BOX
La Villita Dam

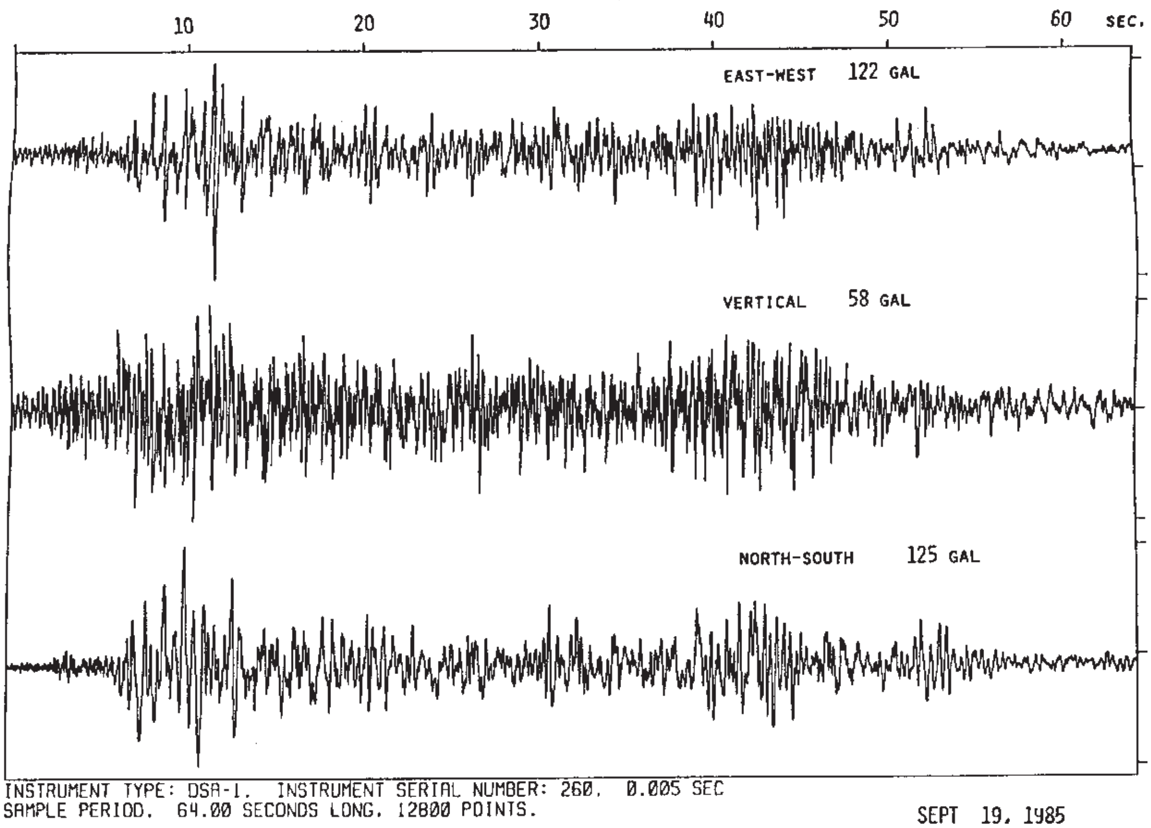


Figure 6
BEDROCK ACCELERATION RECORDS
La Villita Dam

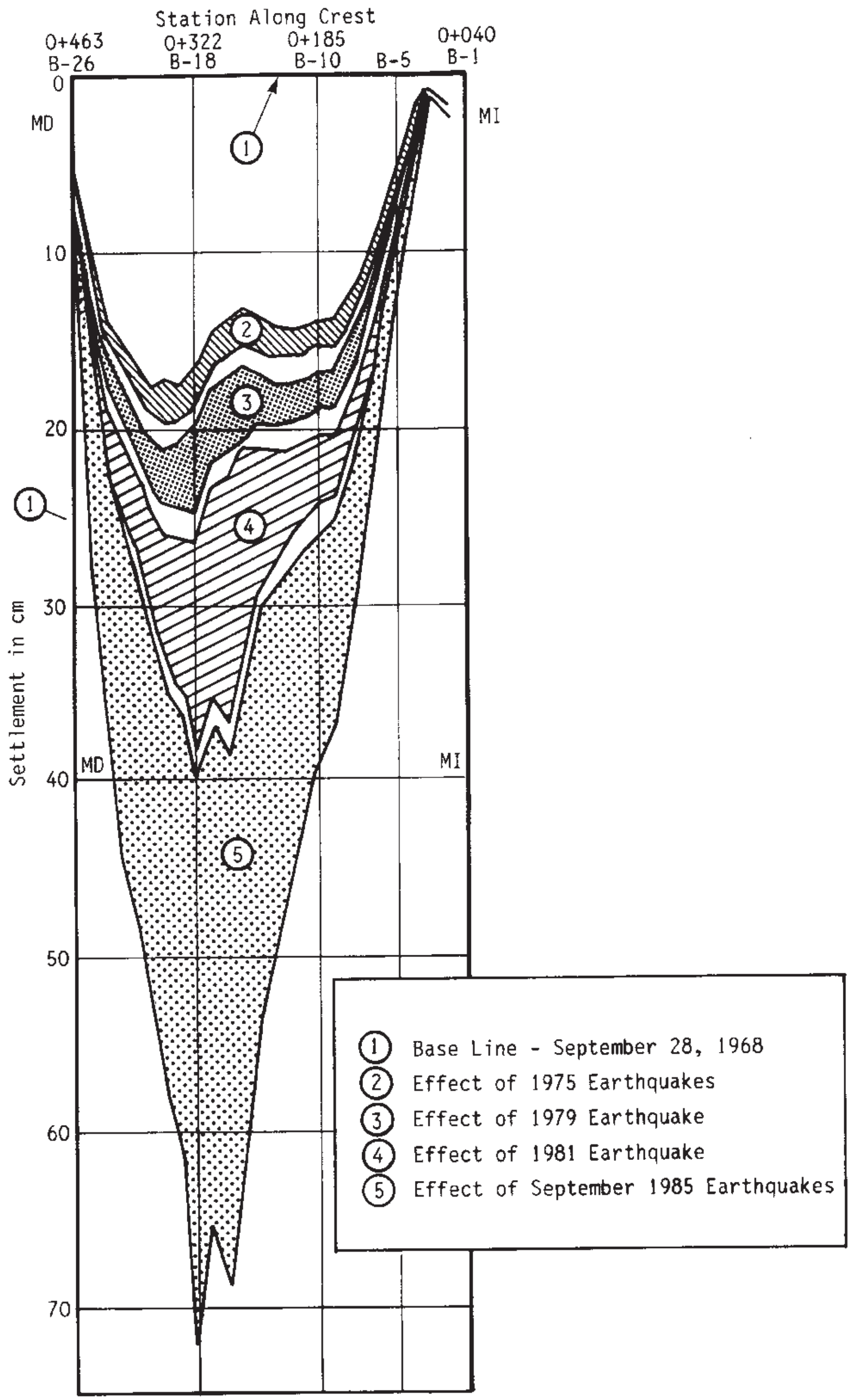


Figure 7
HISTORIC CREST SETTLEMENTS
La Villita Dam

LOS ANGELES DAM, CALIFORNIA

The January 17, 1994, Northridge Earthquake (Mw 6.7) affected the dams of the Van Norman Complex (VNC), owned and operated by the Los Angeles Department of Water and Power (LADWP). Epicentral distance was about 10 km. The VNC includes the decommissioned Upper and Lower San Fernando dams and their replacement, Los Angeles Dam (LAD), built in 1979. LAD, a 155-foot high modern compacted embankment, experienced up to 3.5 inches of crest settlement and surficial cracking of the asphalt concrete facing along its upstream slope. Since 1971, the VNC has been extensively instrumented. These instruments have provided an exceptional crop of local strong motion records at and near the dam for the Northridge event, with peak ground accelerations (PGA) approaching the acceleration of gravity (1g). Especially strong motions were recorded at the dam crest of Los Angeles Dam, at the west abutment and in the lower part of the foundation. Peak ground accelerations were 0.32g at foundation level and 0.33g at the right abutment. Crest acceleration was 0.60g.

Los Angeles Dam

The VNC serves as the terminal for the first and second Los Angeles aqueducts and includes Los Angeles Dam and the North Dike, a saddle dam, the two decommissioned Upper and Lower San Fernando dams, and miscellaneous other facilities. Los Angeles Dam is the replacement for the two San Fernando (Van Norman) dams that were severely damaged during the February 9, 1971, San Fernando Earthquake.

Los Angeles Dam was completed in 1979 and impounds Los Angeles Reservoir, of 10,000 acre-feet capacity (Figure 1). It is a modern, well-instrumented embankment of maximum height 155 feet, founded on bedrock of the Sunshine Ranch Member and the upper member of the Saugus Formation. The foundation rock consists of claystone, siltstone, sandstone and interbedded pebble-cobble conglomerates, with average shear wave velocities measured at about 3,200 ft/s in the foundations of the nearby Upper and Lower San Fernando dams.

LAD includes upstream and downstream shells of compacted silty sand, a central vertical chimney drain, a downstream near-horizontal drainage blanket, and a silty clay core upstream from the chimney drain. A small zone of blended cobbles, gravel and sand is also located immediately downstream from the lower part of the chimney drain. The dam crest is 30-feet wide and the slopes were constructed at about 3:1 (downstream, h to v) and 3.5:1 (upstream, h to v). The embankment materials were compacted to 93 percent relative compaction (33,750 foot-pounds/cubic foot). Compaction was carefully controlled during construction. The interior slopes of Los Angeles Reservoir, including the upstream face of the dam, are lined with an asphalt concrete membrane. The reservoir bottom is unlined. Figure 1 shows the reservoir layout and the maximum cross section of LAD.

The January 17, 1994, Earthquake

At 4:31 a.m. local time on January 17, 1994, the Northridge Earthquake (moment magnitude Mw 6.7) affected the greater Los Angeles area. The earthquake was centered along a blind thrust segment of the Oakridge Fault, about 10 km southwest of the VNC, at a focal depth of about 19 km. The earthquake produced some of the largest peak ground accelerations ever recorded, many in the range of 0.50g to 1.0g, and Modified Mercalli Intensities of up to IX were assigned at several locations. Some of the recorded response spectra were twice as large as the building code spectrum over a significant part of the period range.

Casualties included 57 deaths and at least 5,000 persons injured. Structural damage included numerous cases of partial or total failure, including steel and concrete buildings, apartment buildings, parking structures, highway overpasses and lifelines. This event ended up being the costliest natural disaster in United States history, with over $20 billion in estimated property damage. As a result of this earthquake, the attention of the public and engineering community focused on extensive damage caused to welded beam-to-column connections in steel moment-resisting frame (SMRF) buildings. Of about 1,500 SMRF buildings in Los Angeles, at least 137 sustained connection failures during the Northridge Earthquake.

Thirteen dams in the area were found to have experienced cracking or some movement (Sanchez, 1994). Most of the cracking and movement was concluded to be minor. Most significant observations were large longitudinal open cracks at the decommissioned Lower and Upper San Fernando dams, and the 2-inch opening of a joint between the left abutment and the concrete thrust block of Pacoima Dam. This joint opening was accompanied by about half a inch of movement of the thrust block downstream relative to the dam crest.

Earthquake Effects and Observed Performance

Davis and Sakado (1994) and Davis and Bardet (1996) have described in considerable detail the performance of LAD during the Northridge Earthquake. Extensive surficial cracking of the asphalt concrete crest roadway and 3-inch thick asphalt lining that covers the upstream slope of the dam was observed (Figure 2). Most cracks were of the shear type and, near the left abutment, were associated with waving, bulgy surfaces caused by compression of some of the embankment materials. Trenching of the largest cracks indicated that they did not extend deep within the body of the dam. A few cracks existed prior to the seismically-induced cracks, but Davis and Bardet concluded that most cracks were probably caused by transient stresses and deformations during the earthquake.

Immediately after the earthquake, ten survey profiles were taken along the crest axis and downstream slope. The embankment experienced a maximum crest settlement of 3.5 inches and about one inch of horizontal nonrecoverable crest displacement (Figure 3). The downstream slope settled up to 3/4 inch, and moved laterally slightly in excess of two inches downstream.

Seepage levels, piezometers and observation wells indicated increases in pore pressure in and around the LAD. Such increased pore pressures returned to normal within a short time after the earthquake.

Instrumentation and Strong Motion Records

LAD is extensively instrumented with survey monuments, strong motion accelerographs, and piezometers. Strong motion instruments recorded the dam response during the Northridge Earthquake. These strong motion records have been corrected by Professor Trifunac of the University of Southern California. Stations 2 (west abutment) and 3 (foundation) are on bedrock (Figure 1). Station 4 (crest) is at the maximum cross-section. Peak accelerations of the corrected dam records were 0.27g (transverse), 0.32g (longitudinal) and 0.12g (up) at foundation level; 0.60g (transverse), 0.42g (longitudinal) and 0.38g (up) at the crest; and 0.42g (transverse), 0.33g (longitudinal) and 0.32g (up) at the right abutment. Peak ground accelerations recorded at the dam are lower than elsewhere in the VNC. PGAs of 0.85g and 1.00g were recorded on alluvium 4,400 feet south and 8,400 feet north of LAD, respectively (Bardet and Davis, 1996). The records of the Northridge Earthquake obtained at the foundation and abutment of LAD are shown in Figure 4.

Using the extensive design data available for LAD, post-earthquake nonlinear response studies funded by the National Science Foundation indicated calculated deformations and crest acceleration response consistent with those recorded (Bureau, et al., 1996).

Conclusions

The example of LAD confirms that dams built of well-compacted cohesive clays and dense sands can perform satisfactorily during very severe earthquake shaking. Of particular interest is the fact that strong motion records obtained at LAD were generally significantly less severe that those recorded elsewhere in the VNC at distance of less than 10,000 feet away. This is perhaps the only dam were the base motion below the maximum section of the embankment was recorded. Comparisons of such motions with those obtained at the abutment indicate less severe shaking at depth that at the abutment, and this especially for the vertical component of motion. Lastly, observed performance of LAD during the Northridge Earthquake was found to reasonably match the settlement and acceleration histories subsequently calculated through nonlinear analysis procedures, thereby providing another verification of the validity of modern dam evaluation procedures.

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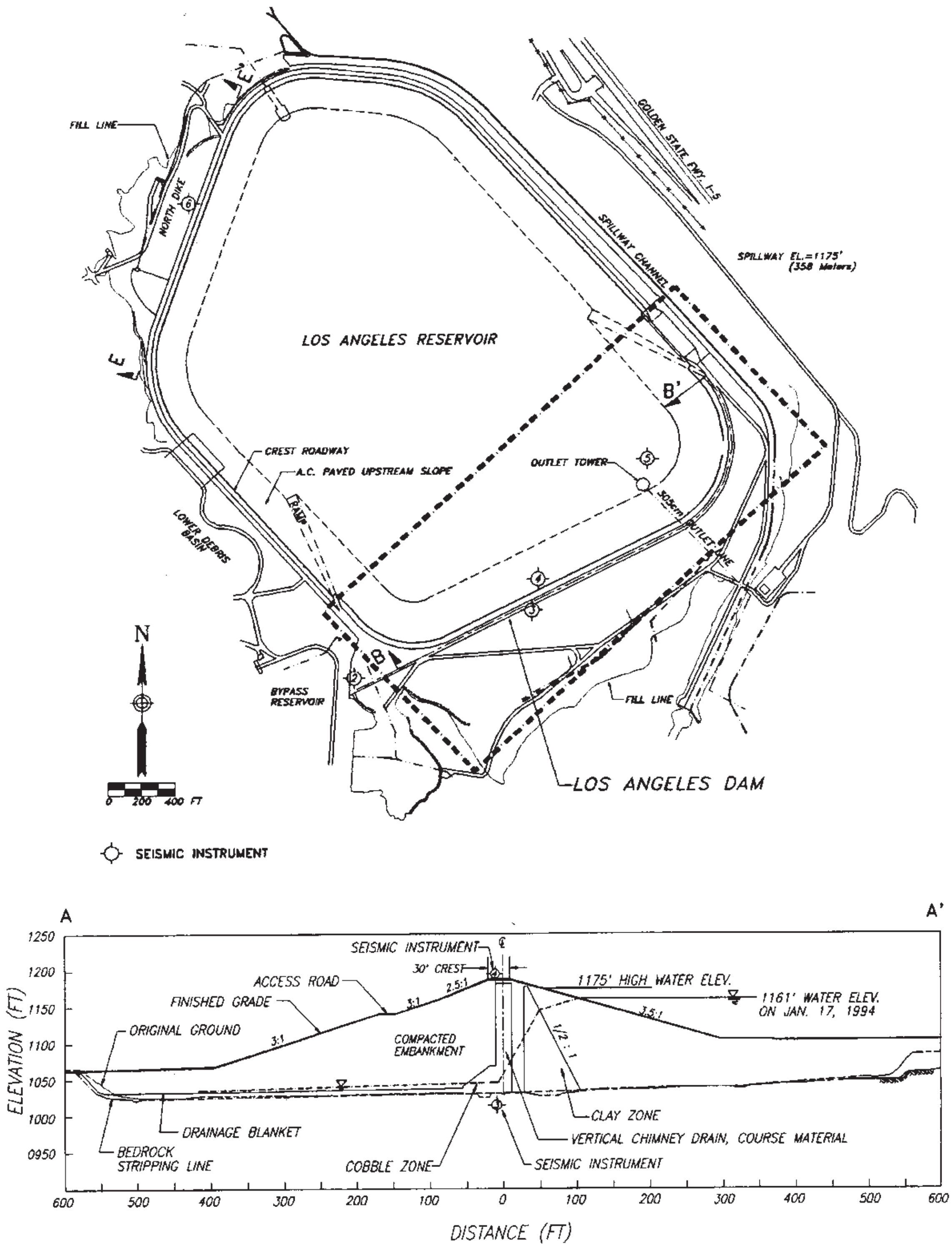


Figure 1
LA RESERVOIR LAYOUT AND MAXIMUM CROSS SECTION
Los Angeles Dam

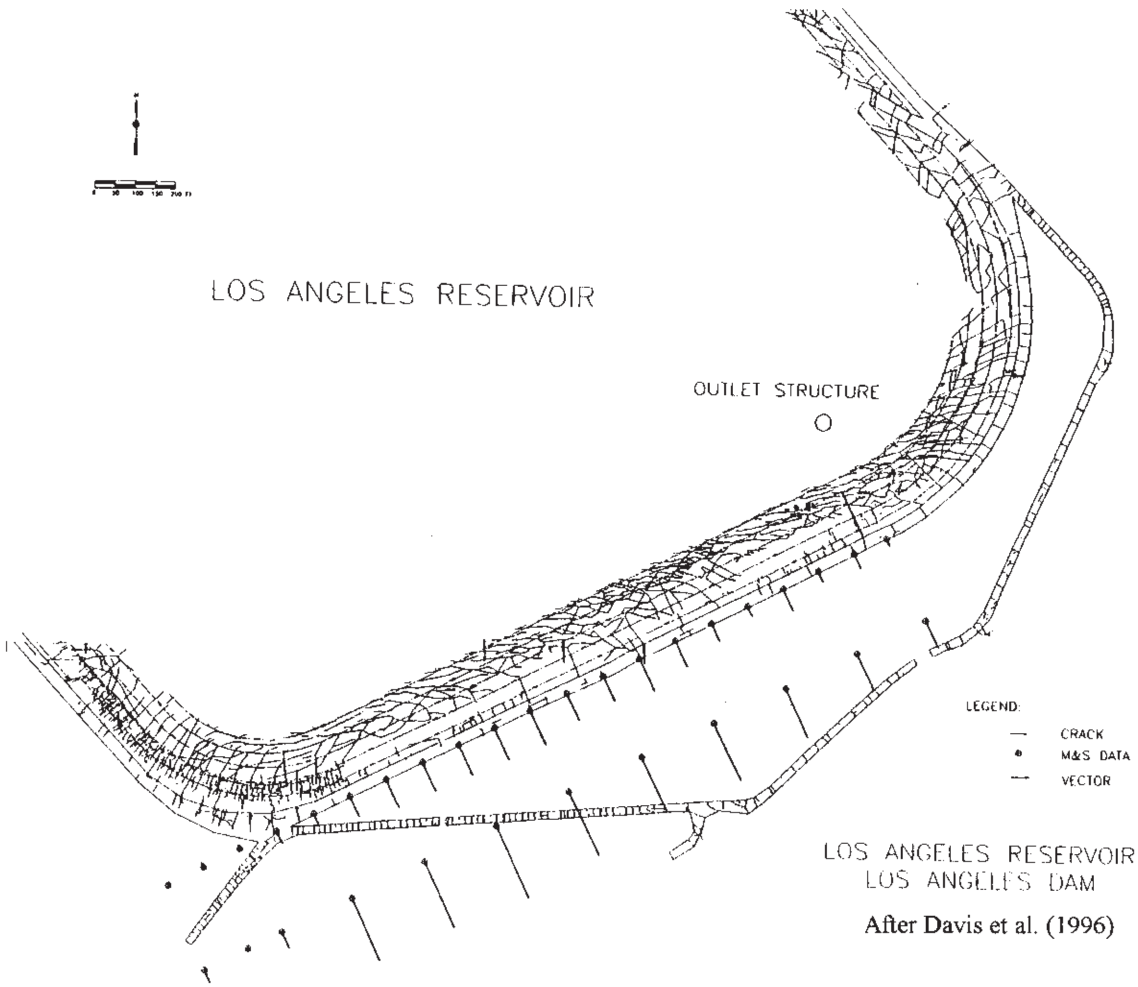
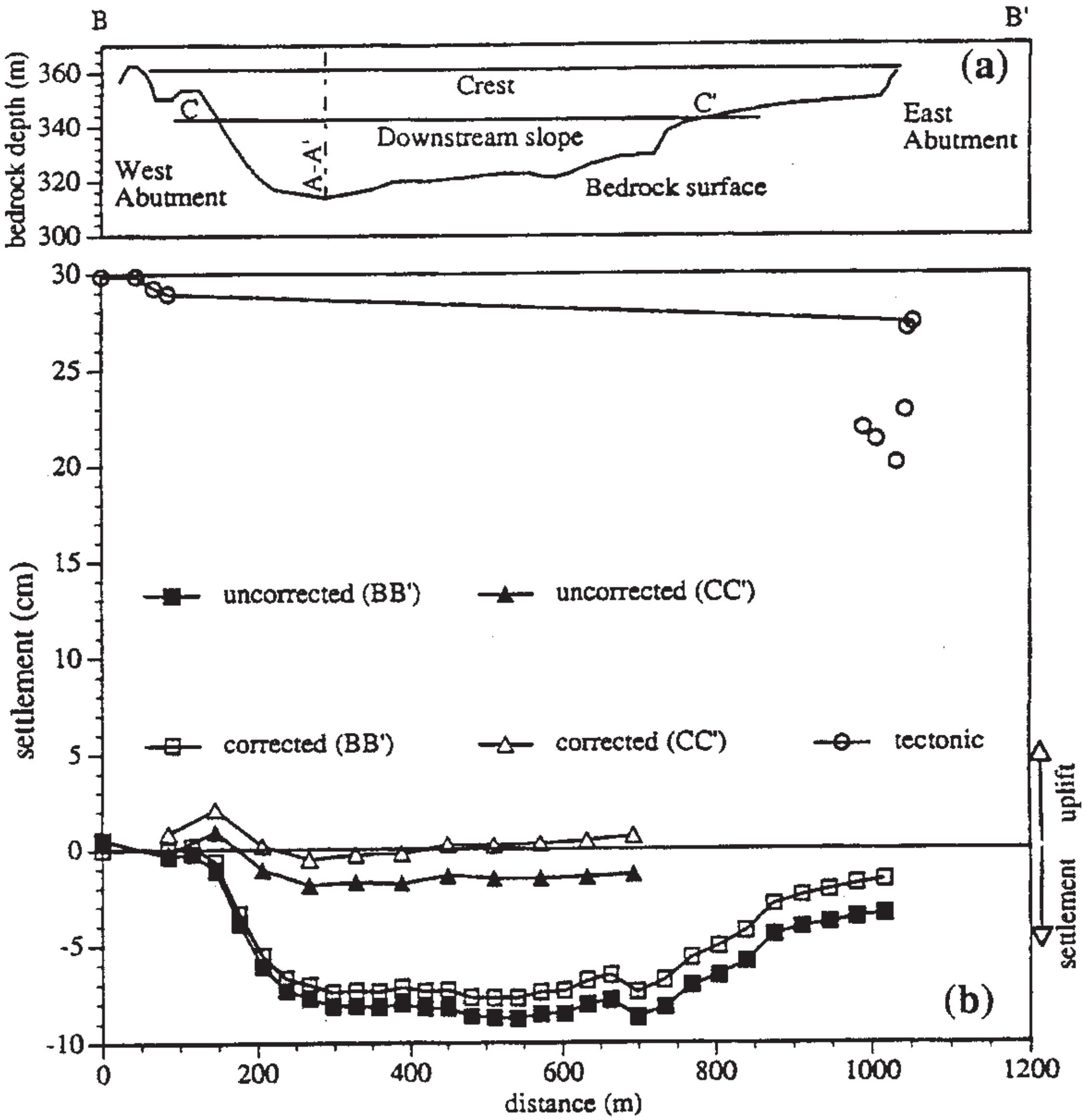


