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FOREWORD

This publication was prepared by the USCOLD Committee on Earthquakes, chaired by Joseph L. Ehasz. Gilles Bureau wrote the introductory section and coordinated the Committee's efforts in preparing this report. Principal contributors to dam write-ups were: Donald H. Babbitt, Gilles Bureau, Gonzalo V. Castro, Anil K. Chopra, Joseph L. Ehasz, Richard L. Kramer, C. Eric Lindvall, Robert B. McDonald, and Ram P. Sharma. The other members of the Committee reviewed the final manuscript and made numerous helpful comments and contributions. Dames & Moore, Oakland, California, provided word processing and technical illustration support.

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OBSERVED PERFORMANCE OF DAMS DURING EARTHQUAKES

Historically, few dams have been significantly damaged by earthquakes. On a world-wide 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 small earthfill embankments of older, and perhaps, inadequate design. About half a dozen embankment or concrete gravity dams of significant size have been severely damaged. Several of these experienced near total failure and were replaced. In the United States alone, over 5,200 dams are higher than 50 feet; over 740 exceed 100 feet and over 300 exceed 200 feet (USCOLD, 1982). Hence, if one considers the total number of existing large dams 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 local duration and intensity sufficient to jeopardize their structural integrity. Except for several well-known examples, most existing dams have not been tested by levels of ground motion equivalent to the Design Basis Earthquake (DBE, USCOLD, 1985). Conversely, a few dams have experienced significant damage under shaking less demanding than what had 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 very technical and not easily accessible to dam owners or the general public. This has created a need for this publication, which provides a brief overview of the seismic performance of dams of all types.

First, an inventory is presented of the principal dams that have experienced significant earthquake shaking. This information is summarized in Table 1 and includes, where available, principal earthquake parameters, dimensions and dam types, epicentral distances, and crude indicators of the severity of the damage incurred, if any has been reported. Next, eleven case histories have been selected for more detailed coverage. These examples were chosen based on 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 reliable strong-motion records near or on the dams, 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 performances were observed.

At this time, it is not possible to include all of the dams that could justifiably be included in this publication. The USCOLD Committee on Earthquakes anticipates that a sequel will include other case histories of interest to the Profession.

PERFORMANCE OF EMBANKMENT DAMS

The October 17, 1989 Loma Prieta, CA Earthquake (M[1] = 7.1) affected a wide region of the San Francisco Bay Area and induced strong shaking in about a dozen embankment dams located within the epicentral area. Over 100 dams of various sizes, most of them embankment dams, were located within 100 km from the epicenter. This recent event once more documented the ability of well-designed embankment dams to safely withstand severe ground motion. 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 must withstand during their expected life earthquakes of higher intensities and longer durations than were experienced during the October 17, 1989 event. The strong phase of shaking during that earthquake

[1] M = Richter magnitude

lasted less than eight seconds at rock and firm soil sites in the epicentral area, a relatively short duration for a magnitude greater than 7.0. Also, at the time of the earthquake, most of the reservoirs were at between 10 and 50 percent of their maximum capacity, due to several consecutive years of low precipitation. Hence, the drought may have been a beneficial factor for the seismic resistance of earthfill dams in which phreatic surfaces were 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 and similarly to what had generally been predicted in prior evaluation studies.

Austrian Dam, a 200 foot high earthfill dam located about 12 km from the Loma Prieta epicenter, with a reservoir water level only at mid-height at the time of the earthquake, experienced substantial abutment cracking and a maximum crest settlement of nearly three feet. The non-recoverable earthquake-induced deformations of Austrian Dam remained well below the 10 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. But the 1989 earthquake was considerably less demanding than a local DBE in overall duration and seismic energy content. The observed settlements of this gravelly clayey sand embankment might not have been predicted under loading conditions similar to those which occurred in October 1989, based on some of the frequently used numerical methods of dam safety evaluation. While Austrian Dam was safe, this experience reminded us of the constant need to learn from actual performance of dams, so that seismic safety can be improved.

Prior to the Loma Prieta earthquake, 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, CA (1906); Kanto, Japan (1923); Kern County, CA (1952); Hebgen Lake, MT (1959); Tokachi-Oki, Japan (1968); San Fernando, CA (1971); Chile (1971, 1985); Mexico (1979, 1981, 1985) and Edgecumbe, New Zealand (1987) earthquakes.

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 and that their performance has been closely related to the nature of the materials used for construction. While most well-built earthfill dams are believed to be capable of withstanding substantial earthquake shaking with no detrimental effects, those 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 inadequately compacted sands or silts, and tailings dams represent nearly all the known cases of failures, primarily as a result of the liquefaction of these materials. Therefore, hydraulic fill dams, a type of construction virtually abandoned, and tailings dams represent the most hazardous types of embankment dams. Conversely, rockfill dams or concrete face rockfill dams (CFRD) are generally considered to be inherently stable under extreme earthquake loading, and represent desirable types of dams in highly seismic areas.

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 5 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 degrees 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. Ono Dam settled nearly one foot with longitudinal cracks 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 dams in southern California during the 1952 Kern County 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, CA earthquake (M 6.5) 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. A major catastrophe was narrowly avoided in a highly developed urban area. 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 of an area involving over 70,000 downstream residents was barely avoided, and 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 the potential vulnerability of embankments constructed of poorly compacted saturated fine sands and silts to the attention of engineers and public agencies involved in dam safety. It also triggered numerous state-mandated re-assessments of dam safety and led to significant advances in the development of numerical methods of dynamic analysis of dams.

Another recent event of interest is the 1985 Mexico earthquake (M 8.1), that involved two large earth-rock and rockfill dams, La Villita (197 foot high) and El Infiernillo (485 foot high). While neither of these dams experienced significant damage during the 1985 earthquake, they have been shaken since 1975 by a unique sequence of closely spaced events, five of which being larger than magnitude 7.2. Cumulative earthquake-induced settlements of La Villita Dam, an earth-rockfill embankment with a wide central impervious clay core, now approach one percent of its original height. La Villita Dam's settlements have shown a tendency to increase in magnitude with more recent events, perhaps due to some progressive weakening of part of the embankment materials. Similar performance has not been observed at El Infiernillo Dam, an earth core rockfill dam (ECRD), the deformations of which have remained small and consistent from one event to the next.

Lastly, recent events of moderate magnitude, such as the 1987 Edgecumbe earthquake (M 6.2) in New Zealand, which damaged the 259 foot high Matahina Dam, or the 1987 Whittier Narrows, CA 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 high technical quality of performance data and strong-motion records collected as a result of these events.

PERFORMANCE OF CONCRETE DAMS

No concrete dam is known to have failed as the result of an earthquake. Perhaps one hundred or more concrete dams have been shaken by earthquakes susceptible of being felt at the damsites, but only about a dozen have experienced peak accelerations recorded or estimated at 0.20g or greater. These 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 experienced substantial ground motions. During the 1971 San Fernando, CA, earthquake (M 6.5), the 372 foot high Pacoima Dam was subjected to estimated base accelerations of a maximum of about 0.70g; an unprecedented peak acceleration of 1.25g was recorded on rock at the left abutment, slightly above the dam crest; however, this large acceleration is 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 that earthquake, except for partial opening of the joint between the arch and the left abutment block.

Ambiesta Dam, a 194 foot high arch in Italy, 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, 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, CA; Barcis and Maina diSauris, in Italy; Kariba, in Zambia; Monteynard and Granval, in France; and Kurobe, in Japan, were located 50 km or less from epicenters of various events of magnitudes between 4.9 and 6.6, but were undamaged. However, the local intensities of shaking at those sites were probably moderate.

Concrete gravity and buttress dams have been, to date, affected more severely by earthquakes than have arch dams. 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 reported to have been damaged by an earthquake (1957). The event, rated at 8 on the British Intensity scale of a maximum of 10, was estimated to be centered about 6.4 km from the dam site. It resulted in cracking of 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, in India, a 338 foot high straight gravity dam, and Hsinfengkiang Dam, in China, a 344 foot high buttress dam, were shaken as the result of nearby earthquakes of magnitudes 6.5 (1967) and 6.1 (1962), respectively. Both of these dams developed substantial longitudinal cracking near the top. Damage was attributed to design or construction details that would now be avoided in modern structures. The two dams were repaired and are still in service.

Lower Crystal Springs Dam, a 127 foot high curved concrete gravity dam built of interlocking concrete blocks, withstood the 1906 San Francisco earthquake (M 8.3, estimated) without a single crack. The primary fault rupture was located less than 600 feet from the dam; a right-lateral slip of about ten feet was measured nearby. Lower Crystal Springs Dam was shaken again, by moderate motion, during the 1989 Loma Prieta earthquake, but was once more 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 bout 60 km away from the epicenter of the 1977 Romanian earthquake (M 7.2), but performed satisfactorily.

SELECTED CASE HISTORIES

The following case histories of dam performance during earthquakes have been selected for detailed coverage in this publication:

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

TABLE 1
HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES

AME	LOCATION	ID	HT. [ft]	EQK NAME	DATE	M	DIST [km]	DAMAGE	Principal References Consulted
UGUSTA	GA., USA	E	---	Charleston	31-Aug-1886	7.0	180	Collapse	Duke, C.M. (1960)
. 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)
. 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)
AKE RANCH	CA., USA	E	38	San Francisco	19-Apr-1906	8.3	0.1	None	Ambraseys (1960)
EAR GULCH	CA., USA	E	45	San Francisco	19-Apr-1906	8.3	3.2	None	Ambraseys (1960)
ILARCITOS	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)
. HOWELL	CA., USA	E	38	San Francisco	19-Apr-1906	8.3	0.2	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. HOWELL	CA., USA	E	36	San Francisco	19-Apr-1906	8.3	0.2	Moderate	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
AKE RANCH	CA., USA	E	36	San Francisco	19-Apr-1906	8.3	0.1	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ROCKER	CA., USA	E	45	San Francisco	19-Apr-1906	8.3	2.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
URLINGAME	CA., USA	E	24	San Francisco	19-Apr-1906	8.3	1.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
MERALD 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)
ROCKER	CA., USA	E	45	San Francisco	19-Apr-1906	8.3	2.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
OTRE DAME	CA., USA	E	50	San Francisco	19-Apr-1906	8.3	3.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. CRYSTAL SPRINGS	CA., USA	E	75	San Francisco	19-Apr-1906	8.3	0.0	Moderate	Ambraseys (1960)
. CRYSTAL SPRINGS	CA., USA	GA	127	San Francisco	19-Apr-1906	8.3	0.4	None	ICOLD (1974)
AGUNITA	CA., USA	E	15	San Francisco	19-Apr-1906	8.3	6.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ELVEDERE	CA., USA	E	48	San Francisco	19-Apr-1906	8.3	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
.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)
AGUNITAS	CA., USA	E	48	San Francisco	19-Apr-1906	8.3	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
OWELL	CA., USA	E	50	San Francisco	19-Apr-1906	8.3	19.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
STATES	CA., USA	E	93	San Francisco	19-Apr-1906	8.3	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ERRYMAN	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)
AKE CHABOT	CA., USA	E	135	San Francisco	19-Apr-1906	8.3	46.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ACIFIC GROVE	CA., USA	E	20	San Francisco	19-Apr-1906	8.3	41.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
AKE RALPHINE	CA., USA	E	35	San Francisco	19-Apr-1906	8.3	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EMESCAL	CA., USA	E	105	San Francisco	19-Apr-1906	8.3	29.0	Minor	Ambraseys (1960)
. SAN LEANDRO	CA., USA	E	125	San Francisco	19-Apr-1906	8.3	37.0	None	Ambraseys (1960)
IEDMONT 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)
ORT COSTA	CA., USA	E	45	San Francisco	19-Apr-1906	8.3	44.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ORREST LAKE	CA., USA	E	60	San Francisco	19-Apr-1906	8.3	44.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
AKE HERMAN	CA., USA	E	50	San Francisco	19-Apr-1906	8.3	51.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. 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)
. 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)
AKE CAMILLE	CA., USA	E	30	San Francisco	19-Apr-1906	8.3	52.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
AKE FREY	CA., USA	E	83	San Francisco	19-Apr-1906	8.3	59.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
OLCANO LAKE	Mexico	E	12	Imperial V.	22-Jun-1915	5.3	0.0	Collapse	Ambraseys (1960)
URAYAMA SHIMO	Japan	E	52	Kanto	01-Sep-1923	8.2	18.0	Moderate	Nakayama, A. (1964)
. MURAYAMA	Japan	E	79	Kanto	01-Sep-1923	8.2	18.0	Moderate	Ambraseys, N.N. (1960)
. MURAYAMA	Japan	E	101	Kanto	01-Sep-1923	8.2	18.0	Moderate	Ambraseys, N.N. (1960)
NO	Japan	E	161	Kanto	01-Sep-1923	8.2	98.0	Serious	ICOLD (1974)
OKYO W.S.	Japan	E	79	Kanto	01-Sep-1923	8.2	24.0	Minor	Duke, C.M. (1960)
HEFFIELD	CA., USA	E	25	Santa Barbara	29-Jun-1925	6.3	11.2	Collapse	Seed, H.B.; Lee, K.L.; Idriss, I.M. (1969)
ARAHONA	Chile	T	200	Talca	01-Oct-1928	8.4	160.0	Collapse	Smith, E.S. (1969)
HATSWORTH NO. 2	CA., USA	HF	44		30-Aug-1930	5.3	1.0	Moderate	Sherard, J.L.; et al (1963)
ALPASO	Peru	CCRD	255		10-Oct-1938			Minor	Ambraseys, N.N. (1960)
isc. Embankmts	Japan	E	50/60	Ojika	-1939	6.6		Severe	Akiba, M.; Semba, H. (1941)

HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES (CONT'D)

NAME	LOCATION	ID	HT. [ft]	EQK NAME	DATE	M	DIST [km]	DAMAGE	Principal References Consulted
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	Chile	CFRD	275	Illapel	04-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	AB	98	Nankai	21-Dec-1946	7.2	50.0	Minor	ICOLD (1975)
HOSOROGI	Japan	E	28	Fukui	28-Jun-1948	7.3	4.8	Collapse	Ambraseys, N.N. (1960)
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)
HAYWEE	CA., USA	HF	81	Kern County	21-Jul-1952	7.7	152.0	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	NE., USA	E	125	Fallon	23-Aug-1954	6.7	48.0	None	Ambraseys (1960)
COLEMAN	NE., USA	COMP	--	Fallon	23-Aug-1954	6.7	24.0	Collapse	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ROGERS	NE., 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)
OUEDD FODDA	Algeria			Orleansville	09-Sep-1954	6.8			
ARCATA	CA., USA	E	55	Eureka	21-Dec-1954	6.6	8.0	Moderate	Ambraseys (1960)
ST MARY'S	CA., USA	M	50	Daly City	23-Oct-1955	5.4	3.0	Minor	Sherard, J.L.; et al (1963)
BLACKBROOK	England	G	100		11-Feb-1957	5.6	6.4	Moderate	Walters (1964)
PINZANES	MEXICO	CFRD	220	Mexico	28-Jul-1957	7.5		None	Ambraseys (1960)
HEBGEN	MON., USA	E	90	Hebgen Lake	17-Aug-1959	7.1	16.0	Serious	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
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	- -1962	4.9	4.0	None	
KARIBA	Rhodesia	CA	420		23-Sep-1963	6.1		None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
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	218	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 COBRE	Chile	T	--		28-May-1965	7.1	35.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
KOYNA	India	CG	338	Koyna	11-Dec-1967	6.5	3.0	Serious	Chopra, A.K.; Chakrabarti, L. (1971)

HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES (CONT'D)

ME	LOCATION	ID	HT. [ft]	EQK NAME	DATE	M	DIST [km]	DAMAGE	Principal References Consulted
R	India	E	79	Koyna	11-Dec-1967	6.5	8.0	Moderate	ICOLD (1974)
YAGAKENUMA	Japan	E	40	Tokachi-Oki	16-May-1968			Collapse	Shibata et al (1971)
CHIRIGOYA	Japan	E	26	Tokachi-Oki	16-May-1968			Collapse	Shibata et al (1971)
MAI-BASHI	Japan	E		Tokachi-Oki	16-May-1968				Shibata et al (1971)
ATTAI	Japan	E		Tokachi-Oki	16-May-1968				Shibata et al (1971)
MANOSAWA	Japan	E	34	Tokachi-Oki	16-May-1968			Collapse	Shibata et al (1971)
OREY	Peru	T		Peru	1969			Collapse	M-K Engineers (pers. comm.)
ZURYU	Japan	ECRD	419	Gifu	09-Sep-1969	6.7	40.0	None	Nose, M.; Baba, K. (1981)
ISENYAMA	Japan	ECRD	312	Gifu	09-Sep-1969	6.7	--	None	Takahasi, T.; et al (1977)
UACHOPOLCA	Peru	T		Peru	1970			Collapse	Smith, E.S. (1971)
OPEZ	CA., USA	E	166	San Fernando	09-Feb-1971	6.5	8.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
RNELL DEBRIS	CA., USA	E	49	San Fernando	09-Feb-1971	6.5	8.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
RY CANYON	CA., USA	HF	66	San Fernando	09-Feb-1971	6.5	10.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
N 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)
HANNEL 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)
NSEN	CA., USA	E	97	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
REEN VERDUGO	CA., USA	E	117	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
INKWATER	CA., USA	E	105	San Fernando	09-Feb-1971	6.5	14.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ORTER ESTATE	CA., USA	E	41	San Fernando	09-Feb-1971	6.5	16.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
UQUET CANYON	CA., USA	E	190	San Fernando	09-Feb-1971	6.5	17.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ESERVOIR NO. 1	CA., USA	E	35.0	San Fernando	09-Feb-1971	6.5	22.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ATSWORTH	CA., USA	HF	44	San Fernando	09-Feb-1971	6.5	23.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EPULVEDA	CA., USA	E	57	San Fernando	09-Feb-1971	6.5	24.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ROLD	CA., USA	E	30	San Fernando	09-Feb-1971	6.5	26.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
IEDERICH	CA., USA	E	60	San Fernando	09-Feb-1971	6.5	27.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
CINO	CA., USA	E	168	San Fernando	09-Feb-1971	6.5	26.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ANTA FELICIA	CA., USA	E	200	San Fernando	09-Feb-1971	6.5	26.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
NKLE	CA., USA	E	41	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. HOLLYWOOD	CA., USA	E	87	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
FRANKLIN	CA., USA	E	55	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. 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)
ENOAKS	CA., USA	E	62	San Fernando	09-Feb-1971	6.5	30.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
TONE CANYON	CA., USA	E	185	San Fernando	09-Feb-1971	6.5	30.4	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
IRMONT	CA., USA	HF	121	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. FRANKLIN	CA., USA	HF	103	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
GLE ROCK	CA., USA	E	113	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
OWENA	CA., USA	E	29	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
BIO DIV. DEBRIS	CA., USA	E	55	San Fernando	09-Feb-1971	6.5	32.0	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
AWTELL	CA., USA	E	34	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
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)
ILVER LAKE	CA., USA	HF	53	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
TON WASH	CA., USA	E	63	San Fernando	09-Feb-1971	6.5	33.6	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
LYSIAN	CA., USA	E	72	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
OD RANCH	CA., USA	E	146	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SCOT	CA., USA	E	73	San Fernando	09-Feb-1971	6.5	35.2	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
EVY CHASE	CA., USA	E	35	San Fernando	09-Feb-1971	6.5	28.8	None	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
. SAN FERNANDO	CA., USA	HF	140	San Fernando	09-Feb-1971	6.5	11.2	Major	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
SAN FERNANDO	CA., USA	HF	80	San Fernando	09-Feb-1971	6.5	11.2	Serious	Seed, H.B.; Makdisi, F.; De Alba, P. (1978)
ACOIMA	CA., USA	CA	372	San Fernando	09-Feb-1971	6.5	5.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
G TUJUNGA	CA., USA	CA	251	San Fernando	09-Feb-1971	6.5	32.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
ANTA ANITA	CA., USA	CA	251	San Fernando	09-Feb-1971	6.5	27.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC

HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES (CONT'D)

NAME	LOCATION	ID	HT. [ft]	EQK NAME	DATE	M	DIST [km]	DAMAGE	Principal References Consulted
EL COBRE	Chile	T		Chile	08-Jul-1971	7.5	80.0	Serious	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
SALAMANCA	Chile	T		Chile	08-Jul-1971	7.5	110.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
ILLAPEL	Chile	T		Chile	08-Jul-1971	7.5	100.0	Collapse	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
LIMAHUIDA	Chile	T		Chile	08-Jul-1971	7.5	100.0	Moderate	Eisenberg, A.; Husid, R.; Luco, J.E. (1972)
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)
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)
BAIHE	China	E	216	Tangshan	28-Jul-1976	7.8		Serious	Liu, L; Li,K.; Bing, D. (1980) 7WCEE
PAIHO	China	E	213	Tangshan	28-Jul-1976	7.8	150.0	Moderate	CSCPRC, Report 8 (1980)
TOUHO	China	E	72	Tangshan	28-Jul-1976	7.8		Serious	CSCPRC, Report 8 (1980)
AMBIESTA	Italy	CA	194	Friuli	06-May-1976	6.5	22.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
LUMIEI	Italy	CA	446	Friuli	06-May-1976	6.5	30.0	None	EDF (1987)
MAINA DISAURIS	Italy	CA	446	Friuli	06-May-1976	6.5	43.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
BARCIS	Italy	CA	164	Friuli	06-May-1976	6.5	48.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
IZVORUL MONTELVI	Romania	CG	417	Vrancea	04-Mar-1977	7.2	100.0	None	EDF (1987)
POIANA USULUI	Romania	CB	262	Vrancea	04-Mar-1977	7.2	60.0	None	Hansen, K.D.; Roehm, L.H. (1979) - WP & DC
PEREZ CALDERA	Chile	T		Argentina	24-Nov-1977	7.4	350.0	Minor	Smith, E.S. (pers. comm.)
MOCHI-KOSHI	Japan	T	98	Nr Izu-Oshima	14-Jan-1978	7.0	35.0	Collapse	Marcuson, W.F. et al (1979)
MOCHI-KOSHI	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	CA., USA	E	150	Mammoth Lakes	27-May-1980	6.2	21.0	None	Leps, T.M. (Pers. Comm., 1987)
LONG VALLEY	CA., USA	E	126	Mammoth Lakes	27-May-1980	6.2	5.0	None	Seed, H.B. (1985) - EERI Newsletter, Vol.9
LA VILLITA	Mexico	ECRD	197	Mexico	25-Oct-1985	7.3	31.0	None	Singh, S.K.; Suarez, G. (1987)
EL INFIERNILLO	Mexico	ECRD	485	Mexico	25-Oct-1985	7.3	55.0	None	Singh, S.K.; Suarez, G. (1987)
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 Newsletter (1985)
CERRO NEGRO	Chile	T		Chile	03-Mar-1985	7.7		Collapse	Castro, G. (pers. comm., 1986)
VETA DE AGUA	Chile	T		Chile	03-Mar-1985	7.7		Collapse	Castro, G. (pers. comm., 1986)
RAPEL	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	Mexico	ECRD	485	Michoacan	19-Sep-1985	8.1	75.0	Minor	Bureau, G.; Campos-Pina, M. (1986)

HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES (CONT'D)

AME	LOCATION	ID	HT. [ft]	EQK NAME	DATE	M	DIST [km]	DAMAGE	Principal References Consulted
A VILLITA	Mexico	ECRD	197	Michoacan	21-Sep-1985	7.5	61.0	None	Bureau, G.; Campos-Pina, M. (1986)
L INFIERNILLO	Mexico	ECRD	485	Michoacan	21-Sep-1985	7.5	80.0	None	Bureau, G.; Campos-Pina, M. (1986)
UAI DIV.	New Zlnd	CG	17	Bay of Plenty	02-Mar-1987	6.2	11.0	None	Robinson, R.; Benjamin, H.L. (1987)
ATAHINA	New Zlnd	ECRD	259	Bay of Plenty	02-Mar-1987	6.2	23.0	Moderate	EQE (1987), Gillon (1988)
ARVEY RESERVOIR	Ca., USA	E	160.0	Whittier	01-Oct-1987	6.1	3.0	None	Horowitz, Ehasz (USCOLD Newsletter, 1987)
RANGE COUNTY RES.	Ca., USA	E	114.0	Whittier	01-Oct-1987	6.1	23.0	None	Horowitz, Ehasz (USCOLD Newsletter, 1987)
HITTIER-NARROWS	Ca., USA	E	94.0	Whittier	01-Oct-1987	6.1	4.0	None	Horowitz, Ehasz (USCOLD Newsletter, 1987)
USTRIAN	Ca., USA	E	185.0	Loma Prieta	17-Oct-1989	7.1	11.5	Serious	Bureau et al (USCOLD Newsletter, 1989)
LMADEN	Ca., USA	E	110.0	Loma Prieta	17-Oct-1989	7.1	15.5	Minor	Bureau et al (USCOLD Newsletter, 1989)
UADALUPE	Ca., USA	E	142.0	Loma Prieta	17-Oct-1989	7.1	18.1	Minor	Bureau et al (USCOLD Newsletter, 1989)
EWELL	Ca., USA	E	182.0	Loma Prieta	17-Oct-1989	7.1	18.4	Moderate	Bureau et al (USCOLD Newsletter, 1989)
LMER J. CHESBRO	Ca., USA	E	95.0	Loma Prieta	17-Oct-1989	7.1	19.0	Moderate	Bureau et al (USCOLD Newsletter, 1989)
EXINGTON	Ca., USA	E	205.0	Loma Prieta	17-Oct-1989	7.1	20.6	Minor	Bureau et al (USCOLD Newsletter, 1989)
ASONA PERCOLATION	Ca., USA	E	34.0	Loma Prieta	17-Oct-1989	7.1	24.5	Minor	Bureau et al (USCOLD Newsletter, 1989)
INCONADA	Ca., USA	E	40.0	Loma Prieta	17-Oct-1989	7.1	26.2	Minor	Bureau et al (USCOLD Newsletter, 1989)
EROY ANDERSON	Ca., USA	E	235.0	Loma Prieta	17-Oct-1989	7.1	26.9	Minor	Bureau et al (USCOLD Newsletter, 1989)
ODA LAKE	Ca., USA	E	35.0	Loma Prieta	17-Oct-1989	7.1	28.2	Moderate	Bureau et al (USCOLD Newsletter, 1989)
ILL CREEK	Ca., USA	E	76.0	Loma Prieta	17-Oct-1989	7.1	30.2	None	Bureau et al (USCOLD Newsletter, 1989)
OASTWAYS	Ca., USA	E	46.0	Loma Prieta	17-Oct-1989	7.1	37.9	Minor	Bureau et al (USCOLD Newsletter, 1989)
. CRYSTAL SPRINGS	Ca., USA	CG	127.0	Loma Prieta	17-Oct-1989	7.1	69.0	None	Bureau et al (USCOLD Newsletter, 1989)
SAN ANTONIO	Ca., USA	E	160.0	Pomona Valley	28-Feb-1990	5.5	3.0	Minor	Jephcott, D.K. (EERI Newsletter, 1990)

Note 1:

ID Descriptors: E = Earthfill Dam
GA = Gravity Arch Dam
T = Tailings Dam
HF = Hydraulic Fill Dam
EHF = Earthfill and Hydraulic Fill Dam
CFRD = Concrete Face Rockfill Dam
AB = Arch Buttress Dam
CA = Concrete Arch Dam
ECRD = Earth Core Rockfill Dam
CG = Concrete Gravity Dam
COMP = COMPOSITE

Note 2: Information when left blank could not be found in references consulted.

Performance of dams subjected to earthquakes posterior to February 1990 will be reported in a sequel to this publication, currently being prepared by the Committee on Earthquakes.

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An extensive bibliography on the performance of dams during earthquakes was published in December 1984 by the United States Committee on Large Dams. The USCOLD bibliography supplements the present report and refers to most publications on the subject anterior to 1984. The references listed below were the principal sources consulted in the compilation of the preceding table.

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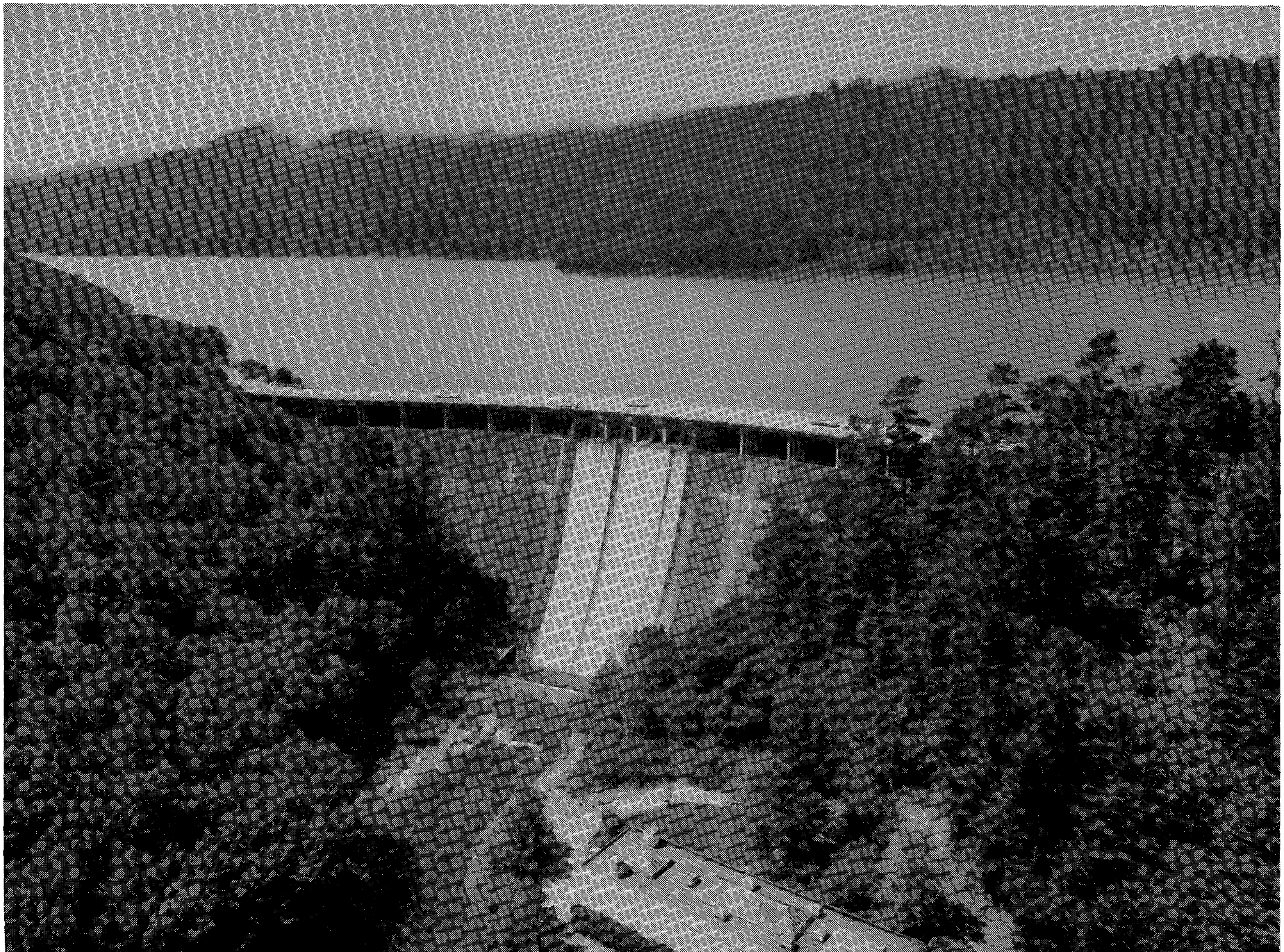
LOWER CYRSTAL SPRINGS DAM, CALIFORNIA, USA

Lower Crystal Springs Reservoir, San Mateo County, California, previously part of the main water supply terminus for the City of San Francisco, is now used as a catchment basin for local runoff and excess water storage. The reservoir lies within the rift valley formed by the San Andreas Fault and is located about 15 miles south of San Francisco. On April 18, 1906, the Great San Francisco Earthquake (estimated magnitude 8.3) caused heavy damage in the City and in its surrounding areas. Lower Crystal Springs Dam, a concrete gravity dam, survived the earthquake undamaged.

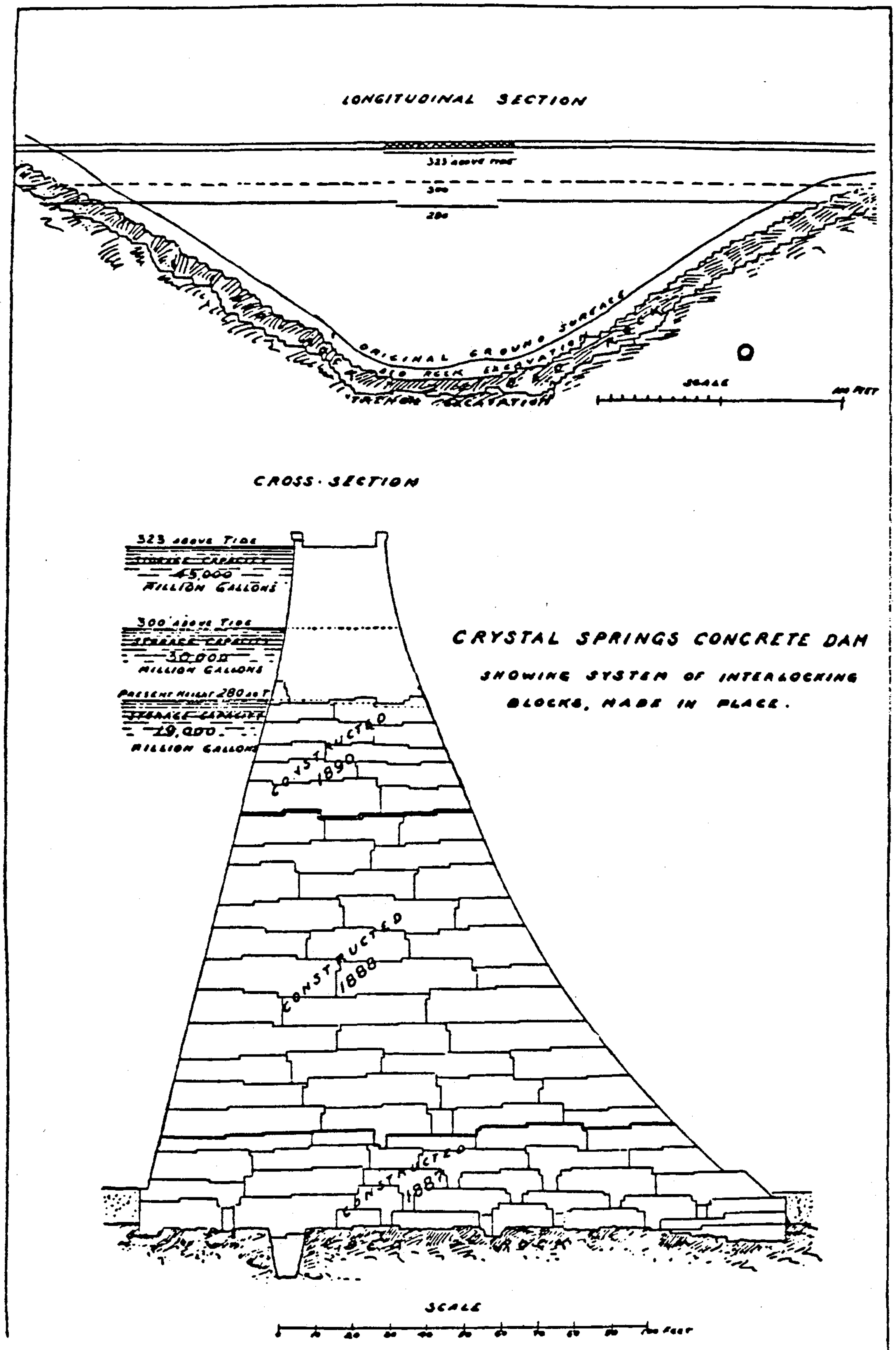
LOWER CRYSTAL SPRINGS DAM

Lower Crystal Springs Dam is a curved, concrete gravity structure constructed across San Mateo Creek (Figure 1). It is 145 feet high and creates the 67,000-acre-feet Lower Crystal Springs Reservoir. Construction of the dam started in 1887 and was completed to its present height in 1890. The original design called for the crest to be 43 feet higher than built. In 1911, a 4-foot high parapet wall was added, raising the dam crest to 11 feet above spillway elevation (Figure 2). To increase reservoir storage, the effective height of the center overflow spillway can be raised by installing flashboards.

The dam was constructed using interlocking blocks of placed concrete, each averaging approximately 34 feet by 34 feet in plan by about 8 feet deep. These blocks were staggered, so that there would be no continuous vertical or horizontal joint through the dam. Most of the cement was imported from England. This is one of the first important structures featuring a carefully designed concrete mix. Rigidly enforced construction methods and control were implemented by the Chief Design Engineer, Hermann Schussler. Centerline cross-sectional dimensions gradate from 43 feet at crest to 176 feet at base, with a curved



GENERAL VIEW
Lower Crystal Springs Dam
Figure 1



From Schussler, 1906

CROSS SECTION
Lower Crystal Springs Dam
Figure 2

downstream face and a 1:4 (horizontal to vertical) batter on the upstream face. The crest is 601-foot long.

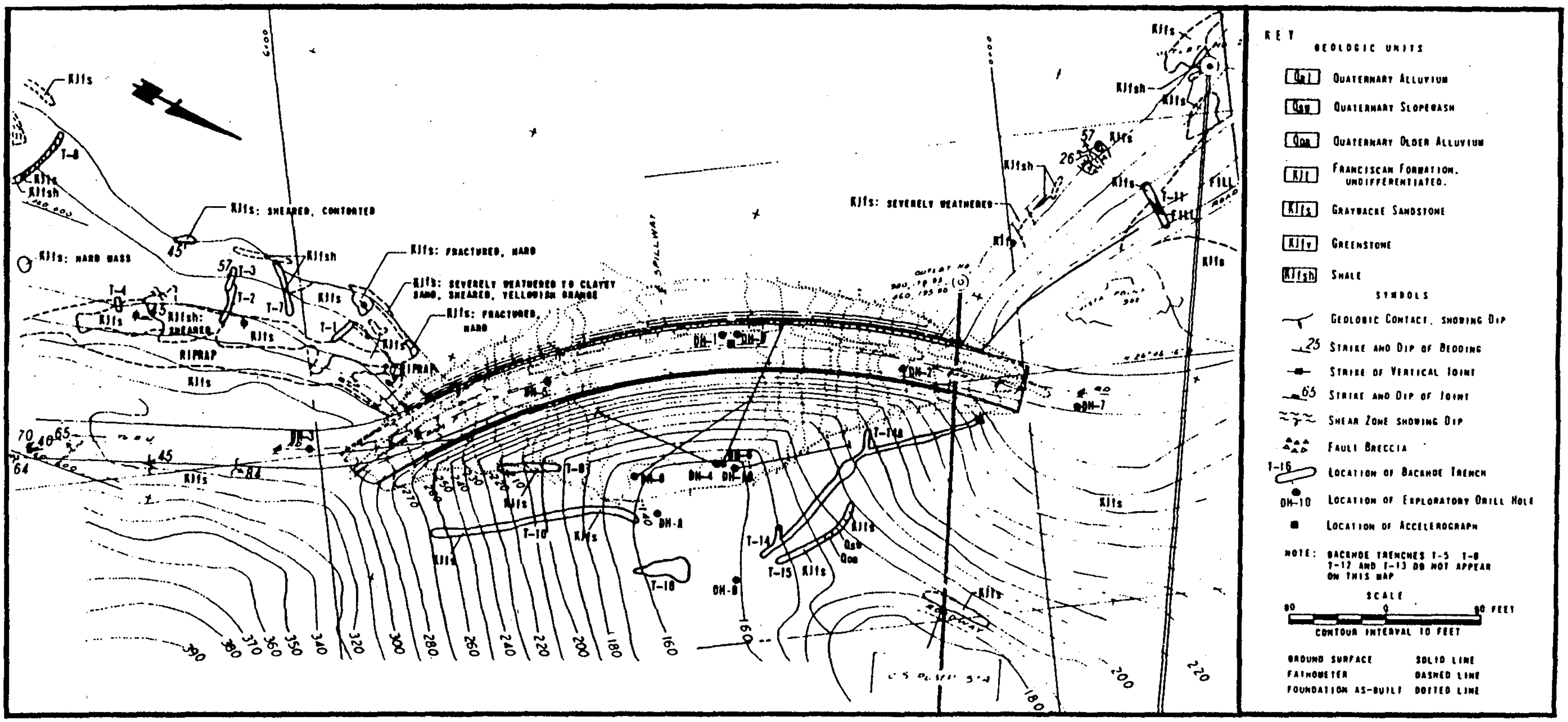
In 1923, the California Highway Commission authorized construction of a two-lane highway bridge across Lower Crystal Springs Dam, supported on concrete bents typically 32.5 feet on center and approximately 20 feet above dam crest.

The outlet works for the dam are located in the left abutment and consist of two outlet towers with separate tunnels. The original brick-lined outlet tunnel and brick masonry tower feed the Crystal Springs pump station, immediately below the dam. A second reinforced concrete outlet tower and tunnel system were constructed in 1934 to meet the growth demands of San Francisco.

The dam is founded on Cretaceous/Jurassic graywacke of the Franciscan Formation, which varies considerably in its physical condition. The foundation rocks located at the edge of the San Andreas fault zone are more highly deformed than typical Franciscan Melange. The dam foundation is characterized by zones of crushed, fractured and sheared rock (Figure 3). In places, the graywacke appears fresh, but the rocks are generally moderately to strongly weathered. Typical cores recovered in 1977 from the foundation showed 2 to 20 fractures per foot, and intact pieces rarely exceeded four inches. Even strong and intact appearing graywacke proved to be fractured and brecciated, when observed with a microscope. Healing of fractures and cracks by deposition of secondary minerals accounts for the fresh and intact appearance of the rock.

THE APRIL 18, 1906 SAN FRANCISCO EARTHQUAKE

The San Andreas Fault has a total length of more than 700 miles and extends from Cape Mendocino in northern California south beyond the Mexican border. It is an active boundary between two major tectonic plates, the Pacific and North American plates.



SITE GEOLOGY
Lower Crystal Springs Dam
Figure 3

Continued relative movement along this boundary has the potential for generating major earthquakes (magnitude 8+) with long duration of strong shaking.

The 1906 event on the San Andreas Fault is one of the three greatest earthquakes in recorded California history, comparable in size to the 1857 Fort Tejon and the 1872 Owens Valley earthquakes. A Richter Magnitude of 8.3 was estimated from early seismograms for the 1906 event. Total fault rupture extended over 435 km from Humboldt County in northern California south to the vicinity of San Juan Bautista, in San Benito County.

Right-lateral offsets documented by breaks in roads, fences, etc. ranged from small amounts south of San Jose up to a maximum of about 20 feet north of San Francisco. Heavy damage occurred in the City of San Francisco and in the surrounding area, as well as north and south near the fault rupture trace. The earthquake ruptured the City water mains, and fires that had developed raged out of control for days and contributed to a large part of the overall losses.

Heavy damage occurred in some communities well to the east of the San Andreas Fault, notably at Santa Rosa and Los Banos, and was probably linked to local geologic conditions. Shaking was felt over most of northern California, southern Oregon and part of Nevada.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Lower Crystal Springs Dam survived the 1906 earthquake with no damage. Engineers who inspected the dam following the earthquake could find no evidence of any distress. Charles Derleth, Jr., an Associate Professor of Structural Engineering at the University of California and quoted in Schussler (1906), reported that "...The intake works, Crystal Springs pumping station and all other accessory construction in the neighborhood of the dam were left intact by the earthquake...." Three of the most prominent cracks still existing in the dam today had been reported prior to the 1906 earthquake. These cracks and several other smaller cracks have probably been caused by shrinkage stresses in the concrete, due to uneven dissipation of the heat of hydration immediately following construction.

The primary fault rupture passed through the Lower Crystal Springs Reservoir approximately 600 feet west of the dam. Eight to ten feet of relative displacement and predominantly right-lateral "en échelon" surface ruptures were reported in that area. Sympathetic movements or localized slope failures were also observed along the eastern shore of the reservoir, about 300 feet away from Lower Crystal Springs Dam. Estimated intensities were IX on the Rossi-Forel Scale for the dam site area.

In 1977, a state-of-the-art seismic safety evaluation that included detailed field and laboratory testing programs and three-dimensional finite element response analysis, concluded that the dam would perform satisfactorily when subjected to a postulated Maximum Credible Earthquake along the San Andreas Fault with sixty seconds of strong motion and a peak horizontal acceleration of 0.70g. Another detailed evaluation, performed in 1984 for the original brick masonry outlet tower, led to the same conclusions as for the dam. The level of shaking to which the dam was subjected in 1906 can only be estimated, but the ground motions at the damsite during that event were probably comparable in severity to the reevaluation acceleration time-histories, which thereby confirmed the outstanding field performance of the dam and its appurtenant facilities.

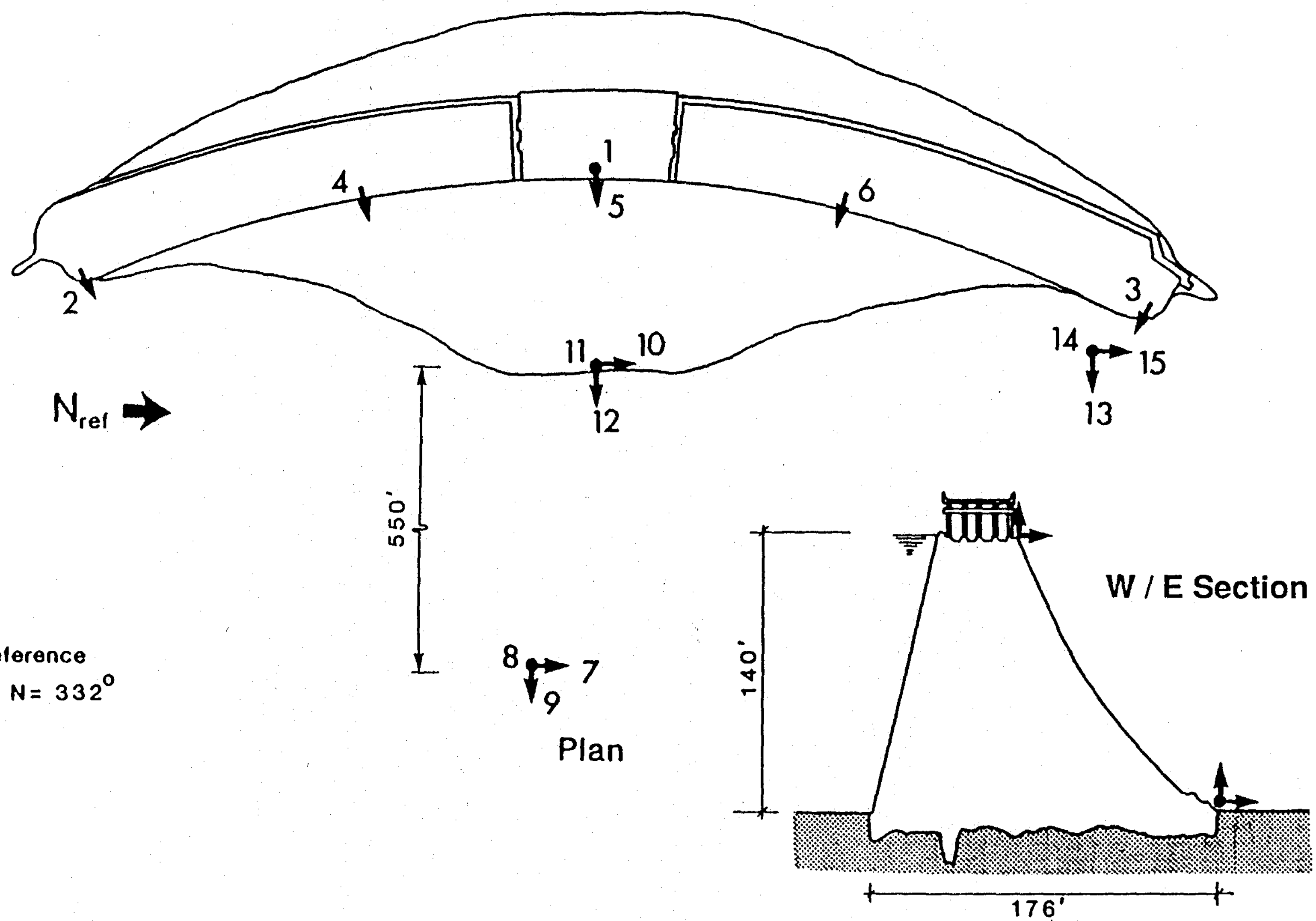
INSTRUMENTATION AND STRONG-MOTION RECORDS

No instruments recorded the 1906 earthquake at the dam or in its vicinity. Fifteen sensors including three SMA-1 strong-motion instruments with common triggering are currently installed at the dam. One of the strong-motion instruments is on the crest, one on the left abutment, and the third is several hundred feet downstream of the dam (Figure 4). These instruments were triggered during the October 17, 1989 Loma Prieta earthquake, where peak accelerations of 0.05g and 0.10g were recorded at the base and crest of the dam, respectively (Figure 5). The corresponding epicentral distance was 69 km.

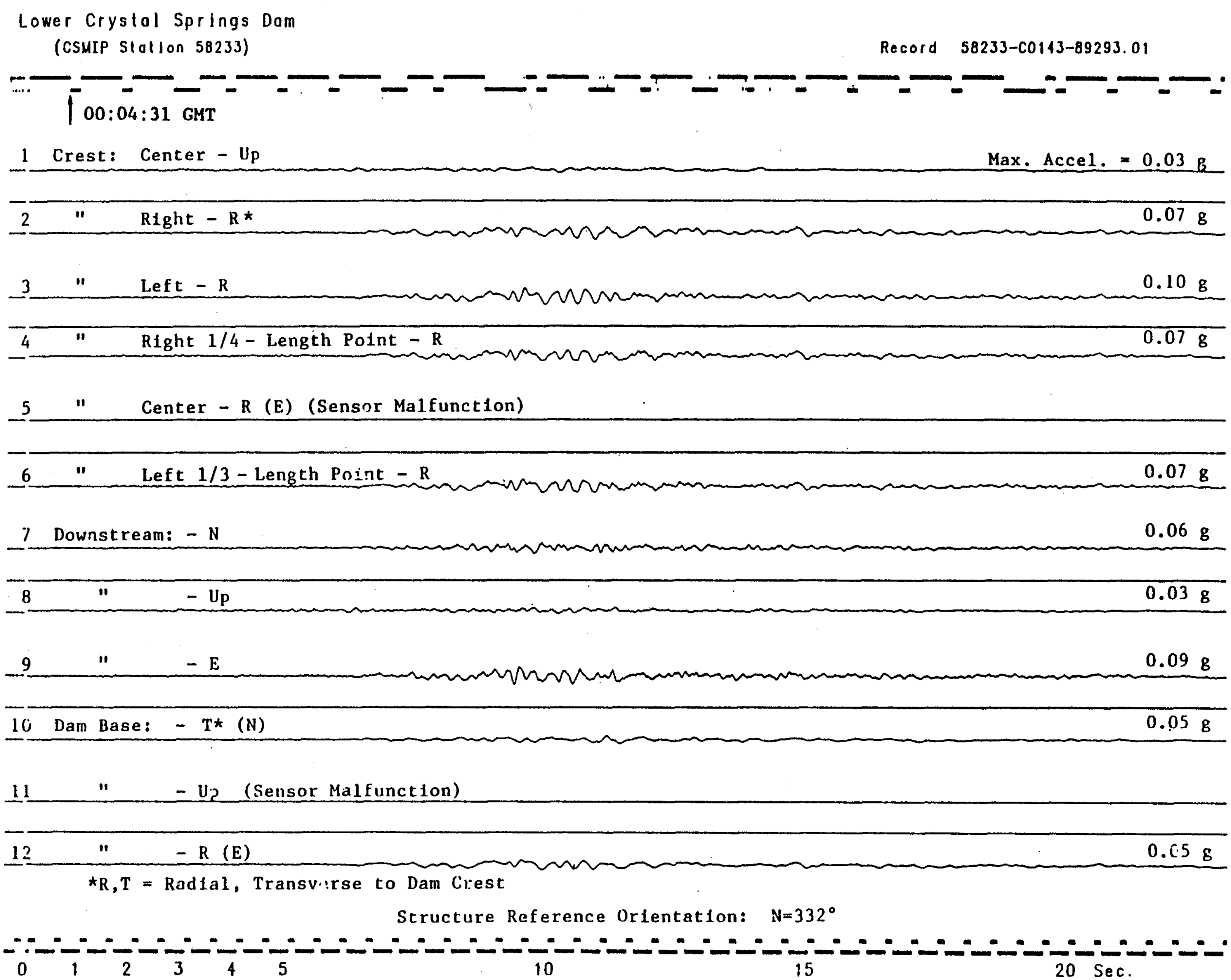
Lower Crystal Springs Dam

(CSMIP Station No. 58233)

SENSOR LOCATIONS



Structure Reference
Orientation: N= 332°



OCTOBER 17, 1989
ACCELERATION RECORDS
Lower Crystal Springs Dam
Figure 5

CONCLUSION

Lower Crystal Springs Dam survived the Great San Francisco Earthquake of 1906 with no damage. Modern seismic safety evaluations concluded that the dam and its appurtenances would perform satisfactorily if shaken by another earthquake of magnitude 8 or greater along the nearby San Andreas Fault. Several factors have contributed to the excellent past and present performance of the 98-year old dam. Most credit should go to the unique design and careful selection of construction materials. The designer also demanded meticulous care and quality control during construction, resulting in a dam of superior quality, and well ahead of its time.

Another significant feature, not often mentioned, is the protection to the dam that may have resulted from the highly deformed, brecciated, and fractured rock foundation. While such a foundation would clearly be considered poor by today's standards and was untreated, the rock performed admirably and may have absorbed some of the high intensity energy generated at the damsite by the earthquake.

Having survived the great earthquake of 1906, Lower Crystal Springs Dam continues to serve the water supply system of the City and County of San Francisco to the present day. In 1976, the dam was designated a California Historic Civil Engineering Landmark and a plaque was dedicated to its designer, Hermann Schussler of the Spring Valley Water Company.

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HEBGEN DAM, MONTANA, USA

On the night of August 17, 1959, several earthquakes, the largest being of magnitude 7.1, shook a large area of southern Montana and northern Wyoming in the vicinity of Yellowstone Park. A huge block of the earth's crust, 125 square miles in area, including Hebgen Lake, subsided during the earthquakes along recognized faults north of the reservoir. The epicenter of the main shock was about 8 miles north of the town of West Yellowstone and 12 miles southeast of Hebgen Dam. Although the dam did not fail, it was severely damaged. The major damage consisted of two feet of embankment settlement and major slumping and spreading of the shells. The spillway training walls were badly cracked and the bottom slab of the spillway channel was completely demolished. Only a series of fortunate circumstances accounted for the fact that the dam did not fail completely.

HEBGEN DAM

Hebgen Dam is an earthfill structure with a central concrete core wall. The dam is located on the Madison River, a tributary of the Missouri River, in Montana, near the western boundary of Yellowstone National Park. The embankment is founded on streambed deposits and the concrete core wall extends into bedrock. Over most of its length and in the left abutment, the concrete wall is founded on sound bedrock.

Hebgen Dam was constructed to a maximum height of about 90 feet, with a crest length of 720 feet. The embankment was built as a rolled fill from gravelly clay, with side slopes of 3:1 (horizontal to vertical) upstream and 2.5:1 downstream. The core wall consists of mass concrete, unreinforced, varying in thickness from 16 feet at the bottom to 3 feet at the top. The right abutment consists of nearly impervious clayey gravelly sand. There has never been noticeable leakage in that area of the dam.

The reservoir volume is 340,000 acre-feet. The lake has two arms and extends for a maximum length of about 20 miles (Figure 1). The spillway, located on the right abutment, has a control structure with six bays for stop logs and a concrete-lined open channel flume, founded on the natural soils of the right abutment. The outlet from the reservoir is a 12-foot diameter pipe, with invert elevation about 65 feet below the reservoir high water level. The outlet is founded on the hard rock of the left abutment. Figure 2 shows a general layout of the dam and spillway.

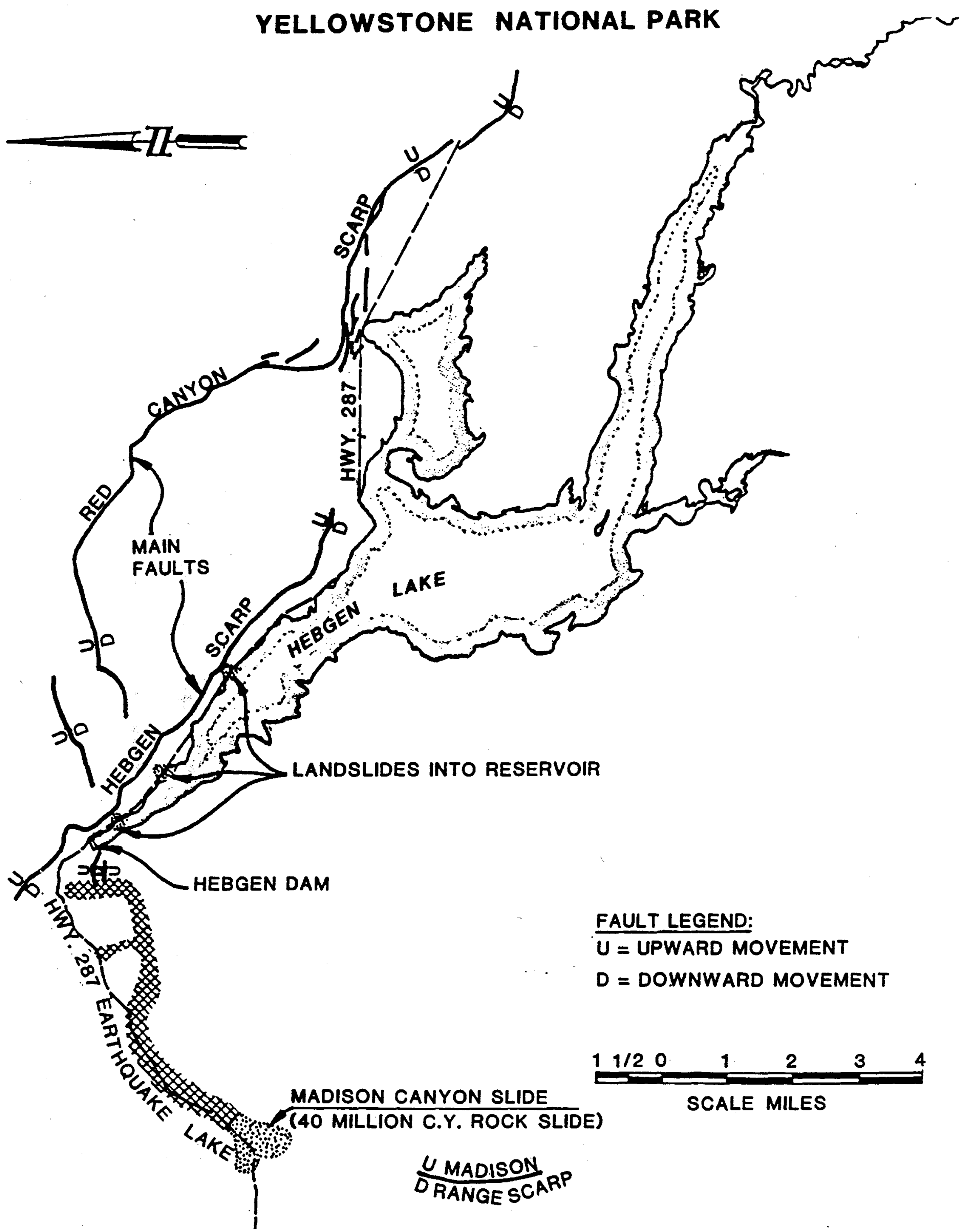
THE MARCH 17, 1959 YELLOWSTONE EARTHQUAKE

The March 17, 1959 earthquake had a magnitude of about 7.1. It is the most severe earthquake on record in Montana and the only major earthquake in historic times in the Hebgen Lake region. Eighteen persons lost their lives and estimated economic losses were about four million in 1959 dollars. Damage to buildings was low, due to the sparse population, and failures occurred primarily in roads and bridges, or as a result of landslides.

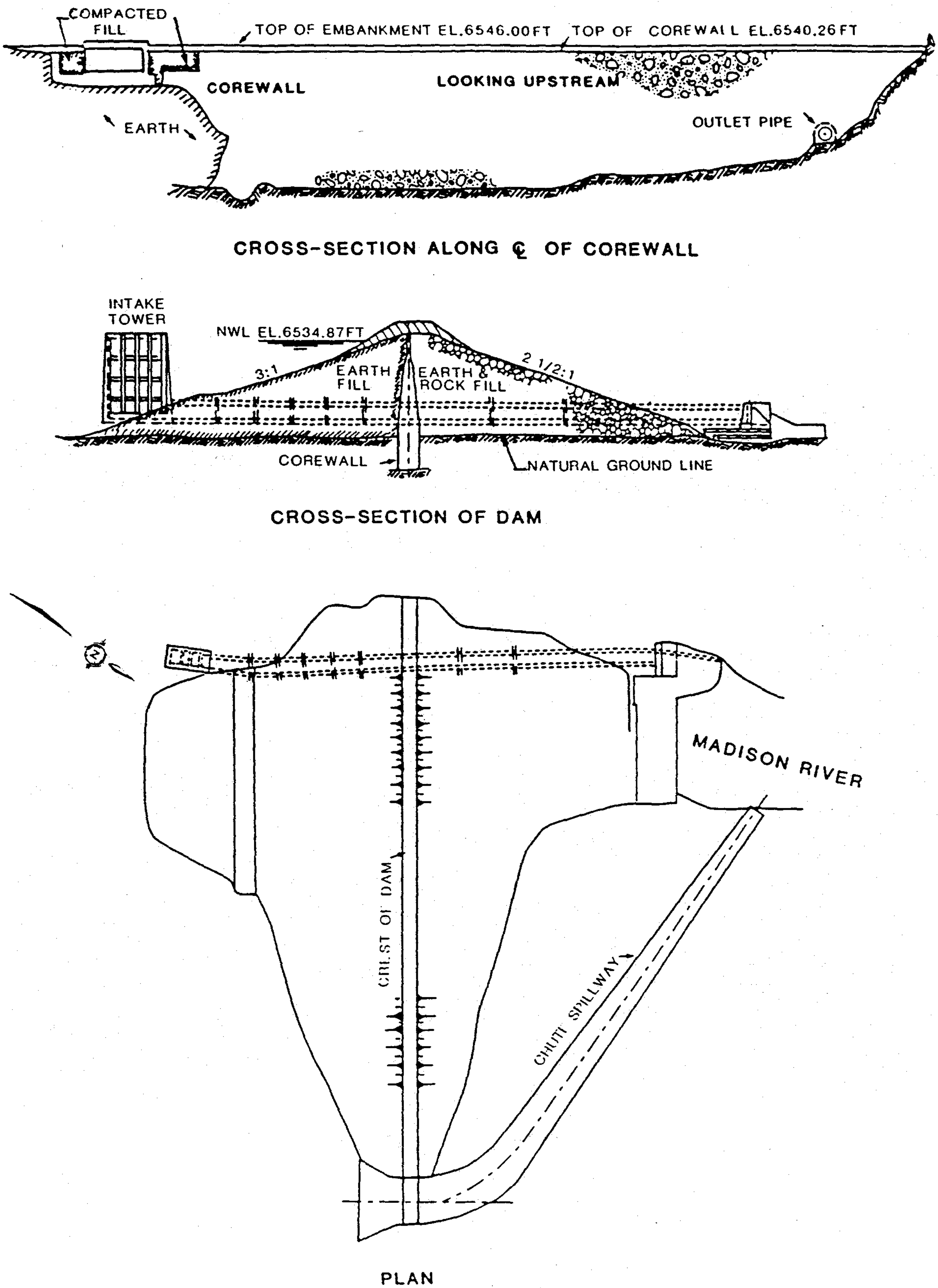
From instrument measurements, the epicenter was calculated to be from 10 to 12 miles due southeast of the dam. The shock was felt well over 500,000 square miles. Movements occurred on a number of major faults, with primarily vertical displacements. The aggregate length of surface ruptures of all the major faults of the area was more than 50 miles. The main trace of the Hebgen fault, which extends along the north side of the valley occupied by the reservoir, is located within about 700 feet of the right abutment of the dam, and experienced a vertical displacement of about 16 feet at that point.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Major landslides occurred along the banks of Hebgen Reservoir and downstream, where 40 million cubic yards of rock and soil filled the Madison River canyon for a distance of nearly one mile, forming a natural dam 230 feet high (Figure 1).