Figure 2
CRACKS & TRANSVERSE MOVEMENTS
INDUCED BY NORTHRIDGE EARTHQUAKE
Los Angeles Dam



After Davis et al. (1996)

Figure3

PROFILE OF CREST SETTLEMENTS

Los Angeles Dam

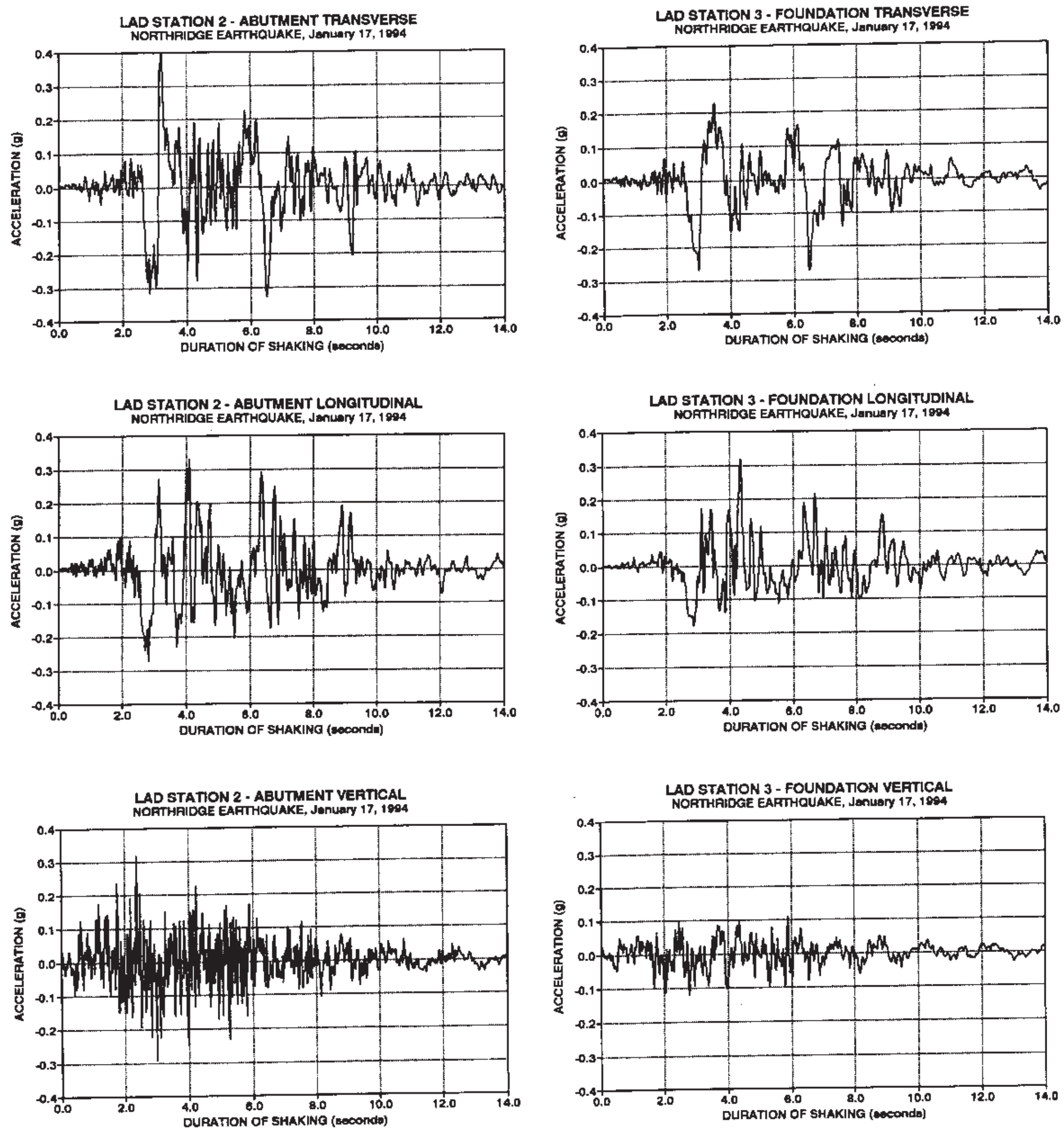


Figure 4
RECORDED FOUNDATION AND ABUTMENT MOTIONS
Los Angeles Dam

LOS LEONES DAM, CHILE

Los Leones is one of the largest conventional dams designed to impound mine tailings. It is a compacted earth and rockfill dam, built to Stage III at the time of preparation of this case history. It impounds copper mine tailings and a shallow “clear water” reservoir. In Stage IV, the dam will be raised to 650 feet (about 200 m) above streambed.

The March 3, 1985, Chilean earthquake (Ms 7.8) shook the Stage II Dam, 354 feet (108 m) high, which was instrumented at crest and toe with strong motion accelerometers. This makes Los Leones Dam one of only a few large dams with recorded seismic performance during a major earthquake. These records were used to verify a numerical model of the dam and calibrate the constitutive relationships used for the design of the final raising.

Los Leones Dam

Los Leones Dam is owned and operated by Codelco Chile, Division Andina. It is located in the Andes Mountains of central Chile (Figure 1) and within an area of strong seismic activity. The dam impounds copper mine tailings and a shallow clear fluids pond. Before construction, the Los Leones River was diverted upstream from the reservoir through a 2.2-mile (3.5 km) long tunnel, designed to pass the 1,000-year flood. The tailings flow from the concentrator, located at El. 2,800 m (El. 9,184 feet), through a 9.4-mile-long (15 km) pressure conduit discharging at the upstream end of the reservoir, and fill the glacial valley of the Los Leones River. Excess clear pond fluids and watershed runoff spill through a multi-port inclined intake structure and an outlet tunnel.

The dam was built across a relatively narrow gorge, where the river has eroded thick banks of randomly deposited streambed alluvium, glacial till, and landslide materials. The embankment was designed to be built in four stages as a rolled earthfill, with a planned final crest at El. 7,013 feet (2,138 m). The Stage I dam was started in October 1978. Final completion to the top of Stage IV (Stage IV-B) started in December 1997, with a final dam height of 525 feet (160 m) at centerline and 650 feet (198 m) at the downstream toe. The Stage IV reservoir has a capacity of 120,000 acre-feet and impounds 222 million metric tons of tailings, representing 20 years of mining activities. Los Leones Dam will reach a final volume of 16 million cubic yards. Pictures of the upstream and downstream slopes of the dam taken in November 1993, are shown in Figures 2 and 3, respectively.

The dam is a conventional, compacted, earthfill embankment. It has an upstream core built of morainic deposits, an inclined chimney drain and filter and transition zones on the downstream side of the core, and a downstream drainage blanket. The downstream shell is built of compacted earthfill, borrowed from alluvial fans in neighboring canyons. Near the toe, a secondary drainage system collects seepage from the underlying aquifer and paleostreams in the dam foundation. The chimney drain slopes toward upstream and is connected to a drainage blanket, placed on top of the foundation surface. The drainage blanket underlies part of the upstream and all of the downstream half of the dam. The dam section is shown in Figure 4.

Laboratory testing of the embankment materials was conducted in Chile. The tests included moisture, density, gradation, compaction and two series of triaxial compression tests on isotropically consolidated, drained specimens (TX/ICD tests) of materials representative of the core (Zone A) and shell (Zone B). The effective friction angle of the embankment materials ranged from 35.5 to 46 degrees, depending on the level of confinement (the friction angle decreases at higher confining pressures). The core materials (Zone A) have a cohesion measured at between 0.20 and 1.60 metric tons/m2. The shell materials are cohesionless. The permeability of the core materials was measured at 0.5 x 10-5 cm/s.

The tailings were estimated to have an effective cohesion of 0.2 tons/m2 and a friction angle of 28 degrees, based on strength data published in the literature for Chilean copper tailings. Their measured coefficient of permeability, or 0.5 x 10-6 cm/s, is one order of magnitude smaller than the coefficient of permeability of Zone A. This is indicative that Zone A acts as a drain with respect to the less pervious tailings. Saturation of parts of Zone A and the upstream Zone B only occurs when clear water is impounded above the solid tailings.

The March 3, 1985, Earthquake

The March 3, 1985, earthquake (Ms 7.8) was centered near the coast of central Chile. At least 180 people were killed and 2,500 injured. Extensive damage occurred in the cities of San Antonio, Valparaiso, ViÒa del Mar, Santiago and Rancagua. The earthquake was felt in Chile along a stretch extending 2,000 km from Copiaco to Valdivia. A few modern structures, including reinforced concrete buildings in Reneca and ViÒa del Mar, suffered significant damage. Damage to adobe structures was extensive. A portion of the port of Valparaiso experienced over 16 inches (41 cm) of lateral spreading as a result of liquefaction. Bridge damage, in the form of subsidence and spreading of approach fills and pier and span collapses, was observed. Minor damage occurred at industrial facilities. Two tailings dams failed, including the Veta de Agua tailings impoundment, near the town of El Cobre, and the Cerro Negro Dam near the town of the same name.

Earthquake Effects and Observed Performance

Stages I and II of Los Leones Dam were built as a single job. The Stage II dam, located 90 miles (144 km) away from the epicenter, had reached a height of 354 feet (110 m) at the time of the 1985 earthquake, and the tailings pond was 98 feet (30 m) below the dam crest. Los Leones Dam responded elastically to this event, as post-earthquake surveys indicated no measurable crest settlements. No cracking nor other disturbances of the dam slopes were reported.

Instrumentation and Strong Motion Records

Strong motion acceleration records were obtained at both the base and crest of the dam, with a peak horizontal acceleration of 0.13g at the toe (PGA) and 0.21g at the crest (PCA). Overall duration of felt shaking exceeded 100 seconds, with a bracketed duration exceeding 40 seconds. Figure 5 shows the significant phase of the 1985 time histories and the large amplifications from the dam crest response. Figure 6 shows the crest response spectrum. Of significance is a very large peak spectral amplification ratio, which was almost five at crest level, and the relatively short period (0.55 s) at which it occurred, which confirmed the overall stiffness of the dam materials.

Back-Calculated Response

The recorded 1985 base motion provided seismic input to analyze the Stage II dam and calibrate a numerical model of final Stage IV-B. This model is shown in Figure 7. The recorded crest motion was used to compare calculated and recorded responses of the Stage II dam.

A Mohr-Coulomb, elastic-plastic nonlinear model and a time-dependent, semi-coupled pore pressure generation scheme were used to represent Los Leones Dam numerically (Bureau, et al., 1994). A two-dimensional grid represented the largest section of the dam. To include the effect of the staged construction on the initial static stresses, a nine-step incremental procedure was used. The zones representing Stages III and IV and the upper tailings layers were not activated in the analyses of the 1985 response.

As Los Leones Dam has been built across a narrow valley, three-dimensional effects were expected. Such effects were successively approximated using two different approaches. The first one used the maximum section of the dam ("full" model) and an increase in material stiffness to simulate the shift in response toward higher frequencies due to the narrow shape of the valley section. The second one used the "geometric adjustment" method (Edwards, 1990). The geometric adjustment consists of entering the seismic input at some intermediate level above the model base, and parametrically adjusting this level until optimal comparison between measured and calculated responses is achieved. The true stiffness is used. The geometrically adjusted model simply represents an "average" dam section across the width of the canyon.

The calibration analysis consisted of fine-tuning the analysis parameters in order to reproduce the characteristics of the recorded response. Properties were kept within the range of values measured in the laboratory and were selected consistently with current dam engineering practice. Various indicators were used to compare calculated and recorded responses, including peak crest acceleration, peak crest spectral acceleration, Arias intensity (a measure of the energy content of the record), effective and bracketed durations, Root-Mean-Square acceleration, and overall spectral shape calculated at the crest of the dam.

Maximum calculated settlement was less than 0.2 inch (0.5 cm), in acceptable agreement with observed performance (no measurable deformations). Both calibration methods provided consistent results. Calculated peak spectral acceleration and Arias intensity were within 15 percent of the reference values. The comparison between calculated and recorded response spectra (Figure 6), was satisfactory. The calibration analysis provided a basis for the development of a numerical analysis model for the design of the final raising of the dam.

Conclusions

Tailings dams can reach dimensions which place them among the largest of all embankments. They should be designed with the same care and concerns for safety and the environment as large water storage dams.

The calibrated seismic design analyses of Los Leones Dam indicated that modest deformations may be expected under the specified design earthquake. The chimney drain and the pervious blanket, by keeping most of the shell dry, should prevent significant deformations of the dam, despite liquefaction of the tailings. The tailings pressure will restrain any movement toward upstream in the lower part of the dam. The dam is expected to be safe after final raising and when the site will be closed.

The 1985 records provided a unique opportunity to calibrate a model of the dam and verify the design concepts implemented. It led to greater confidence in the final design of a facility that will become one of the largest embankment dams in the world. The availability of strong motion records and the detailed analyses undertaken by the owner and its consultants facilitated regulatory approval by Chilean authorities. They provided a vivid example of the utmost importance of instrumenting dams to record earthquake motions. The availability of the records has been, in this case, of direct benefit to the dam owner. But most importantly, by verification of observed performance, it qualified the use of one of the advanced analysis tools that are now available for the design and safety evaluation of embankment dams.

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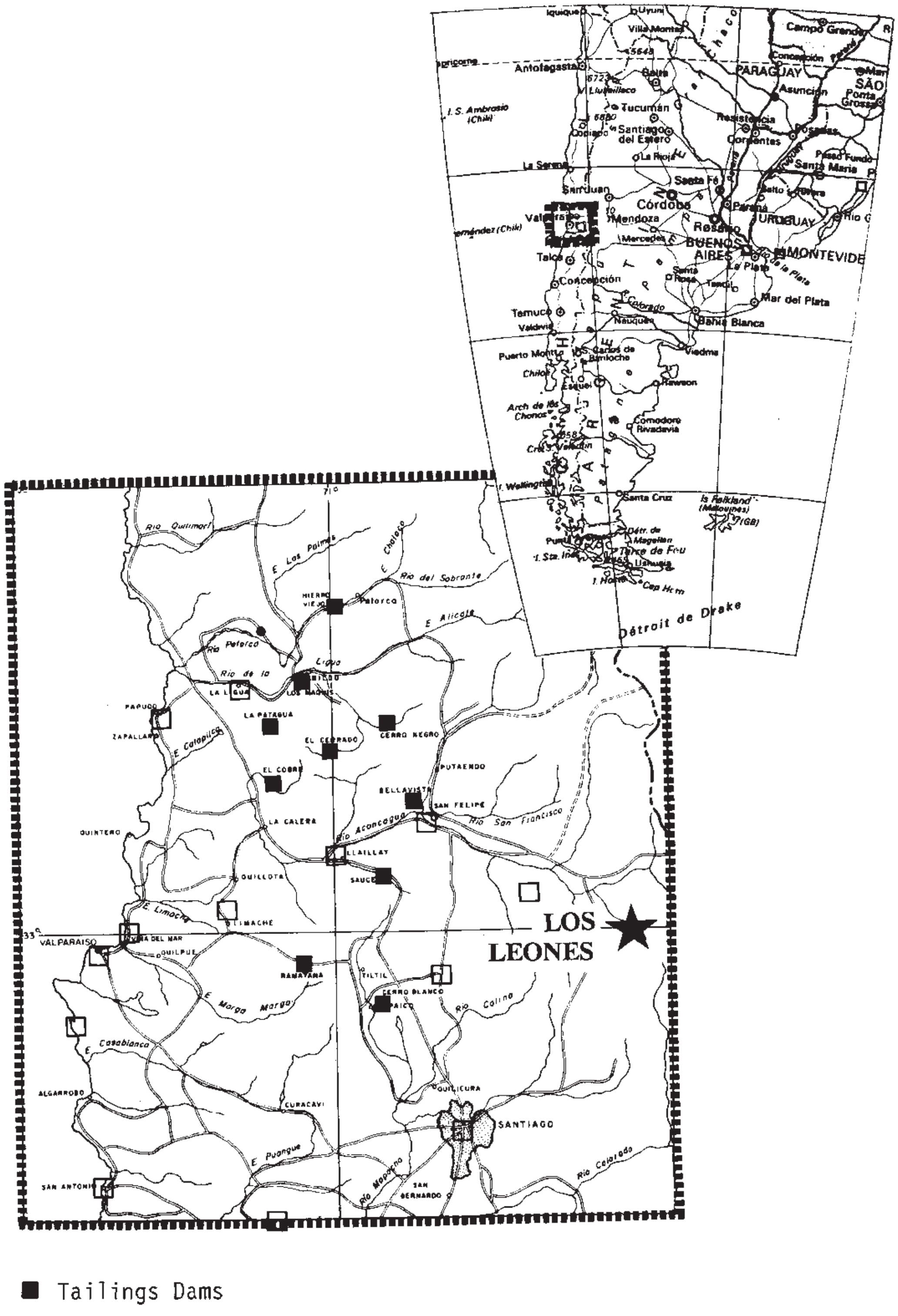