33 **FAULTING AND LANDSLIDES AT RESERVOIR**
Hebgen Dam
Figure 1

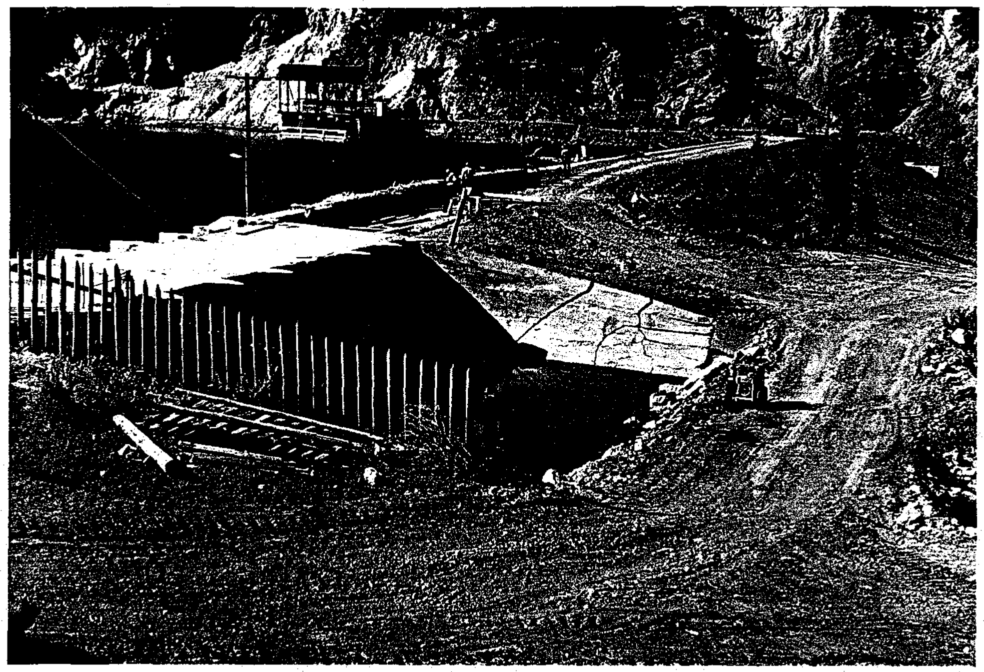


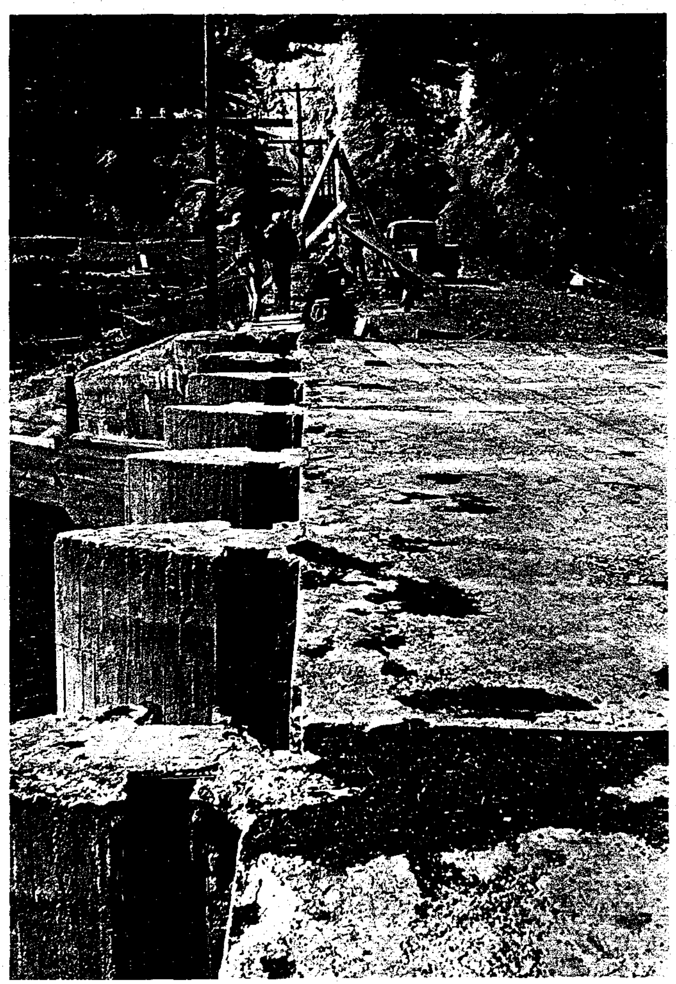
LAYOUT
Hebgen Dam
Figure 2

Associated with the faulting was a general subsidence and tilting of the bedrock underlying the reservoir. The measured subsidence varied between 5 and 22 feet in the vicinity of the reservoir. The bedrock underlying the dam went down approximately 9.7 feet in a remarkably uniform fashion. Levels taken on the top of the concrete core wall, after the earthquake, indicated that the crest of the portion of the wall that was founded in bedrock remained no more than 0.1 foot off perfectly level.

Large earthquake-induced reservoir waves resulted from the shaking and the drop in elevation in different parts of the lake. Hebgen Lake was within 2 feet of being full and was discharging about 1,000 cfs through the overflow spillway when the quake occurred. The dam was overtopped by several seiches. The earthquake set the reservoir water body into oscillation, apparently with a major component of motion in the longest dimension of the reservoir -- toward and away from the dam. Since the earthquake hit in the middle of the night, these waves might have not been observed; however, there was bright moonlight, and the caretaker reported what happened. From his account, at least four large waves, with a period on the order of 10 to 20 minutes, broke over the crest. The downstream face of the fill was eroded by the overtopping water, but from later inspection of the relatively small erosion on the crest and the downstream slope, it became apparent that little water went over the top and that it caused no real threat to the dam. The water continued oscillating for about 12 hours with decreasing amplitudes.

Although Hebgen Dam did not fail, the structure was severely damaged (Figure 3). From the right abutment of the dam to a point about 400 feet toward the center, the top of the concrete core wall was deflected downstream a maximum distance of 0.8 feet in approximately the shape of an arch. From that point to the edge of the spillway, about 220 feet, the arch continued on with a maximum deviation upstream of 2.8 feet from the original center line. The right end of the core wall sloped down a maximum of 0.17 foot in a distance of about 86 feet. Three major cracks, from 2 to 4 inches wide, opened in the top of the core wall, as well as numerous hairline cracks. The cracks in the core wall elongated the wall about 7 inches.





EARTHQUAKE DAMAGE, AUGUST 1959
Hebgen Dam
Figure 3

RICHARD W. KRAMER
5926 URBAN STREET
ARVADA, CO 80004

The main damage to the dam embankment was from settlement relative to the concrete core wall (and relative to the bedrock). The earthfill settled on both sides of the core wall. The high magnitude earthquake consolidated the relatively uncompacted material of the dam and the glacio-fluvial soils near and under the spillway. The upstream fill, which was saturated and more clayey than the downstream fill, suffered the most compaction. However, cross sections taken every 25 feet across the dam after the earthquake indicated little or no evidence of rotational failure of the slopes as a whole.

The cross sections showed a bulging of the lower embankment slopes and a spreading of the base, indicating that large shear strains may have been responsible for a major share of the crest settlement. A number of vertical longitudinal cracks also developed on the crest, but there were no transverse cracks. No cracking was found on the downstream slope of the embankment.

The maximum settlement of the downstream shell of the embankment with respect to the concrete core wall was about 2 feet at the right abutment, where the dam adjoins the spillway. This settlement was at a point where the dam is not very high. It was due to compression of the natural soils comprising the right abutment. The maximum settlement of the upstream shell was about 6 feet and occurred near the midpoint of the dam. About 5 feet of settlement were measured at the right end of the upstream shell near the spillway.

The concrete core wall remained intact and essentially uncracked over most of its length. At the right abutment, where it was not founded on rock, a number of vertical cracks opened to a maximum width of several inches, and the wall moved upstream about one foot. Most of the wall contained a few hairline cracks, however, and there was some question whether these were caused by the earthquake. Except for a few concentrated leaks which developed at the right abutment, and which were later definitely attributed to losses from the cracked spillway channel, no significant leakage developed through or under the dam.

The spillway moved to the left about 13 inches, creating an overlap of about 20 inches where the spillway structure met the core wall. The floor of the spillway dropped several inches. The spillway training walls were badly cracked, and the bottom slab of the spillway channel was completely demolished over large areas and subsequently washed away by the flowing water. Soon after the earthquake, stop logs were put in the spillway control structure, and no more water flowed down the spillway channel. It would have been dangerous to continue spilling, since the water would have eroded the soil under the broken spillway slab. The 12-foot diameter outlet pipe on the left abutment was not damaged and could be used to control the water level adequately.

On the right bank of the reservoir, upstream from the dam, a number of large landslides went into the water. These were not close enough to the dam to cause any trouble. The largest slide occurred about one mile upstream; it had an estimated volume of 350,000 cubic yards and destroyed about a thousand feet of the highway which ran along the edge of the reservoir.

The dam was repaired by adding earthfill at the crest and rebuilding the embankment slopes to their original shape. This required about 13,000 cubic yards of material. A new spillway was constructed, with a design similar to the old one. The concrete core wall was repaired by drilling a series of holes to intersect the visible cracks and pumping water-cement grout into them. A total of 365 feet of core was drilled and 412 sacks of cement were used for grouting.

INSTRUMENTATION AND STRONG-MOTION RECORDS

Hebgen Dam was not instrumented. The closest strong-motion accelerograph was 60 miles away at Bozeman, Montana, where a peak ground acceleration of 0.07 g was recorded.

From observations of damage to relatively small buildings in the sparsely populated epicentral area, the maximum intensity of the March 17, 1959 earthquake was estimated by experienced investigators at about VIII on the Modified Mercalli scale. Based on the extensive surface faulting and landslides, however, a local intensity rating of X would appear to be more appropriate for the Hebgen Dam site.

CONCLUSION

A spectacular and perhaps fortunate aspect of the Yellowstone earthquake, from an engineer's point of view, was the fact that the bedrock foundation of Hebgen Dam dropped down uniformly about 9.7 feet. At the same time, the average subsidence of the entire reservoir bottom was even more. The elevation of the surface of the water dropped by about 10.2 feet, so that the freeboard between the top of the concrete core wall and the reservoir actually increased by about 0.5 foot during the earthquake.

Although Hebgen Dam did not fail completely, the earthquake caused extensive damage to the embankment. The dam was repaired successfully, but at a cost of nearly $150,000 in 1959 dollars. Of particular interest is the fact that Hebgen Dam is one of the few dams known to have been overtopped by earthquake-induced waves from the reservoir. Part of the damage was attributed to overtopping. The earthquake occurred at high reservoir level when the dam was spilling, a highly improbable combination of events that differs from common analysis assumptions, where maximum earthquake load is assumed to occur under normal operating water level condition.

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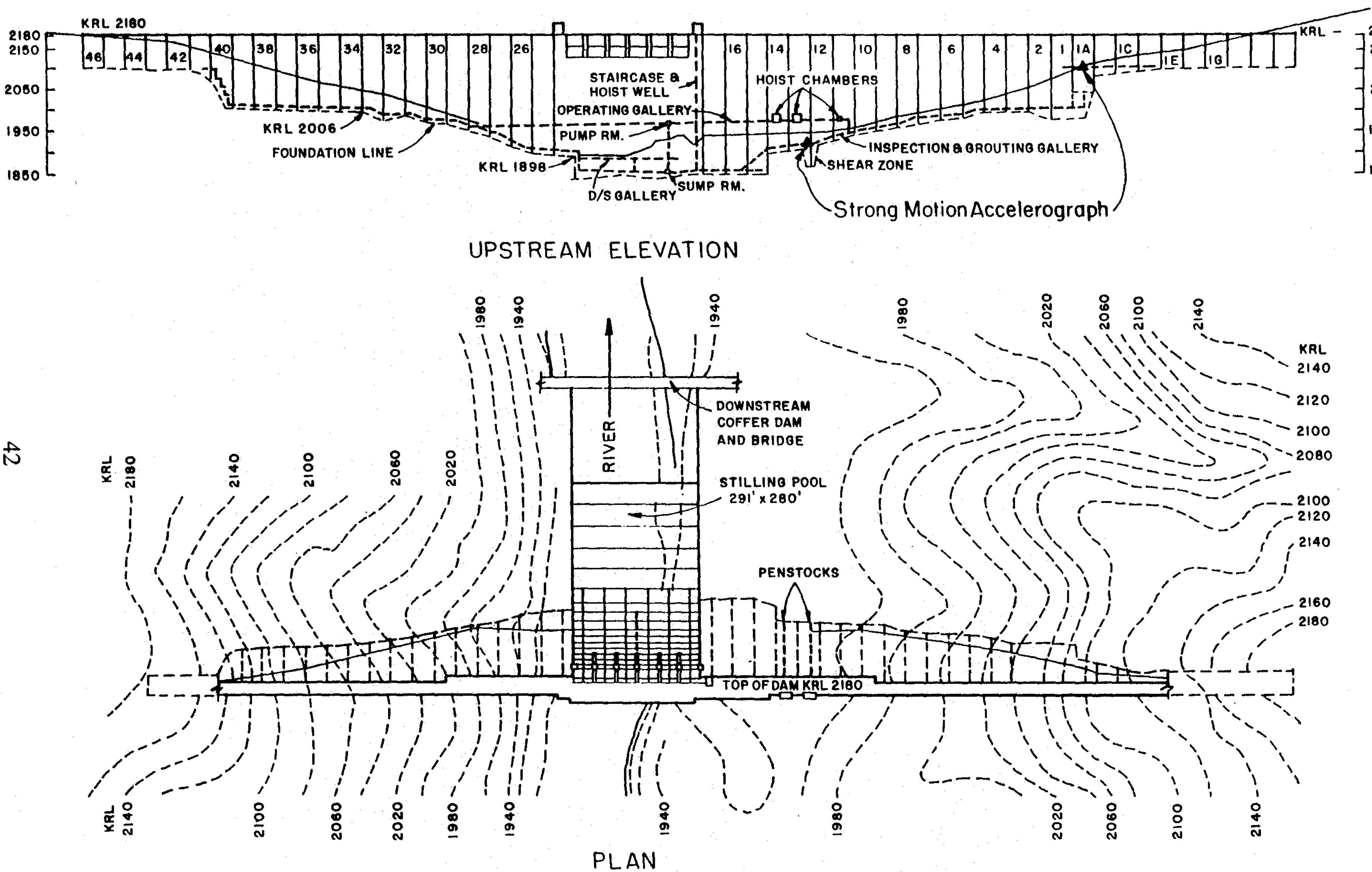
KOYNA DAM, INDIA

On December 11, 1967 an earthquake of unprecedented size occurred in the seemingly aseismic Indian peninsula. The epicenter was within a few miles of Koyna Dam, a major concrete gravity structure. The resulting motion caused significant structural damage to the dam.

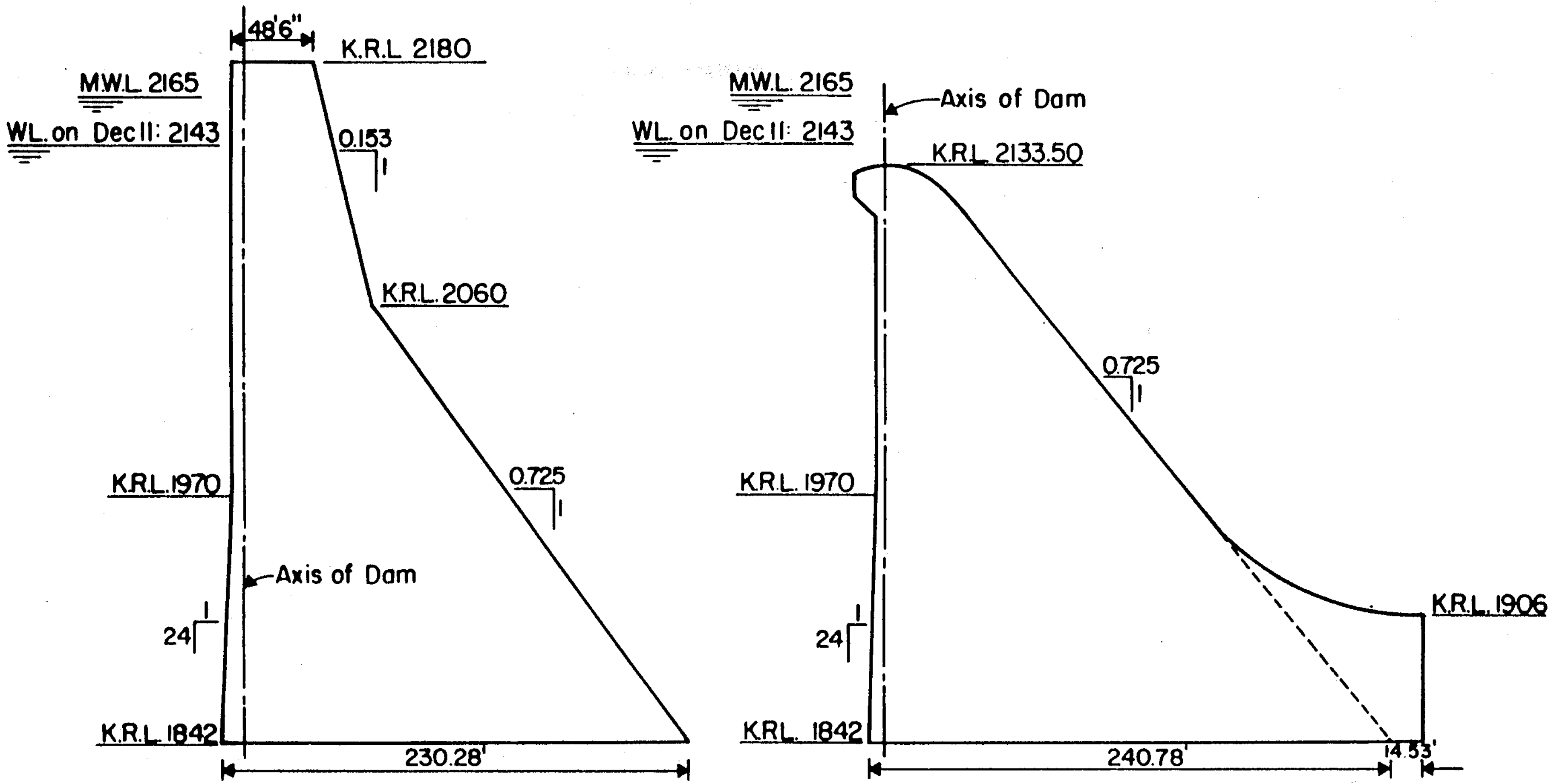
KOYNA DAM

Koyna Dam is located on the Koyna River in the western margin of the Indian peninsula. The dam, constructed during the years 1954 to 1963, is a straight gravity structure made of rubble concrete. It is about 2,800 feet long, 280 feet high above the river bed, and 338 feet high above the deepest foundation. The dam is constructed in 50-foot wide monoliths, and the contraction joints between monoliths are provided with copper water seals. The spillway portion of the dam is about 300 feet long (Figure 1).

The dam was designed by the then prevailing standard procedures, similar to the practice in the United States. The design forces included earthquake forces defined by a horizontal seismic coefficient of 0.05, uniform over the height, and water pressures in addition to the hydrostatic forces computed by standard formulas. The nonoverflow and overflow sections of the dam (Figure 2) were designed to satisfy the following criteria: (1) no tension in the section; (2) maximum compressive stress to be less than allowable stresses for the concrete; and (3) shear friction factor to be more than allowable minimum. Although this design procedure was similar to standard, worldwide practice at that time, the resulting Koyna cross-section is not typical of gravity dams (Figure 3). Departure from a typical section was the result of changes in design that had to be introduced while construction was in progress, because it was decided to combine the originally planned two stages of construction into one. At first sight, the Koyna section may appear to be much



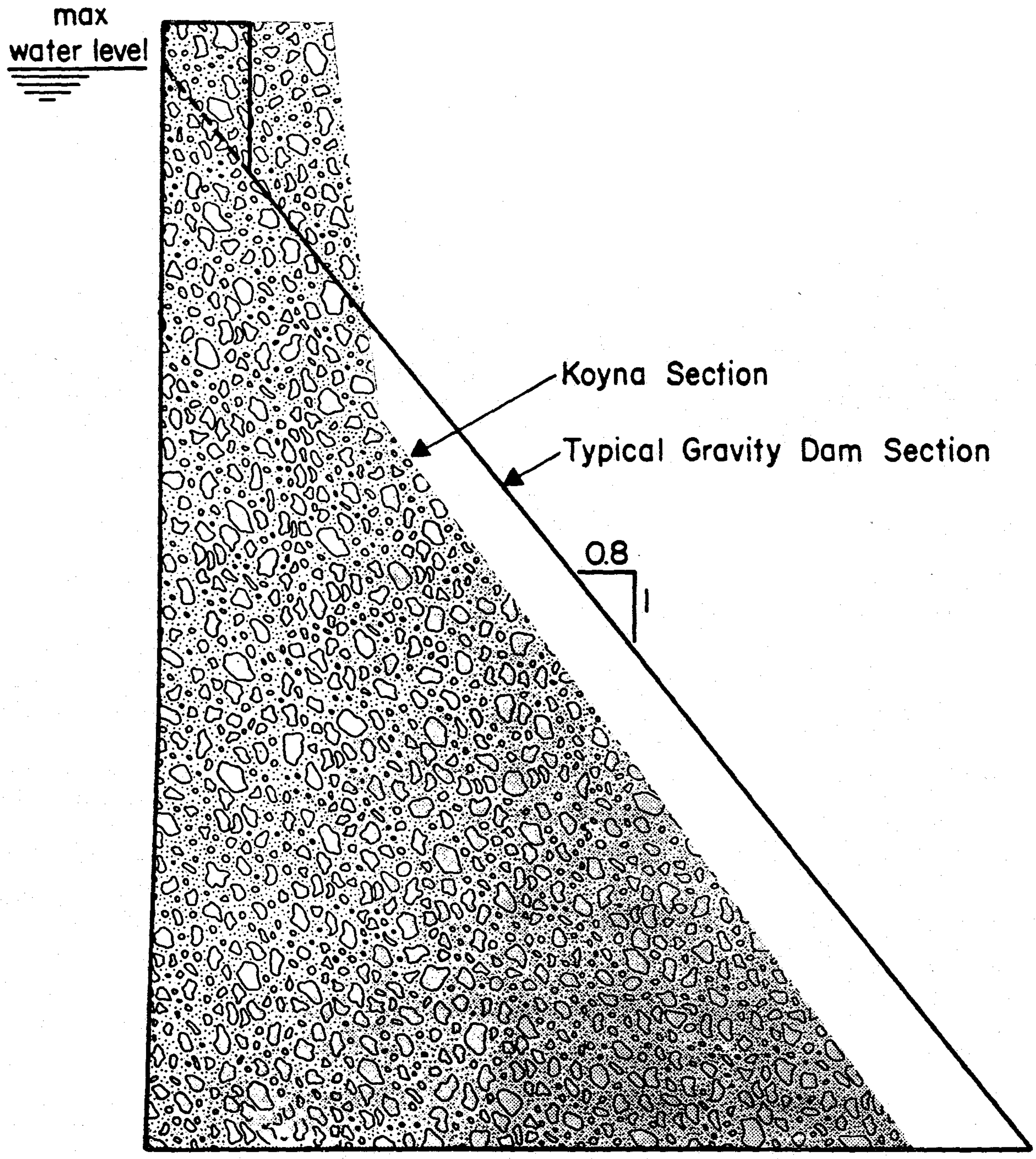
PLAN AND ELEVATION
Koyna Dam
Figure 1



Non Over Flow Section
Foundation K.R.L. 1842 to 1900
Monolith Nos. 15 to 17, 18/2

Over Flow Section
Monolith Nos. 18/2, 19 to 23, 24/2

SECTIONS
Koyna Dam
Figure 2



KOYNA AND TYPICAL GRAVITY DAM SECTIONS
Koyna Dam
Figure 3

more vulnerable to earthquake damage than a typical section, but this intuition has not been supported by dynamic analysis (Chopra and Chakrabarti, 1972).

THE DECEMBER 11, 1967 EARTHQUAKE

Some twenty earthquakes strong enough to have been felt have occurred from 1954 to 1967 in the western margin of the Indian peninsula, which includes the Bombay-Poona-Koyna region; fewer earthquakes have been reported in other parts of the Indian peninsula. Despite these earthquakes, the Indian peninsula was widely considered to be stable and nearly aseismic. This was reflected in the 1967 Seismic Zoning Map of India, in which zone 0 of minimum risk was assigned to almost the entire Indian peninsula.

After the Koyna reservoir started filling during the monsoons of 1962, there were frequent reports of small earthquakes in the area, especially near the dam site. More than one hundred earthquakes of magnitude ranging from 1.5 to 3.5 were recorded during the three-year period beginning September 1964. These earthquakes originated at focal depths of about two miles. Two relatively large shocks occurred on September 13, 1967. Their magnitudes were reported to be in the range of 5.0 to 5.5, the epicenters were in the vicinity of the dam, and their focal depth was estimated to be in the range of two to six miles. These shocks were felt over a radius of about seventy-five miles, causing some damage to poorly constructed buildings and creating fissures in soft soil in Koynanagar. A major earthquake ($M = 6.5$) occurred on December 11, 1967 (December 10, GMT). The epicenter was within eight miles of the dam, and the focal depth was estimated in the range of five to thirteen miles. This earthquake was felt over a radius of 375 miles. It demolished much of Koynanagar, affected the power plant, and also caused structural damage to Koyna Dam. About 180 people were killed and 2,200 were injured. The earthquake of December 11, 1967 is probably the largest earthquake known to have occurred on the Indian peninsula.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

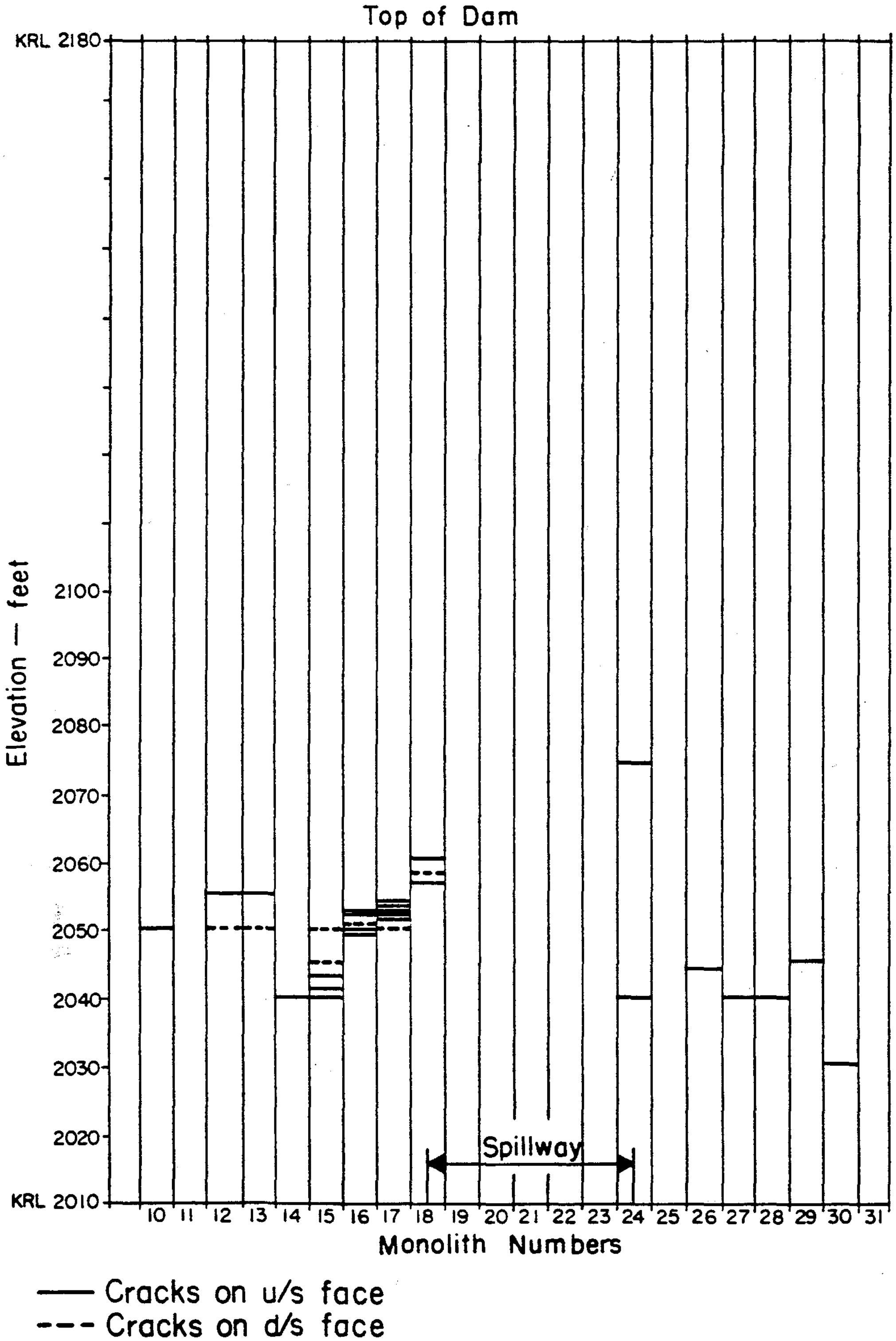
Koyna Dam

The most important structural damage to the dam was horizontal cracks on either the upstream or the downstream face or on both faces of a number of monoliths. On the downstream face of monoliths 13 through 18 and 25 through 30, the level at which the slope of the downstream face changes abruptly (Figure 2), an approximately horizontal crack developed near KRL 2060. Horizontal cracks were observed between KRL* 2040 and KRL 2084, especially near KRL 2060, on the upstream face of monoliths 10 through 18 and 24 through 30. Monolith 18, which is unsymmetrical with half of it being an overflow section, and the remaining nonoverflow with an elevator tower extending fifty feet above the roadway, suffered the worst cracking. Significant leakage of water was observed on the downstream face of monolith 26 near KRL 2060, and traces of seepage water were observed on monoliths 18, 19, 28,29, and 31.

There was evidence of relative movement between adjacent monoliths: spalling of concrete along the vertical joints between adjacent monoliths, and a considerable increase in seepage through the contraction joints between adjacent monoliths after the earthquake, especially from between monoliths 18 and 19 and also 26 and 27.

The damage caused to Koyna Dam by the earthquake of December 11, 1967 was soon repaired in two ways. First, the major cracks were repaired by injecting epoxy resin. Second, the taller nonoverflow monoliths were prestressed along the height from the roadway down to KRL 1990 which is 70 feet below the major cracks. In view of the increase in seismic activity in the vicinity of Koyna Dam and the weakening of the dam by the December 11, 1967 shock, it was considered necessary to strengthen the entire dam. The nonoverflow monoliths were strengthened by increasing the section over the entire

* KRL = Koyna Reference Level



CRACKS INDUCED BY THE DECEMBER 11, 1967 EARTHQUAKE
Koyna Dam
Figure 4

width of the monolith from the base up to KRL 1970, and providing a buttress of width varying between 20 and 30 feet above this level.

Power Plant

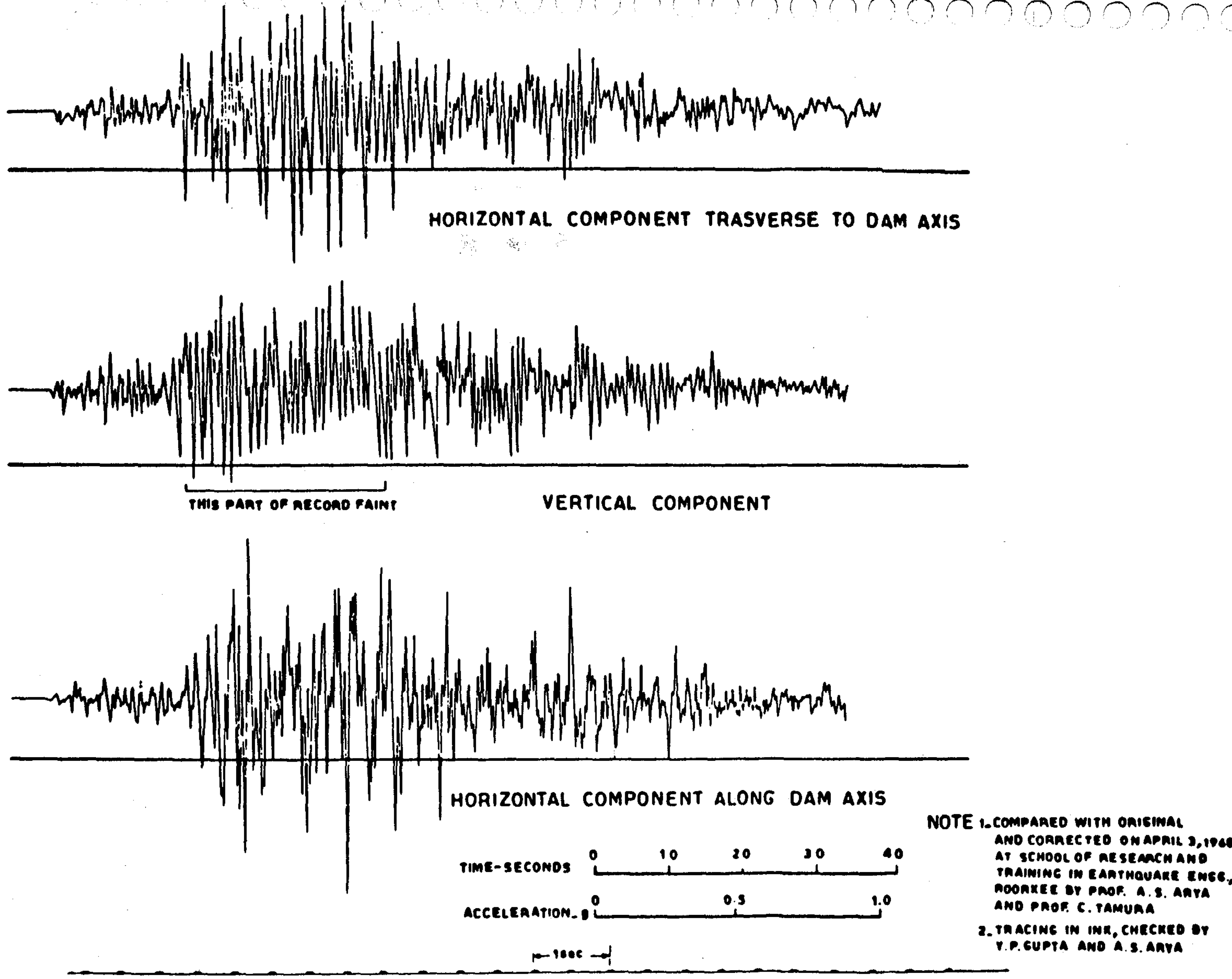
The power plant located about five miles from the dam is underground with several hundred feet of overburden and bedrock above it. Six of the eight generators were operating at the time of the earthquake and they tripped with an indication of overspeed although there was no actual overspeeding. The alignment of the turbines and generators was significantly disturbed and needed adjustment, but there was no damage. A few fine cracks were noticed in the walls of the generator room. Porcelain columns of two bays of switch gear in the outdoor switchyard were broken.

Intake Structure

The intake structure, located three miles from the dam, is a reinforced concrete framed structure rising 214 feet from the invert of the head race tunnel to the level of the approach bridge. The basic structural system consists of three piers and nine columns stiffened by braces. An eleven-span approach bridge connects the intake tower to the shore: the last span rests on the central pier of the intake tower. The intake tower was designed for dead, live, and wind loads and temperature effects, but apparently not for earthquake forces. Except for a few cracks at the junction of the tower and the approach bridge, and some fine cracks in the panel walls where they join to the frame members, the intake tower and bridge were essentially undamaged.