Figure 1
TAILING DAM
Los Leones Dam

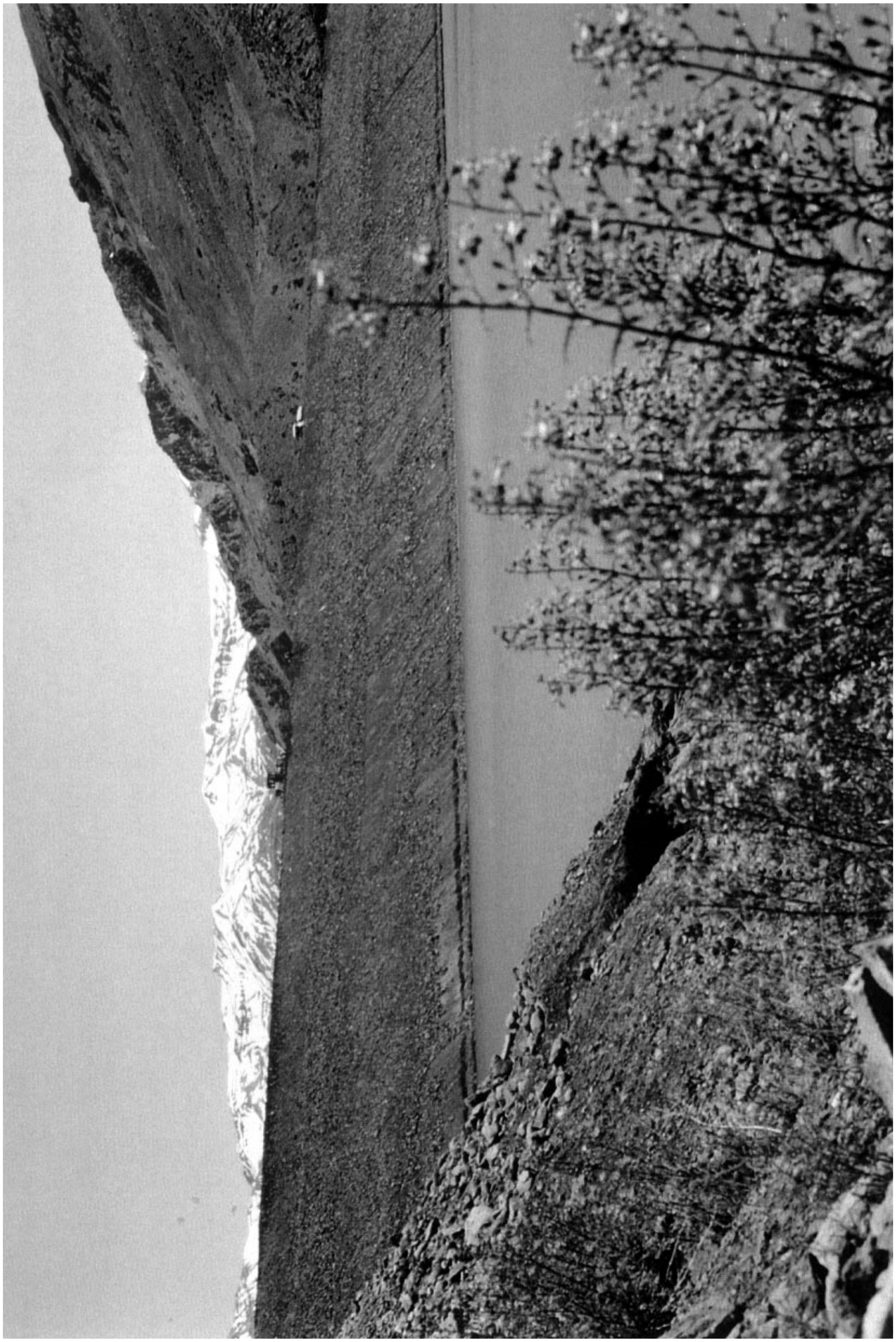


Figure 2
UPSTREAM FACE (November 23, 1993)
Los Leones Dam

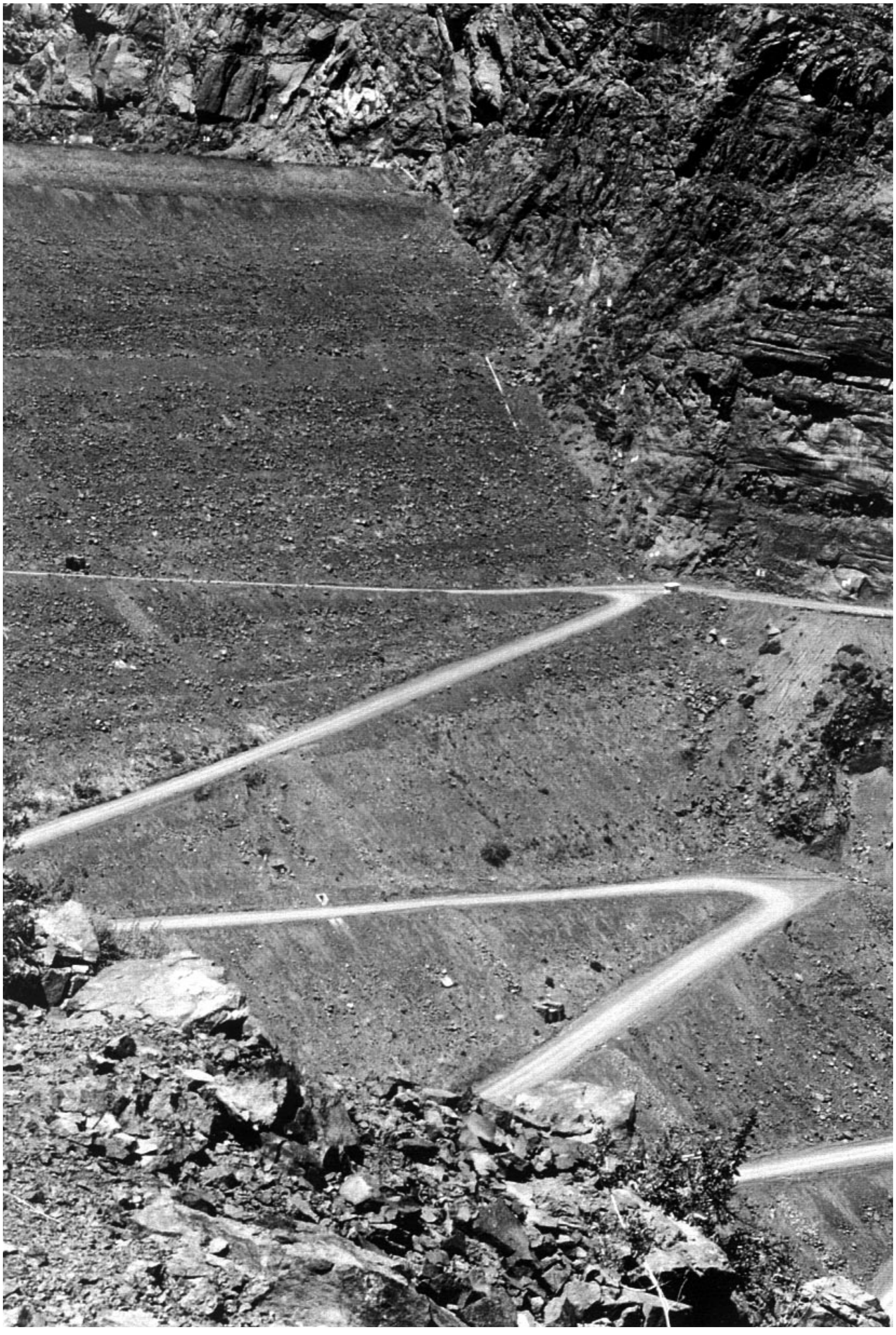


Figure 3
DOWNSTREAM FACE (November 23, 1993)
Los Leones Dam

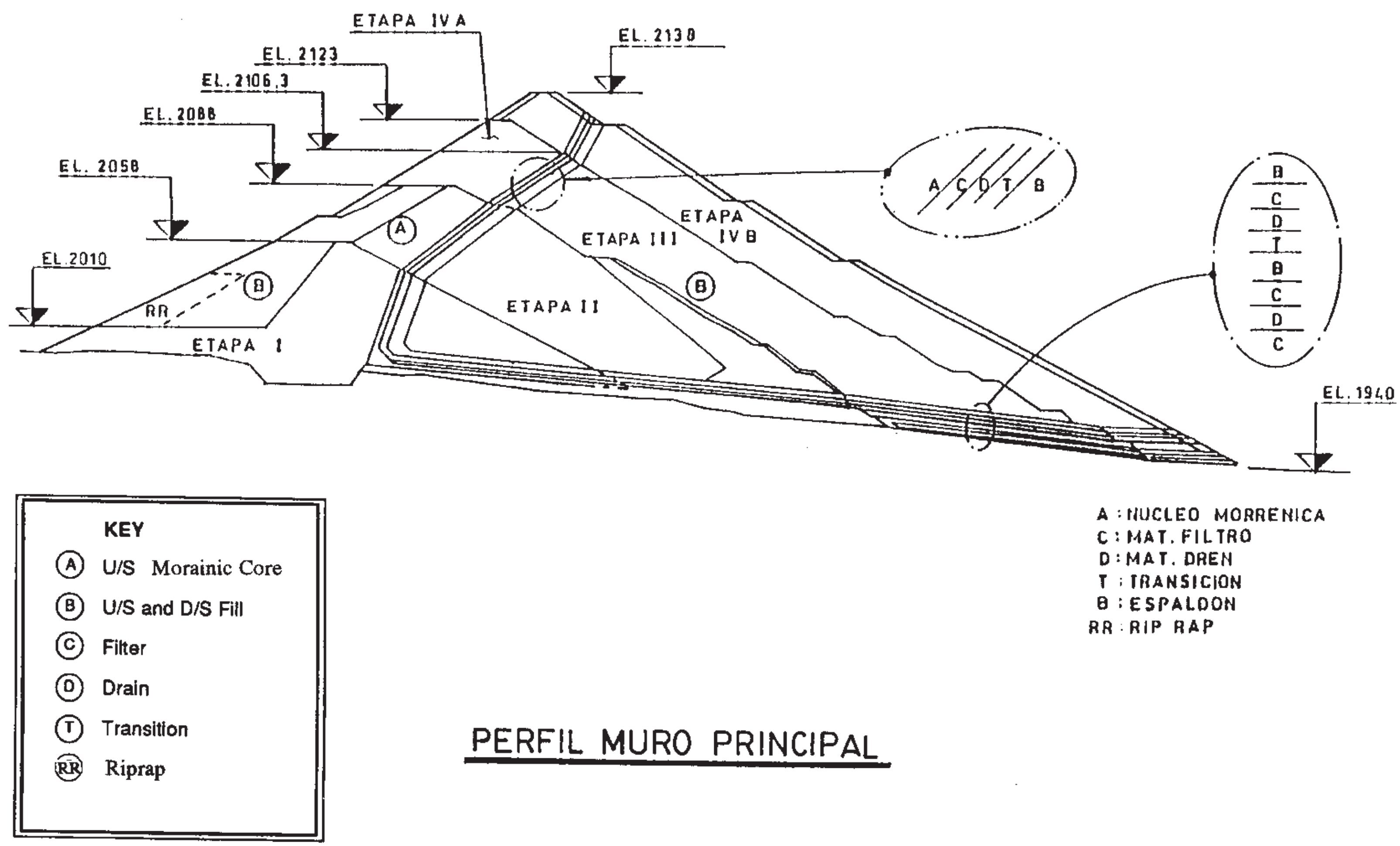
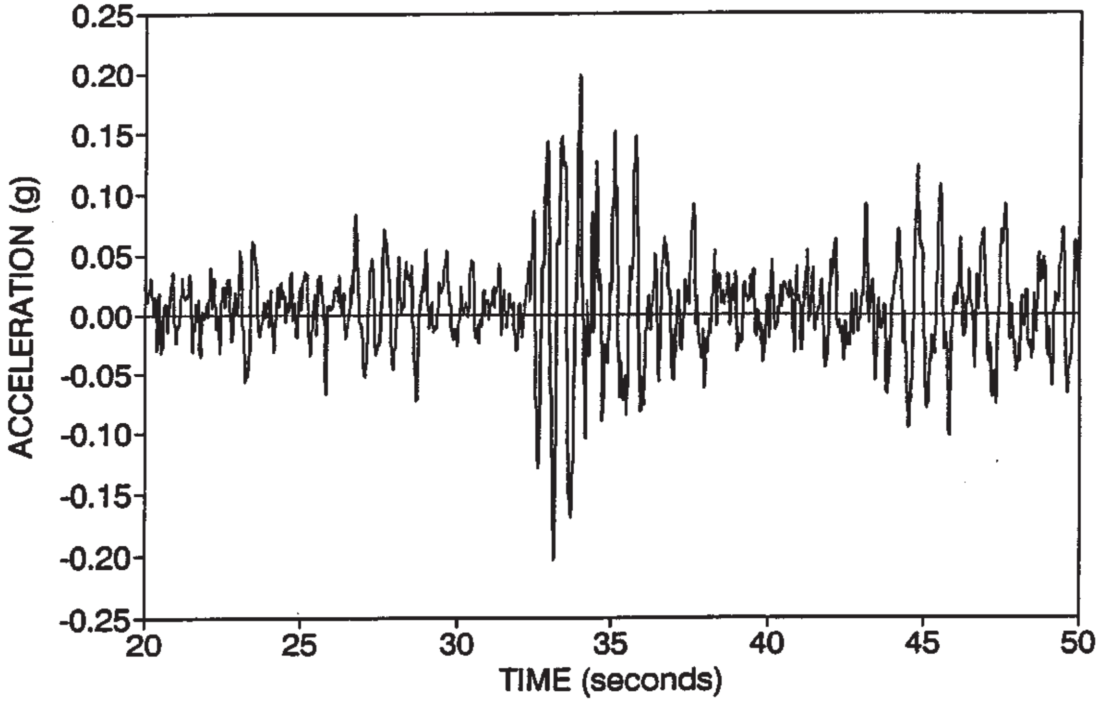
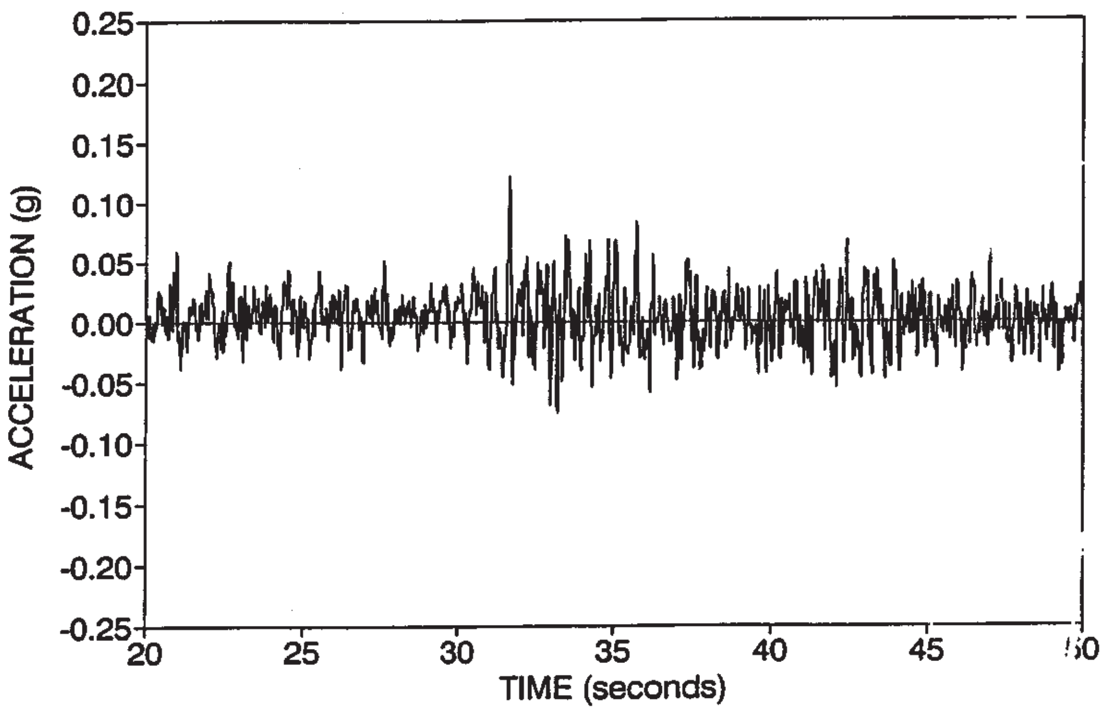


Figure 4
EMBANKMENT ZONING DETAILS
Los Leones Dam

March 3, 1985 - Crest Motion [H]



March 3, 1985 - Base Motion [H]



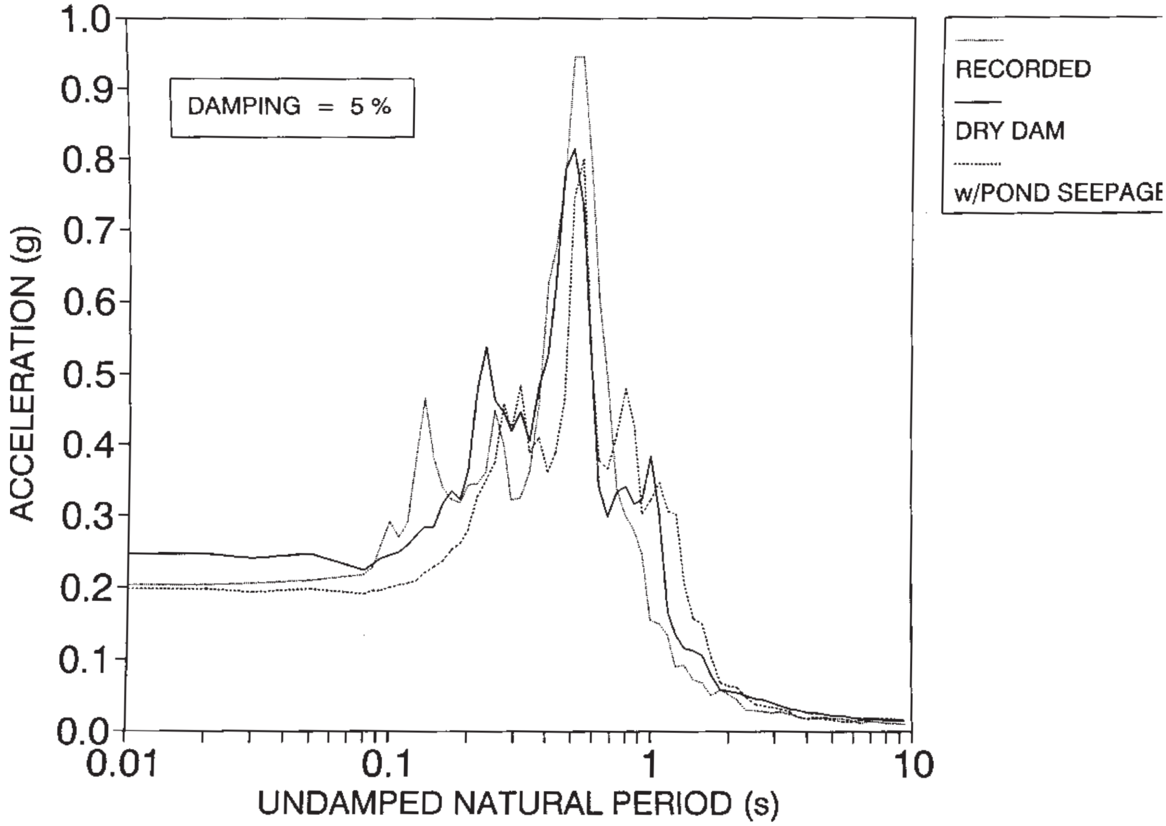


Figure 6
CREST RESPONSE SPECTRUM COMPARISON (March 3, 1985 EQK)
Los Leones Dam

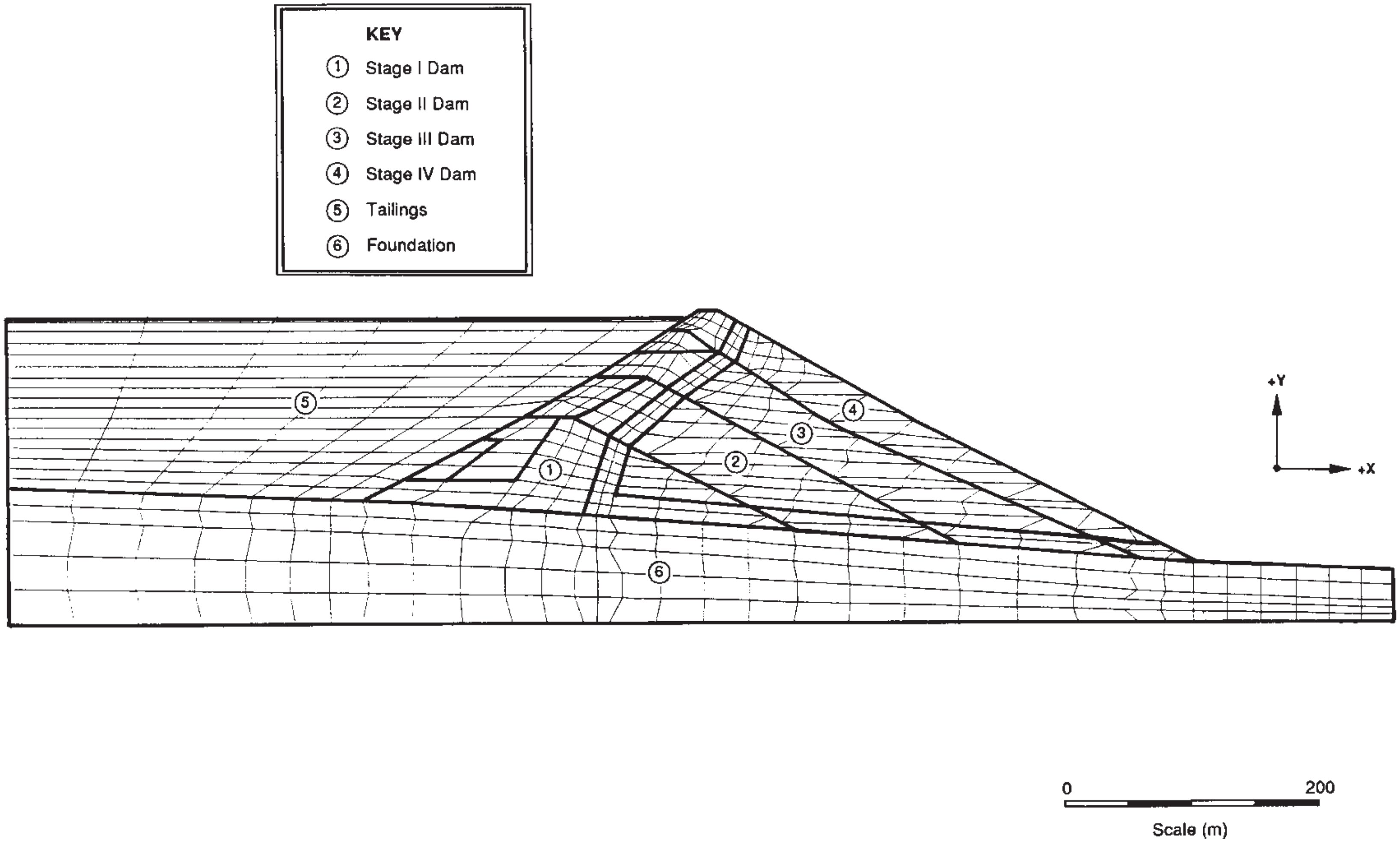


Figure 7
FINITE DIFFERENCE MODEL
Los Leones Dam

MASIWAY DAM, PHILIPPINES

On July 16, 1990, a large earthquake (M 7.7) struck the Philippines. Masiway Dam, owned by the country's irrigation and power administration, the Philippines National Power Corporation, is one of six dams that were located within a short distance from the epicenter.

The 82 feet (25 m) high embankment dam suffered extensive damage, including probable liquefaction in the upstream shell. The upstream shell slumped up to two meters horizontally and settled about one meter. Various nearby slopes suffered significant failures. Estimated peak ground acceleration at the dam site was probably 0.65g or higher.

Masiway Dam

The dam regulates releases from the 351-foot-high Pantabangan Dam, which is located three miles upstream. The layout of the dam and appurtenant facilities is shown in Figure 1. Masiway Dam is an 82-foot-high (25 m) zoned earthfill dam with a central clay core (Figure 2). It has a crest length of 1,400 feet and crest width of 33 feet. The shells consist of alluvial material and conglomerate. The upstream shell of the dam consists of random alluvium, with a permeability of less than 10-3 cm/s and slopes at 2.3:1 (horizontal to vertical). The downstream shell slopes at 2:1 (h to v). Filter zones were placed on both sides of the core, and a chimney drain and a horizontal drainage blanket are located below the downstream shell.

Other project features include a concrete chute service spillway with three radial gates, an unlined, 260-foot-wide auxiliary spillway with a fuseplug, an intake and semi-outdoor 12 MW single-unit powerhouse. The Masiway Hydroelectric Project was placed in service in 1981. On July 16, 1990, the day of the earthquake, the reservoir elevation was El. 128 m. The reservoir elevation on August 6, 1990, was El. 125.99 m.

The July 16, 1990, Earthquake

On July 16, 1990, the heavily populated Island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 20,000 square miles. At least 1,700 people were killed and perhaps 1,000 were missing. At least 3,500 persons were severely injured. Over 4,000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake and many of the better hotels were damaged.

Seismologically, the July 16, 1990, earthquake is particularly difficult to characterize since it appears to have had two centers of energy release that were apparently triggered within a few seconds of each other. The first one was located on the Philippine Fault near the city of Cabanatuan; the second center of energy release was on the Digdig Fault, which belongs to the same system as the Philippine Fault and branches off northeast from that feature. The two faults broke along a combined length of about 75 km. The fault displacements were left-lateral strike- slip. The maximum mapped displacement was on the order of six meters.

The energy released in the combination of the two events has been reported to correspond to a Richter magnitude of 7.7. In the years that followed the earthquake, seismologists have been continuing studies related to defining better the magnitude level, because of the difficulties resulting from the superimposition of two distinct events.

Masiway Dam was perhaps located as close as 5 km to the segment of the Philippine Fault that ruptured on July 16, 1990. It is the closest to the source of energy release among several dams that were shaken by strong motions from the earthquake. That distance is approximate and based on discussions with staff members from the Philippines National Power Corporation (PHILVOCS), the owner of the dam.

Earthquake Effects and Observed Performance

Reservoir level. On July 16, 1990, the reservoir elevation was El. 128 m. Controlled drawdown was initiated at a rate of about 10 cm per day following the earthquake. The reservoir elevation on August 6, 1990 was El. 125.99 m.

Dam. This 25-meter-high embankment dam suffered extensive damage. The upstream shell slumped up to two meters horizontally and one meter vertically. Locations of observed cracks and directions of movement are shown in Figure 3. All the principal cracks were parallel to the dam axis. The largest of these cracks extended along most of the crest access road, about five feet from the centerline, and to a depth of about 5.5 feet, as observed in test pits. Major cracks were also observed at between 3 and 11 feet below the crest elevation, especially along the upstream slope. The shell ravelling appeared to approach the natural angle of repose of the constituent materials at several locations.

On the upstream side of the crest, several aligned sinkholes provided further evidence of movement of the upstream shell along the core. The dam settled by almost one meter along the left spillway training wall. Based on observations and reports, settlements gave the impression of being of the same order of magnitude over the entire length of the embankment. A subsequent report (Swaisgood and Au-Yeung (1991) indicated overall settlements between 6 inches to over 3 feet (Figure 4). The difference in behavior between the upstream and downstream shell pointed to the probable occurrence of liquefaction in the upstream shell.

Spillways. There was little observable damage to the spillways, except longitudinal cracking of maximum width of about four inches along the entire length of the fuseplug. The dam owner reported that the spillway remained fully operational after the earthquake. Slope failures were also observed along the training dike which connects the left abutment of the main dam and the right extremity of the fuseplug.

Powerhouse. An excessive quantity of seepage water was observed to flow into the powerhouse through drains in the upstream wall of the control room. The incoming seepage exceeded the capacity of the drainage evacuation system, thereby causing some wetting of the powerhouse floor. The powerhouse operator reported that the seepage inflow varied with the reservoir elevation, although no precise correlation was established with the actual elevation of the entrance points of the drain pipes.

A switchyard area, on fill material, is located upstream from the powerhouse. There may have been some slight settlement in that area. The concrete water conduit which connects the intake structure to the powerhouse is located below this area. This water conduit is under full reservoir pressure. Some deformation of the powerhouse/water conduit may have caused leakage from the pressure conduit to enter the drains, causing an excessive flow of seepage water into the powerhouse. It was found necessary to dewater the water conduit to check for possible cracks in the lining at its junction with the powerhouse and plug them against leakage.

Side slope stability. The slopes surrounding the powerhouse parking area suffered various slides. Some slope stabilization work was required to restore a safe access to the powerhouse.

Instrumentation and Strong Motion Records

No strong motion records of the event of July 16, 1990, were recorded in the vicinity of Masiway Dam. The dam was not instrumented for earthquake loading.

Conclusions

The 25-meter-high embankment dam suffered extensive damage. The difference in behavior between the upstream and downstream shells indicated probable occurrence of liquefaction in the upstream shell. The settlement, cracks and deformations experienced by Masiway Dam are related to the strong level of shaking that resulted from its short distance from the causative fault. The dam appears to have responded similar to other embankment dams that were exposed to ground motions of comparable local intensity levels and probable duration.

Extensive repair work was necessary to bring the dam and reservoir back to full operation. Additional fill materials were placed on the crest to bring the embankment back to its original elevation. The upstream slope was regraded and a stabilizing berm was constructed at he base of the left training wall of the spillway approach.

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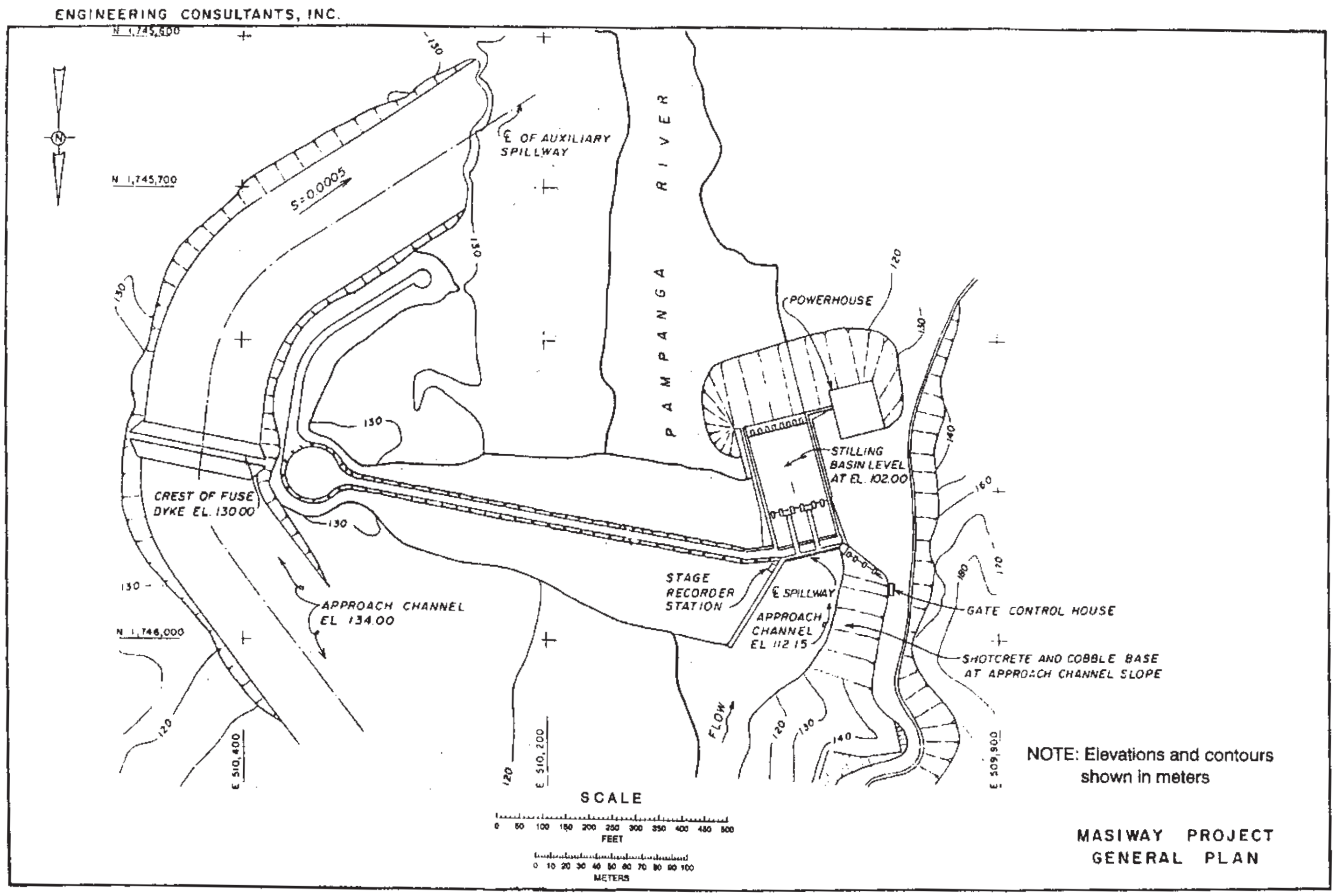

Figure 1
PLAIN VIEW
Masiway Dam

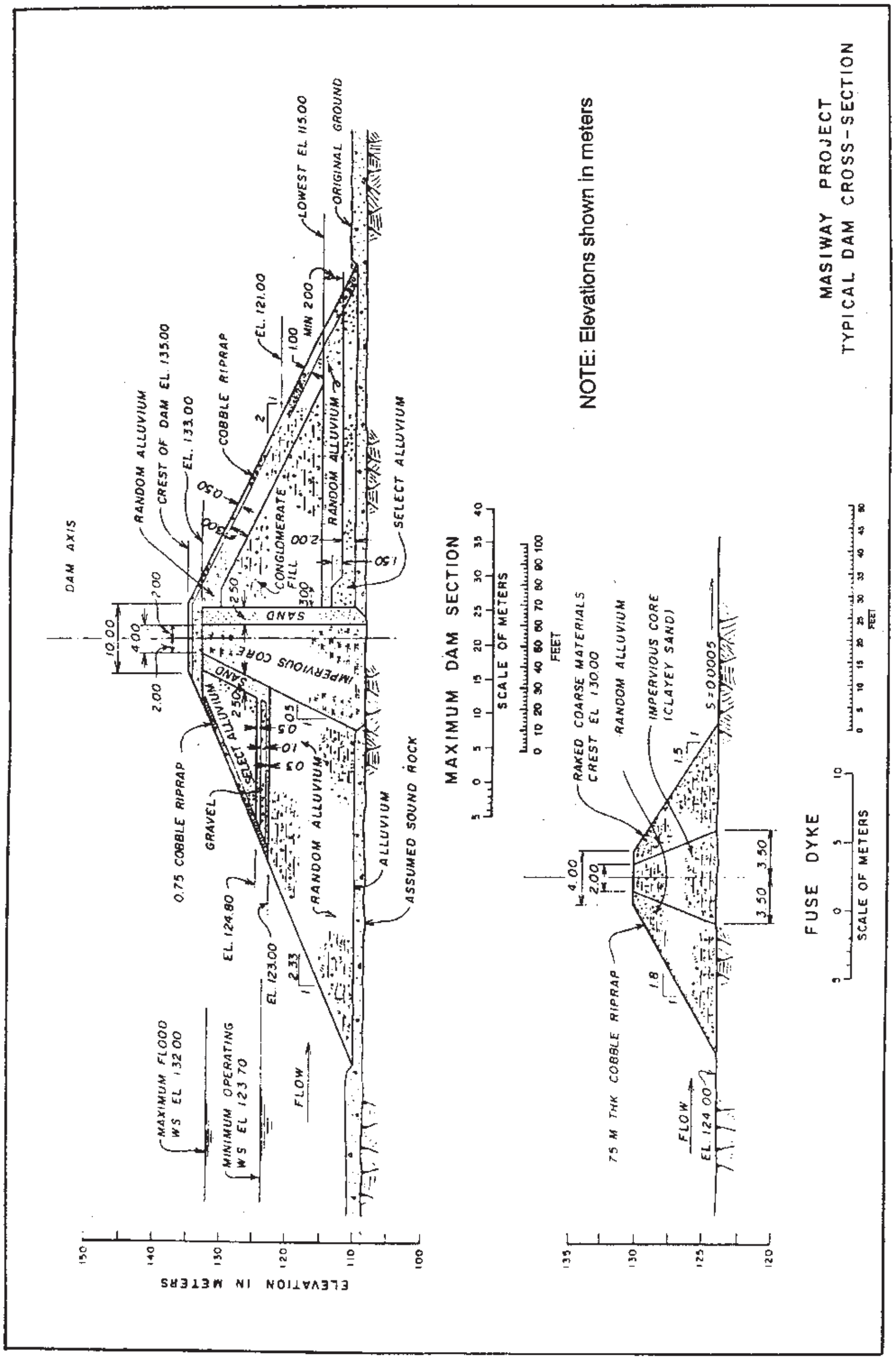

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Figure 2
MAXIMUM SECTION
Masiway Dam

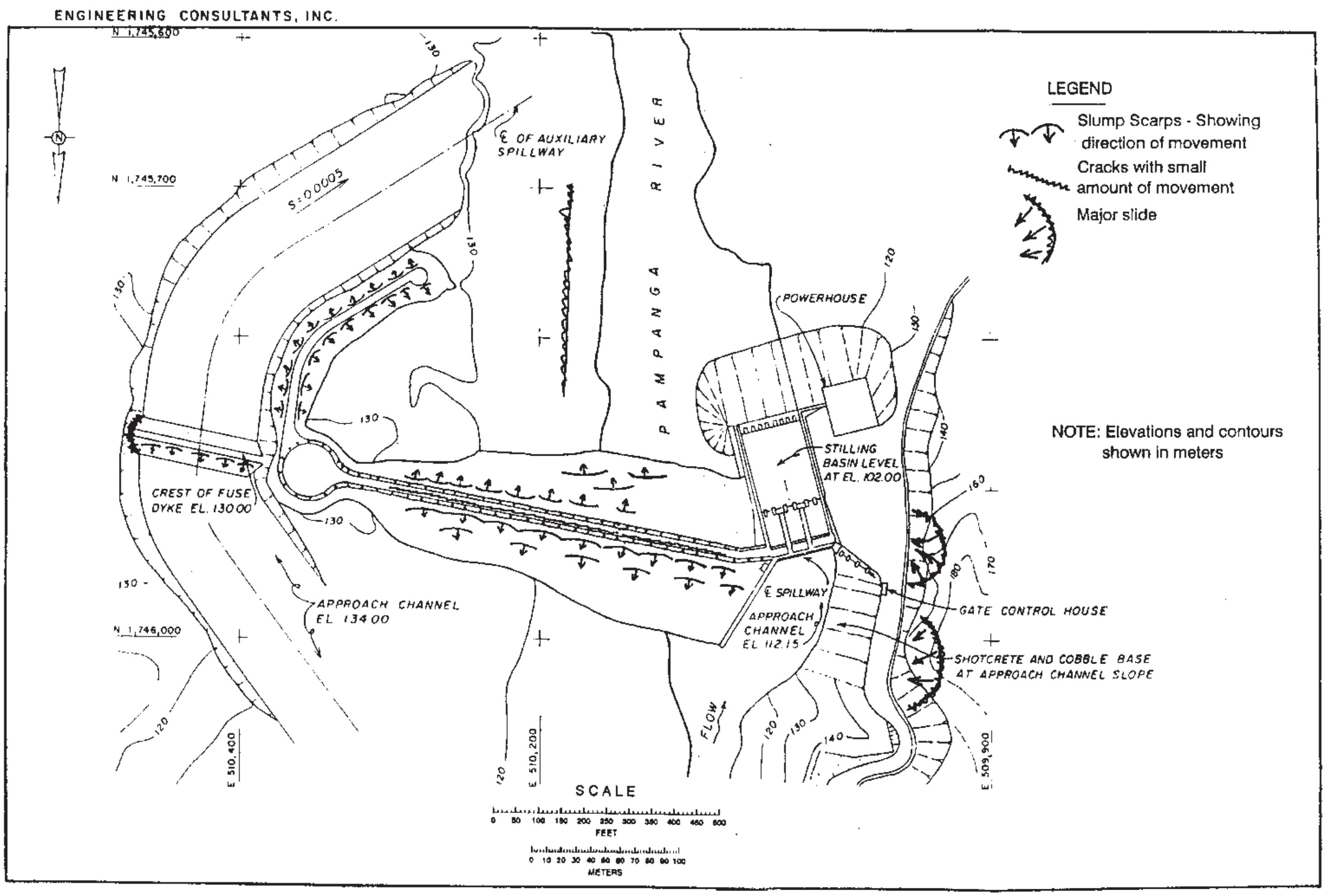

Figure 3
EARTHQUAKE DAMAGE
Masiway Dam

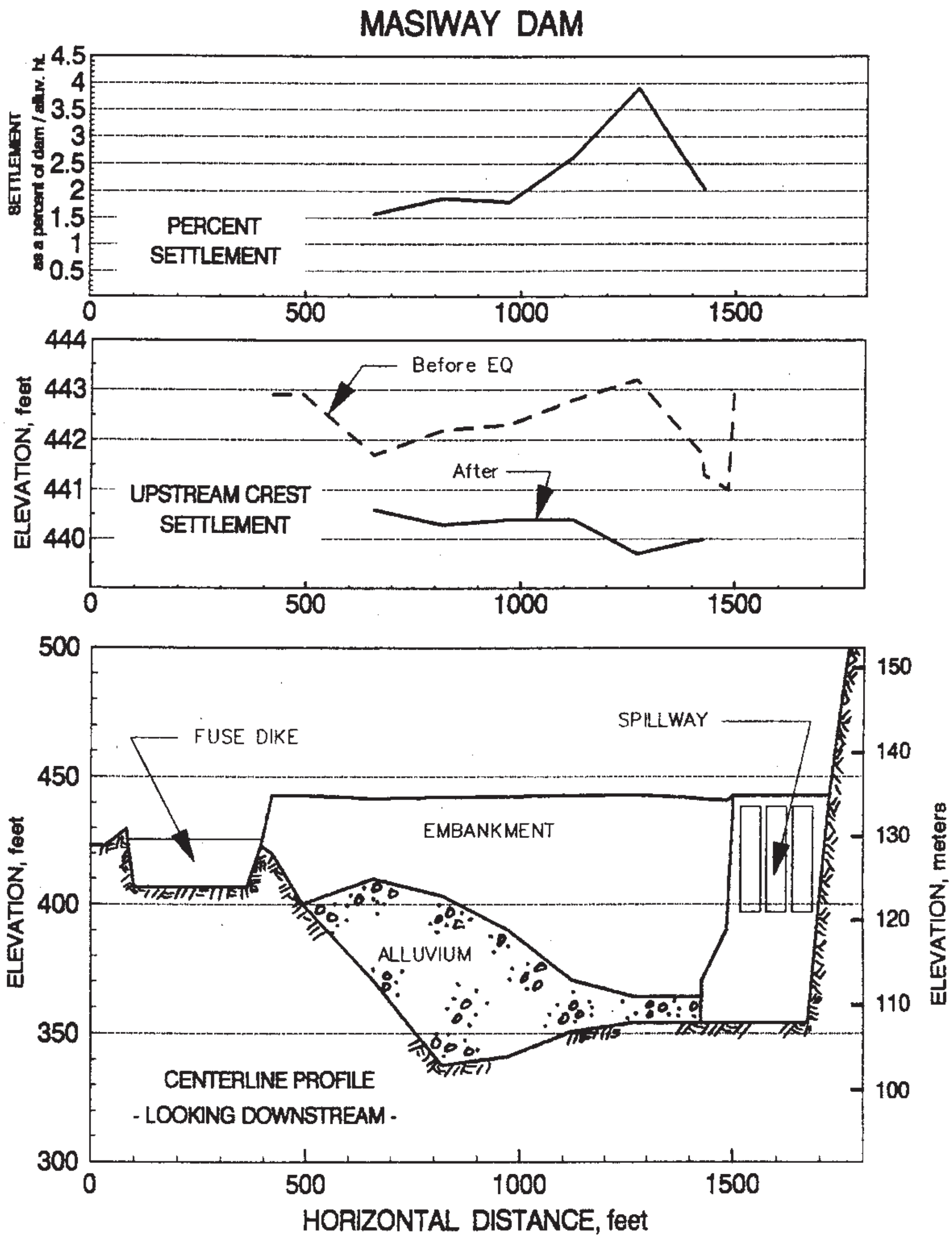

Figure 4
CREST SETTLEMENT
Masiway Dam

MOCHIKOSHI TAILINGS DAM

The Hozukizawa tailings disposal pond at the Mochikoshi complex in Japan was impounded by three dikes (Dikes No. 1 to No. 3). As a result of the 1978 Izu-Ohshima-Kinkai Earthquake, Dike No. 1 failed immediately and Dike No. 2 failed about one day after the earthquake, without further shaking.

The Hozukizawa Disposal Pond

The following descriptions of the impoundment and the earthquake effects are a summary taken from Ishihara (1984). The Hozukizawa disposal pond at Mochikoshi was constructed in a bowl-shaped depression on top of a mountain by sealing off the periphery with three dikes (Figure 1). A detailed plan view is shown in Figure 2. The site originally consisted of a weathered deposit of tuff with cobble inclusions, deposited by a series of neighboring volcanic eruptions.

At the initial stage of construction, the highly weathered natural surface layer was stripped off and the less weathered tuff was exposed. Saw tooth shaped rock asperities provided a rough foundation surface for the starter dam. The starter dam was constructed in 1964 by spreading local volcanic soils with bulldozers. During construction, the soils were compacted by several passes of the bulldozers. In order to provide drainage for water seeping from nearby natural springs, a system of drainage conduits was installed at the bottom of the starter dam. However, because of the relatively high degree of permeability of the original mountain deposits, no drainage system was installed over the bottom of the pond for draining excess water resulting from consolidation of the tailings sludge.

The mine’s milling operation to extract gold was conducted in a processing concentration plant located beside the Mochikoshi River (Figure 1). The tailings were pumped as a slurry through discharge pipes up 600 m to the disposal pond, which was located on top of the mountain. The slurry was delivered either to the top of Dike No. 1 or Dike No. 2, and discharged toward the pond through three pipes at each dike location. The dikes were successively raised by placing local volcanic soils at a rate of approximately 2 m per year by the upstream method of tailings dam construction (the downstream slope is maintained fixed, while the crest is raised in the upstream direction).

Cross sections before and after the failures of dikes No. 1 and No. 2 are shown in Figures 3 and 4. These cross sections include the logs of borings drilled after the failures. Typical gradations of the dike and tailings materials are shown in Figures 5 and 6, respectively.

The tailings deposit is a stratified sequence of silts and sandy silts. The void ratio of these silts and sandy silts was 0.98 and 1.00 and their specific gravity 2.72 and 2.74, respectively. The plasticity index was 10 for the silts, while the sandy silts were classified as non-plastic.

The dikes were constructed of a mixture of the weathered tuffs and volcanic ashes that covered a widespread area of the mountains in the vicinity. These materials are a mixture of gravel, sand and silt, as shown by the wide range of the grain size distribution curves shown in Figure 5. The wet unit weight of these soils ranged between 14 and 19 kN/m3 ; the natural water content was 30 to 60 percent; and the void ratio was 1.1 to 2.6.

Permeability coefficients obtained from the in-situ grouting method were on the order of 10-4 cm/s for the bulldozer-compacted dike materials and 10-3 cm/s for the original bedrock (weathered tuff). The permeability coefficient of the tailings was estimated to be about 7 x 10-4 cm/s horizontally. Due to their highly stratified nature, the vertical permeability of the tailings was probably one thousandth to one hundredth times lower.

The Izu-Ohshima-Kinkai Earthquake of 1978

On January 14, 1978, a destructive earthquake (M 7.0) shook the southeastern area of the Izu Peninsula, about 120 km southwest of Tokyo, Japan (Figure 7). The epicenter of this event was located about 15 km off the east coast of the peninsula. The main shock was followed for about a week by a series of aftershocks, with epicenters moving gradually in a westerly direction (Figure 9). The two largest aftershocks, including one of magnitude 5.8, took place at 7:30 a.m. and 7:36 a.m. on January 15, 1978. Their epicenters were located approximately in the middle of the Izu Peninsula, close to the Mochikoshi tailings dam site.

Most of these events were estimated to have had a focal depth of about 10 km. The strong ground shaking produced by the earthquake was recorded at several stations, but outside the area of highest intensity shaking. A survey of the overturning of tombstones in many cemeteries in the epicentral area was used to obtain an approximate estimate of the distribution of shaking intensity. Estimated contours of equal peak horizontal accelerations are shown in Figure 8 (Ohashi, et al., 1978).

Earthquake Effects and Observed Performance

The failure of Dike No. 1 was triggered by the main shock of the Izu-Ohshima-Kinkai Earthquake. A cross-section through the collapsed dam is presented in Figure 3. Dike No. 1, which was the largest (28 m high and 7.3 m wide at crest level) collapsed almost concurrently with the strong phase of the earthquake shaking. An attendant at the pond, who happened to be stationed at a house on the left bank, came out immediately upon perceiving an unusually high level of shaking and watched the failure. According to his account, within about ten seconds after the main shock, the front face of the dike bulged, and a breach occurred in the upper part of the embankment, near the left abutment. It was followed by a huge mass of tailings slimes rushing down the valley with a loud roar, toppling trees and scouring the valley floor in the process. When the rushing slimes reached the Mochikoshi River, they hit masonry walls on the opposite river bank, surging up to a height of about 10 m and leaving near 30-cm-thick deposits over the road beside the river. The slimes flowed down into the Mochikoshi River, leaving 1.0- to 1.9-m-thick sediments in the river bed along a distance of about 800 m from the point of confluence. The flowing slimes traveled further into the Kano River, and contaminated that river to a distance of about 10 km downstream.

The top portion of Dike No. 1 failed totally throughout a height of 14 m from the top level of the embankment down to the top elevation of the starter dam, as illustrated in Figure 3. A volume of 80,000 m3 of materials was released by the dike failure, of which 60,000 m3 were tailings slimes and 20,000 m3 were part of the dike-forming volcanic ashes.

Dike No. 2 did not fail during the earthquake. A series of medium-sized aftershocks rocked the central part of the Izu Peninsula from early morning to about noon on January 15, 1978. The largest of the aftershocks (M 5.8) occurred at 7:31 a.m., immediately followed by the next biggest aftershock (M 5.4) at 7:36 a.m. Following those events, an inspector found at about 8:30 a.m. that five to six cracks were developing along the downstream face of Dike No. 2, parallel with the axis of the dike. Those cracks were reported to be 1 to 3 m long, with openings about 5 mm wide. Subsequently, around 9:30 a.m., another inspector discovered a longitudinal open crack, 5 m long and 5 cm wide, in the middle of the downstream slope of Dike No. 2.

At about 1:00 p.m. on that day, a caretaker standing on the opposite side of Dike No. 2 noticed a gradual sinking of the central part of the embankment. While running to the site, he watched the dike fail suddenly through a crest breach about 20 m wide, which led to the release of the impounded tailings sludge. Later on, the breach size increased to a crest width of 65 m and generated a number of cracks over the sloughing surface. A total volume of 3,000 m3, consisting of 2,000 m3 of tailings slimes and 1,000 m3 of dike materials, flowed down the valley a distance of about 240 m. A cross section of the failure surface, superimposed on the original dike section, is shown in Figure 4.

Instrumentation and Strong Motion Records

No strong motion records were obtained at the Mochikoshi tailings dam sites. Estimated peak ground accelerations in the general area are shown in Figure 8. It may be seen that, at the Mochikoshi site, the peak ground acceleration was estimated to be approximately 0.25 g.

Conclusions

The failure of the tailings impoundment at Mochikoshi was typical of those of impoundments constructed by using hydraulic upstream methods. As the sandy part of the embankment is continuously placed over saturated, weaker slimes, such failures are comparable to weak foundation failures and generally do not initiate within the sandy or coarser fractions. The most unusual characteristic of the Mochikoshi event was the long-delayed (one day) failure of Dike No. 2, presumably as a result of excess pore pressures built up during the shaking, followed by very slow dissipation and redistribution of such pore pressures within weakened parts of the embankment.

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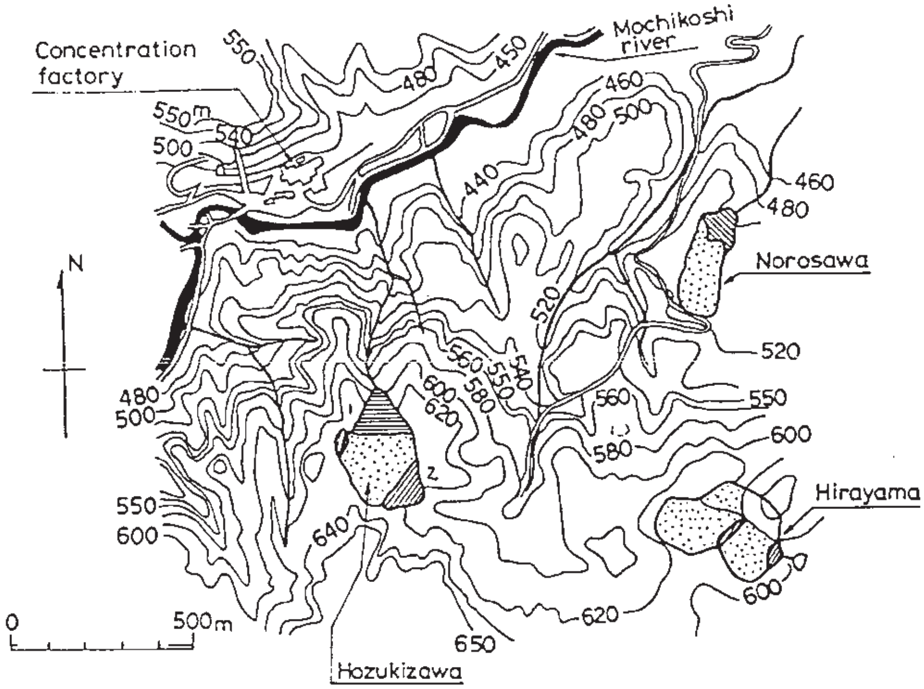

Figure 1
HOZUKIZAWA TAILING POND
Mochikoshi Tailings Dam
From Ishihara, 1984

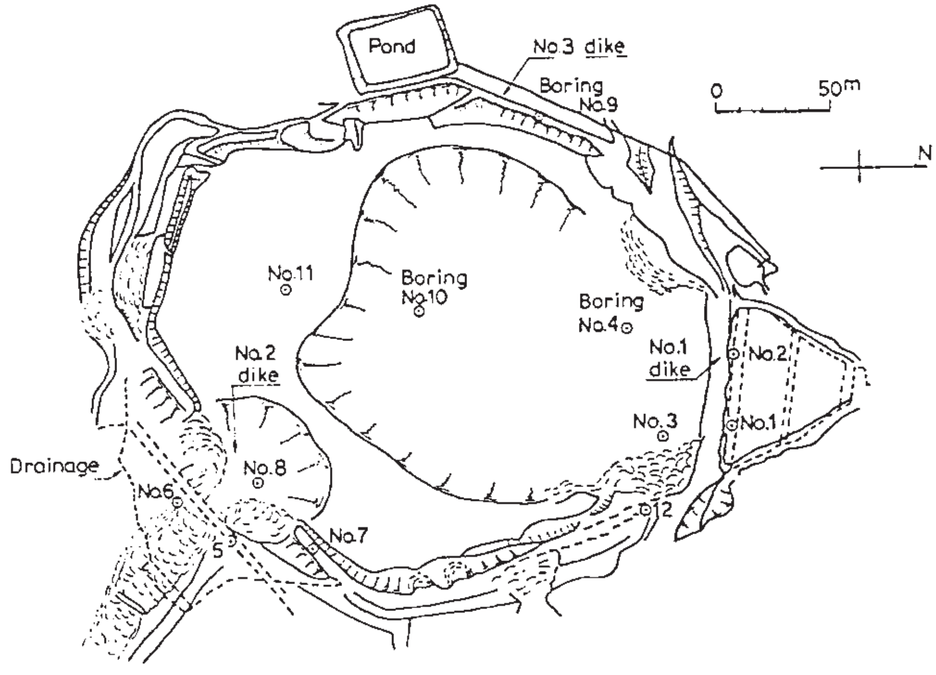

Figure 2
PLAN OF THE HOZUKIZAWA DISPOSAL POND
Mochikoshi Tailings Dam
From Ishihara, 1984