INSTRUMENTATION AND STRONG-MOTION RECORDS

Two AR-240 strong-motion accelerographs, one in monolith 1A and the other in monolith 13, existed in the dam at the time of the earthquake. The motions recorded by the accelerograph located in a gallery of monolith 1A are shown in Figure 5, but the other



ACCELEROGRAMS RECORDED AT BLOCK 1-A, DECEMBER 11, 1967 EARTHQUAKE
Koyna Dam
Figure 5

accelerograph failed to function. The peak accelerations were 0.63g in the longitudinal direction, 0.49g in the transverse direction, and 0.34g in the vertical direction. The recorded ground motion is especially intense in high frequency components, which are especially damaging to short-period structures such as Koyna Dam.

Although there were significant changes between the readings of some stress meters before and after the earthquake, it is not obvious that they were caused by the earthquake, because they seemed to be of the same order as in the long-term stress curves (UNESCO Committee of Experts, 1968). There appeared to be a sudden change in the deflections of the dam caused by the earthquake, e.g. the deflection of monolith 22 changed by about 0.2 feet in the downstream direction during the period from December 7 to 16, 1967. This change in deflection seems to be a consequence of elastic deformations in the monolith and not the result of any rigid body rotation about the base or foundation settlement. Cores drilled from the foundation gallery into the foundation rock through the concrete indicate that the contact between rock and concrete had not suffered any distress. No important changes in the uplift pressures seemed to have been caused by the earthquake. Contraction joints between monoliths had opened at the lower elevations, especially between monoliths 26 through 29, and closed near the top of the dam.

CONCLUSION

The December 11, 1967 Koyna earthquake induced intense shaking and damage to Koyna Dam. Although the dam did not appear to be in danger of failure, the damage was serious enough to result in lowering of the reservoir for inspection and repairs, and to require permanent strengthening. The earthquake experience at Koyna Dam, which represents the most significant information on the performance of gravity dams during earthquakes, provided much impetus for research on this problem during the 1970s.

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LOWER VAN NORMAN DAM, CALIFORNIA, USA

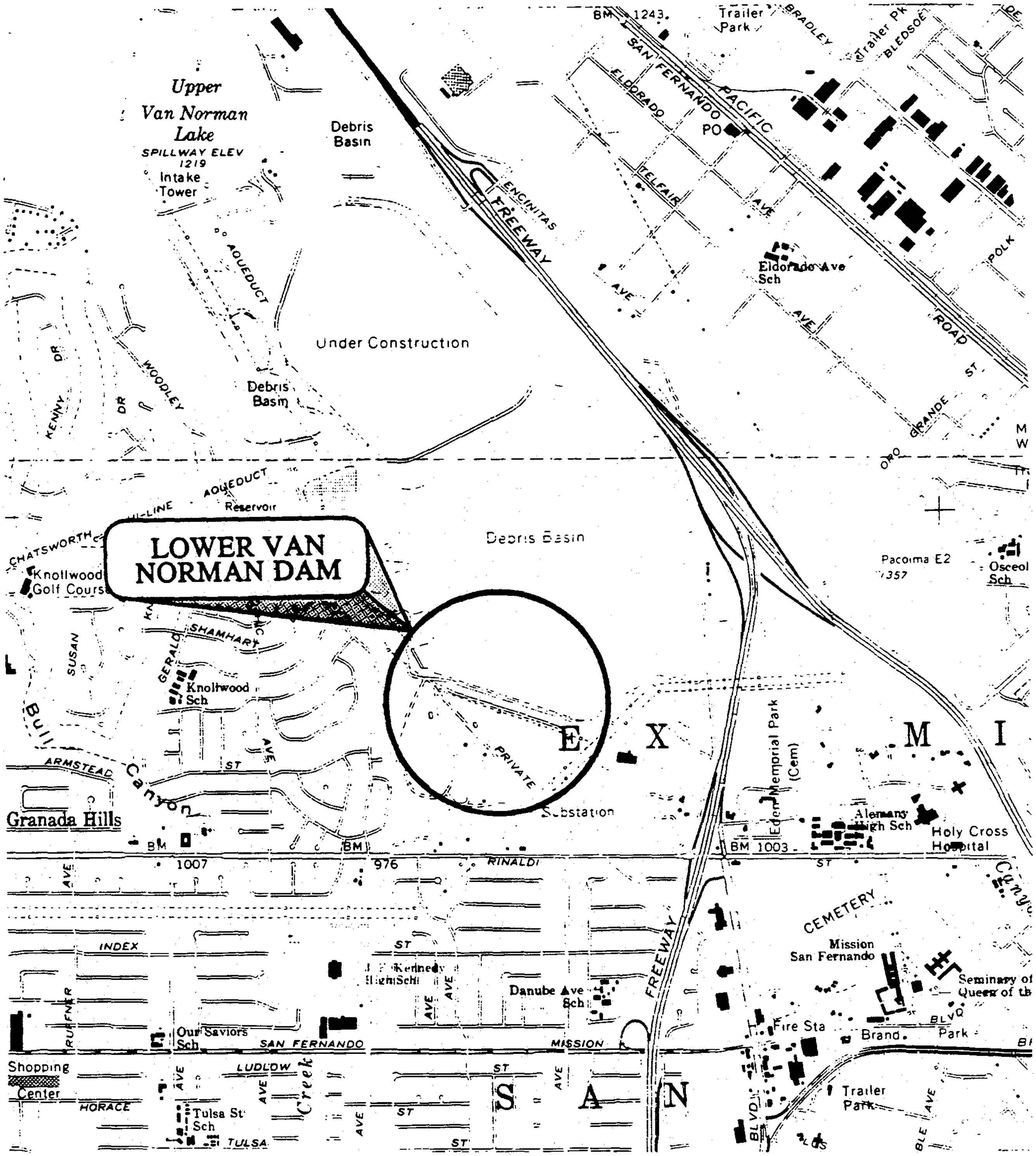
The Lower Van Norman Dam in California, also known as the Lower San Fernando Dam, developed a major slide in its upstream slope and crest as a result of the February 9, 1971 San Fernando earthquake. The liquefaction-induced slide nearly caused a major uncontrolled release of the reservoir. Because of the magnitude of the slide and strong interest of the engineering profession in the evaluation of the seismic stability of earth dams, the Lower Van Norman Dam slide has received considerable attention, beginning with detailed studies immediately following the event. Recent extensive re-evaluations were performed under the sponsorship of the U.S. Army Corps of Engineers (Castro et al, 1989; Seed et al, 1989; Vasquez-Herrera et al, 1989).

LOWER VAN NORMAN DAM

The Lower Van Norman Dam is located in San Fernando, California (Figure 1). A cross-section through the Lower Van Norman Dam showing the major sections of the embankment prior to the 1971 failure is shown on Figure 2. All elevations in this report refer to the National Geodetic Vertical Datum (NGVD).

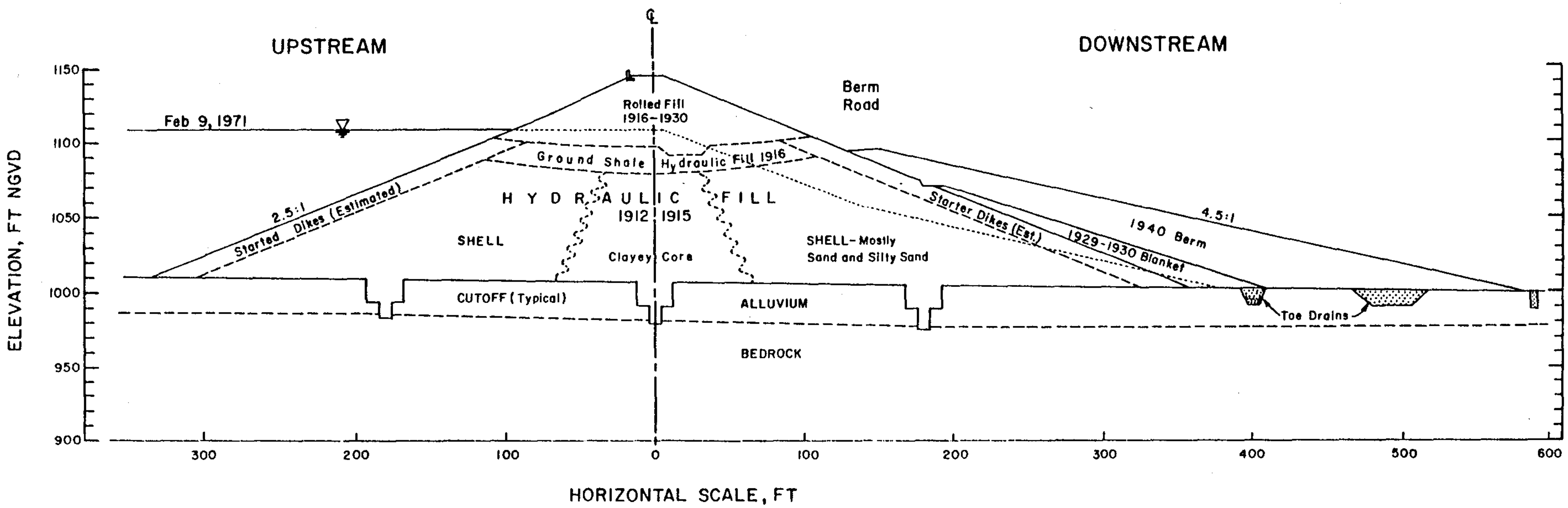
Embankment construction was started in 1912. The embankment was founded on alluvium consisting primarily of stiff clay with lenses of sand and gravel. Old drawings of the dam show three clay-filled cutoff trenches extending through the alluvium to bedrock.

The majority of the embankment consisted of hydraulic fill placed between 1912 and 1915. This material was sluiced from the floor of the reservoir and discharged from starter dikes on the upstream and downstream edges of the embankment. The actual dimensions of the starter dikes are unknown, and therefore, the dimensions shown on Figure 2 are estimates, based on typical hydraulic fill construction practice. The hydraulic fill process



From USGS Quadrangle Map of
San Fernando, California
Photo revised 1972
(Post 1971 Failure)

LOCATION
Lower Van Norman Dam
Figure 1



Based, in part, on
LADWP Drawing, Feb 1969.

SECTION
Lower Van Norman Dam
Figure 2

resulted in upstream and downstream shells consisting of sands and silts and a central core consisting of clayey soil.

Construction photos of the hydraulic fill placement contained in historical records and past reports indicate that the upstream and downstream sections were raised symmetrically and constructed in a similar manner. Therefore, it is reasonable to assume that the general layering of the upstream hydraulic fill shell is similar to that of the downstream hydraulic fill shell.

A 10- to 15-foot-thick hydraulic fill layer consisting of "ground-up" shale from the left abutment was placed in 1916 over the hydraulic fill described above. Records indicate that the maximum size of the ground shale was about 3 inches. In 1985, limited sampling of the ground shale disclosed a broadly graded sand and silty sand.

The embankment was raised a number of times between 1916 and 1930 by placement of rolled fills. The maximum height of the embankment of about 135 feet was reached in 1930. A thin blanket was placed on the lower part of the downstream slope in 1929 and 1930, apparently for seepage control and to provide additional stability due to the raising of the crest. The composition of the blanket was described in a post-construction report as a mixture of shale and gravelly material, placed in 12-inch layers and compacted by trucks.

The final addition to the dam was a 4.5H:1V berm placed on the downstream slope in 1940. Construction records related to the composition of the berm could not be found, but it has been described in early reports as a rolled fill. A photograph of the construction operation shows a roller traveling on the fill.

THE FEBRUARY 9, 1971 SAN FERNANDO EARTHQUAKE

The destructive earthquake (M 6.5) that affected the northern metropolitan area of Los Angeles and the San Fernando Valley on February 9, 1971, was centered approximately

11.2 km from Lower Van Norman Dam. The San Fernando earthquake was generated by thrust faulting along a fault which had received little attention from geologists, and was not shown on most geologic maps. That fault is now considered to be part of the Sierra Madre fault system, which runs along the southern base of the San Gabriel and Sierra Madre Mountains. The San Fernando Valley lies on a portion of the earth's crust that is thrusting under the San Gabriel Mountains. The San Fernando earthquake was caused by a sudden readjustment of the San Gabriel block, in response to pressures associated with seismic straining along the San Andreas Fault "big bend" region, north of the source of the San Fernando earthquake.

Surface fault ruptures were associated with the San Fernando earthquake. Relative displacements along the causative fault plane were estimated to be about five to six feet vertically, accompanied by about five to six feet of horizontal left-lateral slip.

The earthquake was centered on the northern edge of a metropolitan area of over eight million inhabitants and caused severe damage to several large and costly public works, the Lower and Upper Van Norman Dams, highway structures and multistory buildings. It resulted in 58 deaths and about $511 million in damage (in 1971 dollars).

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

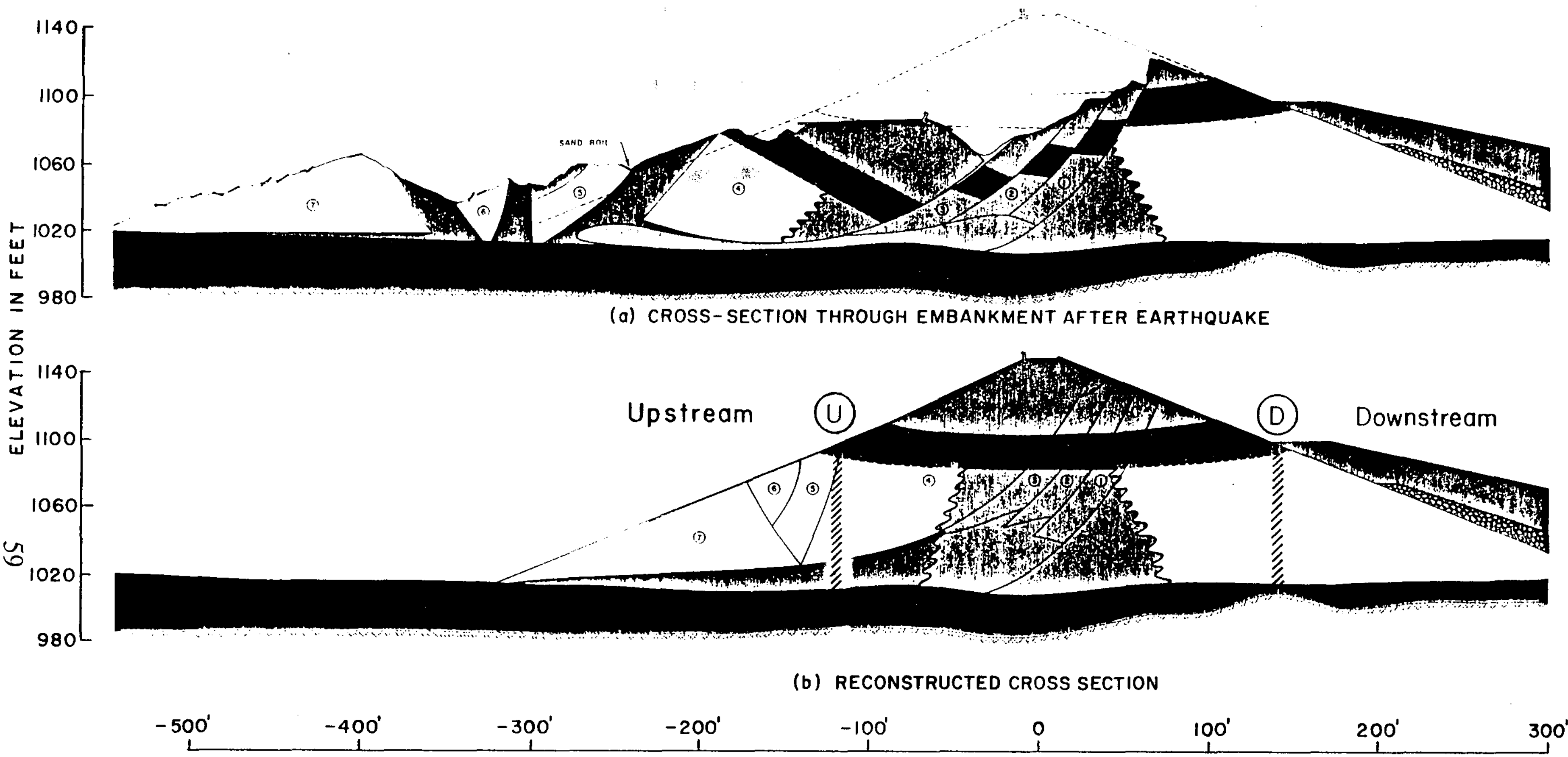
A major slide of the upstream slope and crest of the Lower Van Norman Dam occurred within about a minute after the February 9, 1971 San Fernando earthquake. Also, the two reinforced concrete outlet towers that controlled water release from the Van Norman Reservoir and their access foot bridges were lost. Lowering of the water to safe levels was accomplished through the damaged towers and by bringing emergency pumps to the shoreline.

Early extensive investigations of the slide were performed and reported by Seed et al, 1973; Seed et al, 1975a; Seed et al, 1975b; and Lee et al, 1975. The field investigations showed that the liquefaction slide occurred through the lower part of the upstream hydraulic fill shell. Seed et al, 1973, presented three reconstructed cross-sections of failed portions of the dam based on the logging of a large exploratory trench excavated through the slide area, boring data, and surficial mapping. All three cross-sections indicated that the "liquefied" zone was triangular in shape with its base at or near the bottom of the hydraulic fill. One of these reconstructed cross-sections is presented on Figure 3. The upper part of Figure 3 shows that large blocks of essentially intact soil from the upstream section of the dam moved into the reservoir, riding over the liquefied soil. After movement stopped, the liquefied soil was found to have extruded upwards, between the intact blocks, and to have flowed as far as 250 feet from the toe of the dam. The block of soil which contained the toe of the dam moved about 150 feet into the reservoir.

The downstream shell of the embankment developed settlements and horizontal displacements of up to about one foot but remained essentially intact after the earthquake. Figures 4 and 5 show photographs taken shortly after the earthquake.

A recent exploration program of the downstream shell (Castro et al, 1989) revealed a relatively loose very silty fine sand layer about 15 feet thick at the base of the hydraulic fill shell. This finding is consistent with field observations in trenches and borings made on the upstream side of the dam soon after the 1971 failure (Seed et al, 1973) which showed that the slide occurred through a zone of soil at the base of the upstream hydraulic fill shell. Observations in 1971 indicated that very large strains occurred in this zone.

Construction records of the dam indicate that the same borrow areas and similar construction methods were used for both the upstream and downstream shells, and therefore, it is reasonable to assume that the upstream and downstream hydraulic fill shells were similar in composition. The 15-foot-thick layer of soil found at the base of the downstream shell is the critical layer of the dam from a liquefaction standpoint.

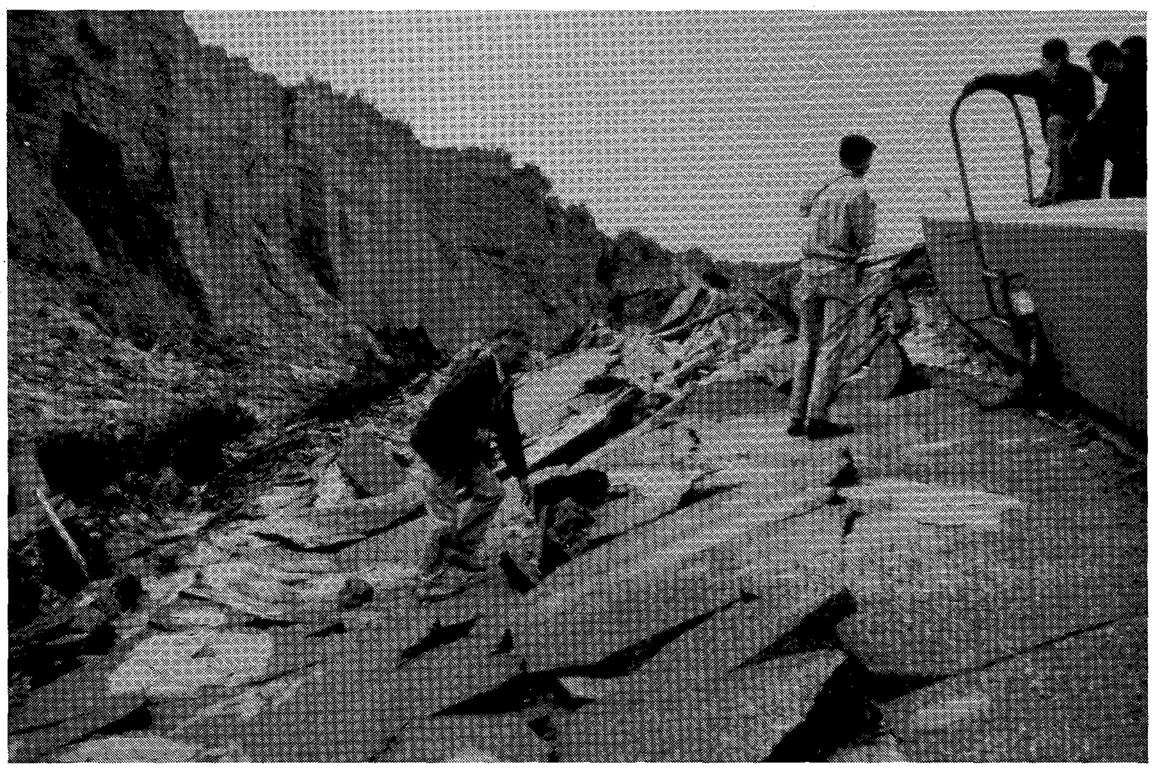


Existing hydraulic fill at Location (D)
represents a mirror image of failed
section at Location (U)

Cross section after Seed, 1973

FAILURE AND RECONSTRUCTED SECTIONS
Lower Van Norman Dam
Figure 3

FEBRUARY 9, 1971, FAILURE
LOOKING WEST
Lower Van Norman Dam
Figure 4



FEBRUARY 9, 1971, FAILURE
COLLAPSED CREST OF DAM
Lower Van Norman Dam
Figure 5

The results of a recent investigation based on steady state concepts (Castro et al, 1989) were consistent with the observed behavior, i.e., a) the dam was susceptible to a liquefaction failure in the upstream direction, b) the dam was not susceptible to a liquefaction failure in the downstream direction once the upstream slope had failed; and c) the strains that accumulated during the 1971 earthquake were sufficient to trigger upstream liquefaction failure.

INSTRUMENTATION AND STRONG-MOTION RECORDS

Seismoscopes located on the bedrock abutment and crest of the dam provided records that were analyzed to estimate earthquake motions at the site. Earthquake motions recorded in the abutment seismoscope had a peak acceleration of about 0.55 to 0.6 g (Seed et al, 1973). Interpretation of the seismoscope records obtained on the crest indicated peak accelerations of the crest of about 0.55 g (Seed, et al, 1973). The seismoscope record from the crest was analyzed to obtain the following time history of the embankment motion:

Time	
0	Start of main shock of earthquake.
-14 sec	Strong motion of earthquake completed, slight tilting of dam crest.
-40 sec	Start of slide movement at crest of dam.
-90 sec	End of main slide movement.

Hence, slide movements of the crest were shown to have started about 26 seconds after the earthquake shaking stopped, and the slide duration was about 50 seconds. Thus, the large slide movements developed in the absence of earthquake loads and were driven only by the static stresses from the weight of the materials and increased pore pressures within the embankment. The downstream shell of the embankment developed settlements and horizontal displacements of up to about one foot, but remained essentially intact after the earthquake.

CONCLUSION

The near-failure of the Lower Van Norman Dam narrowly avoided becoming a catastrophe of unprecedented dimensions in the United States. Had the reservoir been at its normal (and five-foot higher) water level at the time of the earthquake, the 70,000 people that lived immediately downstream of the Lower Van Norman Dam would not have been evacuated in time. This experience has become a most significant event regarding the seismic resistance of earthfill dams, because: (1) it confirmed the high vulnerability of some embankments built of loose and sandy hydraulic fill; (2) it led to significant progress being made in developing methods of numerical analysis of dams; and (3) it triggered the implementation of a systematic re-evaluation program of existing dams in California, which has been conducted since 1971 under the jurisdiction of the California Department of Water Resources, Division of Safety of Dams.

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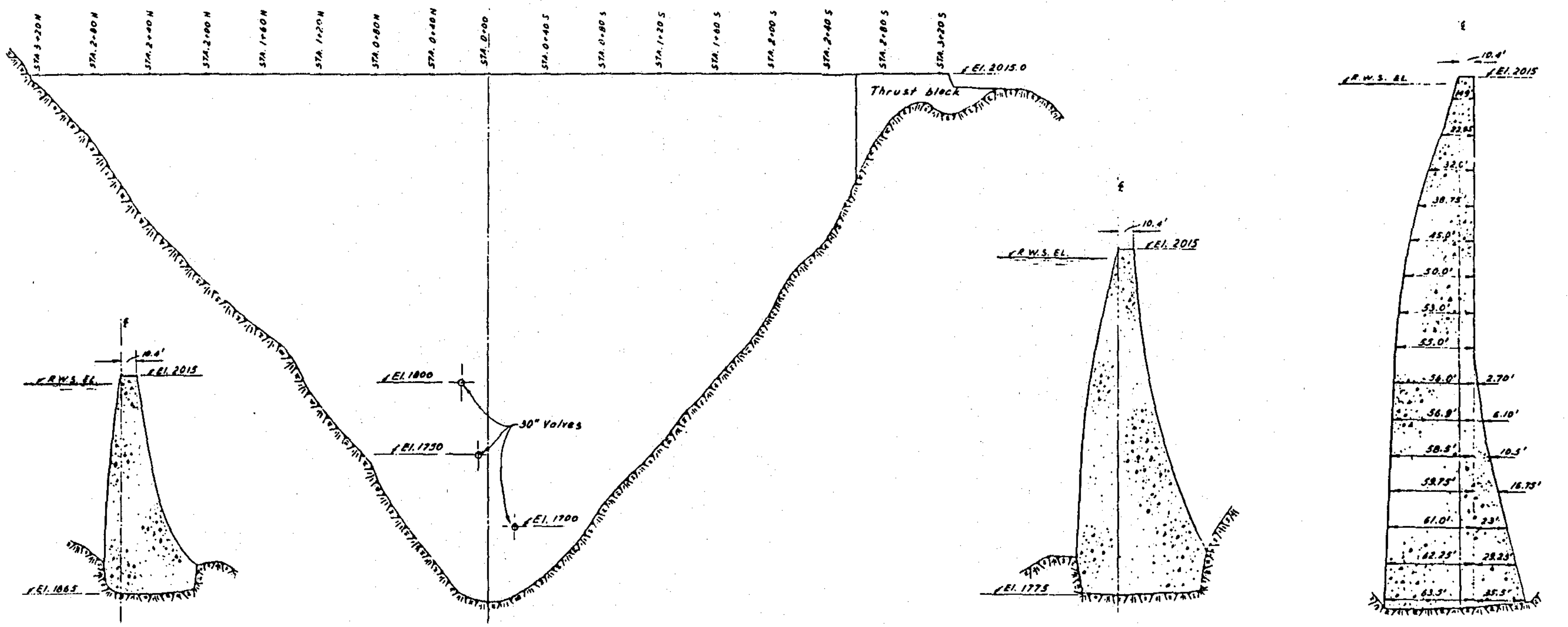
PACOIMA DAM, CALIFORNIA, USA

Pacoima Dam, owned by the Los Angeles County Flood Control District and operated by the Los Angeles County Department of Public Works, is a 372-foot high concrete arch structure, located near San Fernando, California. On February 9, 1971, the dam was severely shaken by an earthquake of magnitude 6.5, one of the most destructive events recorded in that area of California. The concrete arch structure survived undamaged, except for slight tilting and chord shortening. The shaking, however, resulted in partial opening of the joint between the arch and the left thrust block, and caused cracking in the thrust block and downward movement of the rock mass in the upper portion of the left abutment ridge.

PACOIMA DAM

Pacoima Dam is a concrete arch structure constructed across Pacoima Creek, in the San Gabriel Mountains, about 5 miles from San Fernando, California. The dam, completed in 1929, is 372-foot high, has a crest length of 589 feet, and impounds a reservoir of 10,000 acre-feet capacity. The primary functions of the dam are flood control and water conservation. Flood flows are discharged through several outlets in the dam and through a tunnel spillway driven through the left abutment.

The dam was designed as a "constant angle" arch, equivalent to a medium-thick arch dam of moderate double curvature. The thickness at the arch crown is 99 feet at the base and 10.4 feet at the crest (see Figure 1). Because of unfavorable foundation conditions at the left abutment, the dam was built 10 feet lower than originally planned, and a low gravity thrust block was provided at the left abutment to sustain the arch thrust and provide weight for adequate abutment stability against sliding. The dam was designed for full reservoir load only. Earthquake loads were not considered. Vertical contraction joints with keys were



SECTION AT STA. 2+00 N
Scale 1" = 40'

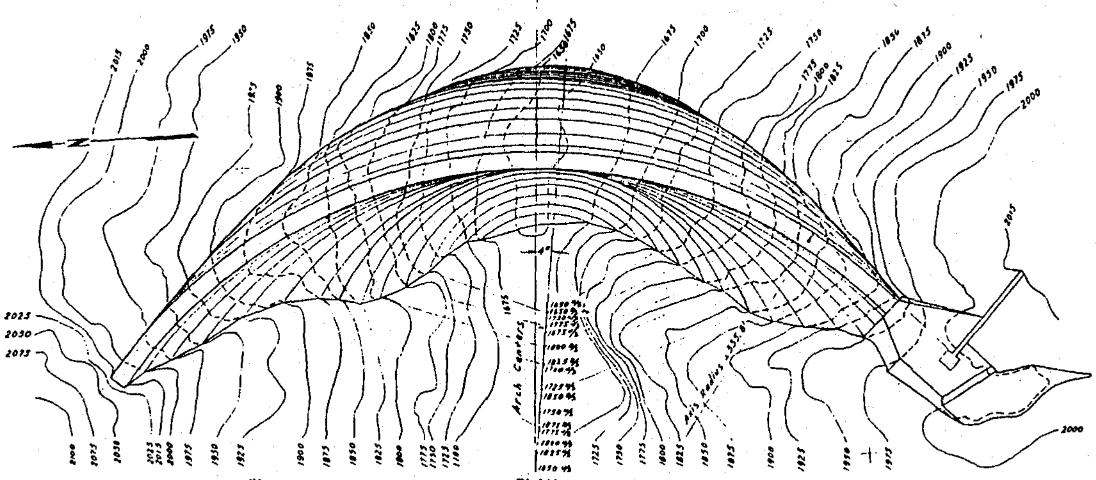
DEVELOPED PROFILE
Scale 1" = 40'

SECTION AT STA. 1+40 S
Scale 1" = 40'

SECTION AT STA. 0+00
Scale 1" = 40'

ELEV.	U/S RAD.	D/S RAD.
2015	335.80	325.40
2000	329.35	314.40
1975	314.75	291.90
1950	298.00	266.00
1925	284.21	238.84
1900	270.40	200.40
1875	257.00	167.00
1850	247.57	145.00
1825	239.00	127.00
1800	234.47	111.30
1775	225.00	94.00
1750	216.35	81.5
1725	203.50	75.0
1700	188.50	68.5
1675	167.50	62.5
1650	155.50	56.5

NOTE:
The geometry of the dam and foundation topography is based on the information shown on drawing No. 64-46



PLAN
Scale 1" = 40'

PLAN, PROFILE AND SECTION

Pacoima Dam

Figure 1

provided in the dam at 50-foot intervals. These joints and the joint between the arch and the thrust block were grouted. The concrete, containing one barrel of cement per cubic yard and with a specified 28-day compressive strength of 2,600 psi, was placed in 5-foot lifts. Consolidation and curtain grouting of the foundation was provided, but no provision was made for drainage of the foundation.

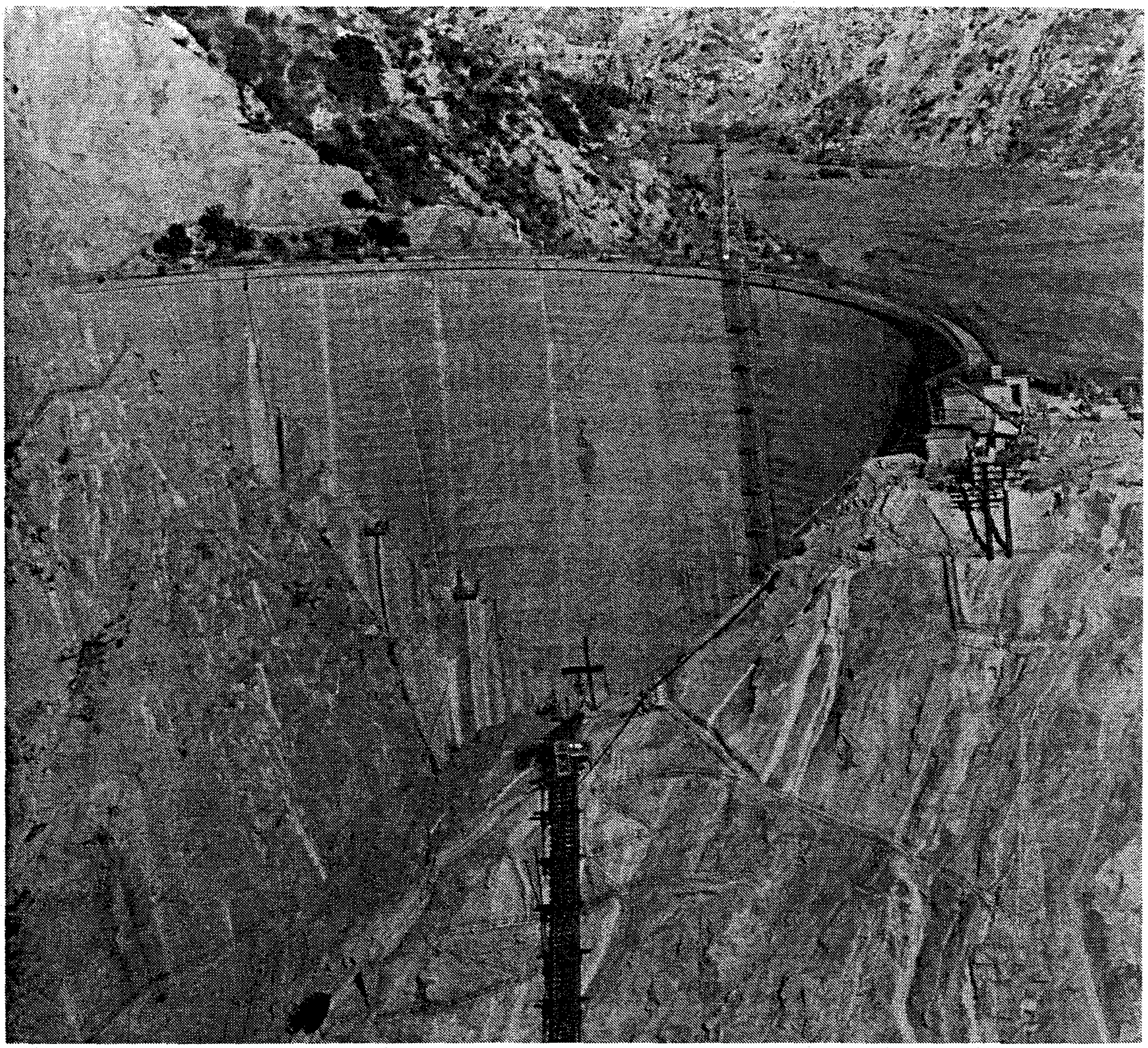
The dam (Figure 2) is located in a narrow steep-walled canyon cutting through the predominantly metamorphic-gneissic formations of the San Gabriel Mountains, which are part of the Transverse Range Geomorphic Province of California. The predominant rock type at the dam site is gneissic quartz diorite. A combined pattern of joints and shears divides the local rock into angular blocks that seldom exceed 4 feet in maximum dimension, and gives the canyon walls a blocky and seamy appearance. A prominent joint system is present in the left (south) abutment. Seven significant faults are located within an 8-mile radius from the dam.