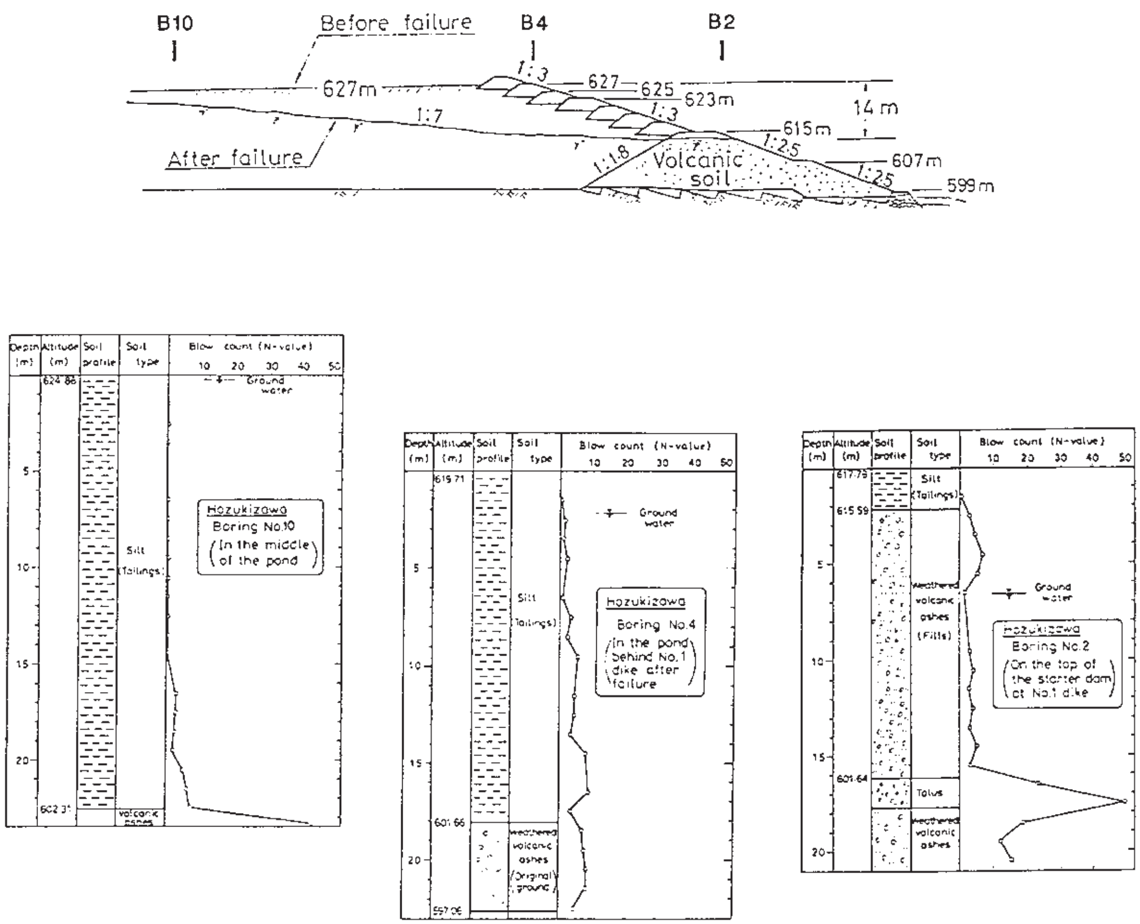

Figure 3

CROSS-SECTION THROUGH EMBANKMENT OF NO.1 DIKE AND BORING LOGS

Mochikoshi Tailings Dam

From Ishihara, 1984

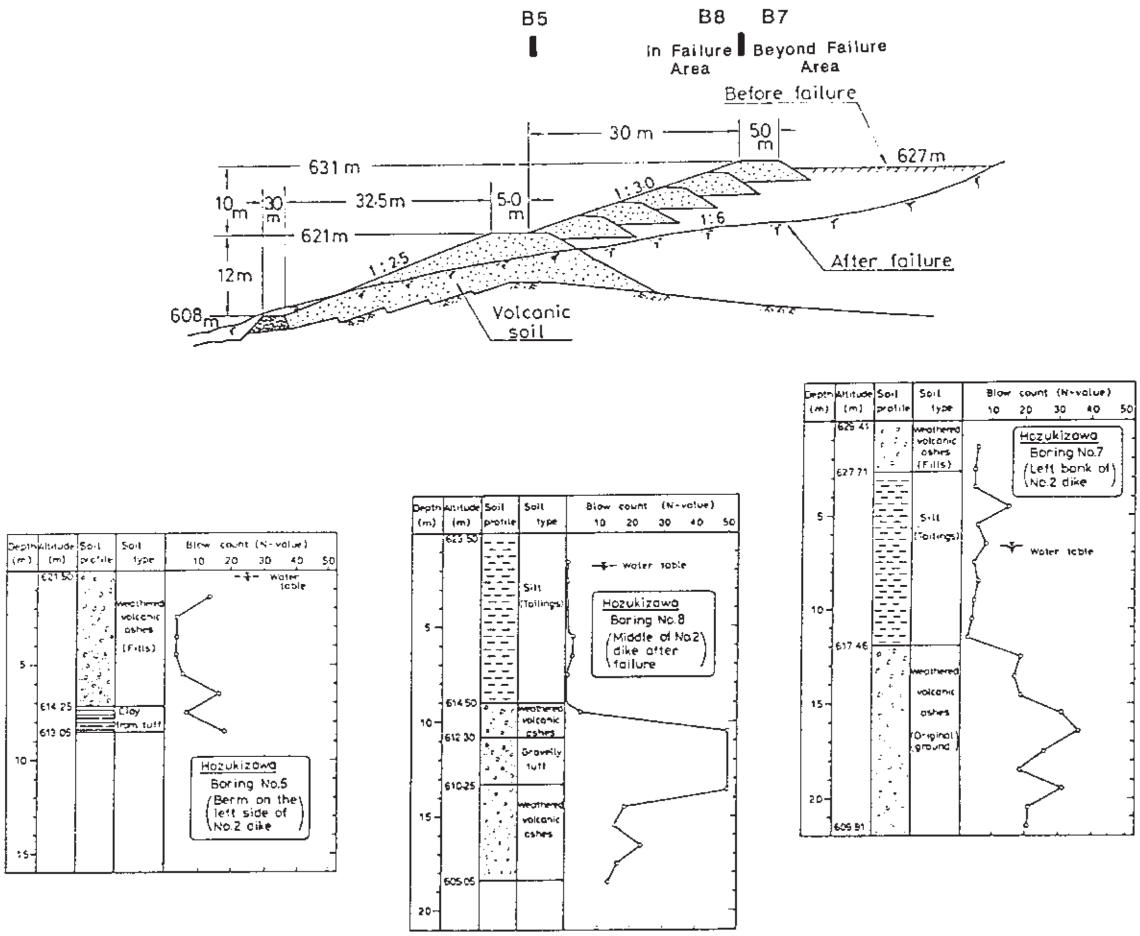

Figure 4

CROSS-SECTION THROUGH EMBANKMENT OF NO.2 DIKE AND BORING LOGS

Mochikoshi Tailings Dam

From Ishihara, 1984

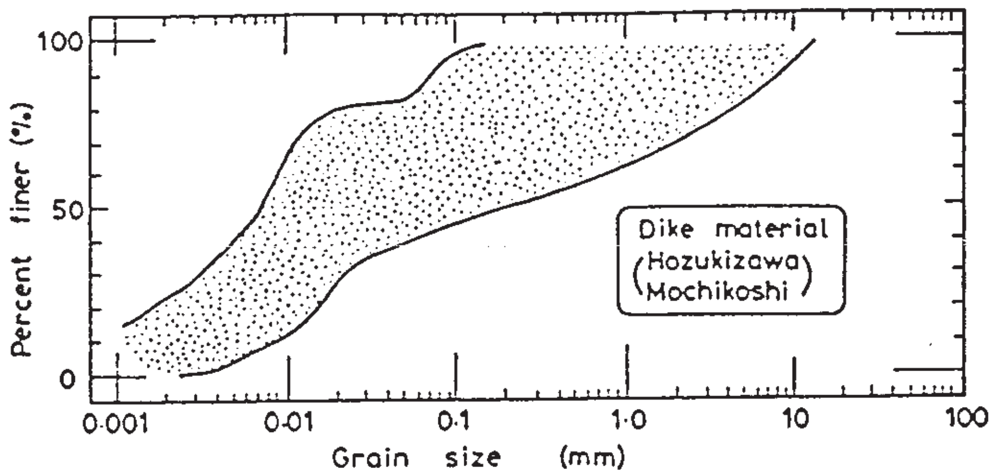

Figure 5
DISTRIBUTION OF GRAIN SIZES IN THE DIKES
Mochikoshi Tailings Dam
From Ishihara, 1984

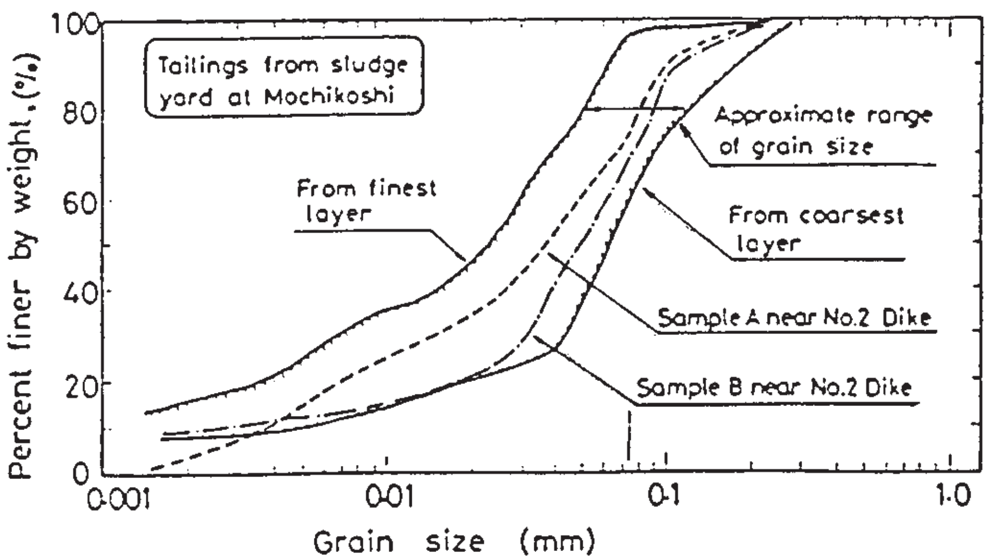

Figure 6
GRAIN SIZE DISTRIBUTION CURVES OF TAILINGS MATERIAL FROM MOCHIKOSHI
Mochikoshi Tailings Dam
From Ishihara, 1984

Figure7
LOCATIONS OF IZU PENINSULA
Mochikoshi Tailings Dam
From Ishihara, 1984

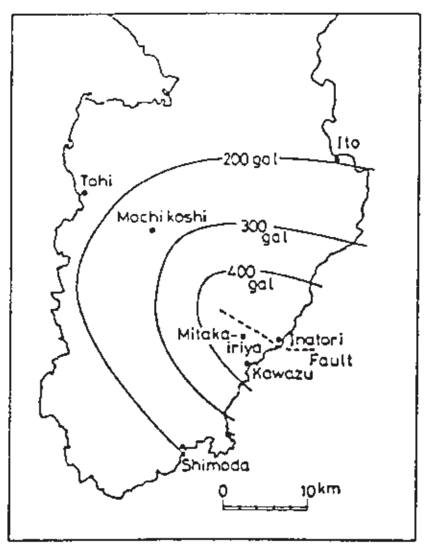

Figure 8
CONTOURS OF EQUAL SHAKING INTENSITY IN TERMS OF ESTIMATED MAXIMUM HORIZONTAL ACCELERATIONS
Mochikoshi Tailings Dam
From Ishihara, 1984

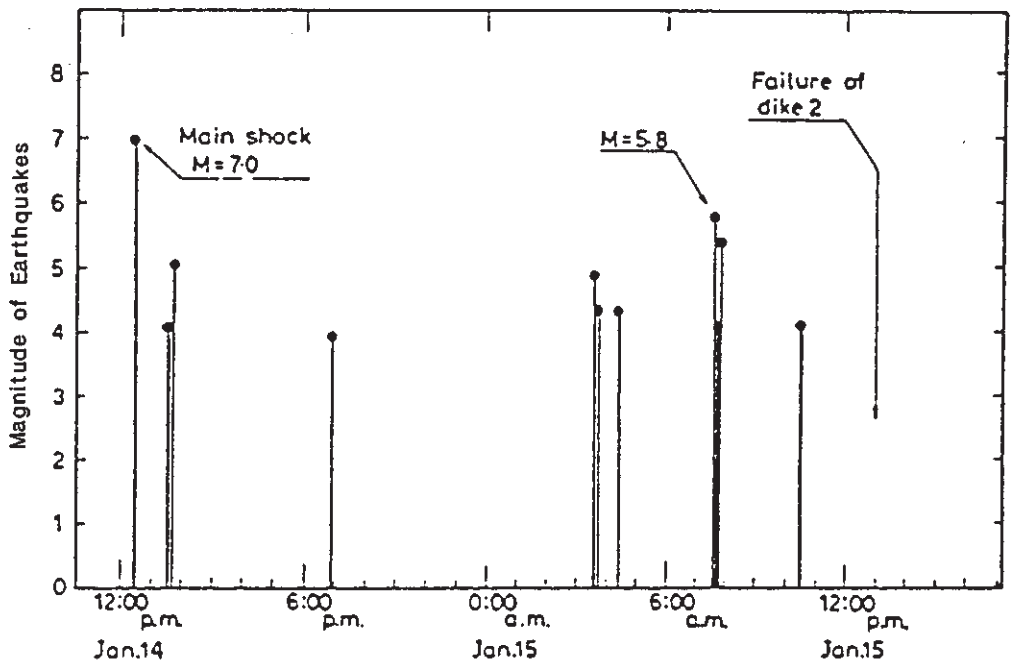

Figure 9

OCCURRENCE OF AFTERSHOCKS WITH MAGNITUDE GREATER THAN 4.0 FOLLOWING THE MAIN SHOCK OF THE IZU-OHSHIMA-KINKAI EARTHQUAKE

Mochikoshi Tailings Dam

From Ishihara, 1984

PANTABANGAN DAM, PHILIPPINES

On July 16, 1990, a large earthquake (M 7.7) struck the Philippines. Pantabangan Dam, owned by the country's irrigation and power administration, the Philippines National Power Corporation, is one of six dams that were located within a short distance from the epicenter. The main dam and Aya Creek Dam, which also forms part of the Pantabangan complex, settled a maximum of about 11 and 8 inches, respectively. Minor cracks were observed on the crests, at the contacts between those dams and their abutments. Estimated peak ground acceleration at the site was 0.65g. The excellent performance of the Pantabangan project was attributed to the low reservoir level that prevailed at the time of occurrence of the earthquake.

Pantabangan Dam

The Pantabangan Project was placed in service in 1977. The Pantabangan impoundment has three components: Aya Creek Dam on the southeast; a low intermediate saddle dam; and the main Pantabangan Dammain dam on northwest (Figure 1). All three dams are zoned earthfill dams. Each of the embankments has a central impervious clayey core with outer shells of alluvial material and weathered conglomerate, a vertical or near-vertical filter and chimney drain, and a horizontal drainage blanket. The maximum sections of the two principal embankments are shown in Figure 2. The impervious core of the main dam is substantially larger at its base than that of the Aya Creek embankment.

Maximum height of the largest embankment is 351 feet (107 meters) and the crest length is 2,400 feet. Slopes are 2.5:1 (horizontal to vertical) upstream, and 2.2:1 (h to v) downstream. The downstream face of Pantabangan Dam is protected with select coarse alluvial material, while most of the upstream slope is faced with a reinforced concrete slope protection.

Aya Creek Dam is approximately 1,400 feet long and has a maximum height of 335 feet. Its slopes are 3:1 and 2.2:1 (h to v), upstream and downstream, respectively. Both of the upstream and downstream faces are protected with select coarse alluvial material.

Other project features include a concrete chute spillway located in rock on the left abutment of the Aya Creek Dam, two intakes towers and two concrete-lined outlet tunnels, 23 feet in diameter (originally used as diversion tunnels), a low level outlet, and a surface powerhouse. The spillway chute is 886 feet long and terminates in a flip bucket.

The July 16, 1990, Earthquake

On July 16, 1990, the heavily populated island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 20,000 square miles. At least 1,700 people were killed and perhaps 1,000 were missing. At least 3,500 persons were severely injured. Over 4,000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake. Many of the better hotels were damaged.

Seismologically, the July 16, 1990, earthquake is particularly difficult to characterize since it appears to have had two centers of energy release that were apparently triggered within a few seconds of each other. The first one was located on the Philippine Fault near the city of Cabanatuan; the second center of energy release was on the Digdig Fault, which belongs to the same system as the Philippine Fault and branches off northeast from that feature. The two faults broke along a combined length of about 75 km. The fault displacements were left-lateral strike- slip. The maximum mapped displacement was on the order of 6 meters.

The energy released in the combination of the two events has been reported to correspond to a Richter magnitude of 7.7. In the years that followed the earthquake, seismologists have been continuing studies related to defining better the magnitude level, because of the difficulties resulting from the superimposition of two distinct events.

The Pantabangan Project is located about 10 km from the Philippine Fault segment that broke on July 16, 1990. That distance is approximate, and is based on discussions with staff from the Philippines National Power Corporation, PHILVOCS.

Earthquake Effects and Observed Performance

Reservoir level. On July 16, 1990, the reservoir elevation was at El. 186.18 m, which is about 35 m below the maximum normal operating pool (El. 221 m). The reservoir elevation increased to 192.47 m by August 6, 1990, as a result of heavy runoff. It continued to rise until the end of the rainy season (December 1990).

Dams. The upstream side of the crest of Pantabangan Dam settled a maximum of 10.25 inches at the maximum section. The settlement decreased proportionately toward the abutments. Settlement profiles are presented in Figure 3. A transverse crack was found in the asphalt pavement of the crest road, at the contact between the embankment and the left abutment. No increase in seepage through the dam was reported.

The only evidence of distress in the saddle dam consisted of diagonal cracks on the paved roadway over the crest, near its left abutment. The cracks were obviously produced by tensile stresses induced by differential settlement, due to the presence of a ridge or a change of geometry in the foundation of the left abutment of the dam. Aya Creek Dam experienced an average settlement of 7.9

inches near its maximum section (Figure 3). A thin crack was found along the crest roadway at the contact between the embankment and the left abutment. No seepage increase was reported.

Spillway. There is no evidence of damage to the Pantabangan main spillway structure. The owner had installed anchored glass plates across the left bridge abutment joints. One plate was installed across the joint between the downstream bridge guides and the bridge left abutment. The other plate was installed across the joint between the parapet walls of the bridge deck and left abutment. The first of these plates remained unbroken, and as installed. The other was cracked, indicating that some slight movement had occurred.

There is no evidence of significant damage to the concrete gravity dam section located to the right of the spillway. One hairline crack was observed at the mid-section on the upstream side of each of the ogee crests. All three radial gates were successfully tested after the earthquake. Both the chute and flip bucket appeared to be in satisfactory condition.

Powerhouse. The Pantabangan Powerhouse is a surface powerhouse with two turbine/generator sets rated at 50 MW each. During the earthquake, the units were not operating. The powerhouse is a reinforced concrete structure, composed of three monoliths with contraction joints separating the monoliths. The only damage that was sustained by this structure was some spalling of the concrete on both sides of the contraction joints at the inside face of the concrete roof. There was no other visible or reported damage to the structure. The units have since been inspected and were operating three weeks after the earthquake. No apparent damage was observed in the access adit to the gate chambers. Water leaks that occurred at the joints of some vacuum valve pipes were rapidly sealed.

Instrumentation and Strong Motion Records

The office of PHILVOCS indicated that no strong motion record of the event of July 16, 1990, had been recovered. A strong-motion accelerograph previously installed at Pantabangan Dam was being repaired at the time of the earthquake.

Conclusions

The Pantabangan hydroelectric project did not sustain significant damage. The minor damage that was observed as modest crest settlement of the two main embankments had no impact on the safety of the dams. Energy production resumed rapidly. Monitoring and surveillance of the embankments were subsequently increased. The leaking fittings in the outlet works were sealed by replacing the damaged flange bolts, packers and O-rings.

The minor damage experienced at Pantabangan was almost certainly related to the low reservoir level at the time of the earthquake and to the fact that the upstream shell materials were most likely not fully saturated, because of the presence of the near-impervious concrete facing. Some small movements of the vertical joints of such concrete facing and occasional spalling of the concrete on either side of the joints were easily repaired and had no impact on the safety of the main dam.

The powerhouse was also subjected to strong motions and performed extremely well. Some concrete spalling observed along the contraction joints in the roof had no impact on the structural integrity of the plant.

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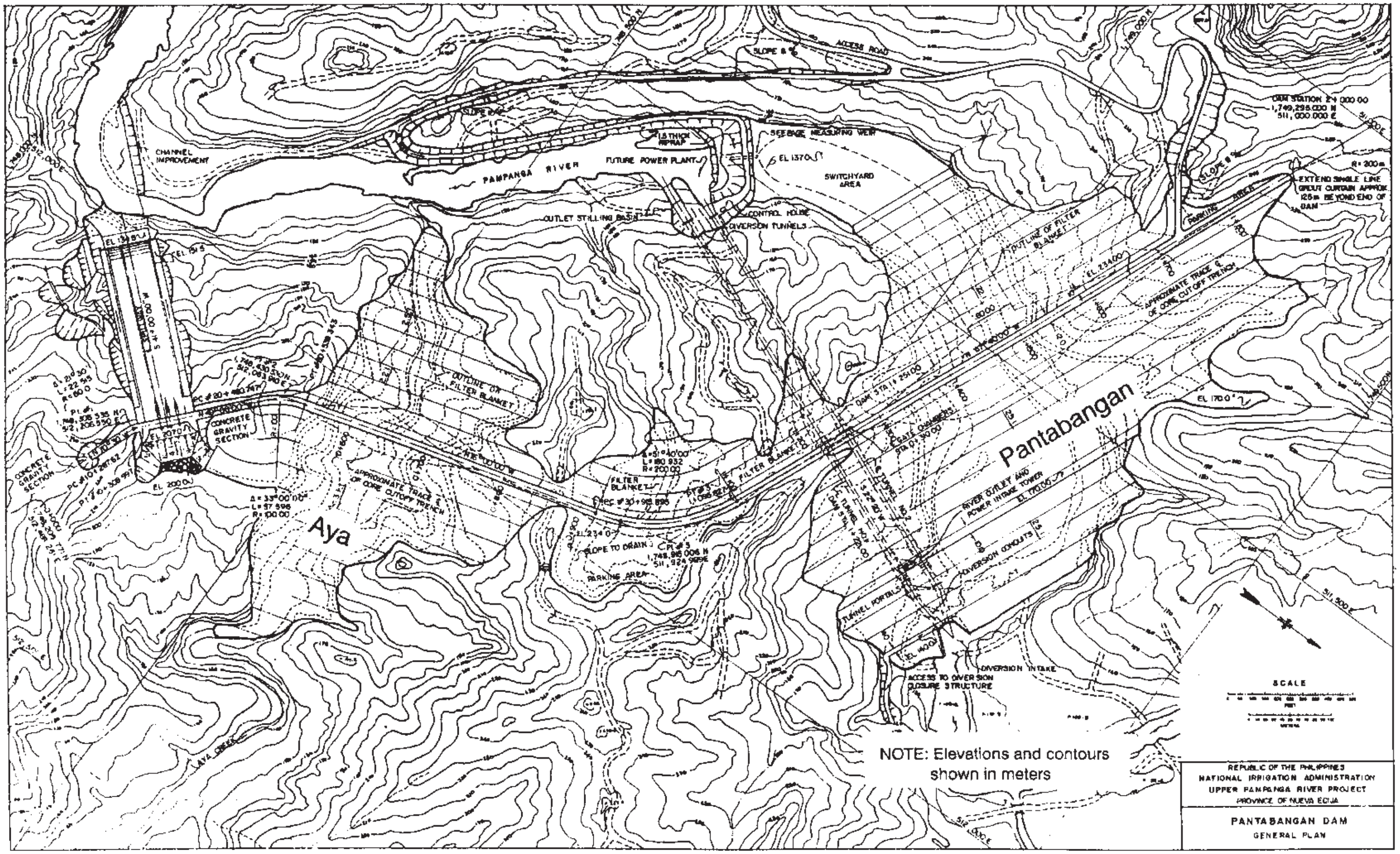

Figure 1
PLAN VIEW
Pantabangan and Aya Dams

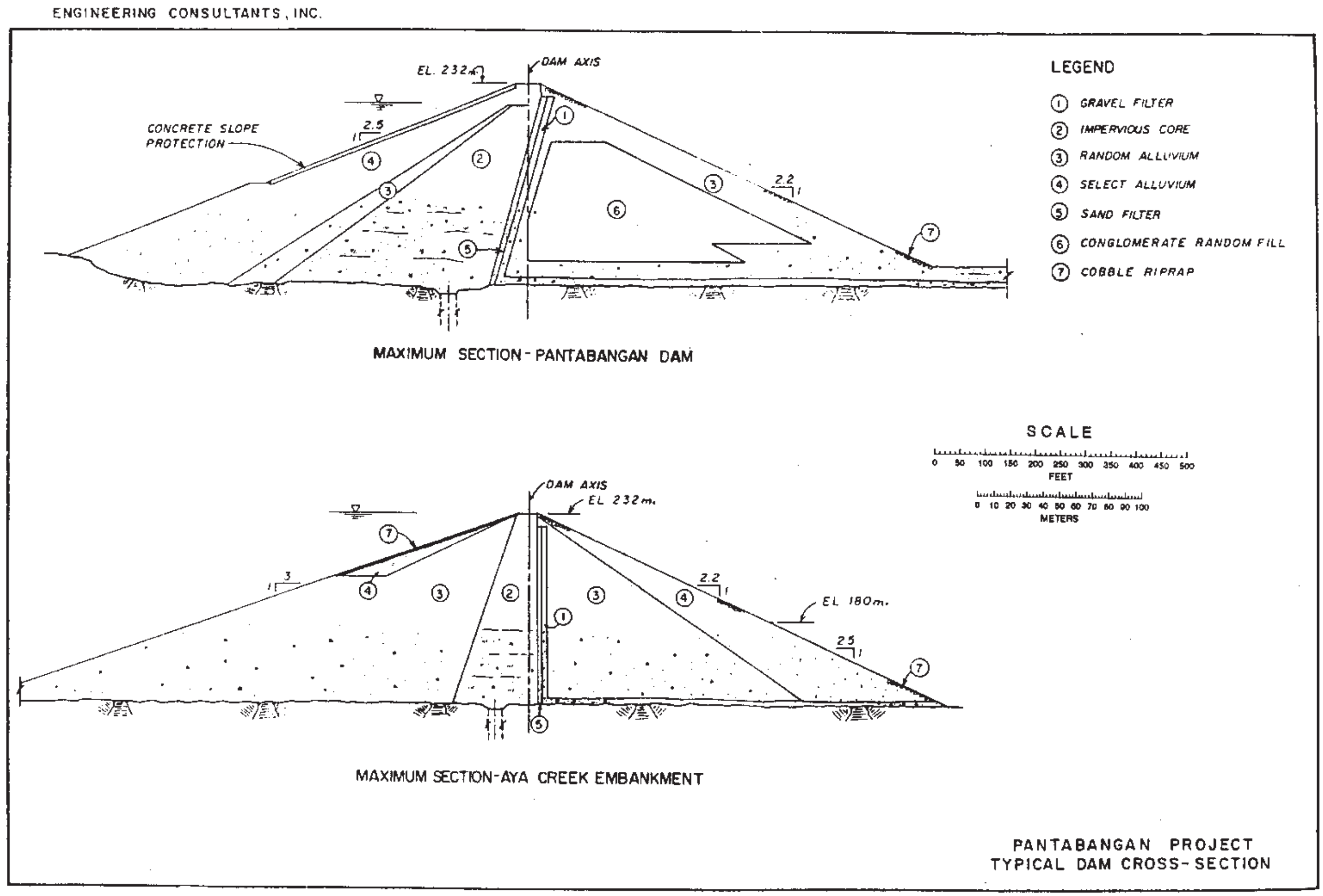

Figure 2
MAXIMUM SECTIONS
Pantabangan and Aya Dams

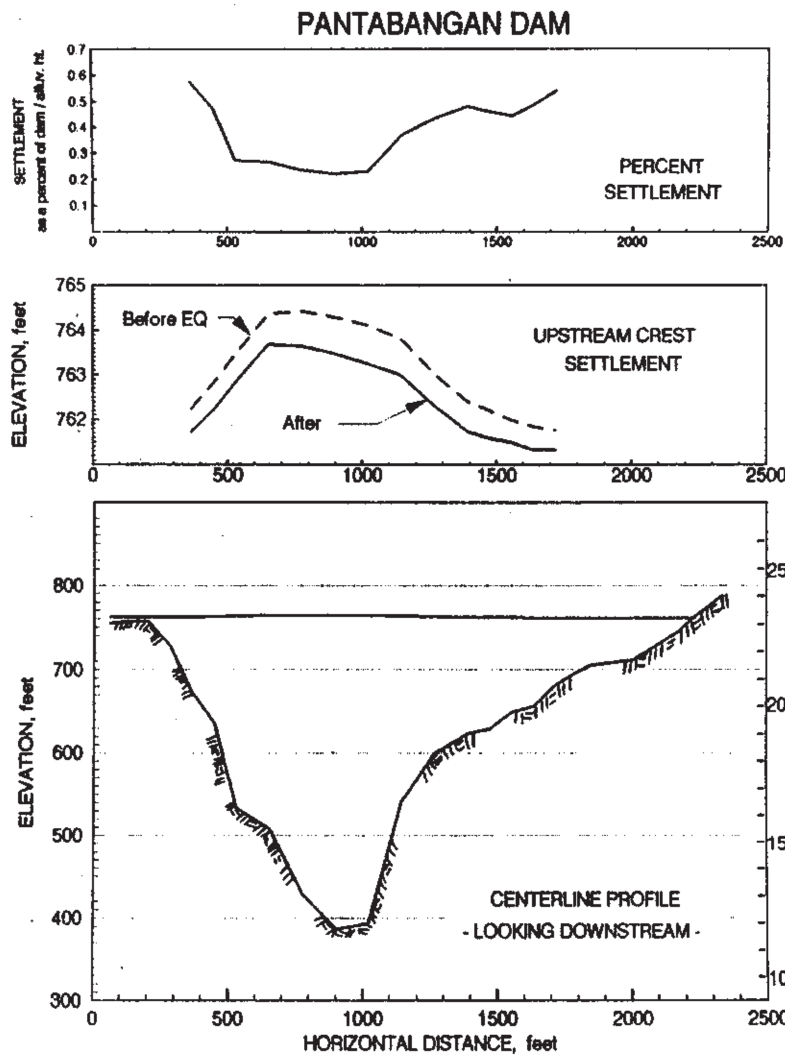

Figure 3
CREST SETTLEMENTS
Pantabangan and Aya Dams

SEFID RUD DAM, IRAN

The Manjil Earthquake of June 21, 1990, occurred in the northern central region of Iran. This magnitude 7.3 earthquake caused heavy casualties and damage to modern structures, their non-structural elements, and equipment. Over 40,000 deaths and 60,000 to 100,000 injuries were reported. Sefid Rud Dam, a large concrete buttress gravity dam, was located less than 20 miles (32 km) from the epicenter and presumably closer to the fault rupture. Peak ground acceleration was estimated at about 0.70g. Subjected to this extremely strong ground shaking, Sefid Rud Dam suffered various forms of damage, including severe cracking in the upper part of the buttresses.

Sefid Rud Dam

Sefid Rud Dam (sometimes referred to as Manjil Dam in the literature) was built from 1958 to 1962 as a 348-feet (106 m) high concrete gravity buttress dam (Figure 1). The dam plays a major role for irrigation purposes. About 50 percent of the rice production of Iran depends on stored water releases from Sefid Rud Dam. The dam has a maximum base width of about 328 feet (100 m) and a crest length of about 1,367 feet (417 m). It is located at the confluence of the Ghezel-Owzan and Shah Rud rivers, and impounds a reservoir of about 1.46 million acre-feet (1.8 billion cubic meters), with a tributary watershed area of about 22,600 square miles (58,000 km2). The dam is composed of 23 buttresses spaced at 46-feet (14 m) center-to-center (Figures 2 and 3). Webs have a constant thickness of 16.4 feet (5 m). The buttresses were designed to act independently from each other. To avoid lateral movements near the right and, especially, the left abutment, buttresses Nos. 6 to 12 and Nos. 18 to 24 were keyed with a series of ground-supported lateral thrust slabs (Figure 4). "Active" joints were provided at the downstream toe of buttresses Nos. 8 to 20 (Figure 5). These joints originally included Freyssinet-type flat jacks, 6.6 ft x 3.3 ft (2 m by 1 m) in size, designed to improve the distribution of foundation stresses between upstream and downstream, and reduce the ratio between shear and normal loads at the foundation-bedrock interface. The joints were grouted once the buttresses presumably reached their final state of stress equilibrium, after a few years of operation.

The spillway is located in the gravity block near the left abutment and has a rated capacity of 70,630 cfs (2,000 m3/sec). A powerhouse with an installed capacity of 87.5 MW was built at the toe of the dam. Five low-level outlets, two on the left abutment side and three on the right abutment side, control irrigation water releases. Those outlets are also used for sediment flushing. Two morning glory spillways in the left abutment provide an additional 56,500 cfs (1,600 m3/sec) of outlet capacity.

Sefid Rud Dam is founded on volcanic rocks of the Tertiary Kara Formation. The right half of the dam is founded on competent andesite and andesitic breccia, while the left half was built on breccia and pyroclastic beds of somewhat lesser quality. A thin continuous basaltic sill was encountered during construction at various locations within and across most of the foundation. Buttress foundation areas and abutment surfaces were prepared by contact and consolidation grouting. Two deep upstream and downstream grout curtains were also provided to control seepage and uplift pressures below the dam.

The June 21, 1990, Earthquake

Iran is located along the Mediterranean-Himalayan seismic belt, within the area where the Arabian and European tectonic plates collide. Historically, Iran has been a seismically very active region, where earthquakes of magnitude 6.0 or greater are frequent. The June 21, 1990, earthquake was centered within the Maku Zanjan seismo-tectonic province, at the edge of the Alborz mountains, at latitude 36.96 N and longitude 49.41 E (Figure 6). It devastated the two Iranian provinces of Gilan and Zanjan. The epicenter was about 124 miles (200 km) northwest of Tehran. The main shock was assigned magnitudes that ranged between 7.3 and 7.7. It was immediately followed by two large aftershocks (M 6.2 and M 6.5), and for months by numerous aftershocks, some of up to magnitude 5.9. The event was felt over an area larger than 232,000 square miles (600,000 km2), see Figure 7. Its focal depth was estimated at between 12.5 and 18.8 miles (20 and 30 km).

Primary ground movements were interpreted to have occurred in the north-northwest direction, hence nearly parallel to the dam axis. Several faults, including the Rudbar and Harzevil fault zones, have been related to the occurrence of this earthquake. Immediately west of the dam, about 30 cm of strike-slip displacement and 50 cm of vertical thrust movement were observed (Figure 8), confirming the compressional nature of the tectonic process.

This event caused widespread damage in one of the most agriculturally and industrially developed regions of Iran. The cities of Manjil, Rudbar and Lushan were extensively damaged. Perhaps 100,000 adobe houses collapsed or suffered damage extensive enough to require their demolition. Adobe housing collapse caused most of the casualties. Moderate to major damage occurred to infrastructure and industrial facilities, including highways, tunnels, a large cement plant, a powerplant, and numerous non-structural elements in residential, office and industrial buildings. Immediately downstream from Sefid Rud Dam, the village of Aliabad had many dwellings collapse and suffered 81 deaths among its inhabitants.

Earthquake Effects and Observed Performance

Spectacular rockfalls were observed in the vicinity of the dam, including sliding along natural joints and toppling failures. The access road to the site and a service road between the morning glory spillways and the left abutment were blocked by rock debris. Cracks developed in the left reservoir bank, 3.3 feet (0.7 m) wide and 3.9 feet (1.2 m) deep.

The Manjil earthquake induced cracks at the horizontal lift joints in the upper part of the central buttresses of Sefid Rud Dam (Figure 9). Those joints were located where the downstream slope of the webs experiences a change in slope. All 23 buttresses were cracked. The principal horizontal cracks ran across entire buttresses and caused some leakage along the downstream face of the dam. Except for buttress No. 5, at least one and as many as four major cracks occurred along each of the buttresses. Principal cracks were accompanied by major concrete spalling, and were up to 0.8 inch (2 cm) in width. No damage was reported in the lower part of the webs. Cracks were most frequent between El. 258.25 m and El. 264.25 m, and at the aforementioned change in web slope (El. 262.25 m). At the dam crest (El. 276.25 m), some of the concrete slabs of the roadway cracked and spalled, including longitudinal cracking along the downstream curbstone. The parapet wall at the top of buttress No. 11 failed and was tilted toward downstream. The guard house at the center of the dam crest was completely destroyed.

In the head gallery below the exit point of the drain pipes, considerable amounts of debris piled up from concrete spalling and from calcite deposits dislodged from the drains. At the left abutment, two paved areas settled about 8 inches (20 cm). No damage to the spillways was reported, but rockfalls blocked part of the left spillway chute channel and the morning glory spillway intakes. No damage occurred to hydraulic and electric hoisting equipment. Minor damage to one of two Tainter gates of the intermediate level spillway occurred on the right side, including buckling of the supporting beam, which caused misalignment of the gate and increased leakage from less than 0.7 to about 3.5 cfs (20 to 100 l/sec). Both gates could be operated after the earthquake. However, gates were closed when the event occurred. The powerhouse suffered minor damage, including failure of one of its columns and occurrence of minor concrete cracks. However, the auxiliary building, which housed the control room, was totally damaged with full collapse of internal brick walls. The nearby switchyard suffered heavy damage and oil leaks. Three of four large transformers were displaced from their rail supports by up to 8 inches (20 cm), and many ceramic insulators were sheared off. Many buildings in the vicinity of the dam experienced severe damage, including collapse of numerous adobe houses in the former construction camp area.

Instrumentation and Strong Motion Records

No permanent strong motion instruments had been installed on the dam or in its immediate vicinity. A portable accelerometer mounted on the dam crest a few months before the earthquake was out of order. Peak ground accelerations (PGA) of 0.65g horizontal and 0.52g vertical were recorded at the Abbar station, in the epicentral area and about 25 miles (40 km) away from the dam. PGA at the dam site was estimated at about 0.7g. The city of Manjil was assigned Intensity X on the MSK scale.

The dam was equipped with plumb lines through five of its buttresses, 38 inclinometer stations, and over 100 joint monitoring stations (Figure 2). The latter were installed to measure any relative movements of the vertical joints between buttresses and between buttresses and thrust blocks. Other instrumentation included uplift pressure cells, piezometers, weirs and seepage measuring devices, and concrete temperature monitoring systems. Unfortunately, no topographic survey control stations had been left in place that would have allowed the monitoring of the global position of the dam with respect to the valley walls.

One of the five plumb lines became non-operational as a result of the failure of fasteners holding its protective tubing. Maximum permanent relative horizontal displacement in the upstream downstream direction, measured at the top of the buttresses, was about 0.4 inch (10 mm). Hence, the earthquake caused non-recoverable movements of some buttress blocks, although of very small magnitude. Hysteresis loops of earthquake-induced crest displacements obtained at plumb lines showed a bi-directional amplitude of about 25 mm. Horizontal movements calculated from inclinometer readings were consistent with those measured from the plumb lines. Relative movements at joint level in directions parallel or perpendicular to the contraction joints largely exceeded the range of the measuring instruments at most of the recording stations. In general, each block of the dam moved toward downstream with respect to the block on its left. Cumulative displacements of between 1.8 and 2.8 inches (45 and 70 mm) were hand-measured. It was concluded that either the entire foundation experienced permanent downward movement between the left and right abutments, or that most dam buttresses became slightly tilted toward the left abutment (Figure 10). Uplift water pressures were found to have strongly decreased after the earthquake, perhaps as a result of the closure of joints or of increased compressive forces across seepage paths. This finding was interpreted favorably with respect to the overall safety of the dam.

Emergency Procedures and Post-Earthquake Repairs

The reservoir was six meters below normal operating level at the time of the earthquake. Controlled lowering of the reservoir was immediately initiated, but at a rate such that all of the water released could be used for irrigation, and would not cause downstream flooding of temporary earthquake relief campsites established close to the Rudbar River.

The primary purpose of long-term repair work was to stop leakage through the buttress blocks and restore shear strength in the cracked sections to assure monolithic action of the buttresses. First, all buttresses were water-tested at 200 kPa above hydrostatic pressure to assess the extent of cracking. It was found that about 80 of the cracks required treatment. For each of those cracks, epoxy-grouting (with Rodur), using an average of 20 boreholes per crack, was accomplished. Rodur can bond wet concrete surfaces in cracks and at low temperature. About 92 metric tons of grout were used. In addition to grouting, twelve post-tensioned VSL anchors with 100 MN capacity were installed through each of the buttresses. The average length of those anchors was 131 feet (40 m), with a maximum inclination of 22 degrees with respect to the vertical. Bonded length of the anchors was 39 feet (12 m). Overall, 234 anchor holes with a cumulative length of 31,000 feet (9,450 m) were drilled, and 738 metric-tons of cement and 36 metric-tons of additives were used for tendon grouting. All repair work was completed within eight and a half months.

Conclusions

There are few precedents of concrete dams located close to the epicenter of an earthquake of magnitude near 7.5. Other concrete dams severely shaken by significant earthquakes have included Hsinfengkiang Dam, China (M 6.1), Koyna Dam, India (M 6.5), Ambiesta Dam, Italy (M 6.5), Lower Crystal Springs Dam, California (M 8+) and Pacoima Dam, California (M 6.5 and M 6.6).

The Sefid Rud buttresses had been originally designed using pseudo-static horizontal loading ranging from 0.10 g to 0.25g. A typical buttress was reanalyzed in 1968, using dynamic analysis and a specified peak acceleration of 0.13g, a damping coefficient of 7.5 percent, and a spectral amplification factor of about 3.1 in the range of frequencies significant to the buttresses (2.1 to 3.6 Hz). It was then concluded that strengthening of the dam would not be required. The Manjil earthquake, therefore, induced seismic loads considerably larger than originally anticipated. It is interesting to note, however, that the design of most old buttress dams generally considered only gravity and water pressure loads and offers little capacity to withstand large accelerations in the cross-canyon direction. However, the buttresses of Sefid Rud Dam were built quite thick, and thus were able to resist substantial cross-canyon accelerations without experiencing unacceptable damage.

The Sefid Rud experience is important because it represents another example of a concrete dam exposed to strong earthquake shaking, substantially more severe than its design loads. The dam suffered some damage, but had an overall satisfactory performance, considering that the Manjil Earthquake was probably the equivalent of the Maximum Credible Earthquake considered for this site.

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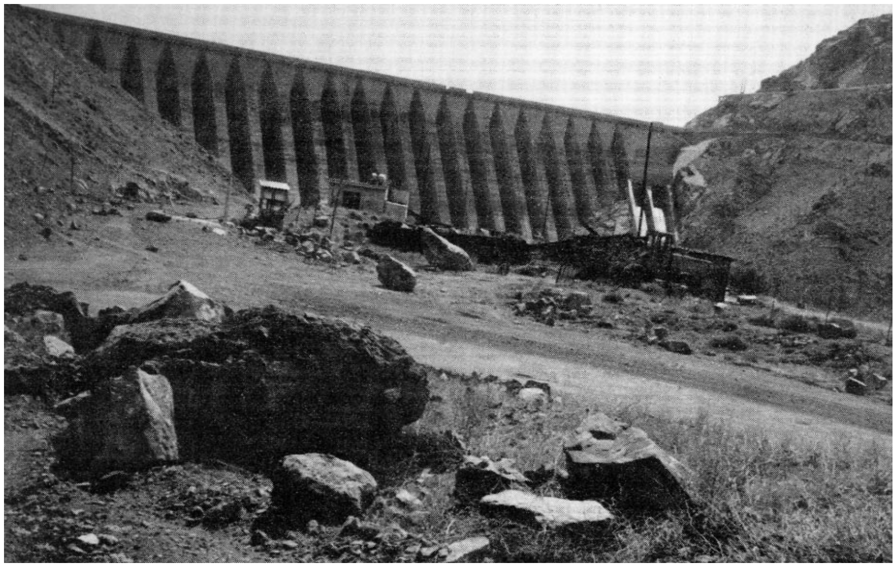