In 1967-1968, a major restudy of the physical condition and structural stability of the dam and its foundation, including detailed three-dimensional trial load stress analysis, concluded that the dam would safely sustain design loads, including 0.15g pseudostatic earthquake loading. However, the stability of the left abutment rock mass was considered to be marginal.

THE FEBRUARY 7, 1971 SAN FERNANDO EARTHQUAKE

Before 1971, little seismic activity had been recorded in the general area of the dam site. The nearest earthquake occurred in August 1952 and had a Richter magnitude of 5.0, with the epicenter located 15 miles northeast of the dam.

The destructive earthquake (M 6.5) that affected the northern metropolitan area of Los Angeles and the San Fernando Valley on February 7, 1971, was centered approximately 4 miles north of Pacoima Dam. The focal depth was about 8 miles. The San Fernando



GENERAL VIEW
Pacoima Dam
Figure 2

earthquake was generated by thrust faulting along a fault which had received little attention from geologists, and was not shown on most geologic maps. That fault is now considered to be part of the Sierra Madre fault system, which runs along the southern base of the San Gabriel and Sierra Madre Mountains. The San Fernando Valley lies on a portion of the earth's crust that is thrusting under the San Gabriel Mountains. The San Fernando earthquake was caused by a sudden readjustment of the San Gabriel block, in response to pressures associated with seismic straining along the San Andreas fault "big bend" region, north of the source of San Fernando earthquake.

The surface rupture associated with the San Fernando earthquake was about 5 miles southwest of the dam. The thrust fault on which major movement occurred slopes to the south, passing below Pacoima Dam at a depth of about 3 miles and surfacing along the southern margin of the San Gabriel foothills in the area of Sylmar and San Fernando. The relative displacement across the fault in its upper parts was estimated to be approximately five or six feet, and was accompanied by about five to six feet of horizontal left lateral slip.

The earthquake was centered on the northern edge of a metropolitan area of over eight million inhabitants, and caused severe damage to several large and costly public works, highway structures, multistory buildings, and to the Upper and Lower Van Norman dams. It resulted in 58 deaths and about $511 millions in damage in 1971 dollars. When the earthquake occurred, Pacoima reservoir had been drawn down to store flood waters during the winter rainfall. The reservoir level stood at 145 feet below the dam crest.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Following the earthquake, extensive inspections, field investigations, and analyses were conducted, that revealed the following effects of the earthquake on the dam and appurtenant structures:

Dam and Thrust Block

- The contraction joint between dam and left thrust block partially opened. The opening ranged from 0.25 to 0.38 inch in width and extended from the crest to a depth of 45 feet, where it ended in a horizontal lift joint.

- A crack in the left thrust block extended 5 feet along a lift line and then angled down to abutment rock (see Figure 3).

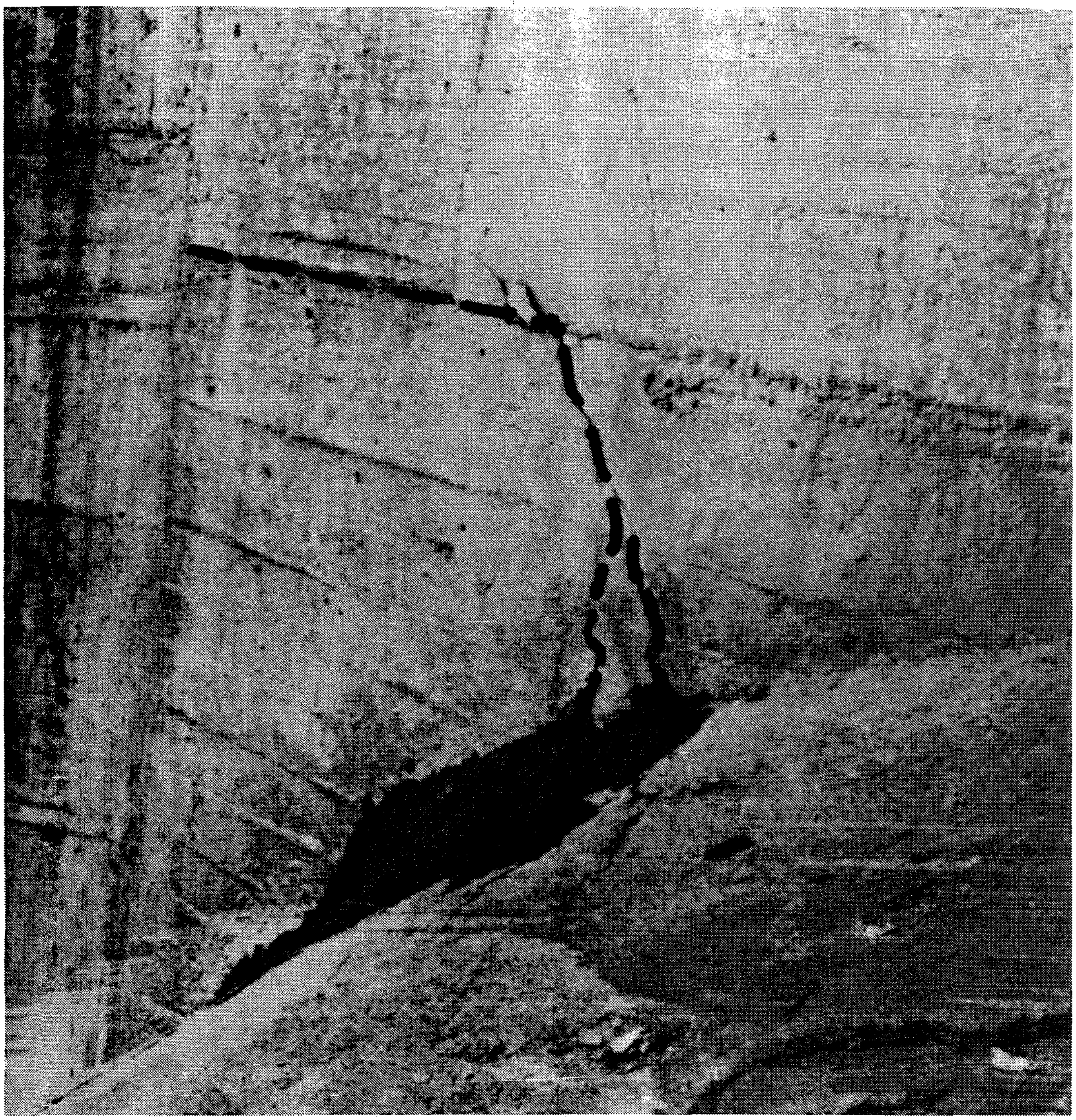
- Permanent slight narrowing of the canyon, due to regional crust adjustment, resulted in chord shortening by 0.94 inch, rotation of the dam axis by 30 seconds and tilting of the dam crest, as the right abutment had dropped 0.68 inch relative to the left abutment.

- No noticeable cracking occurred in the arch dam, nor any relative movement at arch dam-foundation contact. The concrete arch structure thus performed extremely well.

Left Abutment

The upper left abutment was severely affected and suffered extensive gunite cover cracking and slumping of an extensive area on the downstream slope of the abutments.

A major disturbed rock mass consisting of two blocks, namely the lower block (Rock Mass A) and the upper block (Rock Mass B), experienced rearrangement and some downward movement along a sloping failure plane.



CRACK AT LEFT THRUST BLOCK
Pacoima Dam
Figure 3

Spillway Structures

The spillway intake tower suffered slight damage, as well as the spillway chute; cracking occurred at four new locations in the concrete lining of the spillway tunnel.

Seepage Rates and Piezometric Surface

Following the earthquake, seepage rates in drains in both abutments increased, followed by a sudden decrease, but remained higher than pre-earthquake rates for any given reservoir level. Phreatic levels in piezometers increased, and were also followed by an abrupt decline, until they finally stabilized to pre-earthquake levels, suggesting no change in gross abutment permeability. Observed changes were attributed to pressure effects and random rearranging of joint openings and seepage passages in the abutments.

Of immediate importance following the earthquake was the implementation of emergency repairs that would ensure safe operation of the dam for the remainder of the 1970-1971 storm season. Further work was initiated to limit the loads on the dam and the abutments during the 1971-1972 storm season. Final remedial work to repair and rehabilitate the dam and restore full unrestricted service operation included: repair of cracks in gunite cover, stripping of loosened rock from left abutment ridge, installation of abutment relief drains, repair to left abutment grout curtain, consolidation grouting of upper left abutment, repair to spillway tunnel lining, stabilization of rock mass B with post-tensioned steel anchors, repair of joint between dam thrust block and crack in the thrust block.

In 1972, state-of-the-art three-dimensional finite element static and dynamic response analyses of the dam indicated that the arch dam in its post-earthquake condition has adequate seismic structural stability to safely sustain the specified loading combinations, including the Design Basis Earthquake loading, whether or not the open joint between the dam and thrust block and the cracks in the thrust block were repaired. Nevertheless, that joint was grouted and the cracks in the thrust block were repaired prior to restoring the dam to full service.

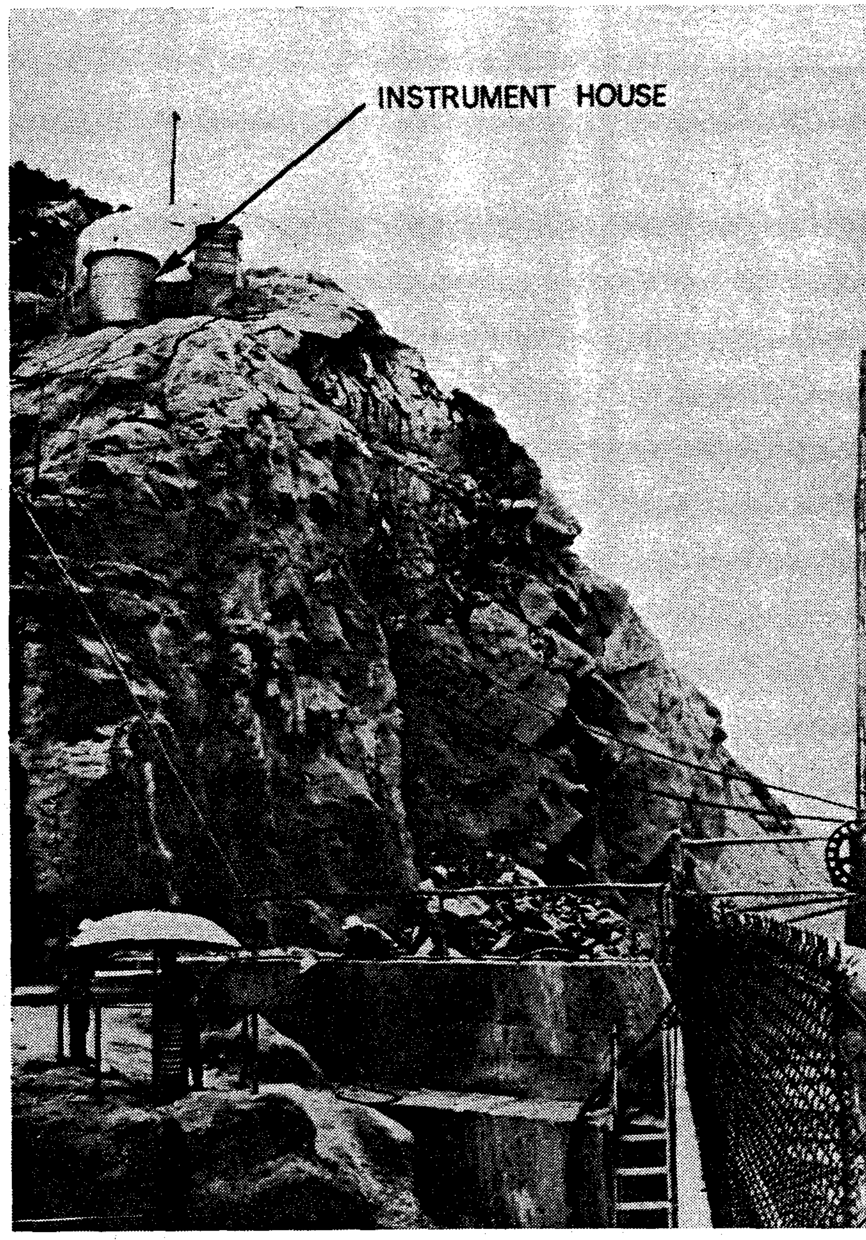
Monitored performance of the rehabilitated dam has confirmed expected satisfactory structural response to normal operating loads.

INSTRUMENTATION AND STRONG-MOTION RECORDS

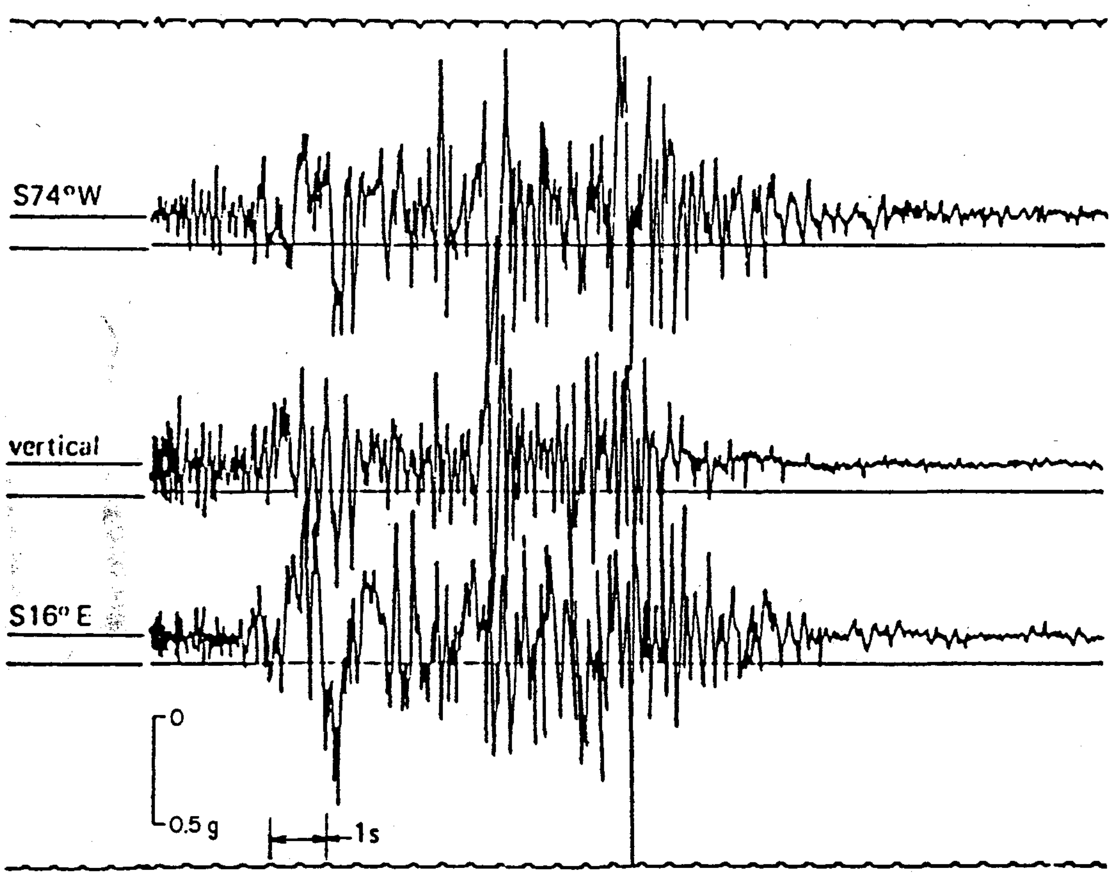
Two strong-motion recording instruments existed on the dam at the time of the earthquake. The first was a Wilmot seismoscope located on the crest of the dam near its center, the other, an AR-240 strong-motion accelerograph located on the rock ridge on the left abutment 120 feet from the arch dam and about 50 feet above its crest (Figure 4). No useable record was obtained from the seismoscope, because the glass record plate was jarred loose during the early part of ground motion. The accelerograph, however, recorded the ground motions (see Figure 5). The peak accelerations obtained then were the highest earthquake accelerations ever recorded, and the first obtained at such a close distance from the epicenter of a significant earthquake generated by thrust faulting. High frequency acceleration peaks of 1.25g for the horizontal components and 0.70g for the vertical components, respectively, were recorded, for a total duration of the strong-motion phase of approximately 8 seconds. However, post-earthquake studies have indicated that these high peak accelerations were probably influenced by the accelerograph's location on the edge of a narrow badly fractured ridge, that may have resulted in unusual amplifications. Subsequent studies suggested that peak horizontal accelerations at the base of the dam might have been in the range of 0.60 to 0.80g.

CONCLUSION

Pacoima Dam, although not originally designed for severe earthquake loading, survived the strong shaking induced by the February 7, 1971, San Fernando earthquake



LEFT ABUTMENT ACCELEROGRAPH
Pacoima Dam
Figure 4



ACCELEROGRAM RECORDED AT LEFT ABUTMENT, FEBRUARY 9, 1971 EARTHQUAKE
Pacoima Dam
Figure 5

without structural distress, thereby attesting to a considerable strength reserve inherent in the arch structure. Modern seismic structural stability and safety evaluations in 1972 and in 1983, using state-of-the-art three-dimensional finite element dynamic response analysis, indicated that the rehabilitated dam should perform satisfactorily if subjected to the Maximum Credible Earthquake postulated for the Sierra Madre Fault segment adjacent to the segment that slipped during the 1971 San Fernando earthquake. Monitoring data indicate that the rehabilitated Pacoima Dam has performed satisfactorily and safely in response to the reservoir loading.

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RAPEL DAM, CHILE

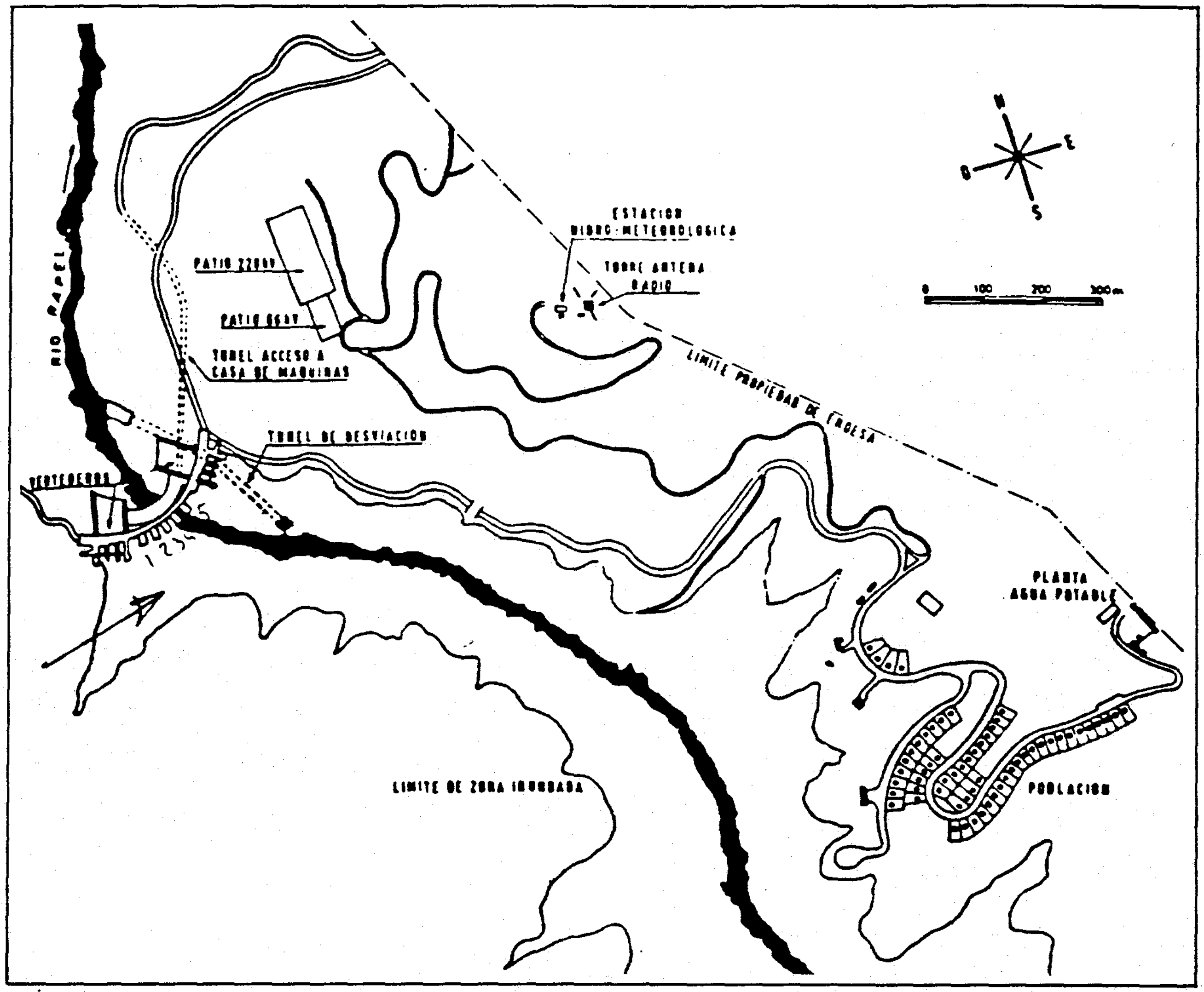
On March 3, 1985, a violent earthquake (M 7.8) affected the central Chilean Coast. Although not as severe as the great 1960 Chile earthquake, the 1985 event resulted in widespread damage inland and along the coast. Rapel Dam, a 364-foot high concrete arch dam, was strongly shaken, but did not suffer any damage. Its appurtenant facilities were, however, somewhat affected and electrical power generation was interrupted for several days.

RAPEL DAM

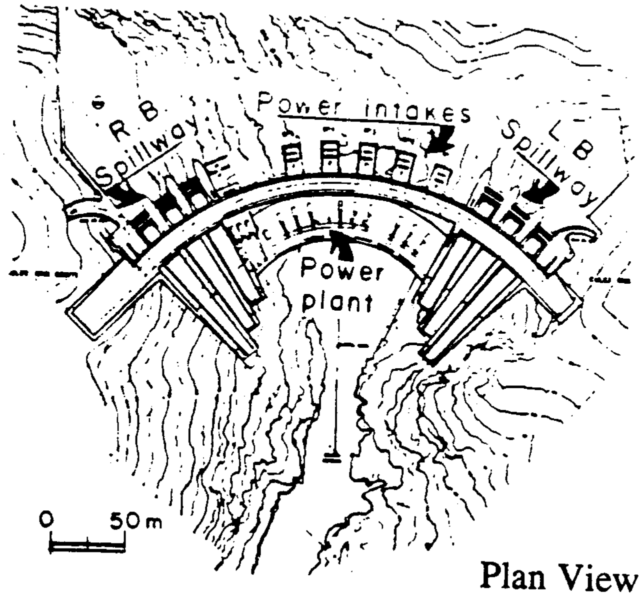
Rapel Dam is a double curvature concrete arch dam with abutment ski jump spillways and power house at the toe (Figures 1 and 3). The reservoir provides head to a 375 MW power plant. The dam is 364 foot-high and has a crest length of 886 feet. Thickness of the arch varies from 18.0 feet at the crest to 62.3 feet at the base, for a total arch wall volume of 340,000 cubic yards (Figure 2). Total volume of the dam, excluding the powerhouse, is 785,000 cubic yards.

Each spillway includes three channels, equipped with 43.3 x 49.2 foot radial gates. Total spillway capacity is 398,800 cfs. Two high head sluices were constructed above the powerplant roof and are controlled by radial gates 9.0 by 14.4 feet in size, with a combined capacity of 24,700 cfs. Low buttresses near the base of the arch protect the powerhouse against high tailwater level during maximum flood.

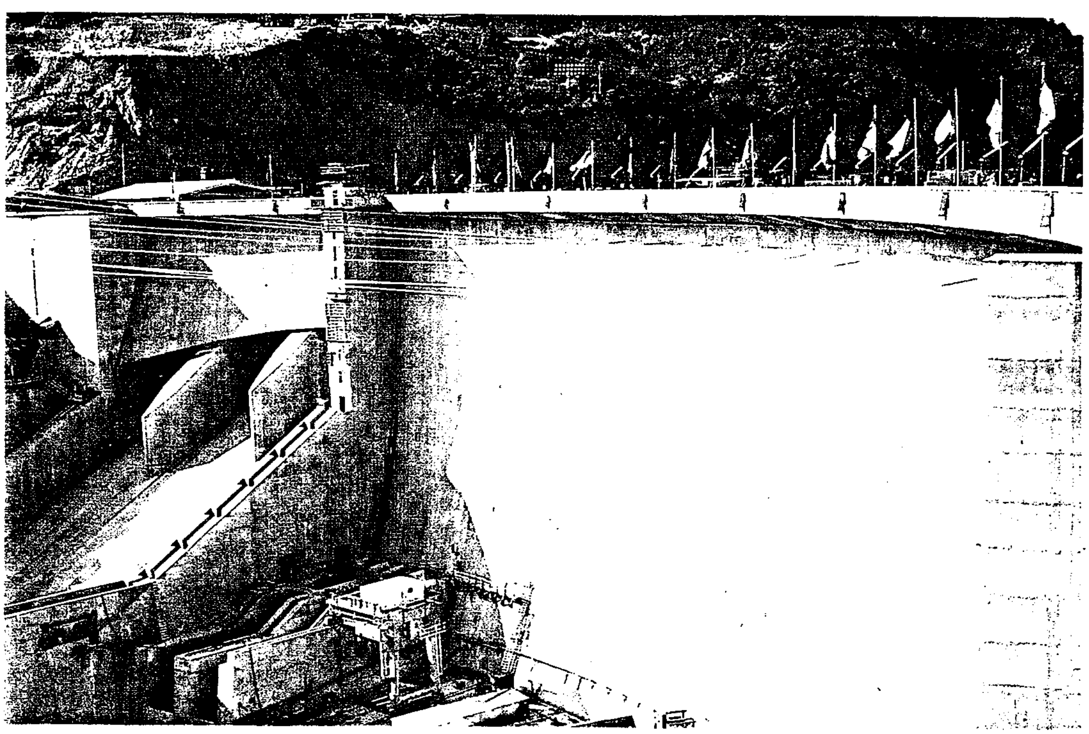
The dam is founded on granite bedrock with intrusive volcanic dikes. A large transverse fault intersects the foundation near the upstream toe and an extensive system of abutment drainage adits was required.



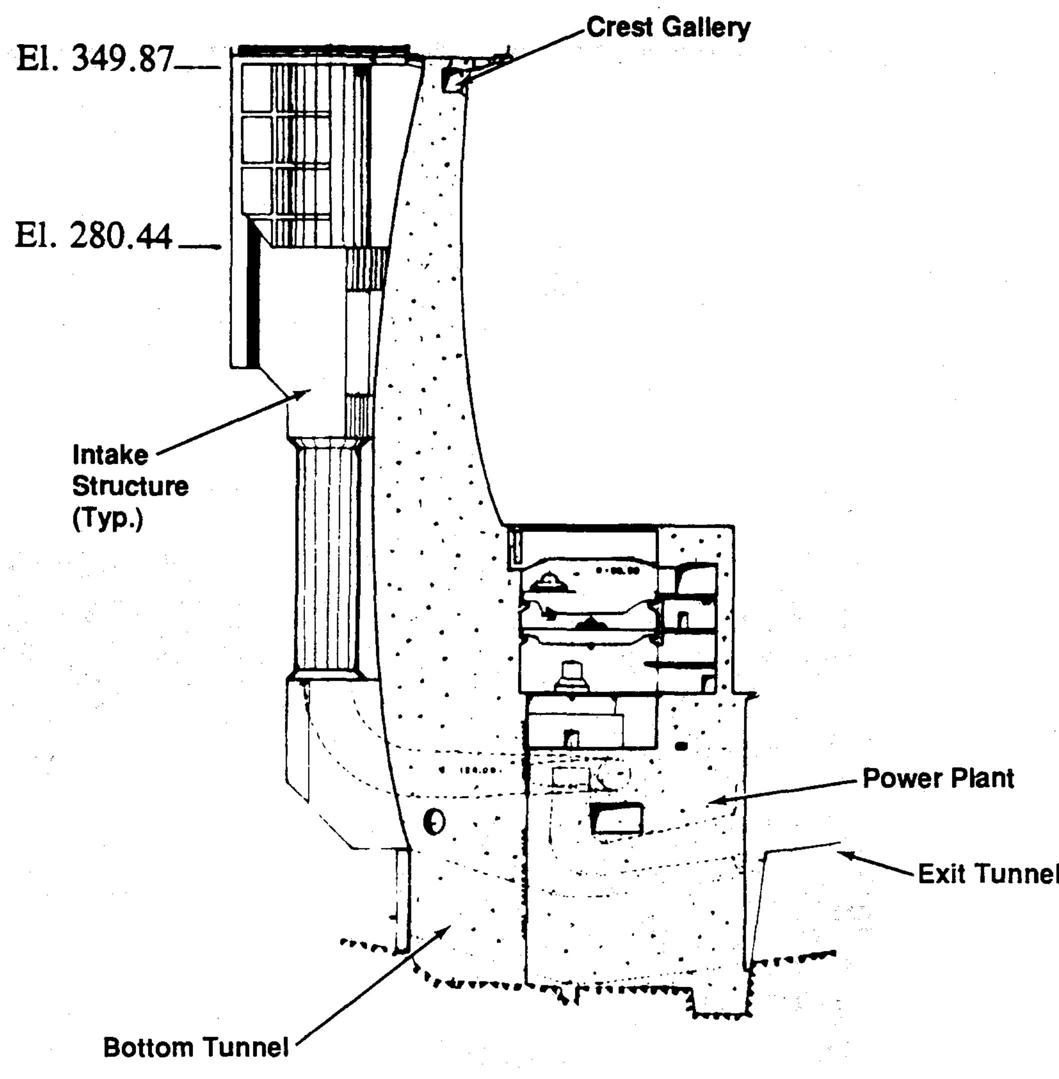
SITE PLAN
Rapel Dam
Figure 1



Plan View



GENERAL VIEW
Rapel Dam
Figure 2



DAM SECTION AT POWER PLANT
Rapel Dam
Figure 3

THE MARCH 3, 1985 EARTHQUAKE

The March 3, 1985 Chilean earthquake (M 7.8) was centered offshore about 35 km northwest of the town of San Antonio. The event occurred at a depth of about 36 km along the subduction (Benioff) zone that underthrusts the Chilean Coast. It was preceded by a foreshock sequence, consisting of over 300 earthquakes, with magnitudes up to 4.5, that began about one month prior to the main shock. The main event and its aftershocks affected an area about 140-km long (north-south) by 70 km-wide (east-west). At least 145 people were killed, nearly 2000 were injured, and extensive damage occurred in Central Chile including the cities of San Antonio, Valparaiso, Vina del Mar, Santiago and Rancagua. Rapel Dam was located about 45 km from the epicenter.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Rapel Dam behaved satisfactorily during the 1985 earthquake. The dam was designed in 1960 using a horizontal seismic load coefficient of 0.12g. The critical load case was found to be a tranverse (U/S-D/S) earthquake motion with the reservoir empty. In order to prevent horizontal arch joint opening as a result of upstream cantilever bending, a curtain of 144 rebars of 1.42-inch size was embedded in the concrete at Elevation 228.84 feet[1]. Although no dynamic analyses were performed before or after the earthquake, comparisons were made with the results of finite element analyses of another dam (Sir Dam, Turkey) of comparable size and geometry. These indicated that Rapel Dam should have performed satisfactorily under seismic loads similar to those induced by the 1985 earthquake.