Figure 1
GENERAL VIEW
Sefid Rud Dam

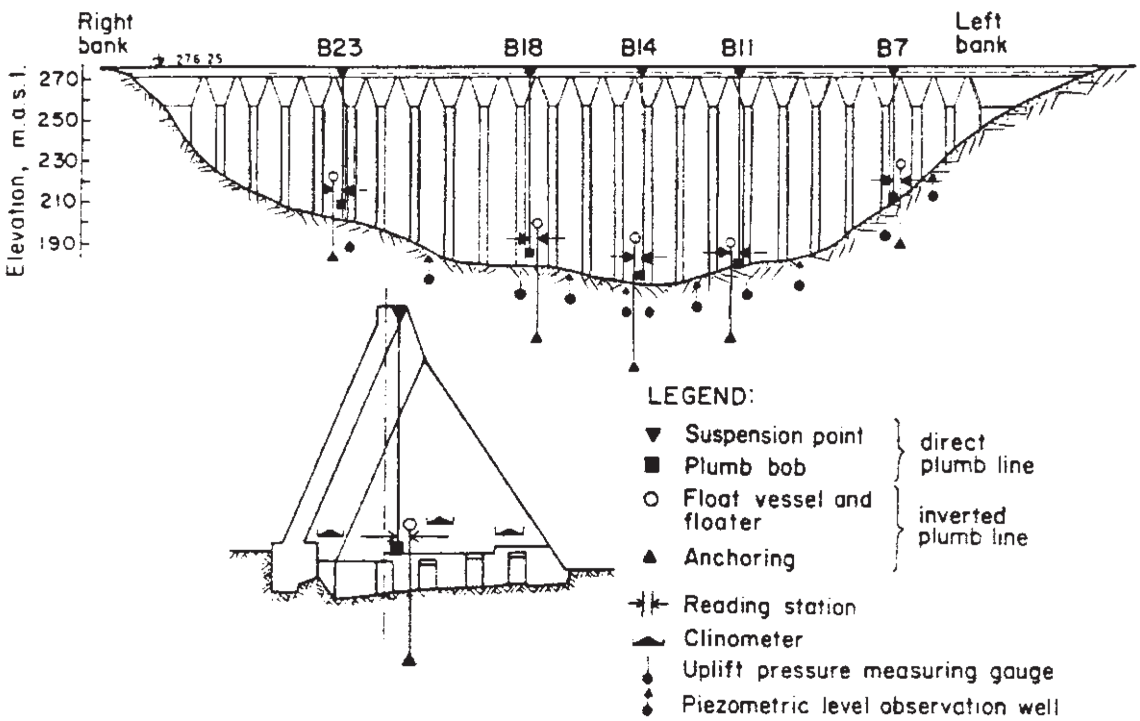

Figure 2
LOCATION OF INSTRUMENTATION
Sefid Rud Dam

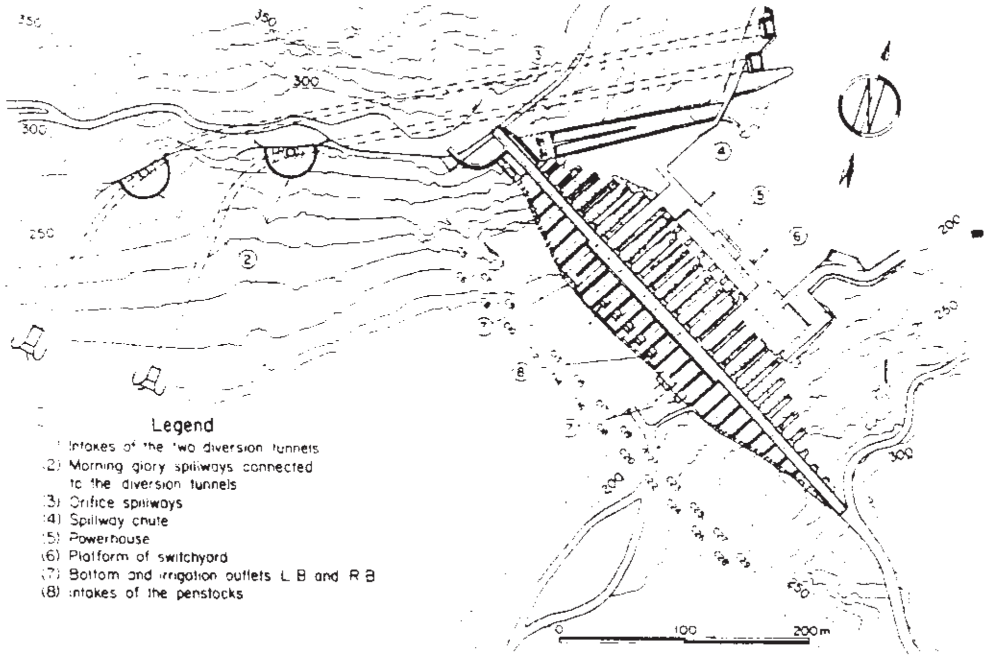

Figure 3
GENERAL LAYOUT
Sefid Rud Dam

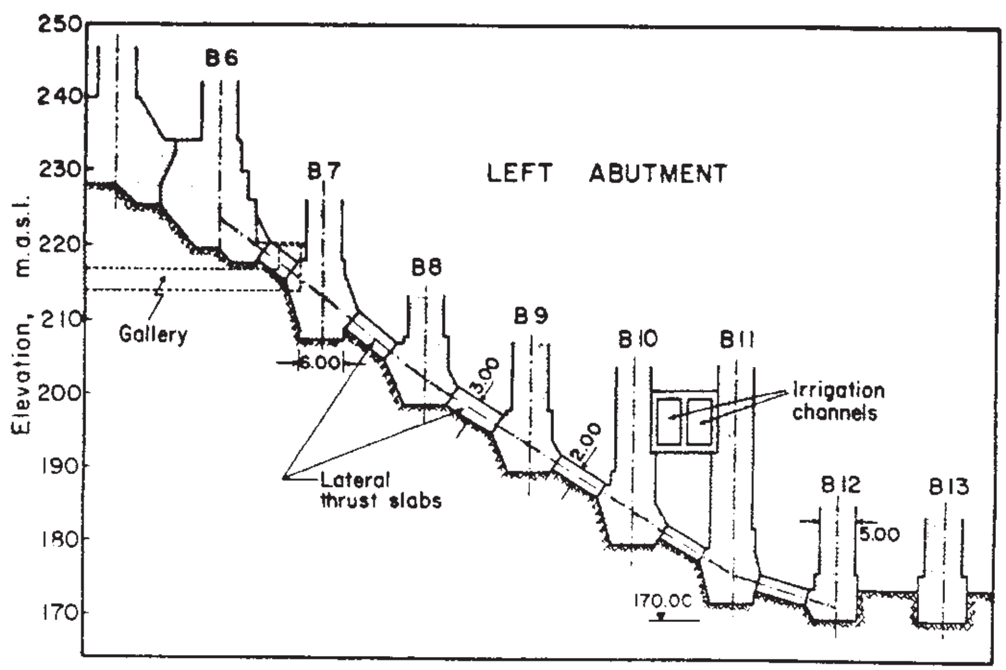

Figure 4
SECTION THROUGH LEFT BANK ALONG AXIS
Sefid Rud Dam

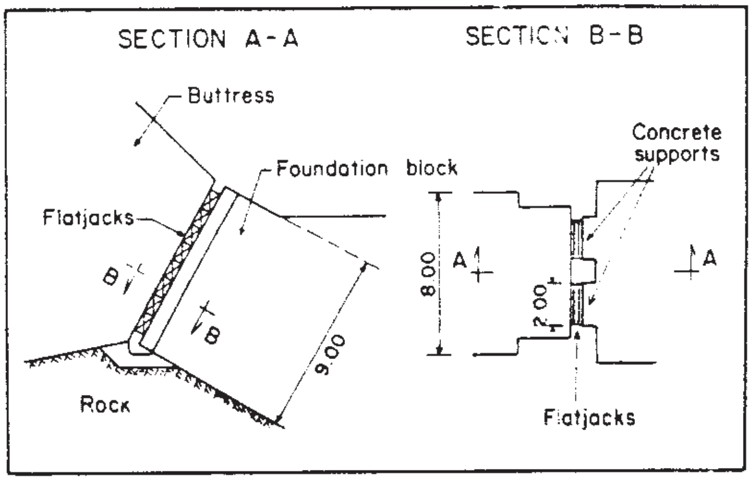

Figure 5
DETAILS OF AN ACTIVE CONSTRUCTION JOINT
Sefid Rud Dam

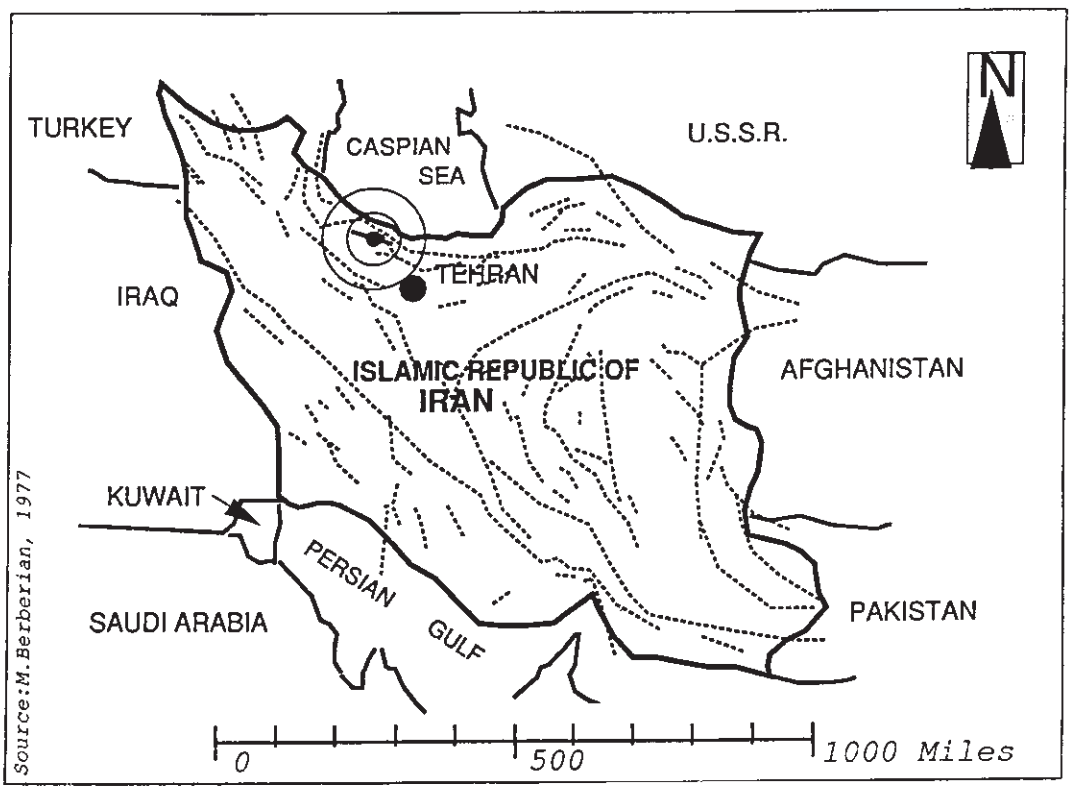

Figure 6
TECTONICS OF IRAN AND JUNE 21, 1990 EPICENTER
Sefid Rud Dam

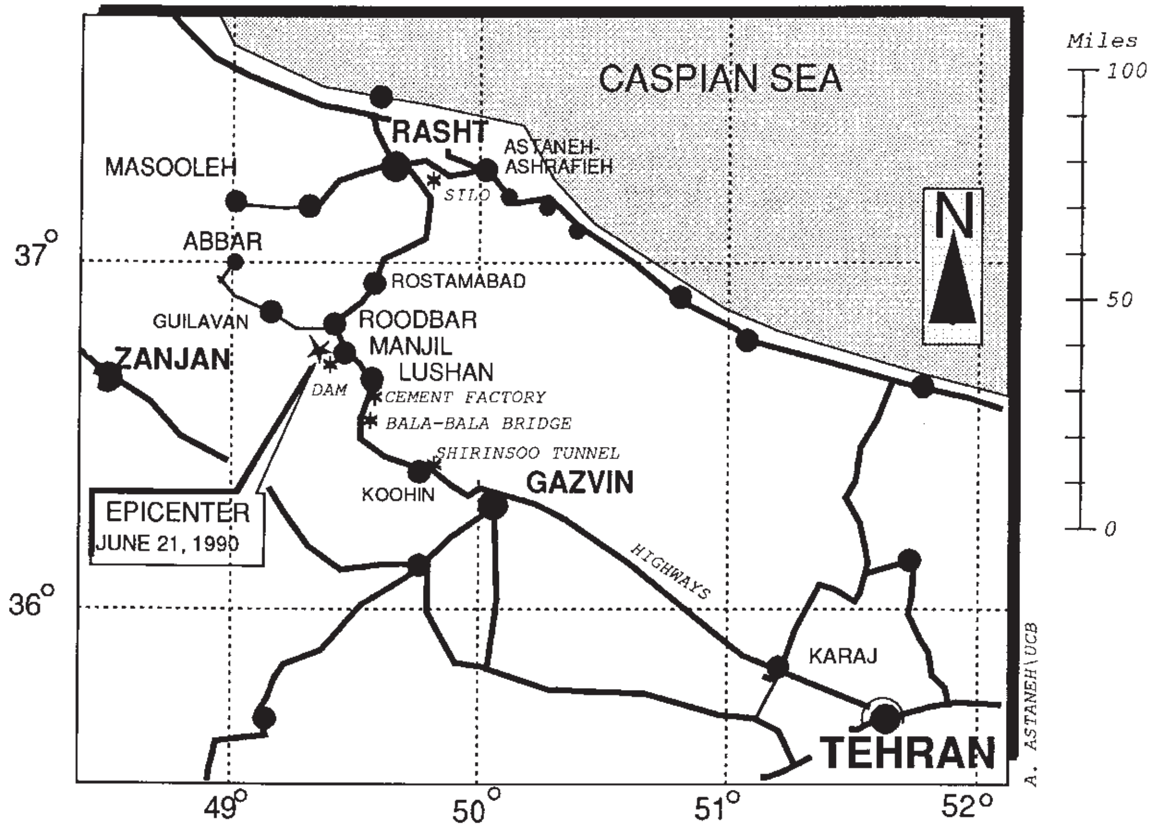

Figure 7
AFFECTED AREAS NEAR EPICENTER
Sefid Rud Dam

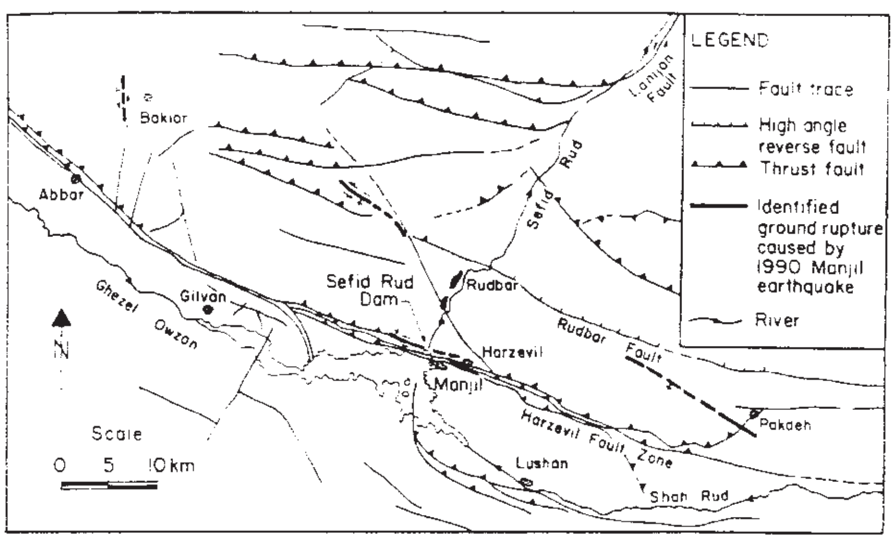

Figure 8
IDENTIFIED FAULT RUPTURE ZONES
Sefid Rud Dam

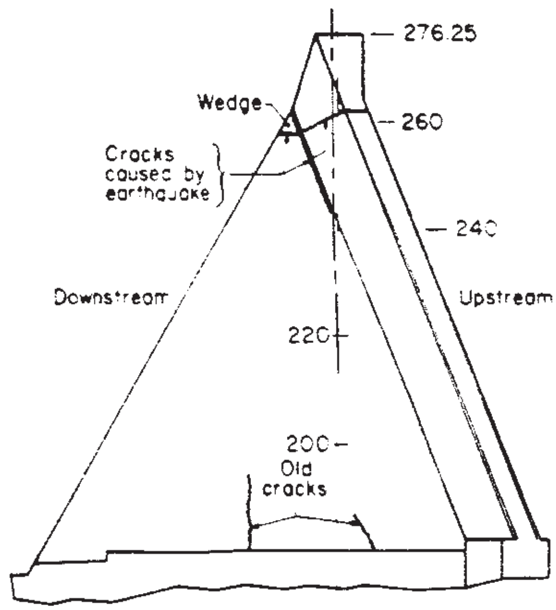

Figure 9
TYPICAL CRACK MAPPING
(Left abutment side, buttress 15)
Sefid Rud Dam

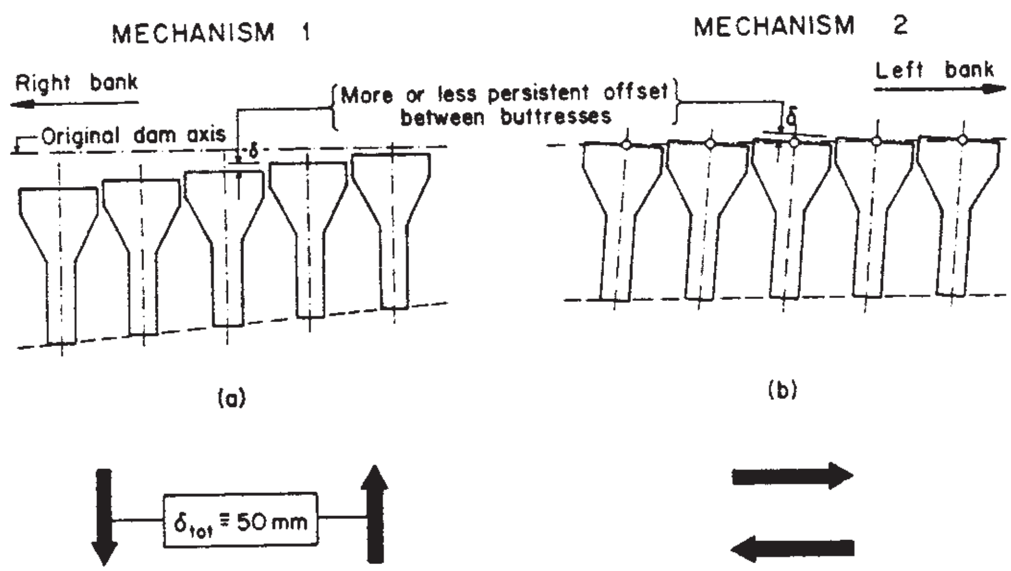

Figure 10
INTERPRETATIONS OF BUTTRESS MOVEMENT
Sefid Rud Dam

SHEFFIELD DAM, CALIFORNIA, USA

On June 29, 1925, a magnitude 6.3 earthquake occurred in the vicinity of the City of Santa Barbara, California. The earthquake resulted in 12 deaths and substantial damage. The earthquake was felt over an area of at least 50,000 square miles. The epicenter was located about seven miles northwest of Sheffield dam, a 720-foot-long embankment, 25 feet high. The dam, which was composed of silty sand and sandy silt, failed during the earthquake. The failure released about 30 million gallons of water, which temporarily flooded the lower part of the city to a depth of about one or two feet before discharging into the sea. Peak acceleration at the site was estimated in 1968 at 0.15g, but was probably at least 0.25g, based on estimates provided by more recent attenuation relationships.

Sheffield Dam

Sheffield Dam was built in 1917 in a ravine north of the City of Santa Barbara. A representative section through the embankment at its maximum height is shown in Figure 1 (Seed et al., 1968). The 720-foot-long embankment had a maximum height of 25 feet. The body of the dam was composed of silty sand and sandy silt, compacted by routing the construction equipment over the fill. The upstream slope was faced with a 4-foot-thick clay blanket, which was extended 10 feet into the foundation to serve as a cutoff. The clay blanket was overlain with a 5-inch concrete facing.

The foundation soils consisted of terrace alluvium, 4 to 10 feet thick, overlying sandstone bedrock. The alluvium was mainly silty sand and sandy silt containing cobbles varying from 3 to 6 inches in diameter, with some thin layers of clayey sand and gravelly sandy clay. It was reported that the upper 1 to 1 1/2 feet of foundation topsoil were somewhat looser than the underlying deposits and that there had been no formal stripping of the upper soil layers prior to construction of the embankment (Seed, et al., 1968; U.S. Army Corps of Engineers, 1949).

Seed et al. (1968) reported that seepage had been noted near the downstream slope and in the area beyond the toe before the earthquake. Seepage around and beneath the cutoff was reported to have resulted in saturating the lower part of the embankment and the foundation (Willis, 1925). At the time of the earthquake, the depth of water in the reservoir was about 15 to 18 ft.

The June 29, 1925, Santa Barbara Earthquake

The main shock of this earthquake occurred at 6:42 a.m. in the morning of June 29, 1925. There were no strong motion instruments in existence at the time but on the basis of records obtained at distant stations, the earthquake has been assigned a magnitude rating of 6.3 with an epicenter located some seven miles northwest of the dam site (Eppley, 1960).

Early reports attributed the earthquake to movement along one of the many faults in the vicinity of Santa Barbara, some of which are quite close to the dam site. However there was no evidence of horizontal or vertical displacement of the ground surface during the earthquake (Eppley, 1960), and a review of more recent studies failed to confirm the existence of a known active fault in the area that could have formed the source of the energy release (Seed et al, 1968). The intensity of ground shaking in and around Santa Barbara was estimated in the usual manner, based on observed damage. Willis (1925) inspected the City, and assigned a maximum intensity of X on the Rossi-Forell scale. By his count the principal vibrations of the earthquakes lasted 15 seconds. Byerley (1955) made an inspection trip through the entire area affected by the earthquake and assigned a Rossi-Forell intensity to each town which he visited. From these data the intensity at the dam site was interpolated to be between Rossi-Forell VIII and IX (Seed et al., 1968).

Earthquake Effects and Observed Performance

The Sheffield Reservoir formed by the dam was about 800 square feet and was capable of impounding a maximum of about 45 million gallons of water. At the time of the earthquake, the depth of water in the reservoir was only about 15 to 18 feet, so that the failure released about 30 million gallons of water which temporarily flooded the lower parts of the city to a depth of about 1 or 2 feet before discharging into the sea.

There were no eye witnesses when the failure occurred. However, after inspecting the damage, O'Shaughnessy (1925) reported that "a great mass of the center, about 300 ft in length, slid downstream perhaps 100 ft." Herbert Nunn (1925), City Manager of the City of Santa Barbara, wrote: "After examination by several prominent engineers, the conclusion has been reached that the base of the dam had become saturated, and that the shock of the earthquake....had opened vertical fissures from base to top; the water rushing through these fissures simply floated the dam out in sections." Willis (1925) reported: "The foundations of the dam had become saturated and the rise of the water as the ground was shaken formed a liquid layer of sand under the dam, on which it floated out, swinging about as if on a hinge."

From these accounts, Seed et al (1968) concluded that sliding occurred on a surface near the base of the embankment, causing a large portion of the dam to move a considerable distance downstream, breaking up as it did so to give the general appearance shown in Figure 2. This sliding was related in some manner to a severe reduction in soil strength resulting from increases in pore-water pressure induced by the earthquake shaking.

Instrumentation and Strong Motion Records

Sheffield Dam was not instrumented at the time of occurrence of the earthquake. In their 1969 reanalysis of the dam failure, Seed, Lee and Idriss used empirical correlations between peak ground accelerations and Rossi-

Forell intensity to estimate the peak ground acceleration (PGA) in the vicinity of the Sheffield Dam and assigned it a value of 0.15g. They estimated the duration of significant shaking at between 15 and 18 seconds. In view of more recent knowledge, modern estimates of mean PGA for a site located only 7 miles (11.2 km) from the epicenter of a magnitude 6.3 earthquake would be of the order of 0.25g, based on a weighted average of five recently published well accepted PGA attenuation equations. Actual PGA may have been higher, since 0.25g represents a mean estimate.

Conclusions

Sheffield Dam, a 25-foot-high compacted silty sand and sandy silt embankment built on a similar foundation, was shaken by a magnitude 6.3 earthquake and failed completely. This is one of the rare known cases of complete failure of a dam as a result of earthquake loading.

The failure was due to liquefaction of the saturated silty sandy soils at the base of the embankment and the upper part of the foundation. Detailed dynamic finite element analyses (Seed, et al., 1968), using the results of laboratory cyclic strength tests on the embankment and foundation materials, provided conclusions that were in reasonable accord with the observed performance. That study of the Sheffield Dam failure was perhaps the first application of dynamic finite element analysis to investigate the response and behavior and embankments dams, and led to the development of procedures and evaluation methods that have been used extensively in the following 25 years and are still in use today.

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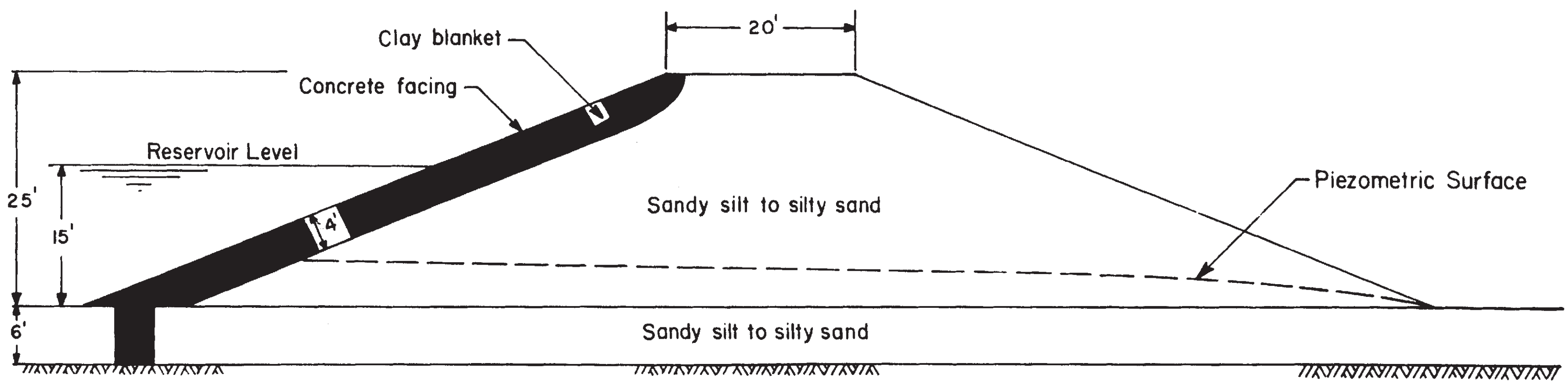

Figure 1
CROSS-SECTION THROUGH EMBANKMENT
Shefield Dam
From Seed et al, 1968

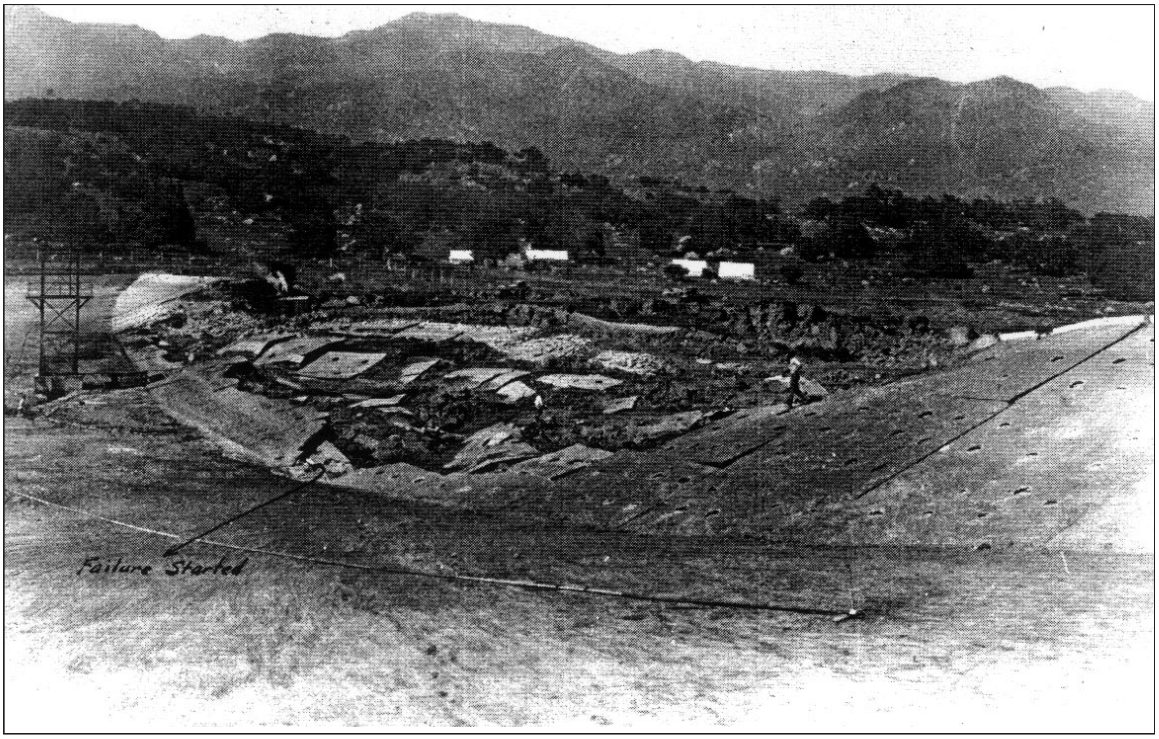

Figure 2
SHEFFIELD DAM AFTER FAILURE
Sheffield Dam

VERMILION DAM, CALIFORNIA, USA

Between May 25 and May 27, 1980, a swarm of substantial seismic events, totaling over ten individual earthquakes, occurred on known active faults in a relatively small area of about 20 km by 10 km at the eastern toe of the Sierra Nevada of California, about 50 km northwest of Bishop, California, and at a relatively short distance from Vermilion Dam. The events of significance to the dam ranged in magnitude from M 5.8 to M 6.4. The nearest major epicenter to Vermilion Dam was that of the M 6.3 event on May 27, at an epicentral distance of about 22 km. The peak ground acceleration recorded at foundation level at the dam was 0.24 g.

No visible damage resulted at this modern, well-compacted, earthfill dam constructed on top of up to 270 feet of coarse, dense alluvium that had been deposited by several advances and retreats of the Mono Creek Glacier during Pleistocene times. Repeated surveys of benchmarks, however, showed that settlements had occurred, the maximum crest settlement having been about 0.17 ft. No appreciable increase in seepage was reported.

Vermilion Dam

Vermilion Dam (Lake Thomas A. Edison) is located on Mono Creek, a tributary of the South Fork of the San Joaquin River, on the western slope of the Sierra Nevada in Fresno County, California (Figure 1). Vermilion Dam is a 165-foot-high, zoned, compacted, sandy earthfill embankment, 4,234 feet long (Figure 2), owned and operated by Southern California Edison Company (Edison). Its slopes (Figure 3) are 2.25 to 3:1 (horizontal to vertical) upstream and 2.0 to 2.5:1 (h to v) downstream. Its crest is at an elevation of 7,650.5 ft above sea level.

Lake Thomas A. Edison provides storage of about 125,000 acre-feet. Final design of Vermilion Dam was prepared in 1952 and construction was completed in 1954. A detailed paper describing the unusual and complex foundation conditions was prepared by Terzaghi & Leps (1960). The foundation of the dam is its most interesting feature. It was reported as consisting of highly varied layers and lenses of fluvial and glacio-fluvial silts, sands, gravels and boulders, all of which have been formed, reworked, and consolidated by several advances and retreats of the Mono Creek Glacier. The sediments are from 100 to 270 feet thick. They are underlain by granodiorite of the Sierra Nevada batholith.

Because of the glacial preloading of the foundation and the generally coarse, granular texture of the thick glacio-fluvial deposits, there was assurance regarding the structural competence of the foundation soils. The problem in relation to creating a safe dam on the site had been to minimize and control the exit of the probable foundation seepage. It is apparent from over 40 years of operating experience that underseepage has been adequately controlled. The maximum seepage flow has not exceeded about 6 cfs and has remained stable. This was achieved both by constructing an extensive, impervious, rolled fill blanket from the core of the dam upstream along the reservoir bottom for distances of up to 1,400 ft, together with upstream cutoff trenches to a shallow but discontinuous impervious stratum of varved silt, and by generous provision of a deep, filtered, toe drain, together with gravity-discharge, relief wells.

The dam has a small, gated spillway on its left abutment, an ungated, auxiliary spillway on its right abutment, and an outlet works through the base of the maximum section formed by a reinforced concrete, well articulated, cut-and-cover conduit, gated at both ends. Each of these facilities is founded on glacio-fluvial soil.

The dam is extensively instrumented, with seepage weirs, piezometers and benchmarks, one strong motion accelerometer (SMA-2) operated by the owner, plus five strong motion instruments operated by the California Strong Motion Instrument Program (SMIP) of the California Division of Mines and Geology (CDMG).

Seismicity

The California Division of Mines and Geology (1991) has reported that, since 1978, the Bishop-Mono Lake area has been one of the most seismically active regions in California, with local magnitudes ranging as high as 6.5. A map of pertinent regional faults is shown in Figure 4. Faults of primary capability with regard to Vermilion Dam are in a zone located 20 to 40 km to the east and northeast of the dam.

The May 27, 1980, Earthquake

Of the many strong events experienced in the period May 25 through May 27, the May 27 event (M 6.3), which occurred at 7:51 a.m., caused the strongest shaking at Vermilion Dam, with a peak ground acceleration of 0.24 g recorded just downstream from the toe of the dam. There was no visible damage to the dam and its auxiliary features.

Earthquake Effects and Observed Performance

For the broad area along State Route 395 east of the Sierra, extensive reports are available, detailing surface rupture, rockfalls, slumps, and building damage, the latter mostly in the area of the City of Mammoth Lake. West of the Sierra crest, no important damage was reported, and only the Vermilion SM A-2 provided a significant source of data. The area is very lightly populated.

It was of some interest that an employee of the Edison Company who had been standing on the left abutment of Vermilion Dam at lake level during the May 27 event, declared in a written statement that the crest of the Dam was "... moving back and forth as much as two or three feet ... and ... was moving in a vertical motion ... two or three feet." He reported the duration of motion to be 15 to 20 seconds, a somewhat more credible statement.

While a subsequent examination of the dam failed to find any visible damage, a resurvey of monuments on the surface of the dam was carried out and carefully reviewed. The survey data indicated that the maximum settlement, occurring at the maximum dam section, was about 0.17 ft. Figure 6 illustrates the chronology of maximum crest settlement since construction of the dam was completed in 1954.

Instrumentation and Strong Motion Records

At the time, there was a minimal amount of seismic instrumentation on the westerly slope of the Sierra Nevada, except at Vermilion Dam. The greatest concentration of such instrumentation had been placed east of the Sierra, in the known, seismically active area between Bishop and Mono Lake, California. At Vermilion Dam, in addition to Edison's SMA-2, there was an array of strong motion instruments which had been placed by the California Division of Mines and Geology (CDMG) as part of the SMIP. The array was located on the dam crest, on berms and at the toe. Unfortunately, the SMIP array malfunctioned in May 1980, and no data were recorded. On the east side of the Sierra, however, many records were obtained. They were published by the CDMG (1980). Material regarding peak recorded acceleration attenuation at various distances from the 1980 epicenters is displayed in Figures 4 and 5, up to epicentral distances of 30 km. The available data appear to check reasonably well with published attenuation equations, such as the Leps-Jansen chart (1984).

Conclusions

From a dam safety standpoint, particularly with regard to the maintenance of adequate freeboard at embankment dams after a major seismic event, the indication at Vermilion Dam was that a properly compacted embankment dam on a dense foundation will not experience major crest settlement as a result of significant seismic shaking. Furthermore, an indication from the May 1980 swarm of events that was particularly valued was the confirmation that the deep deposits of glacio-fluvial sediments under the Dam were, indeed, as heavily pre-consolidated by glacial loading as had been estimated prior to construction by Edison's engineering geology consultants. Such deposits proved to be relatively incompressible.

Acknowledgments

The cooperation of Robert A. Burks and Thomas A. Kelly of Edison in obtaining and providing the basic information presented above is especially appreciated.

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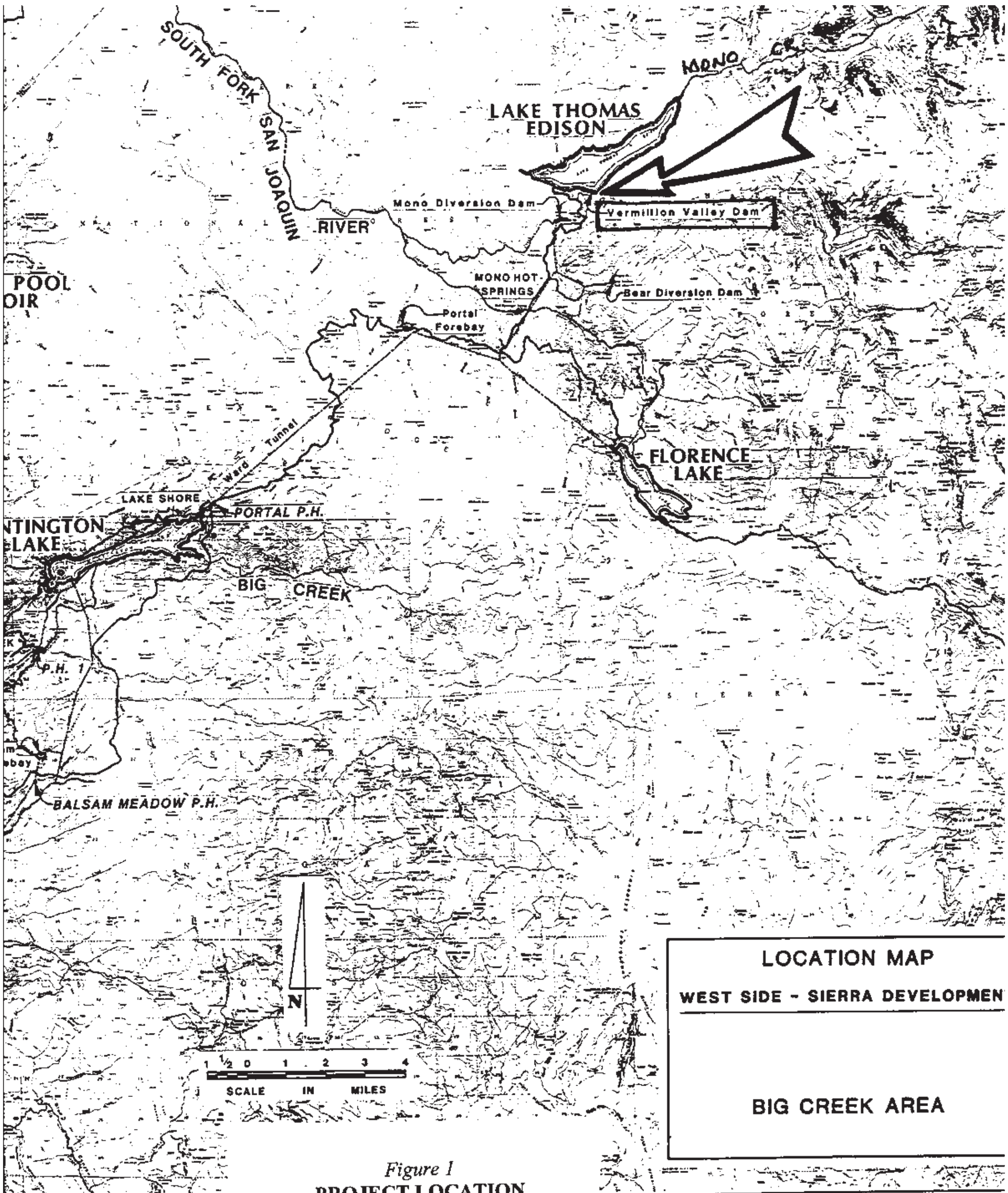
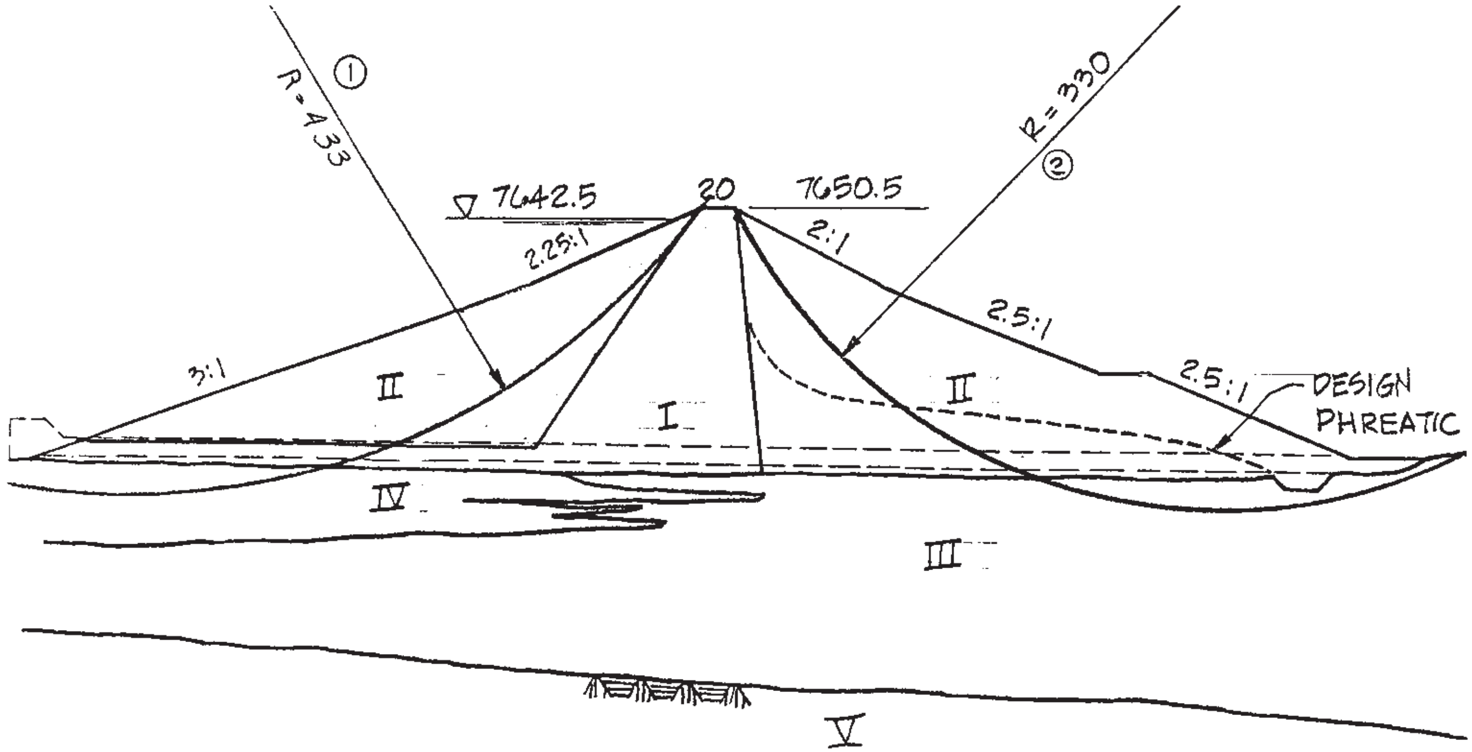


Figure 1
PROJECT LOCATION
Vermilion Dam

Figure 2
VERMILION DAM



MATERIAL PROPERTIES

ZONE	DESCRIPTION	γ (dry) pcf	C_{psf}	$\phi°$	
I	SILTY SAND	120	1200	37	(a)
II	GRAV. SAND	115	0	37	(a)
III	CSE. GLAC. DEPOSIT	127	0	45	(b)
IV	VARVED SILT	107	0	37	(a)
V	BEDROCK	150	4×10^5	—	(b)

(a) FROM TESTS
(b) ESTIMATED

PRINCIPAL RESULTS (RESERVOIR FULL)

CIRCLE	SAF. FACT. (STAT.)	SAF. FACT. a=0.10g	SAF. FACT. a=0.17g
1	2.5	1.9	1.6
2	2.4	1.8	1.6

Figure 3
CROSS-SECTION
Vermilion Dam

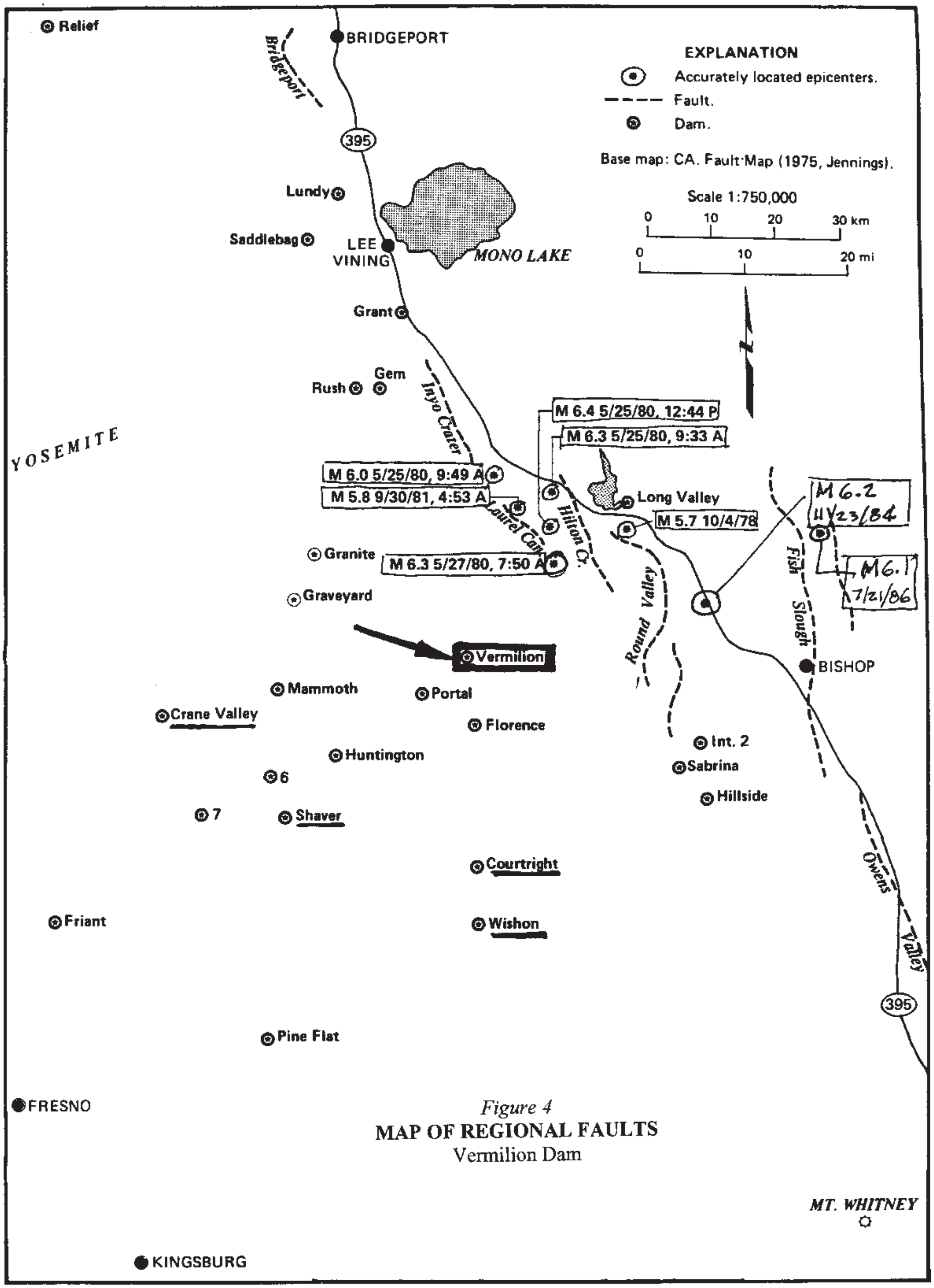
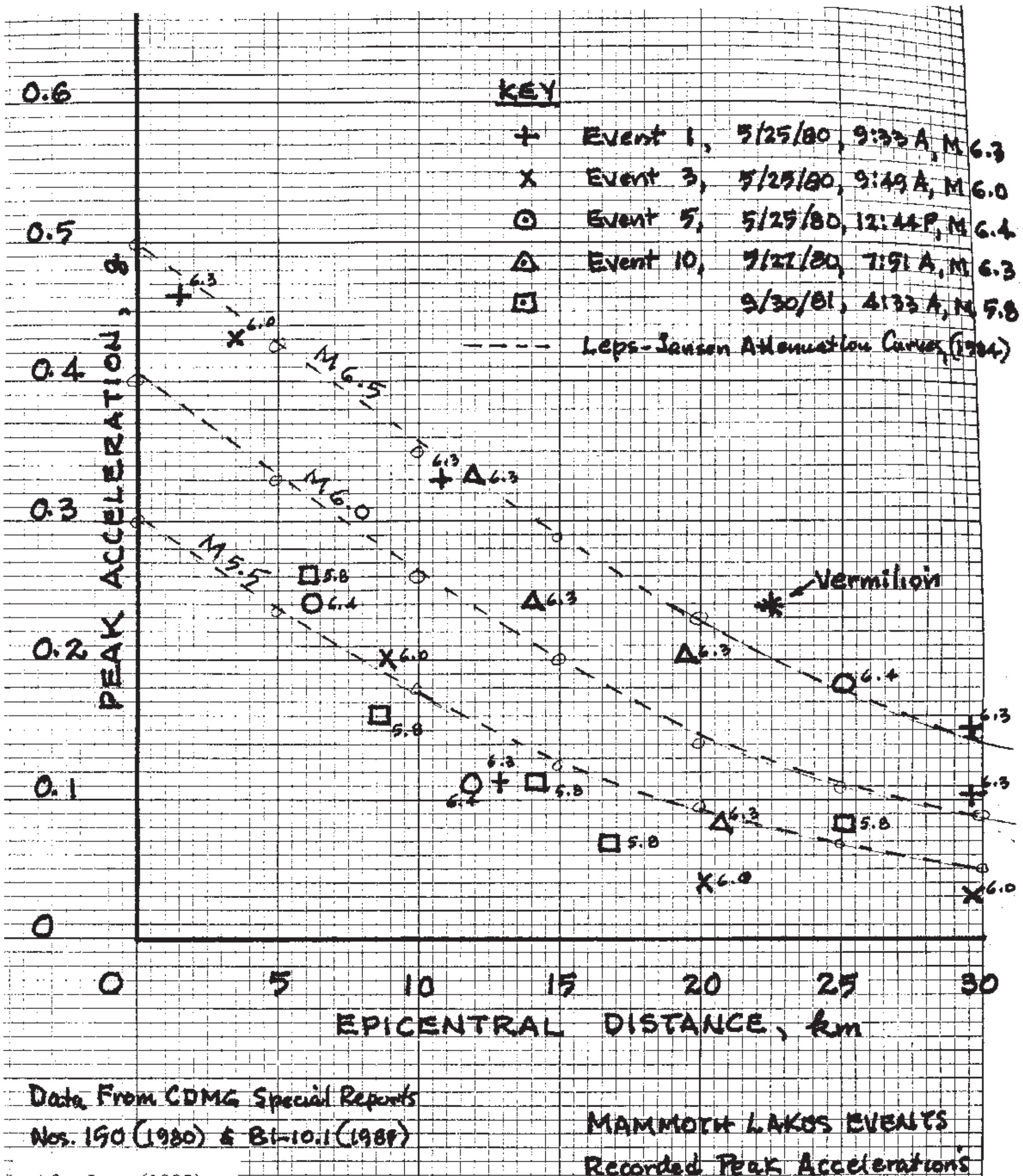


Figure 4
MAP OF REGIONAL FAULTS
Vermilion Dam



After Leps (1992)

Figure 5
ESTIMATED PGA DATA
Vermilion Dam

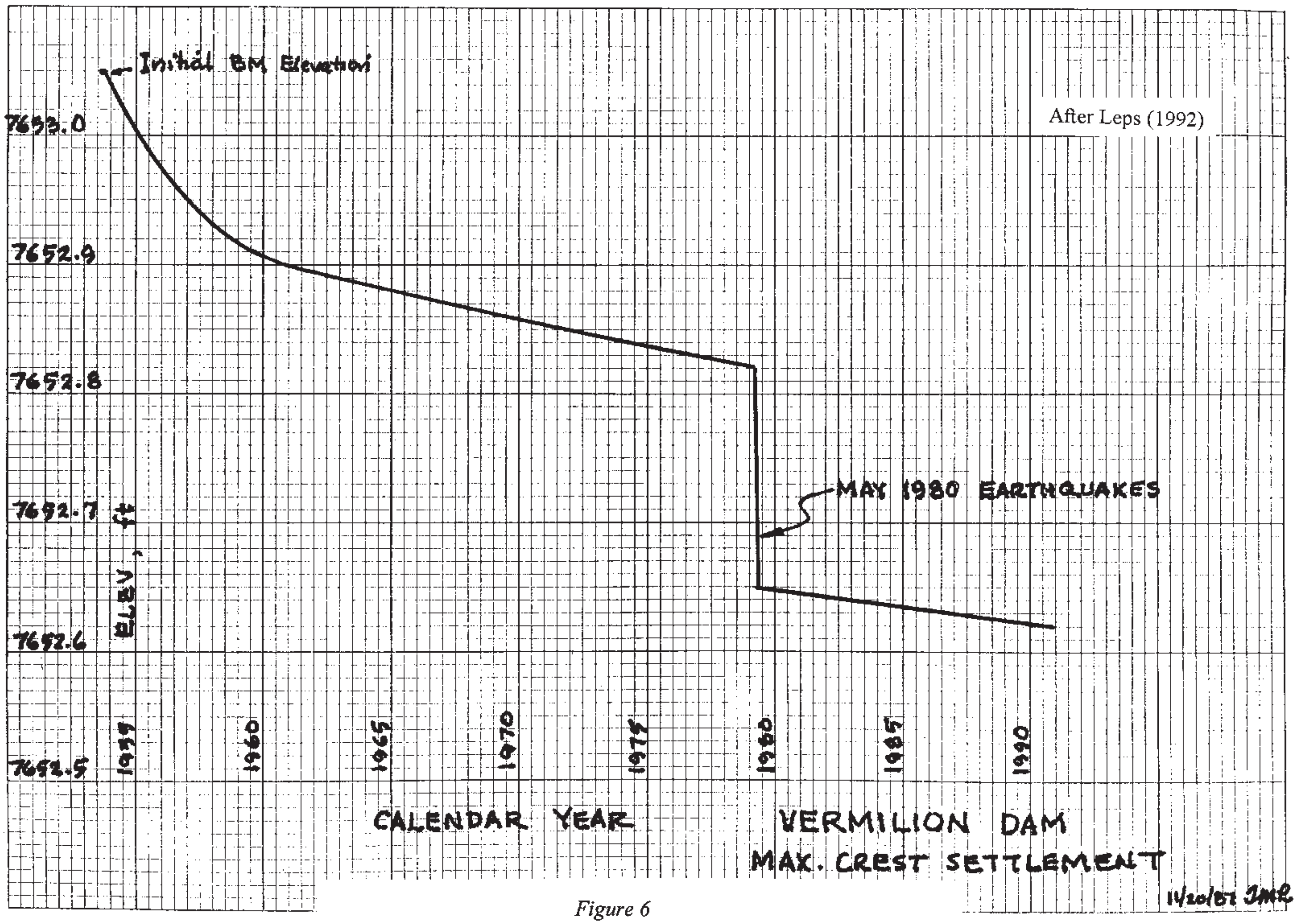


Figure 6
CHRONOLOGICAL PLOT OF CREST SETTLEMENT
Vermilion Dam