Several of Rapel Dam appurtenant structures were damaged. Damage primarily affected two areas, the spillway walls and the upper part of the intake towers. The inside face of the spillway walls was cracked on the upstream side. Leakage was observed at wall joints and significant cracking occurred, near the gate in the left wall of Channel 4, located

[1] Rapel Dam Local Datum.

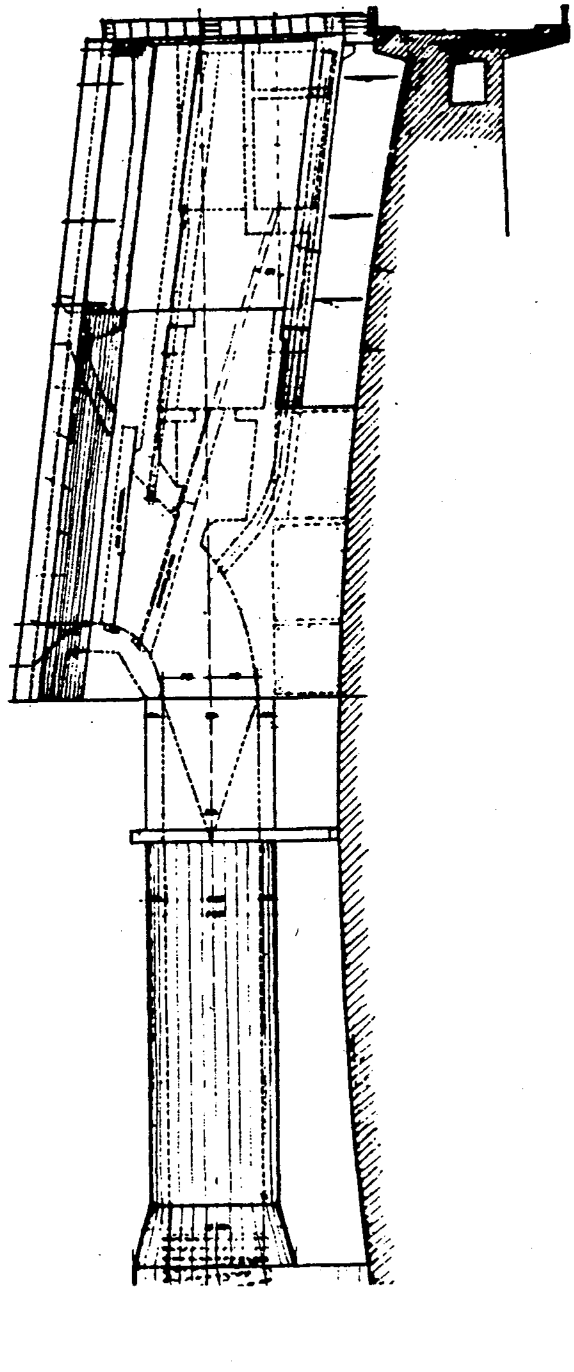
in the right abutment spillway. It was speculated that the large spillway walls may have amplified the (more severe) cross-canyon (east-west) component of the ground motion.

The intake facility consists of five intake structures (numbered I to V from left to right abutment), each structurally independent from the others. They are connected to the dam from their lower elbow level up to Elevation 114.83 feet (Figure 4). From elevations 114.83 feet to 164.04 feet, the intake structures abut against the arch by means of vertical concrete slabs, initially used as construction supports and then disconnected. Four sets of three reinforced concrete beams tie the intake structures to the dam at Elevations 224.74, 239.90, 259.19, and 280.51 feet. Each beam was reinforced with 72 rebars of 1.42-inch diameter. The open space along the vertical slabs was filled with mass concrete at the end of construction and a small horizontal concrete slab, unrestrained horizontally, was placed to provide access from the crest of the dam to the operating platform of the intake structures at Elevation 353.25 feet.

The upper parts of the intake structures cracked immediately above Elevation 280.51 feet where they rejoin the concrete arch. Evidence of relative movement was conspicuous at upper horizontal slab level, where pavement damage occurred. It was concluded, based on the satisfactory performance of the horizontal ties, that the intake structures responded in cantilevered mode above Elevation 280.51 feet, which is also the floor elevation of the inside chambers of the intake structures. Damage was observed at intake structures III, IV and V, the last of these being the most affected. Cracking and concrete spalling occurred along the surface of the vertical slabs between dam and intake structures. Diagonal cracking and leakage were conspicuous along the upstream wall of Intake V. Horizontal cracking was also observed along the side walls of that structure.

INSTRUMENTATION AND STRONG-MOTION RECORDS

Significant records were obtained at the Rapel Dam site. Free-field instruments located near the dam recorded peak horizontal and vertical accelerations of 0.31g and 0.11g,



TYPICAL SECTION - INTAKE STRUCTURES
Rapel Dam
Figure 4

respectively. It was noted, however, that the peak horizontal acceleration occurred in the east-west direction, or more or less parallel to the dam axis. The north-south horizontal component, theoretically more critical to the dam since acting in the transversal direction, was only 0.14g. The dam face and crest were not instrumented.

CONCLUSION

Rapel Dam was subjected to moderate earthquake loading of significant intensity and duration. The satisfactory performance of the dam is consistent with the excellent seismic capacity exhibited to date by concrete arch dams, but the 1985 ground motion was less than could presumably occur at that site. Principal damage occurred at the location where the intake structures become physically separated from the dam, emphasizing the importance of structural drift and possible out-of-phase movements between adjacent structures. Had the intake structures been attached all the way to the crest of the dam, it is likely that less damage, or perhaps none at all, would have occurred.

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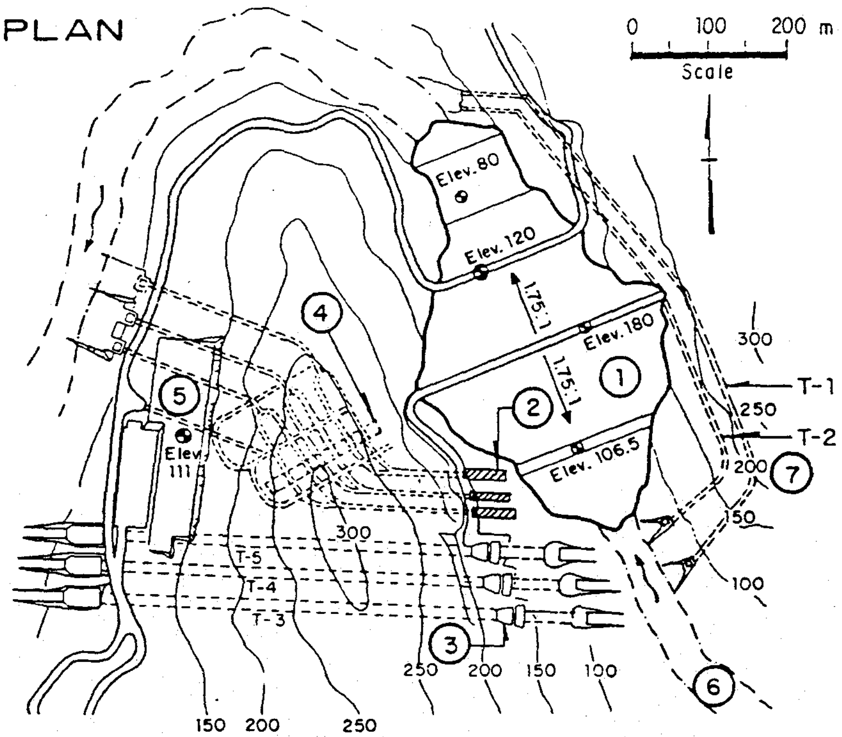
EL INFIERNILLO DAM, MEXICO

On September 19, 1985, a large earthquake (M 8.1) struck the southwestern coast of Mexico. This event resulted in unprecedented damage in Mexico City, located more than 400 km away from the epicenter. It caused perhaps 20,000 deaths in the City and left an estimated 250,000 homeless. El Infiernillo Dam is one of two large embankment dams within 75 km of the epicenter that were affected by the earthquake.

EL INFIERNILLO DAM

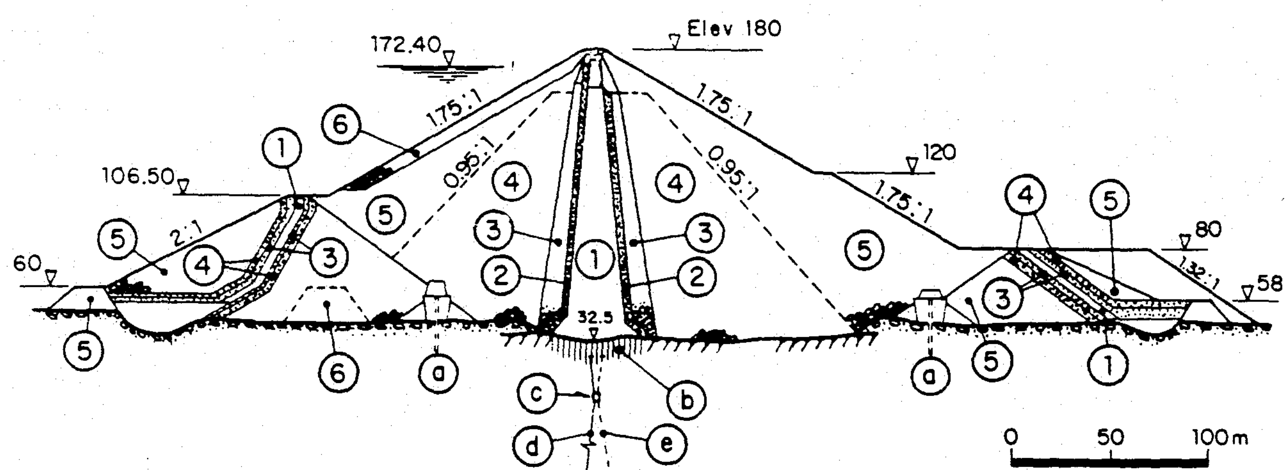
El Infiernillo, an earth core rockfill dam (ECRD), was constructed from 1960 to 1964 on the Balsas River, at about 70 km inland. The principal component of a 1,000 MW hydroelectric development, the dam also provides flood control. The embankment stands 479 feet high and has a crest length of 1,080 feet. It was designed with a thin vertical core of compacted clay, upstream and downstream sand filters and transition zones, and external rockfill shells sloping at 1.75:1 (horizontal to vertical) (Figure 1). The dam has a total volume of 5.5 million cubic meters, 5.0 million of which are granular. The filters, transition zones and inner part of the rockfill shells, which slope at 0.95:1 (h to v), were compacted, but the outer rockfill shells were dumped. Rockfill was processed from three quarries excavated in silicified conglomerate and one in sound diorite. El Infiernillo Dam is founded on breccias and conglomerates, intersected by several basalt dikes. At the time of the 1985 earthquake, construction to raise the crest by 4 m (13 ft) was in progress.

As the dam is located at a meander of the Balsas River, most of its appurtenant facilities were constructed on the left bank (Figure 1). These include three 42.6-foot diameter spillway tunnels with a combined discharge capacity of 362,000 cfs, individually controlled by three radial gates, and three intake tunnels serving the underground power plant. Two other 29.5-foot wide outlet tunnels, originally used to divert the river, pass



(1) Dam
(2) Intake Structures
(3) Spillway Gates
(4) Underground Powerhouse
(5) Switchyard
(6) Balsas River
(7) Contours, in m Above Mean Sea Level

T-1, T-2: Diversion Tunnels
T-3, T-4, T-5: Spillway Tunnels



(1) Impervious Core
(2) Filter Zone
(3) Transition Zone
(4) Compacted Rockfill
(5) Dumped Rockfill
(6) Select Rockfill

(a) Cut-off Wall (Secant Piles)
(b) Grout Blanket
(c) Gallery G-4
(d) Grout Holes
(e) Drainage Holes

LAYOUT AND SECTION
Infiernillo Dam
Figure 1

through the right bank. The powerhouse is 420 feet long by 69 feet wide, and 131 feet high. It is equipped with six Francis turbines with a maximum discharge of 6846 cfs and a design head of 331 feet. Four of these (160 MW) were installed in 1965, and two 180 MW additional units became operational in 1975.

THE SEPTEMBER 19, 1985 EARTHQUAKE

The September 19, 1985 Michocoan, Mexico, earthquake (M 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 Michocoan seismic gap. In this area, subduction is the main tectonic process, the plate contact being delineated by the Mid-American Trench (12km 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 (M 7.5, USGS) further extended the ruptured zone to the southeast on September 21, 1985. The epicenter of the principal shock was located about 75 km from El Infiernillo Dam. The earthquakes of September 19 and 21 produced the most extensive strong motion data sets yet in Mexico.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

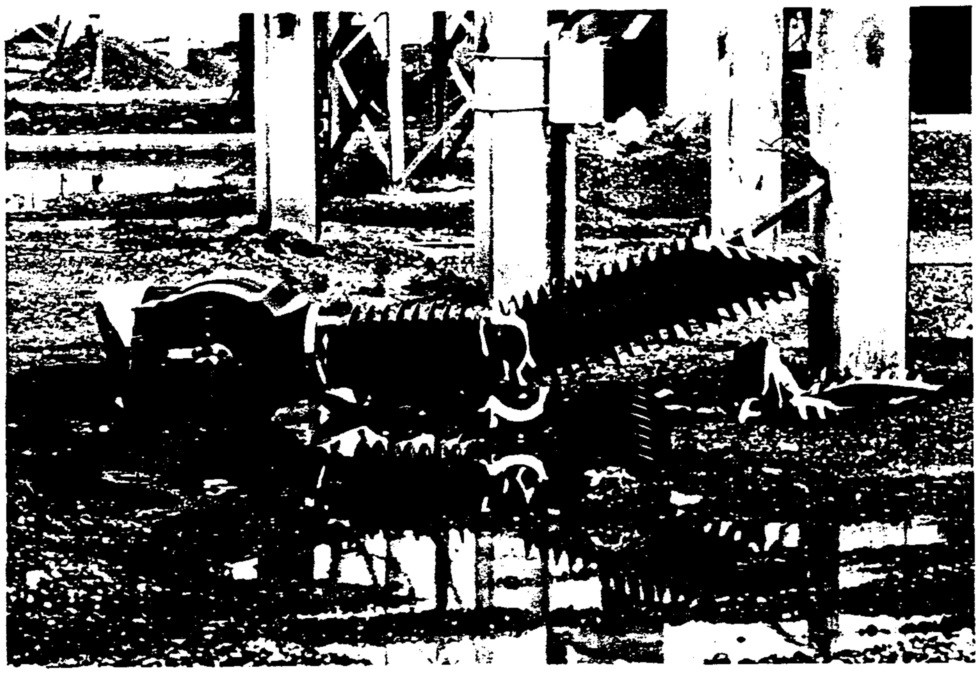
El Infiernillo Dam was subjected to about 60 seconds of strong ground motion during the September 19 earthquake, with probable crest accelerations of the order of 0.50g. Overall, the dam performed extremely well and earthquake effects appeared to be insignificant.

Two longitudinal cracks, 0.08 to 5.9 inches wide, formed on either side of the dam crest along its entire length, or about 1,100 feet. The cracks intersected the base layer of the pavement at the top of the dam, immediately above the interface between the impervious core and the upstream and downstream shells. Additional minor longitudinal cracks, about 30 feet long and with a maximum width of 1.4 inches, formed on the crest along the abutments. Two of these cracks were near the right abutment, and another one near the left abutment. The contractor in charge of raising the dam surveyed the crest immediately after the occurrence of the main shock and found that the embankment had settled a maximum of 3.5 inches. Construction equipment was used to investigate the extent of the principal cracks, which were found to be superficial. The parapets on either side of the crest withstood the earthquake without toppling, but experienced small amplitude horizontal and vertical misalignments, conspicuous to the eye and confirming the occurrence of small amplitude permanent deformations within the embankment.

The powerhouse was unaffected by the earthquake and remained fully operational. It disconnected from the grid for a brief period of time immediately after the earthquake and as a result of protective relay action. No damage, even minor, occurred to the spillway or tunnels, or in any part of the complex network of inspection and dewatering galleries that criss-crosses the left bank. The only damage observed occurred at La Iguana Hill substation, immediately downstream of El Infiernillo Dam, where several out-of-service ceramic insulators were toppled down from their concrete pedestals (Figure 2). Minor rockfalls and rotational slope failures were observed along the access road to the El Infiernillo plant.

INSTRUMENTATION AND STRONG-MOTION RECORDS

Several strong-motion accelerometers, which include AR-240, SMA-1 and DCA-310 recorders, are installed at various locations within the dam, power plant and tunnels. Two stations are located at the maximum embankment section, at the crest and on downstream berms at 60 and 100 m, respectively, below the top of the dam. Two free field instruments, one at each of the abutments, and two underground instruments, one in the power plant machine hall and the other in one of the right bank tunnels complete the El Infiernillo array. Maximum accelerations recorded at the right bank instrument (underground) were



TOPPLED INSULATORS
LA IGUANA SUBSTATION
Infiernillo Dam
Figure 2

0.13g for the September 19 event and 0.06g for the September 21 aftershock. Peak accelerations at the center of the highest berm were 0.38g for the main event, suggesting probable accelerations of the order of 0.50g at the crest of the dam. Several instruments, in particular those at the crest of the dam, did not function, presumably because the recording film was exhausted.

Deformations and internal pore pressures have been monitored since construction at El Infiernillo Dam with numerous instruments, including 4 cross-arm settlement devices of the USBR type, 12 inclinometers, 30 extensometers, one pneumatic piezometer and 95 survey monuments. Post-earthquake surveys and interpretation of instrument readings confirmed the settlements measured by the contractor, and indicated that the measured deformations at El Infiernillo Dam increased regularly in amplitude from bottom to top of the embankment and were mostly localized in the upper third of the dam section. Piezometer data did not record any significant increase in pore pressures, but recorded reservoir elevations indicate that a seiche with a single amplitude of about 1.25 feet occurred in the reservoir as a result of the 1985 earthquakes.

CONCLUSION

The 1985 Michoacan earthquakes induced significant shaking at El Infiernillo Dam. Despite minor damage and occurrence of small-amplitude permanent deformations, the Mexican Comision Federal de Electricidad concluded that the earth core rockfill dam and its appurtenant structures performed extremely well and without evident impairment of its overall safety.

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LONG VALLEY DAM, CALIFORNIA, USA

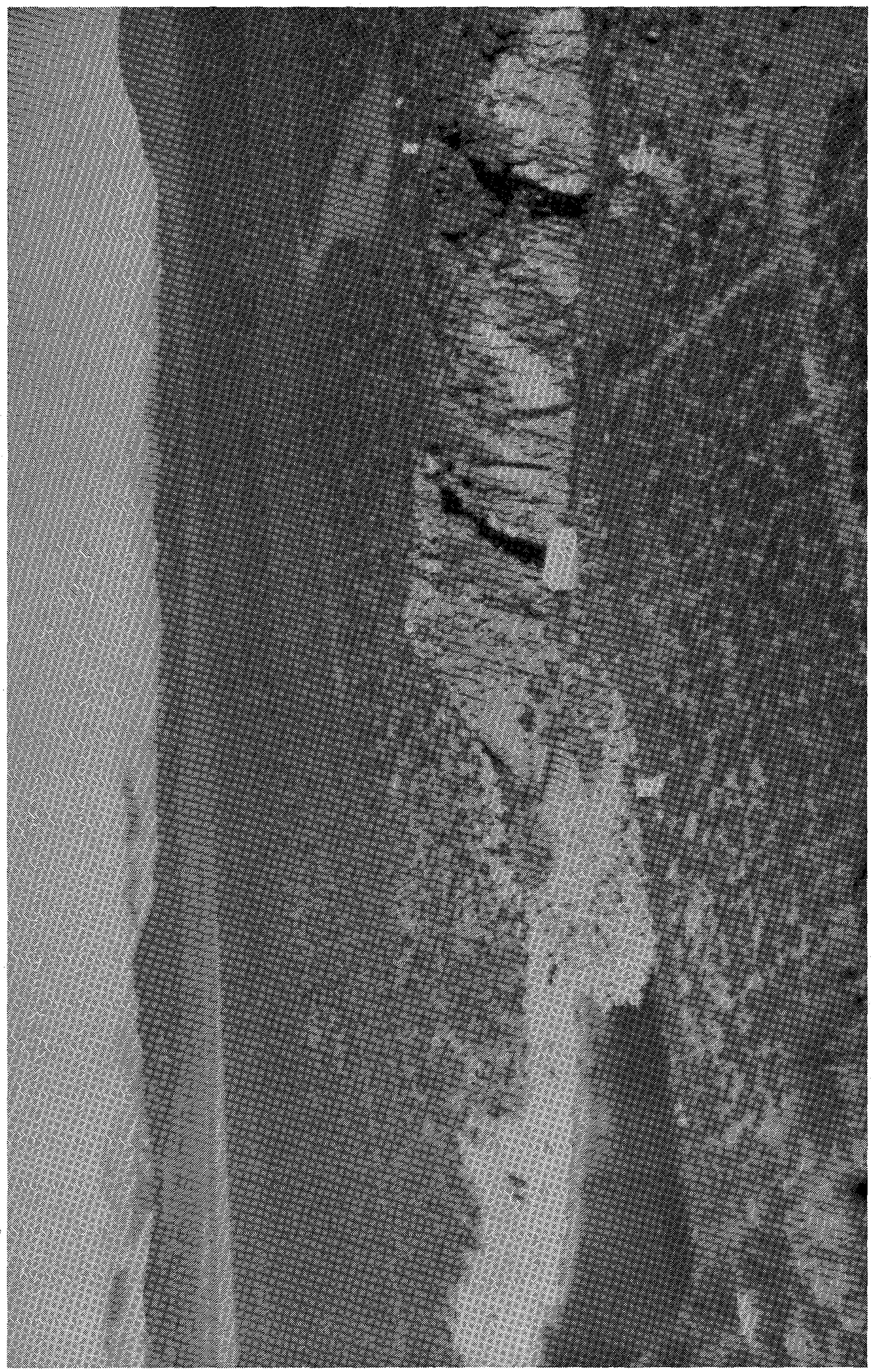
Long Valley Dam, located in east-central California, is noteworthy among dams having experienced earthquakes in three respects. First, numerous local earthquakes of moderate magnitude have shaken the dam. Secondly, well-documented strong motions have been recorded at the dam site. Thirdly, the dam has been used as a model to verify predicted embankment response during strong-motion shaking.

LONG VALLEY DAM

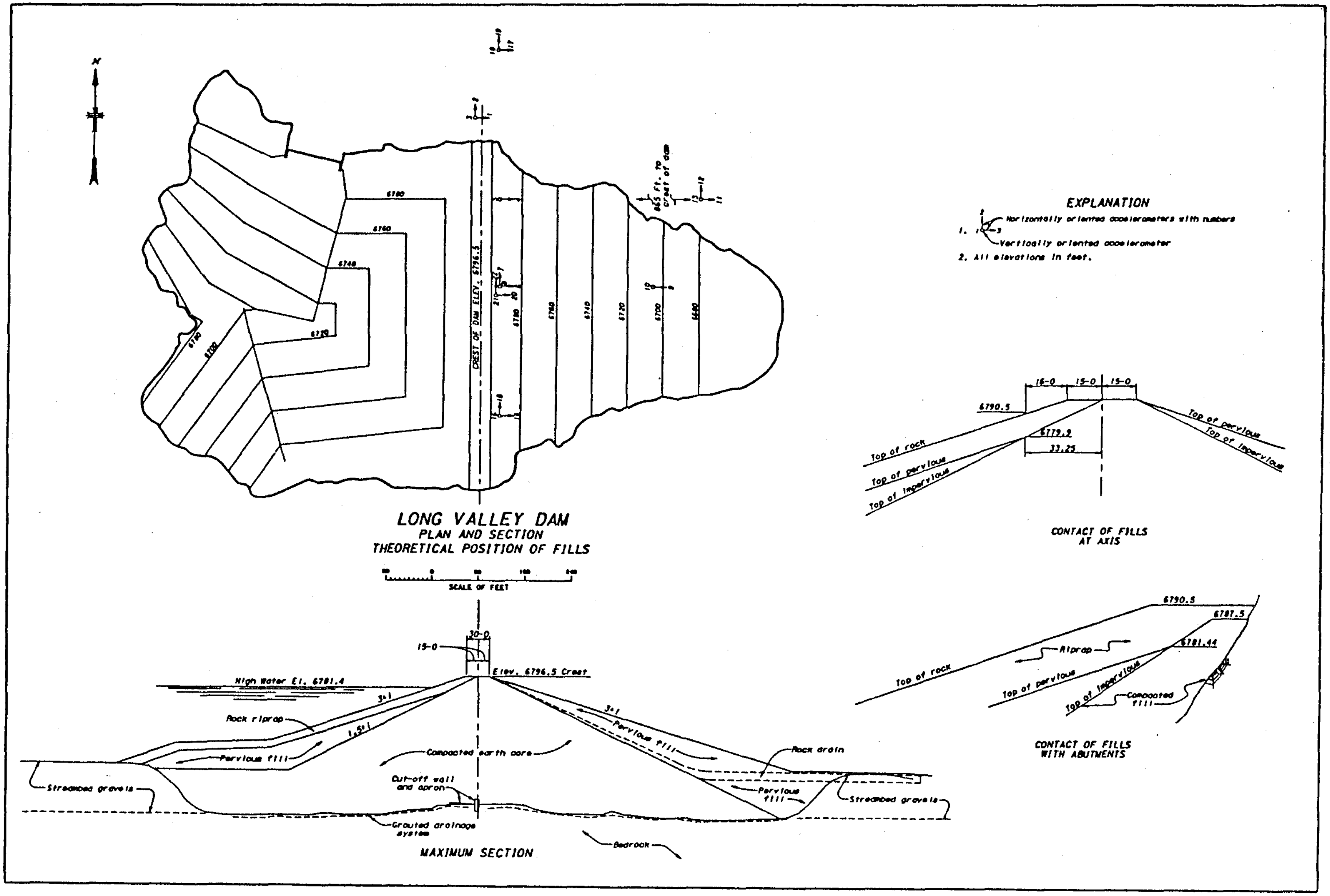
Long Valley Dam is a zoned embankment constructed across the Owens River in Mono County, California. The nearest city is Bishop, California, about 18 miles to the southeast. A 183,000 acre-foot reservoir is contained behind this 181-foot high dam, which rises 126 feet above streambed and has a crest length of 600 feet (see Figure 1).

Construction of the dam was begun by the Los Angeles Department of Water and Power in the late 1930s for water supply storage and runoff regulation purposes. Construction was completed in 1941. The embankment extends along the upstream valley slopes because of the high permeability of the rhyolite tuff that comprises both abutments. Modifications because of high seepage were required in 1965, 1968, 1973, 1974, and 1986 to improve the abutments and toe drains.

The core of the dam consists of homogeneous, roller-compacted earthfill. The upstream and downstream shells of the dam contain dumped and sluiced small rocks, and the outer shell of the upstream face consists of rock riprap (Figure 2). A pressure tunnel



GENERAL VIEW
Long Valley Dam
Figure 1



PLAN AND SECTION
Long Valley Dam
Figure 2

and penstock, as well as an 8-foot diameter diversion tunnel, are used as outlet works. The spillway is a 3,500 cfs side channel type.

Strong-motion instruments were installed at Long Valley Dam in 1975 as part of the State of California's Strong-Motion Instrumentation Program (CSMIP). In this cooperative program between facility owners and the State of California, the Office of Strong-Motion Studies of the State Division of Mines and Geology installs and maintains the instruments. In 1979, because of numerous earthquakes occurring in the area, the number of accelerometers was increased from 9 to 22.

TECTONIC HISTORY AND EARTHQUAKES OF THE LONG VALLEY DAM AREA

Long Valley Dam is only about three miles southeast of the edge of the Long Valley caldera and three miles northeast of surface traces of the active Sierra Nevada frontal fault system. Long Valley caldera was formed slightly less than one million years ago as a result of a catastrophic collapse of a then-existing volcanic center. More or less simultaneously, over 144 cubic miles of magma were erupted. The resultant volcanic rock is the Bishop tuff, a mixture of pumice falls and ash flows, about 4,800 feet thick within Long Valley caldera. Post-caldera volcanic activity both within and outside of the caldera has resulted in significant infilling of the caldera with rhyolites and other volcanics, which are mixed with sedimentary fill. Periodic eruptions from vents peripheral to a resurgent dome within the caldera are estimated to be spaced at 200,000-year intervals. Well-exposed Holocene fault scarps of the Sierra Nevada frontal fault system crossing the caldera indicate that repeated moderate to large ($M > 6.5$) earthquakes have caused surface offsets in the last 10,000 to 20,000 years.

Beginning with an earthquake of local magnitude 5.8 in October 1978, the area of Long Valley Dam was shaken almost continuously by a low- to moderate-magnitude earthquake sequence (or swarm) until late 1986. This intense earthquake activity, including 26 earthquakes above magnitude 5 and 6 earthquakes above magnitude 6, coupled with measured uplift of the resurgent dome in the caldera, indicated a potential for volcanic eruption. The U.S. Geologic Survey, based on an accumulation of hazard warning factors, issued a formal "notice of volcanic hazard" for the general area in which Long Valley Dam is located.

The geologic evidence thus indicates that the area of Long Valley Dam has been and is subject to frequent earthquakes of both tectonic and volcanic origins. Moreover, some of these recent earthquakes may have been associated with a combination of processes, namely volcanic stoping and intrusion into existing fault structures with subsequent release of tectonic stress energy along the faults.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

The timing of strong-motion instrument installation in 1975 was fortuitous in that valuable strong-motion data were collected in 1978. The increase in the number of accelerometers in 1979 was well-planned, because much valuable data has been recorded from the succeeding earthquakes. Although the earthquakes of significance to Long Valley Dam have only been of moderate magnitude, their proximity to the dam has resulted in repeated and relatively high peak accelerations both in the structure and in adjacent natural ground (bedrock).

Recorded peak accelerations (some of these are preliminary accelerations from unprocessed data) from selected earthquake events are as follows:

		Recorded Peak Accelerations (g)			
Earthquake Date and Time (yr mo day hr min) GMT	Event Magnitude ML	Center Crest of Dam	Left Toe of Dam in Tunnel (Rock)	Left Abut at Crest Elev (Rock)	Left Abut Ridge (above Dam Crest)
78.10.04.16.42	5.8	0.17 L	------	------	0.26 L
80.05.25.16.33	6.1	0.23 T,L	0.08 V	0.13 L	0.42 L
80.05.25.16.49	6.0	0.06 L	0.04 L	------	0.19 L
80.05.25.19.44	6.1	0.24 L	0.11 L	0.09 L	0.49 L
80.05.27.14.50	6.2	0.52 T	0.24 L	0.21 T	0.99 L
81.09.30.11.53	5.9	0.11 L	0.07 L	0.09 L	0.12 L
83.01.07.01.38	5.4	0.13 L	0.06 L	------	0.08 L
84.11.23.18.08	6.1	0.08 L	------	------	0.08 T,L
86.07.21.14.42	6.5	0.21 L	0.10 L	0.08 V,L	0.34 L

Notes:

a) GMT = Greenwich Mean Time
b) T = acceleration transverse to dam axis
c) L = acceleration parallel to axis (longitudinal acceleration)
d) V = vertical acceleration
e) ML = Local Magnitude, from University of California, Berkeley seismograph station

Despite several relatively high accelerations, the only earthquake-related damage observations made at Long Valley Dam were minor surface cracks oriented transversely to the dam axis, near the abutments; however, investigations determined that these cracks were never more than a few inches deep. Rockfalls from the canyon walls and abutments during earthquakes have also been reported.

INSTRUMENTATION AND STRONG-MOTION RECORDS

One CR-1 and three SMA-1 strong-motion accelerographs are connected to 22 accelerometers deployed on Long Valley Dam and adjacent bedrock. The locations of these instruments are shown on Figure 2. The highest peak accelerations at selected locations on the dam and on bedrock are shown for nine of the largest and most recent earthquakes. The data are basically what might be expected. The response motion at the center crest of the dam is generally considerably greater than the input motion (to the dam) in the bedrock foundation and in the abutments at the crest level.

The strong motions recorded on the left abutment ridge above the dam level show larger amplitudes than those recorded at all of the other instruments. These anomalous motions probably result from the focusing of earthquake seismic waves and local topographic effects. Use of such records for design purposes could lead to overconservative analyses and design. Instrumentation locations at Long Valley Dam illustrate a careful planning to select strong-motion records at critical locations on and adjacent to the dam.

Strong-motion records from the May 27, 1980 earthquake were used as a field check in a study to verify the equivalent-linear method of dynamic analysis for estimating the seismic response of embankment dams (Lai, S. and Seed, H.B., 1980). The study involved 2-D and 3-D response analyses, using appropriate combinations of dynamic shear modulus and damping coefficients. The study reasonably reproduced the recorded strong motions, and provided a benchmark example that embankment dam seismic design can produce safe structures with verifiable response to earthquake shaking.

CONCLUSIONS

Long Valley Dam has survived unscathed through numerous events of moderately severe earthquake shaking. Strong-motion records at this site are unusually well-documented and have been used in a benchmark study in dam response prediction (Lai et al., 1980).

Long Valley Dam has a relatively large number of accelerometers. This extensive seismic instrumentation provides appropriate coverage for a site having a high exposure to potentially significant seismic events. The records of the instruments on the dam and on bedrock have documented relative motions that are predictable and rational, based on the locations of the instruments. The influence of topographic effects upon ground motion is apparent in the records obtained at the left abutment ridge.

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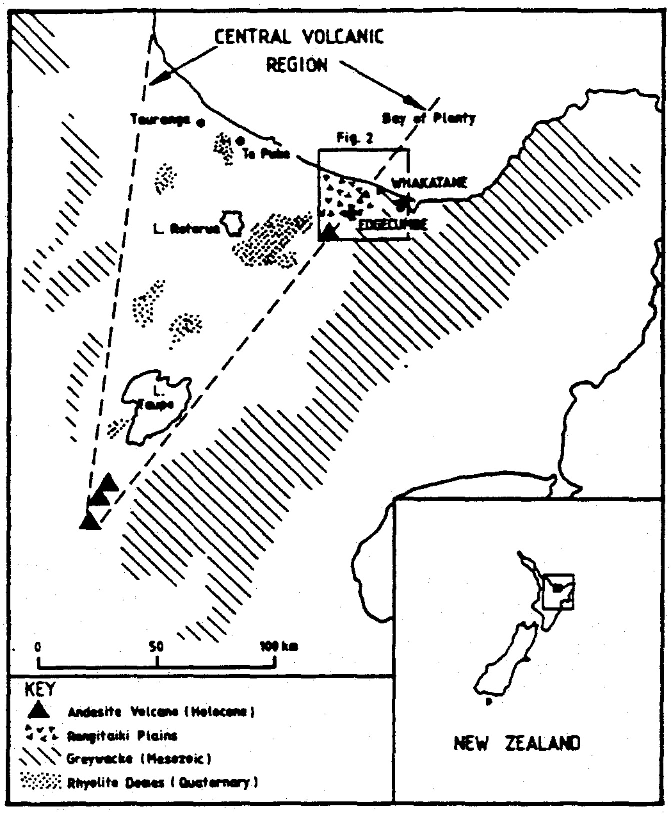
MATAHINA DAM, NEW ZEALAND

On March 2, 1987, an earthquake of magnitude 6.3 struck the Bay of Plenty Region of the North Island of New Zealand. Matahina Dam and power plant were located at a relatively short distance from the epicenter. No damage occurred in the powerhouse or switchyard, but the dam, an earth and rock embankment, settled and experienced deformations of engineering significance. Overall, Matahina Dam performed satisfactorily, considering the severity of the ground motion to which it was exposed.

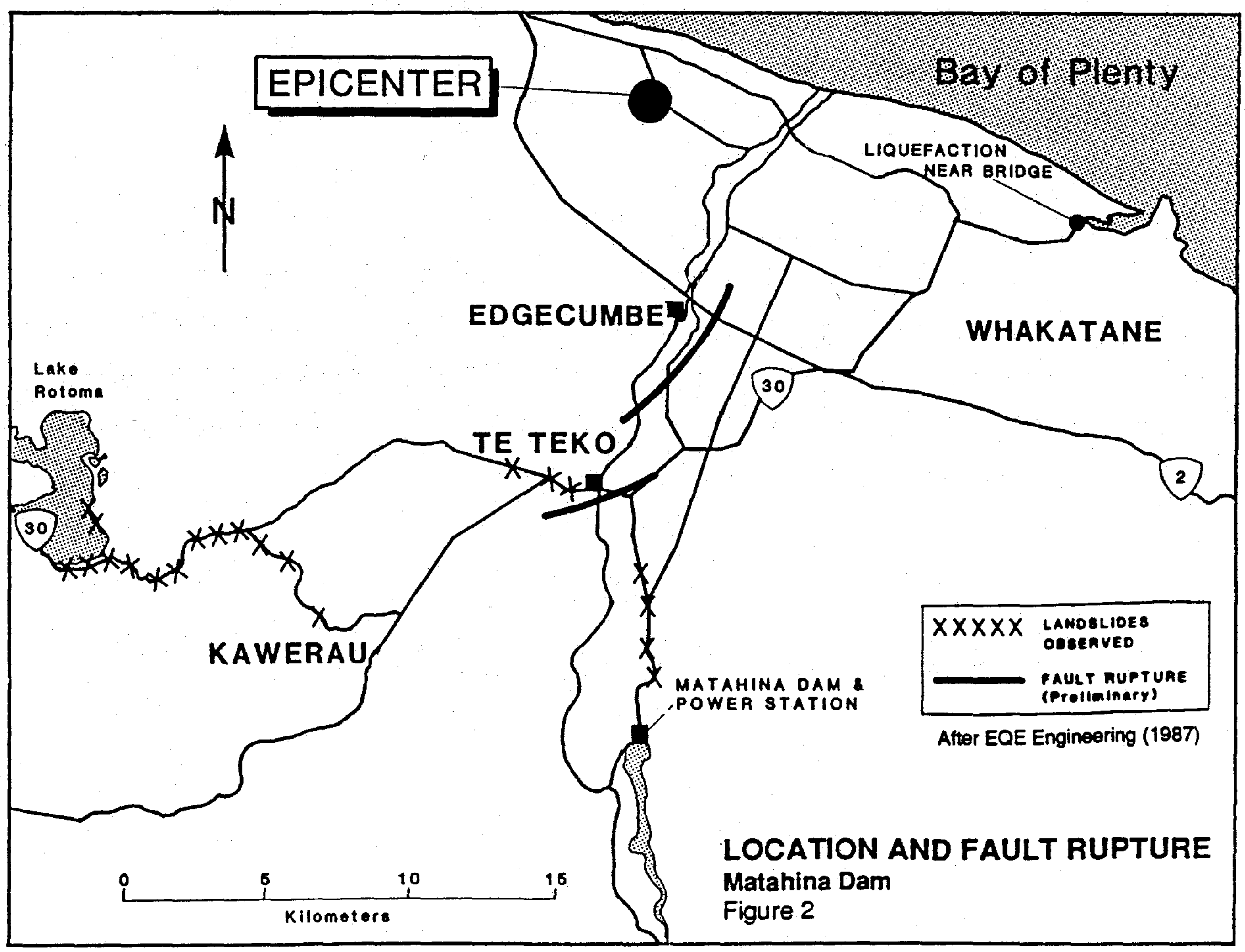
MATAHINA DAM

Matahina Dam, located on the Rangataiki River, is a 282-foot high earth and rockfill dam, with a crest length of about 1,312 feet. The dam, constructed in 1967, is owned and operated by the Electricity Corporation of New Zealand. The embankment has an upstream sloping core of weathered graywacke, with gradation characteristics similar to a low plasticity clayey gravel, transition zones of weathered fine ignimbrites (welded tuff) and compacted ignimbrite rockfill shells (Figure 3). Most of the upstream face slopes at 2.5 to 1 (H to V) and the downstream face at about 2.3 to 1 (H to V). The dam is founded on alluvial sediments, primarily dense gravels, sands, and silty clays of Tertiary origin. Seepage is controlled by a shallow concrete cutoff wall below the core and a 30 m deep drainage curtain discharging into a transverse drainage blanket.

The abutments consist of hard and massive jointed ignimbrite, typical of the near vertical cliffs that shape the canyon of the Rangataiki River in the project vicinity. The left abutment forms a prominent rock ridge that contains the diversion and outlet tunnels. Spillway, penstock and powerhouse are also located at the left abutment. The Waiohau fault zone intersects this abutment about 1,600 feet away from the dam. Related branch faults were encountered at construction in the dam foundation.



REGIONAL GEOLOGY
Matahina Dam
Figure 1



LOCATION AND FAULT RUPTURE
Matahina Dam
Figure 2

THE MARCH 2, 1987 EDGECUMBE EARTHQUAKE

The Matahina dam site is located within the eastern margin of the Central Volcanic Region of New Zealand. Tectonic spreading affects the area and is related to active convergence along the Pacific and Australian plate boundary, east of the North Island. Predominantly normal faulting, oriented northeast-southwest, is common in the area.

The May 2, 1987 Edgecumbe Earthquake (magnitude 6.3) occurred along a previously unmapped fault and caused surface ruptures near the town of Edgecumbe (Figures 1 and 2). Soil failures, liquefaction, settlements and landslides were observed throughout the area and affected the nearby towns of Kawerau, Te Teko and Whakatane. Residences in the Edgecumbe area experienced moderate damage; numerous chimneys collapsed, tile roofs were broken and a few houses were lost. Underground services were widely affected and service interruptions lasted several weeks. Several commercial or industrial buildings suffered minor impairments. Substantial buckling and overturning of stainless steel storage tanks occurred in a dairy factory.

Seismic activity in the general area was preceded by a strong foreshock (magnitude 5.2) and followed by four aftershocks of magnitude greater than 5.0. The main shock was about 12 km deep and centered at about 23 km away from Matahina Dam. It produced a complex extensional surface scarp, about 6 km long and striking southwest from Edgecumbe. About four feet of maximum extension, with a minor component of strike-slip, were measured across the scarp. The area to the northwest of the fault trace was thrown down by about five feet. Secondary normal fault traces and other deformation features, such as compressional rolls and sandblows, were also observed. The dam was located 11 km away from the main trace of the surface faulting. The intensity of shaking at the power station was estimated to be VII (Modified Mercalli Scale).

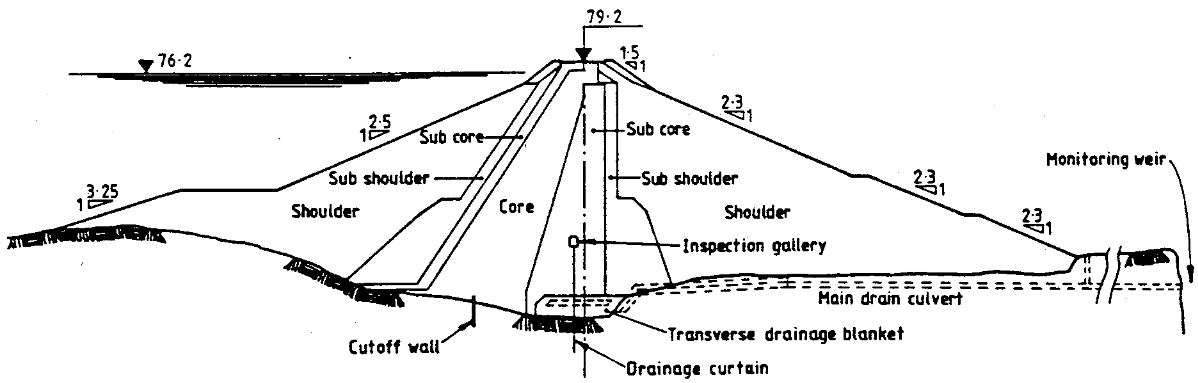
Extensive general regional subsidence with a maximum depth of about seven feet was measured in the Edgecumbe area during the relevelling of the main benchmarks on the low-lying alluvial floodplain of the Rangitaiki River. Related to this subsidence, serious damage occurred to some stretches of the dikes which provide flood protection in this area.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Detailed inspection of Matahina Dam following the Edgecumbe earthquake revealed surface cracking near the abutments and significant settlement and downstream movement of the upstream and downstream rockfill shells (Figures 4 and 5). The abutment areas were investigated using trenching, boreholes and geophysical exploration. The cracks were shown to be shallow and not continuing through the core, but trenching exposed a large cavity below the crest pavement, upstream of the core in the right abutment. This cavity was concluded to be probably related to earlier core cracking, seepage and internal erosion which occurred shortly after the dam first impoundment in 1967. The earthquake shaking compacted loose materials in the former leakage area, thereby creating a cavity below the pavement.

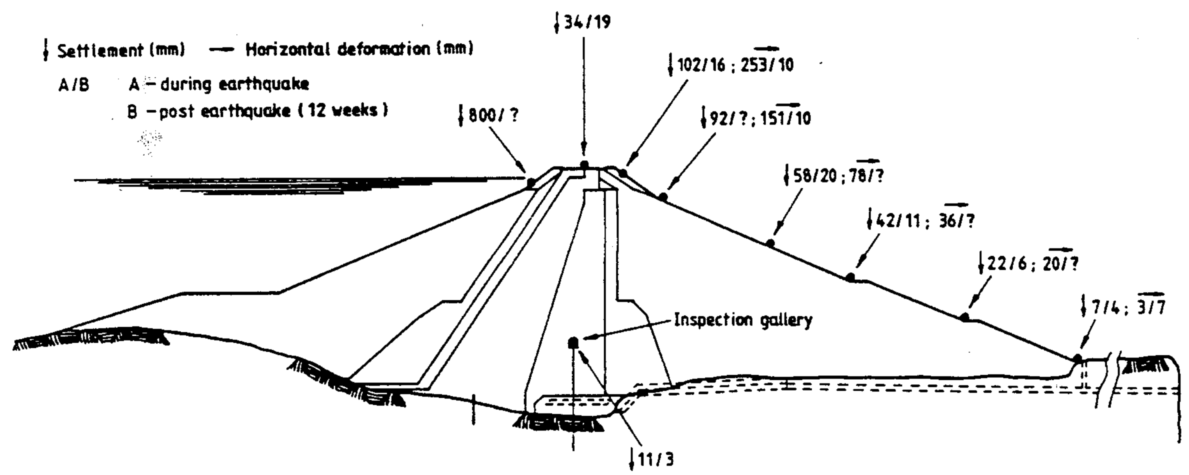
A slight increase in seepage was noticed through the dam. Although immediate inspection did not indicate severe damage, the Matahina Reservoir was drawn down 8.2 feet as a safety measure. Local residents believed that the safety of the dam was of concern and unofficially evacuated the downstream area. During the reservoir drawdown, the flow from the drainage blanket weir increased from about 20 to 170 gpm, but probably as a result of the increased tailwater level during the drawdown rather than of the earthquake. Seepage from the left abutment increased to about four times normal immediately after the earthquake, then decreased, and slowly increased again in the eight months that followed the event. This situation is closely watched by operation personnel.

The extensive network of surface monuments of the dam had been resurveyed about three weeks prior to the earthquake. Post-earthquake surveys indicated that the dam settled significantly as a result of shaking. Most of the settlement occurred immediately, but it continued for several weeks. The crest settled a maximum of 4.02 inches and moved



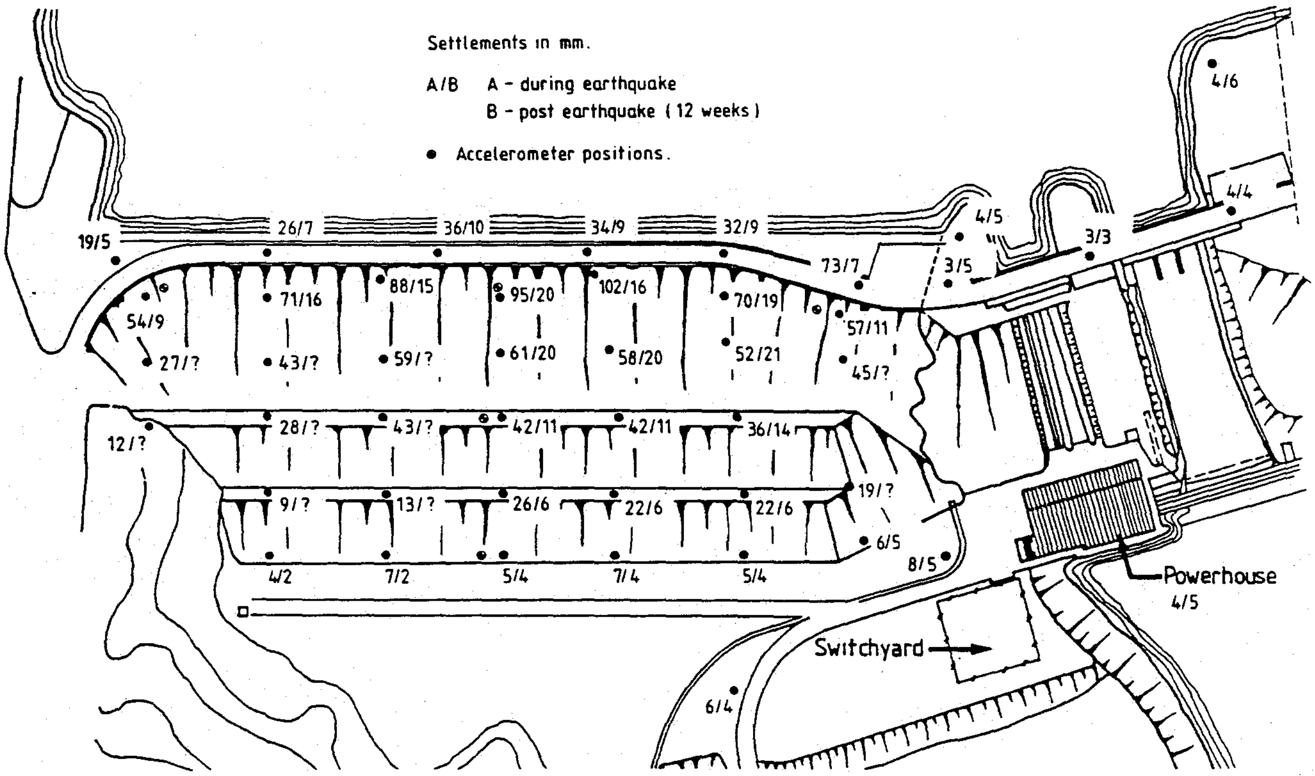
Cross section of the Matahina Dam.

SECTION
Matahina Dam
Figure 3



Deformations at centre of dam

MEASURED SETTLEMENTS (SECTION)
Matahina Dam
Figure 4



Settlements resulting from earthquake.

MEASURED SETTLEMENTS (PLAN)
Matahina Dam
Figure 5

downstream as much as 10 inches, due to the earthquake. Local settlements of the upstream shell of up to about 31.5 inches were common, as evidenced through the discoloration of the previously submerged riprap. No submerged slope failures were found using exploration sonar. Settlements were concluded to primarily result from the earthquake-induced compaction of the rockfill.

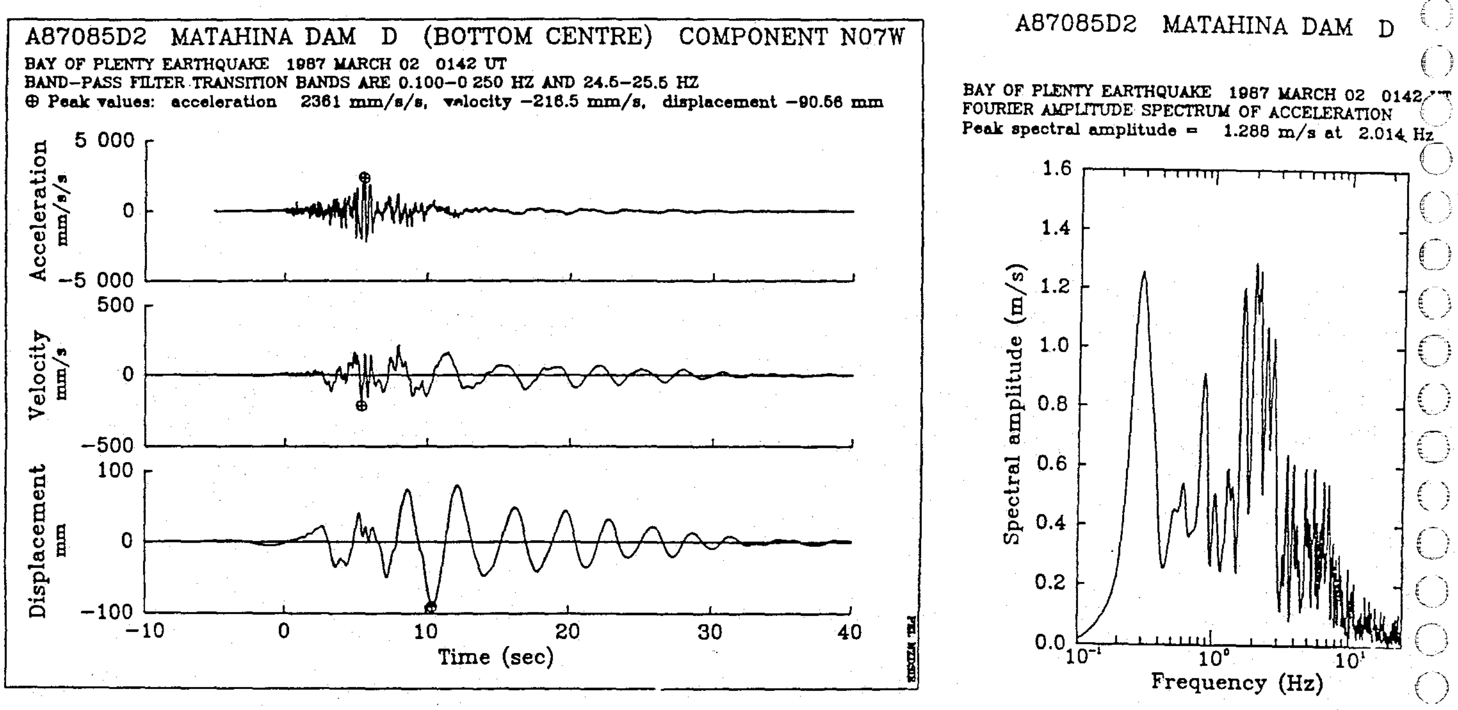
There was no damage to the powerhouse or switchyard.

INSTRUMENTATION AND STRONG-MOTION RECORDS

The dam was equipped with five strong-motion accelerographs, serviced by the DSIR Physics and Engineering Laboratory. These recorded the foreshock and the main shock and some of the aftershocks (Figure 6). Three of these instruments are sited across the crest, one at the center of the base of the dam and one at a mid-height rockfill berm. They recorded the following peak accelerations:

	Horizontal (g)	Vertical (g)
BASE	0.33	0.14
MID-HEIGHT	0.48	0.21
CREST	0.42	0.29

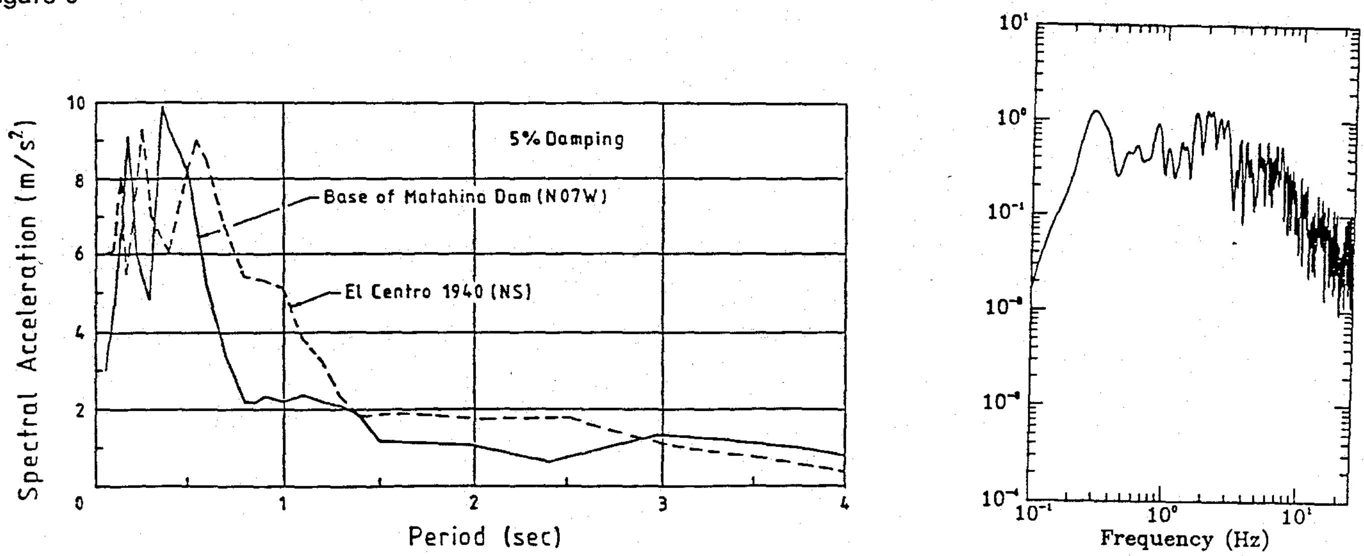
Hence, the largest amplification of peak ground accelerations occurred at mid-height, rather than at the crest of the dam. This may result from the significant plastic deformations that occurred in the upper part of the dam, which would dampen high frequency motion. Vertical accelerations were amplified by a factor of about two when travelling from base to crest.



ACCELEROGRAM RECORDED AT BASE, MARCH 2, 1987

Matahina Dam

Figure 6



Acceleration, velocity and displacement at the base of the Matahina Dam and comparison of the 5% damped acceleration response spectra there with El Centro 1940 (NS).

RESPONSE SPECTRUM COMPARISON, EL CENTRO AND BASE MOTION

Matahina Dam

Figure 7

CONCLUSION

Matahina Dam experienced about ten seconds of strong shaking as a result of a nearby moderate earthquake. Significant settlements of the dam crest and upstream shell occurred. No major leakage was observed. Conditions requiring repairs were identified as a result of the earthquake, although probably caused as a result of an earlier problem during the dam's first impoundment.

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AUSTRIAN DAM, CALIFORNIA, USA

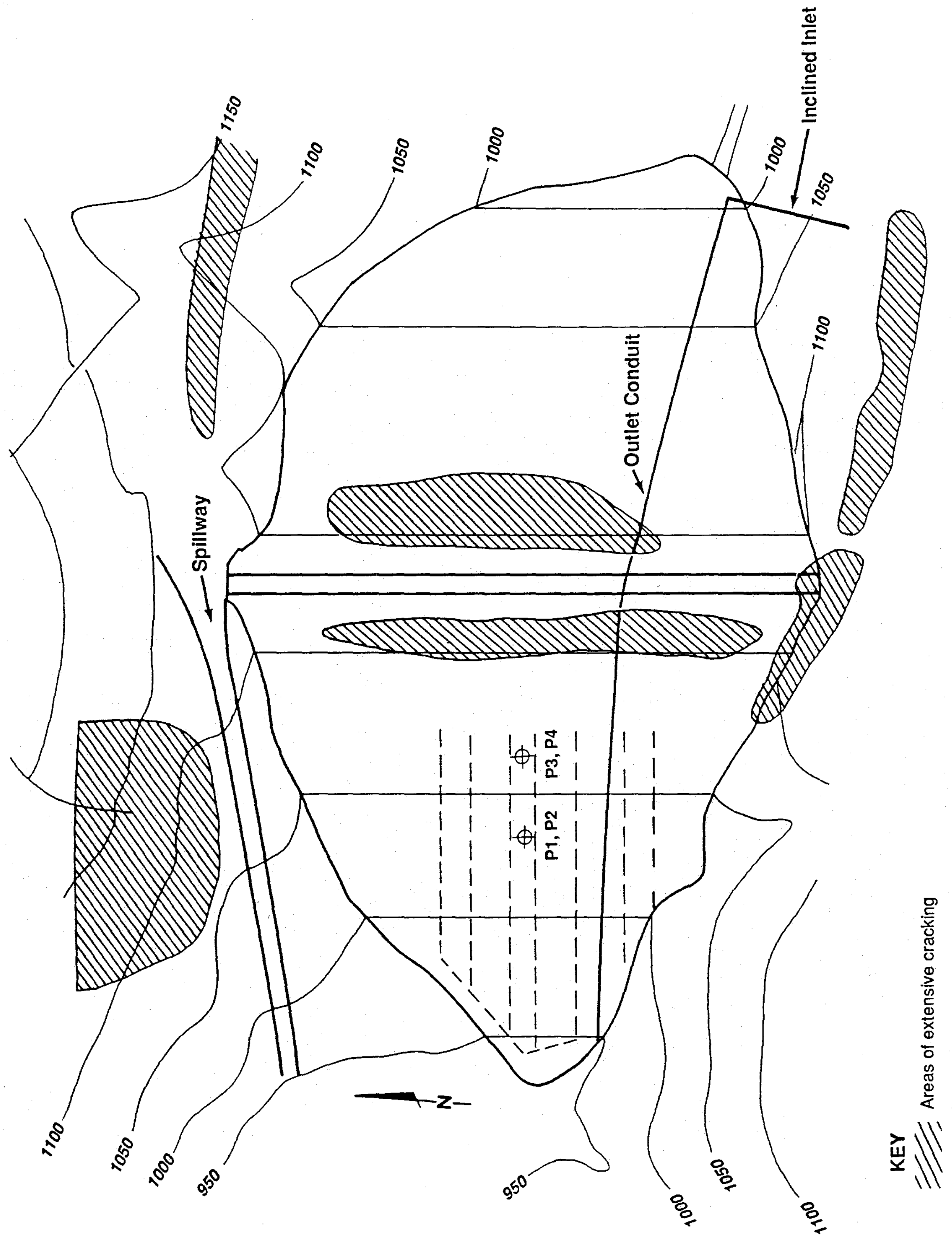
On October 17, 1989, a Magnitude 7.1 earthquake struck the San Francisco Bay Area in California. The epicenter was near the southwestern limit of the metropolitan area, which has a population of 5 million. The earthquake resulted in 62 deaths, over 3,000 injuries to people, and more than $5.6 billion in property damage.

AUSTRIAN DAM

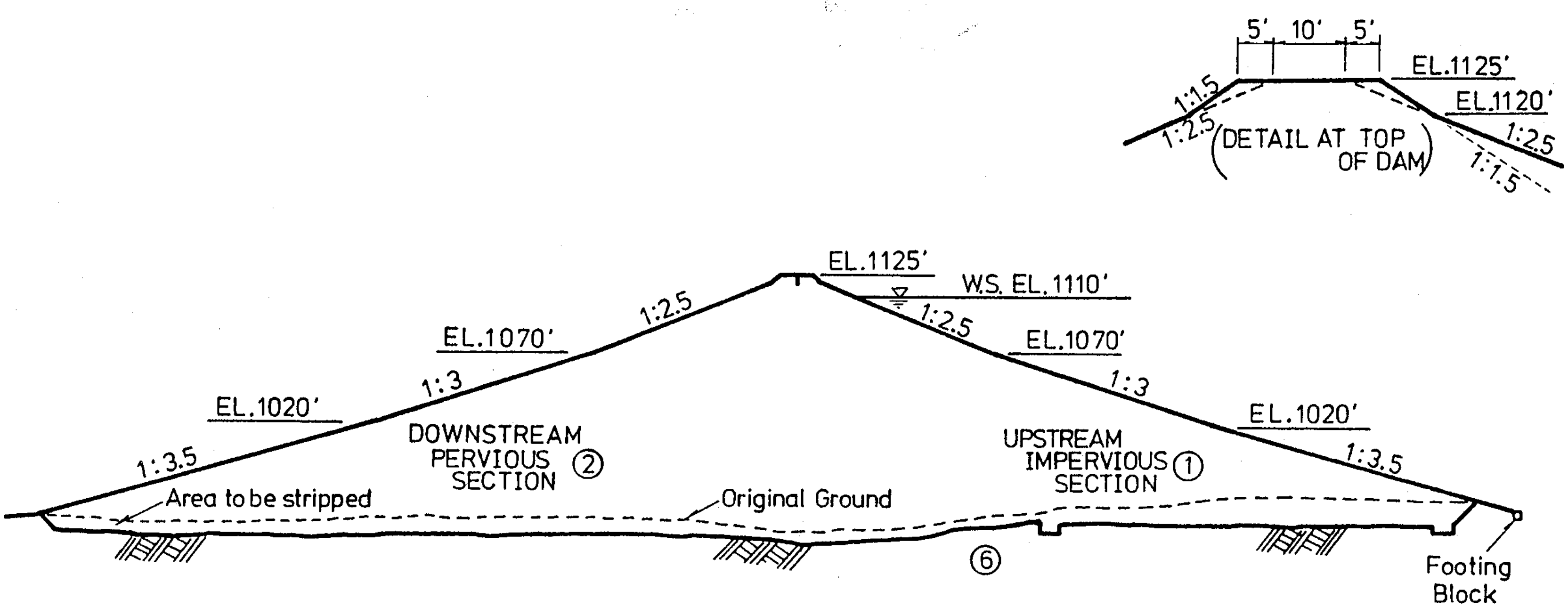
Austrian Dam is a 200-foot high embankment, constructed in 1949-1950 on Los Gatos Creek, near the town of Los Gatos. Figure 1 is a plan view of the dam. The crest length is 700 feet. The design called for an upstream impervious zone, a downstream pervious zone, and highly pervious strip drains located near the old stream channel in the downstream zone, see Figure 2. However, the weathered sedimentary rock at the site broke down during excavation, placement, and compaction, resulting in a nearly homogeneous gravelly, clayey sand embankment, compacted to approximately 90 percent of ASTM D-1557 maximum density. Excavation as a result of repair work since the earthquake has disclosed that drainage from the strip drains was impeded by placement of waste materials. Essentially all soils and highly weathered rock had been removed from the dam footprint prior to the embankment construction.

The dam is in a vee-shaped canyon in the Santa Cruz Mountains, about 2,000 feet northeast of the San Andreas fault zone. The Sargent fault, a feature related to the San Andreas fault, passes through the right abutment within 700 feet of the dam.

The dam impounds a 6,200-acre-foot water supply reservoir, serving a portion of the urban area downstream. The dam crest is at Elevation 1125. At the time of the earthquake



PLAN
Austrian Dam
Figure 1



SECTION
Austrian Dam
Figure 2

the reservoir contained 700 acre-feet of water, which corresponds to a reservoir water surface at Elevation 1023. Storage was low both as a result of the annual operating cycle and of three years of below average rainfall. Mid-October is the usual start of the local rainy season.

A reinforced concrete spillway is located on the steep right abutment, contiguous to the embankment. The ungated control section is located 15 feet below the dam crest, on moderately weathered fractured shale. The chute is founded on highly weathered shale, replaced during construction with shallow compacted fill in some areas.

The reinforced concrete outlet conduit, 4 feet in diameter, was constructed in a trench excavated into bedrock at the base of the left abutment. An inclined outlet facility extends up the left abutment, upstream from the dam. It contains eight ports to allow selective withdrawal of reservoir water.

OCTOBER 17, 1989 LOMA PRIETA EARTHQUAKE

The October 17, 1989 Loma Prieta Earthquake occurred along the San Andreas fault system. The epicenter of the magnitude 7.1 (Ms) shock was centered in the Southern Santa Cruz Mountains near the Loma Prieta Lookout, hence the name. This event is the largest northern California earthquake since the 1906 San Francisco earthquake (M 8.3). The focal depth was 18.5 kilometers, deeper than most earthquakes along that zone. For 7 to 10 seconds after the initial rupture, rupture propagated upward and laterally along the fault plane. It was estimated from seismologic data that a 50 kilometer length of the fault ruptured. The top of the ruptured area was estimated to have extended upward to within 6 kilometers of the ground surface. Damage by extensive ground breakage and fissuring was abundant in the epicentral area, but no surface displacements along the San Andreas, or along the neighboring Zayante-Vergeles, and Sargent faults, were observed. This segment of the San Andreas fault last ruptured in the magnitude 8.3 San Francisco Earthquake of

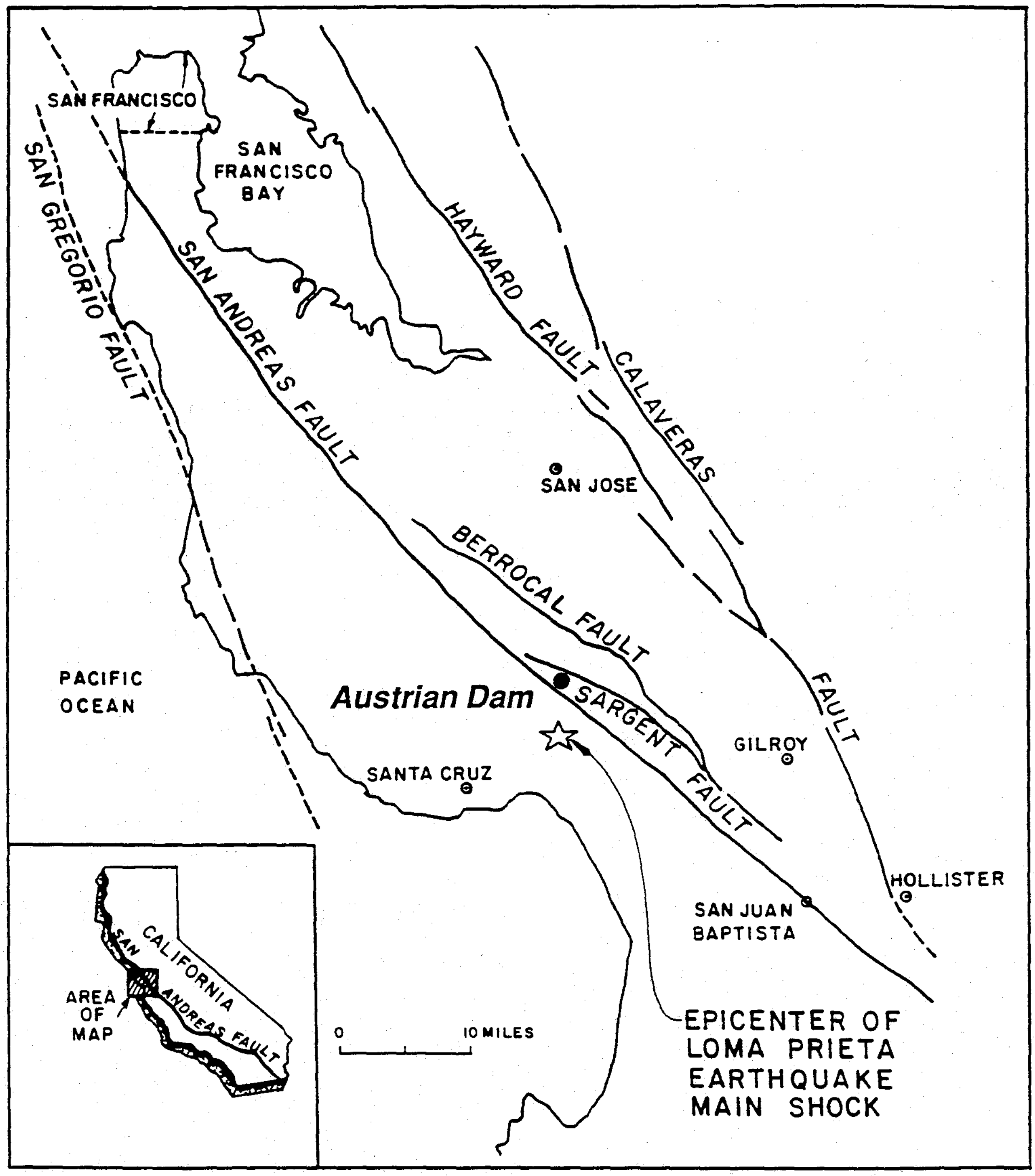
1906, when it was displaced only about 1.2 meters as compared with 3 to 4 meters elsewhere along the fault. The largest aftershock of the October 17 event has been magnitude 5.0.

EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

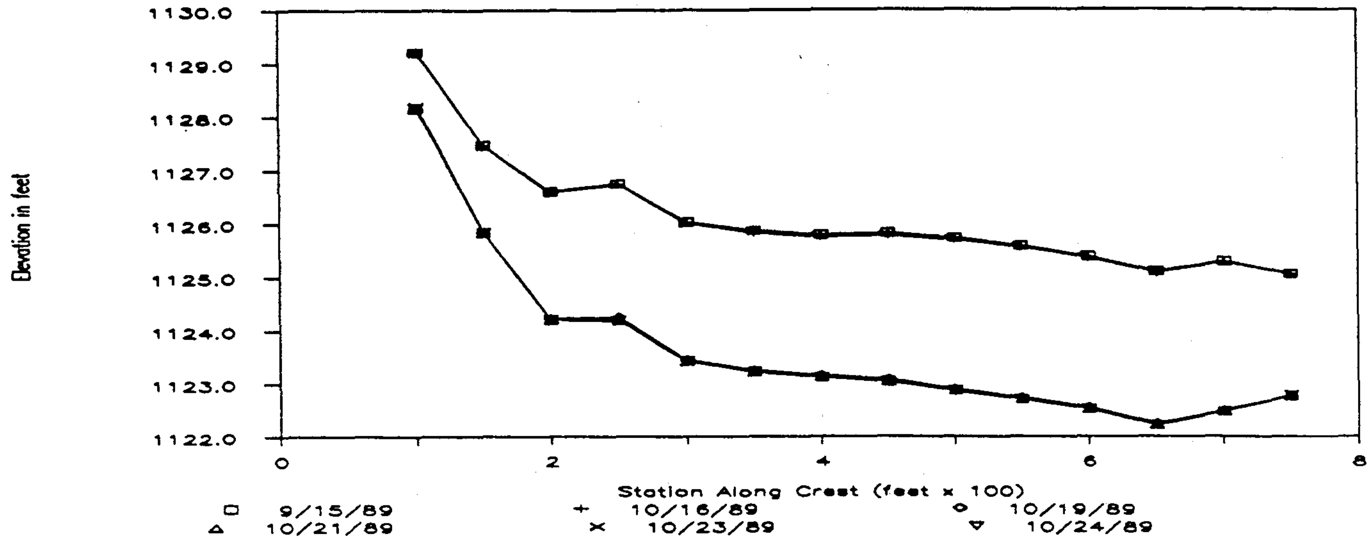
Austrian Dam was located about 11 km from the October 17 epicenter (Figure 3). Embankment crest movements caused by the earthquake accentuated the historic settlement pattern (Figure 4). Maximum settlement was 2.8 feet, with significant deformations occurring over the right two-thirds of the dam. Maximum downstream movement was 1.1 feet near the spillway wall on the right abutment, and maximum upstream movement was 1.4 feet at the left quarter point of the embankment (Figure 5). Longitudinal cracks up to 14 feet deep occurred within the upper 25 percent of the upstream and downstream faces. Shallower longitudinal cracks were found on much of the downstream face. Crest cracking was confined to the abutment contact areas. A transverse crack was traced 30 feet down the left abutment, where the dam had been constructed on weathered, highly fractured rock. Transverse cracking and embankment separation from the spillway structure occurred to a depth of 23 feet on the right abutment. Water levels increased in the open well piezometers in the embankment (Figure 6).

Spillway damage consisted primarily of numerous transverse tension cracks. The structure appears to have elongated about one foot, toppling the end walls in the process. Some cutoff walls were damaged. Voids up to 6 inches wide were observed upstream from other cutoff walls. The walls of the "U" shaped section flexed inward, lifting the base of walls and adjacent portions of the floor slab up to one inch. Exploration and analysis of the structure have not been completed at the time of this writing. The only damage to the outlet works consisted of the tipping of a valve actuator steel tank, located at the top of the inclined facility.

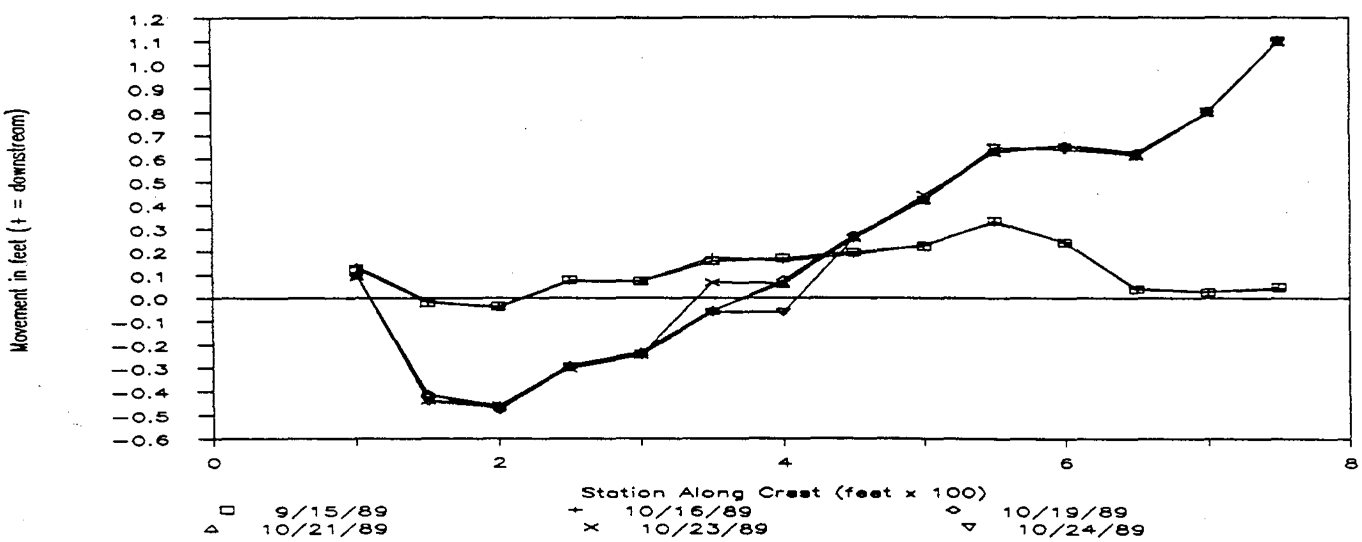
Ground cracking occurred in and above the reservoir area and on the abutments. Cracking was shallow, except for possibly the right abutment ridge, which is being



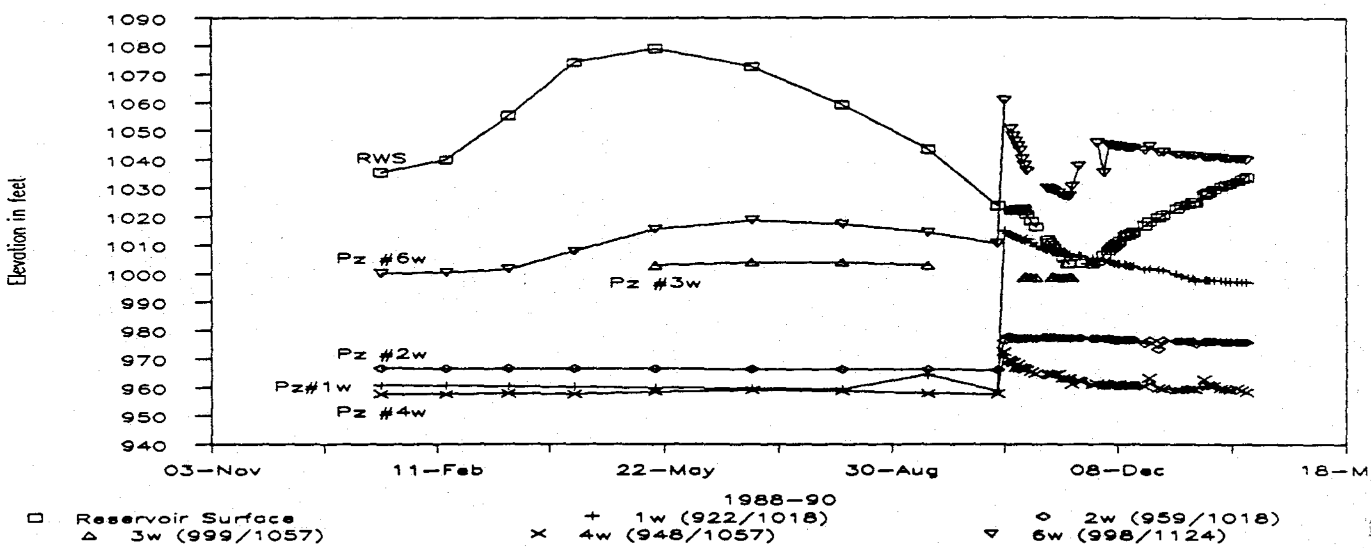
DAM AND EPICENTER LOCATIONS, OCTOBER 17, 1989
LOMA PRIETA EARTHQUAKE
Austrian Dam
Figure 3



CREST SETTLEMENT,
LOMA PRIETA EARTHQUAKE
Austrian Dam
Figure 4



TRANVERSE CREST MOVEMENT,
LOMA PRIETA EARTHQUAKE
Austrian Dam
Figure 5



PIEZOMETER RESPONSE
LOMA PRIETA EARTHQUAKE
Austrian Dam
Figure 6

investigated at the time of this writing. Shallow landslides were triggered by the shaking. More landslides are expected during subsequent periods of heavy rainfall.

Repairs of Austrian Dam began within days following the earthquake, so that the dam could be used during the upcoming rainy season. Cracked embankment and foundation materials have been excavated and replaced with compacted embankment fills which include crack stopper zones near the abutments. Freeboard has been restored. A grout curtain at the left abutment contact has been regrouted. A toe drain has been installed. The cracks in the spillway were epoxy grouted to allow spillway use during the impending rainy season. The principal spillway and earthworks repairs were essentially complete in about an 8-week period.

INSTRUMENTATION AND STRONG-MOTION RECORDS

No strong-motion instruments were located at the dam site or in its vicinity. The strong-motion phase of the shaking lasted for about ten seconds, with recorded peak ground accelerations of about 0.65g near the epicenter. In San Francisco and Oakland, about 100 kilometers from the epicenter, peak accelerations of about 0.10g were recorded on rock sites, and up to 0.30g at several stations located on soft soils or in filled areas.

Accelerations measured on the left abutment and at two locations on the crest of Lexington Dam, a 205-foot high embankment, are the closest recordings to Austrian Dam. Peak horizontal accelerations of over 0.40g were recorded by each instrument. The peaks occurred simultaneously in the transverse and longitudinal directions of the dam. Lexington Dam was 21 kilometers northwest of the epicenter; Austrian was only 11.5 kilometers northwest of the epicenter and is also closer to the fault itself. The peak ground acceleration at Austrian Dam may have been up to 0.60g.

CONCLUSIONS

Austrian Dam was severely damaged by the magnitude 7.1 Loma Prieta earthquake. The 2.8 feet of embankment settlement were reasonably consistent with a conservative estimate of a maximum of 10 feet during a magnitude 8.3 MCE, predicted from an earlier seismic evaluation of the dam (1982), using modern analysis techniques. Embankment pore pressures increased, as expected. The transverse cracking at the abutments was more severe than would ordinarily be expected for the amount of settlement experienced. Such cracking is probably due to a combination of factors that include the contact with the 25-foot high vertical spillway wall located on an already steep right abutment, some poor embankment compaction near the top of the dam, and construction of the embankment on the fractured rock of the upper left abutment.

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SAN JUSTO DAM, CALIFORNIA, USA

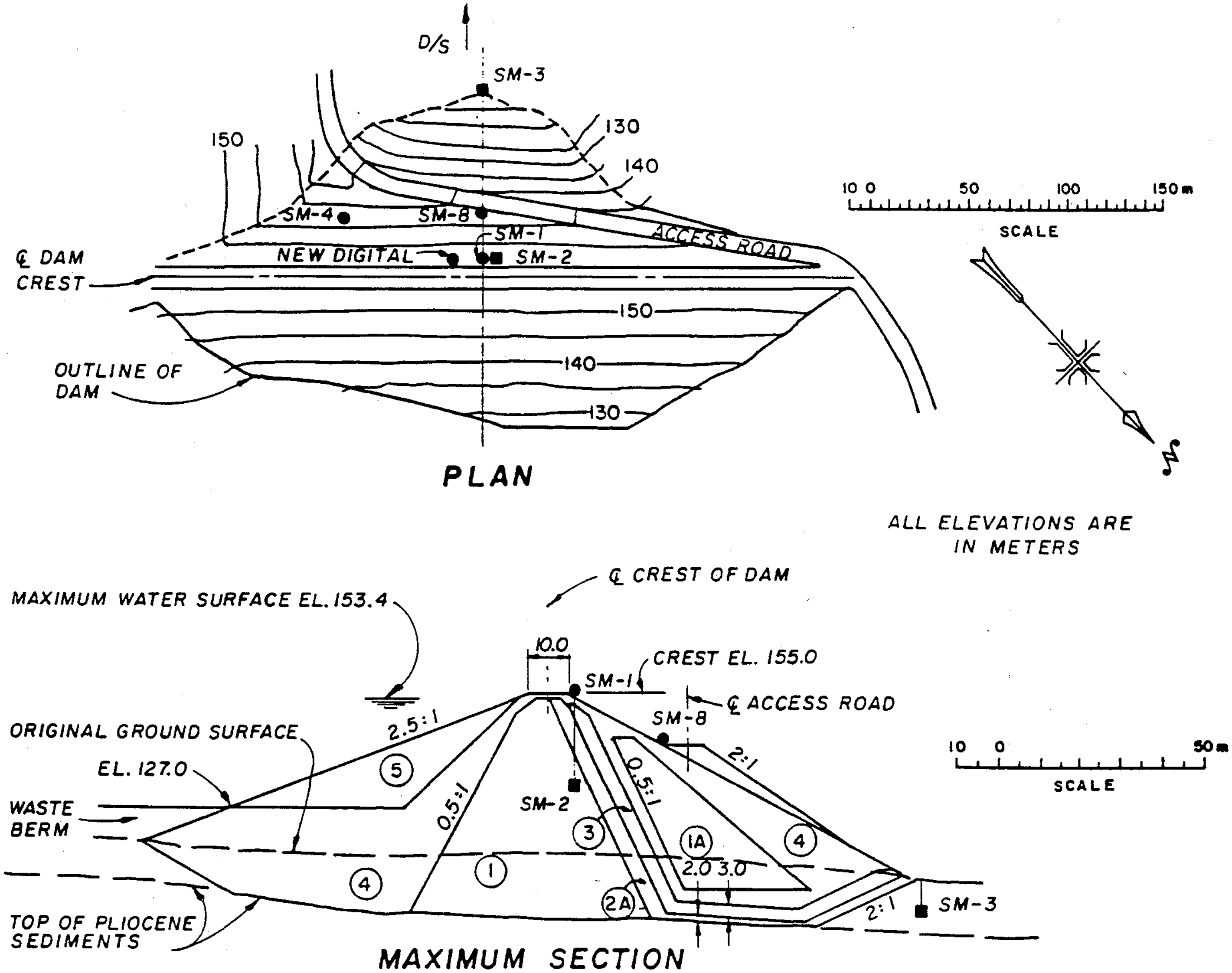
On October 17, 1989, an earthquake of magnitude 7.1 occurred along the San Andreas fault in west-central California. San Justo Dam, located only 1.5 miles (2.4 kilometers) from the trace of the San Andreas fault, just west of Hollister, CA, was designed to withstand such earthquakes. Although earthquake ground motions resulted in significant horizontal acceleration on the crest of San Justo Dam, the dam's performance was satisfactory and no detectable damage occurred.

SAN JUSTO DAM

San Justo Dam (Figure 1) was designed and constructed by the U.S. Bureau of Reclamation and is owned and operated by the San Benito County Water District. The dam is a zoned earthfill embankment with a clayey silty core, and double chimney and blanket drains, one of sand and the other of gravel (Figure 2). The upstream slope is 2.5:1 (horizontal to vertical) and the downstream slope is 2.0:1. The crest length is 1,085 feet, the hydraulic height approximately 131 feet, and the structural height 147 feet. Negligible natural drainage flows into the reservoir created by San Justo Dam and associated dike: reservoir water is supplied through pipelines, tunnels, and a pumping plant from San Luis Reservoir 25 miles to the east. Construction of the dam was completed in 1987.

The foundation of the dam consists of dense, variably-cemented, bedded clays and sands of Pliocene-Pleistocene age, and of both of marine and continental origins. Bedding attitudes in the reservoir area are mostly variable northwest strikes with low dips (less than 30 degrees) to the southwest; however, in the dam foundation, bedding dips range from 34 to 87 degrees southwest because of local folding.

GENERAL VIEW
San Justo Dam
Figure 1



EMBANKMENT EXPLANATION

1. SELECTED CL, SC, SM, AND ML MATERIALS COMPACTED BY TAMPING ROLLERS TO 0.15 LAYERS.

1A. SELECTED CL, SC, SM, AND ML MATERIALS COMPACTED BY PNEUMATIC TIRED ROLLERS TO 0.3 LAYERS.

2A. PROCESSED SP MATERIALS COMPACTED BY VIBRATORY ROLLERS FROM 0.3 LOOSE LAYERS.

3. PROCESSED GP MATERIALS COMPACTED BY VIBRATORY ROLLERS FROM 0.6 LOOSE LAYERS.

4. DECOMPOSED GRANITE MATERIALS COMPACTED BY PNEUMATIC TIRED ROLLERS FROM 0.3 LOOSE LAYERS.

5. LIMESTONE MATERIALS COMPACTED BY VIBRATORY ROLLERS FROM 1.0 LAYERS.

STRONG MOTION INSTRUMENTATION

LOCATION	INSTRUMENTATION
● SM-1 SURFACE	3 ACCELEROMETERS, AND CR-1.
■ SM-2 SUBSURFACE	3 ACCELEROMETERS
■ SM-3 SUBSURFACE	3 ACCELEROMETERS
● SM-4 SURFACE	SMA-1
● SM-8	3 ACCELEROMETERS

PLAN, SECTION AND INSTRUMENT LOCATION
San Justo Dam
Figure 2

THE OCTOBER 17, 1989 EARTHQUAKE

The October 17, 1989 Loma Prieta Earthquake occurred along the San Andreas fault system. The epicenter of the magnitude 7.1 (Ms) shock was centered in the Southern Santa Cruz Mountains near the Loma Prieta Lookout, hence the name. This event is the largest northern California earthquake since the 1906 San Francisco earthquake (M 8.3). The focal depth was 18.5 kilometers, deeper than most earthquakes along that zone. For 7 to 10 seconds after the initial rupture, rupture propagated upward and laterally along the fault plane. It was estimated from seismologic data that a 50 kilometer length of the fault ruptured. The top of the ruptured area was estimated to have extended upward to within 6 kilometers of the ground surface. Damage by extensive ground breakage and fissuring was abundant in the epicentral area, but no surface displacements along the San Andreas, or along the neighboring Zayante-Vergeles, and Sargent faults, were observed. This segment of the San Andreas fault last ruptured in the magnitude 8.3 San Francisco Earthquake of 1906, when it was displaced only about 1.2 meters as compared with 3 to 4 meters elsewhere along the fault. The largest aftershock of the October 17 event has been magnitude 5.0.

EARTHQUAKE EFFECTS AND PERFORMANCE

San Justo Dam was far enough from the epicenter of the October earthquake and the San Andreas Fault to not have been in the area of ground fissuring. No ground offsets and no new landslides were found in the immediate vicinity of the dam. At the time of the earthquake, the reservoir was 48 percent full.

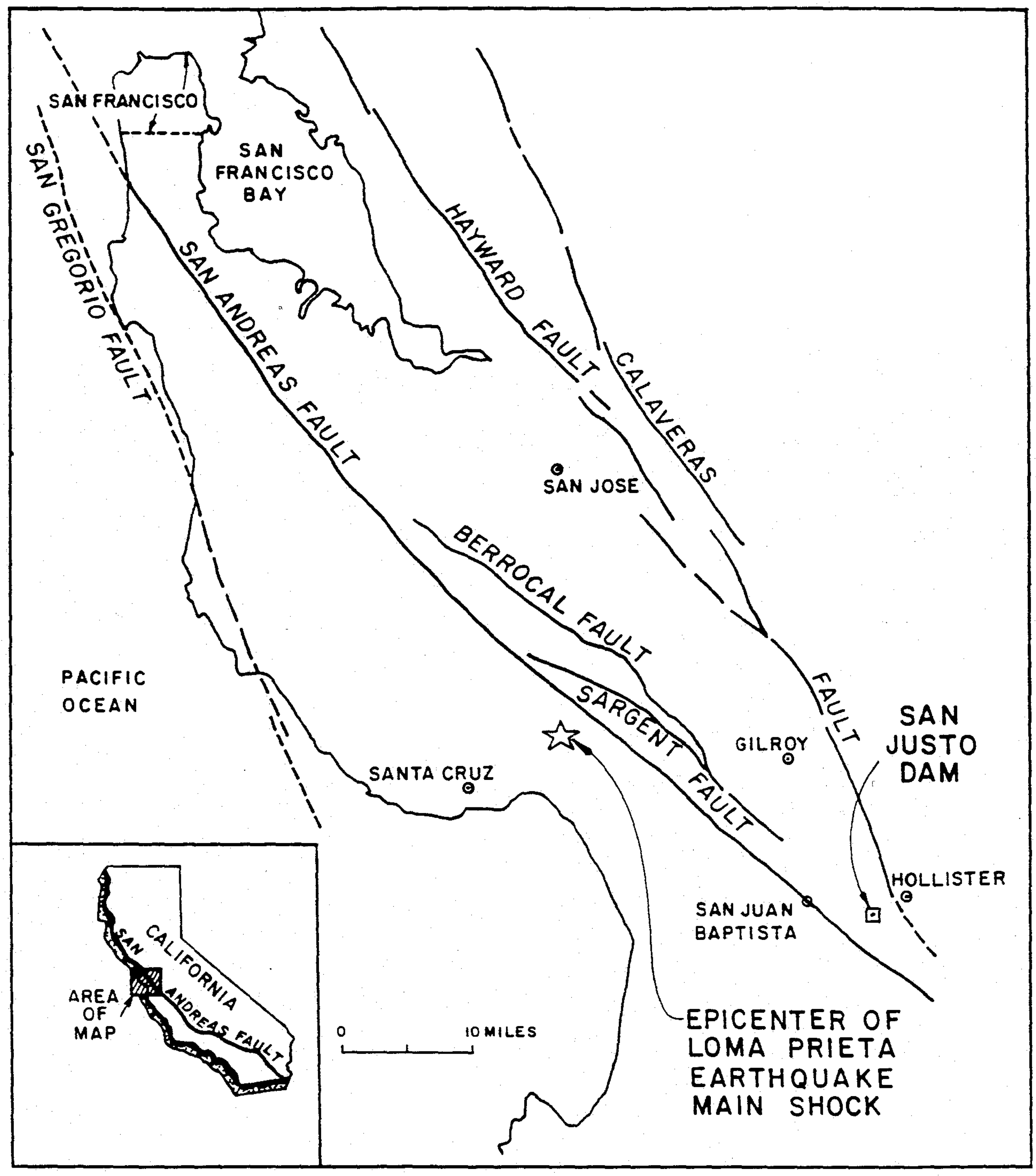
Numerous measurement points, on both the upstream and downstream faces of the dam, have been monitored since mid-1986. Locations both upstream and downstream of the center of the dam crest indicated a slight increase in the rate of crest settlement during the time interval spanning the earthquake. This entire increase is presumed to be due to the earthquake; consequently, earthquake-induced deformations were in the range of 1/2 to 1-1/2 inches (1.5 to 3.8 centimeters) of settlement, an almost negligible effect. Horizontal deflection data were inconclusive.

Less than half of the various types of piezometers in the embankment, in the dam foundation, and in nearby "natural ground," responded to the shaking. Piezometer levels both rose and fell, and differed from their anticipated normal levels by a trace to over 10 meters. Most of the affected piezometers returned to normal levels in 10 to 30 days.

INSTRUMENTATION AND STRONG MOTION RECORDS

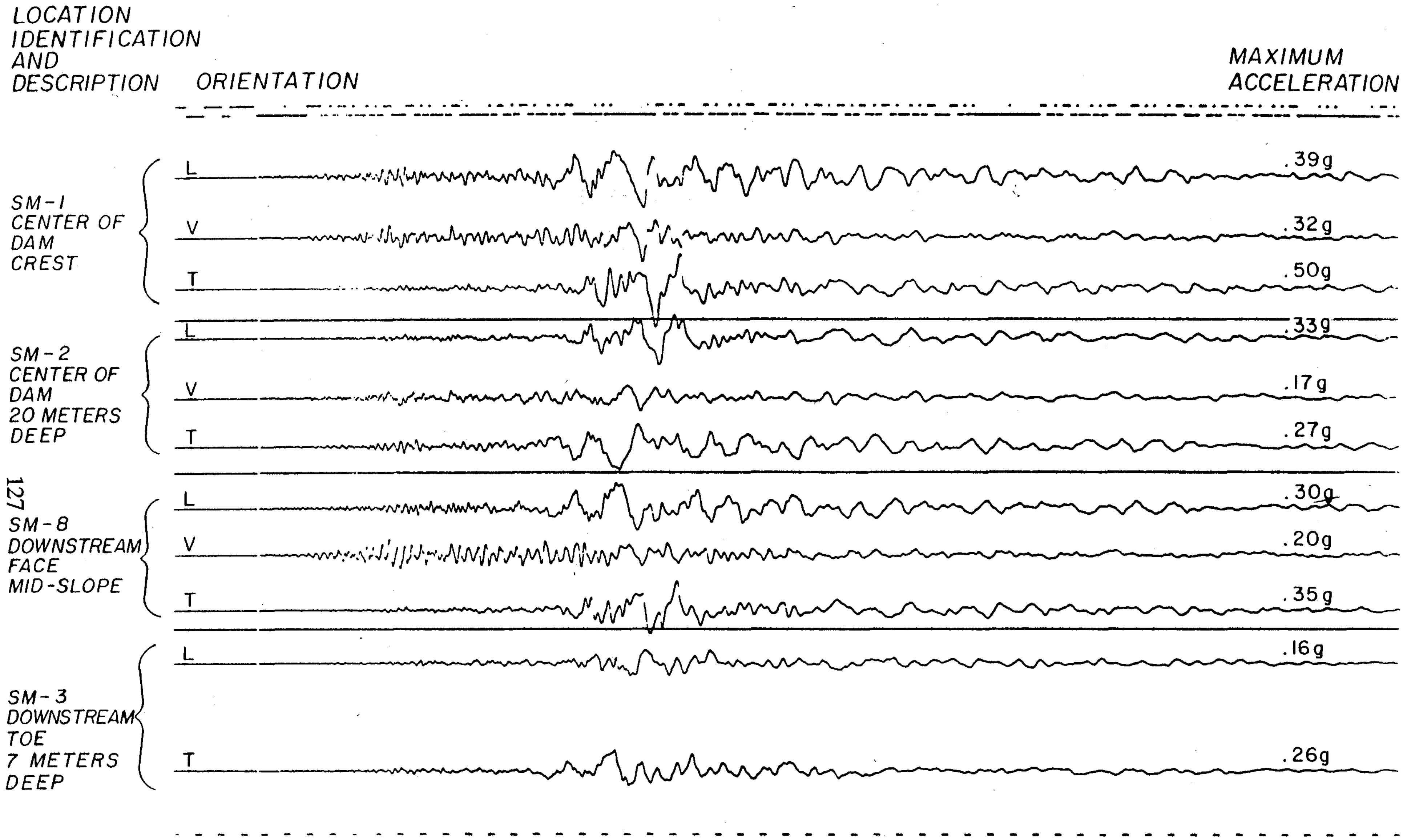
The location of San Justo Dam, between the major, highly active San Andreas and Calaveras faults (Figure 3), suggested that strong-motion occurrences would be more numerous at this dam than any other Bureau of Reclamation structure. Hence, San Justo Dam is well-instrumented with strong-motion recording equipment. The layout of the instruments is shown on Figure 2. Twelve accelerometers located in, on, and at the toe of the dam are hard-wired to a CR-1 central recorder on the crest. One self-contained, three-component system is on the downstream face of the dam near the left abutment, and another is on the reservoir dike (not shown). All of the San Justo Dam records from the Loma Prieta earthquake are in analog form. Since the earthquake, however, a previously planned three component, digital recording system was installed on the dam crest and is now in operation.

The Loma Prieta main shock strong-motion records from San Justo Dam are shown in Figures 4 and 5. The maximum acceleration recorded at the dam was 0.50g at the center of the dam crest. That component of motion was horizontal and transverse to the axis. The ratio of amplification between crest and toe accelerations is about 2. Observations from other instrumented dams indicate that a 1.5 to 2.5 amplification factor of crest response motion to base motion may be typical of the level of shaking experienced by San Justo Dam, when crest and toe records are compared. There is generally a tendency to have higher amplification factors with increased height of dams and lower levels of base motion. The



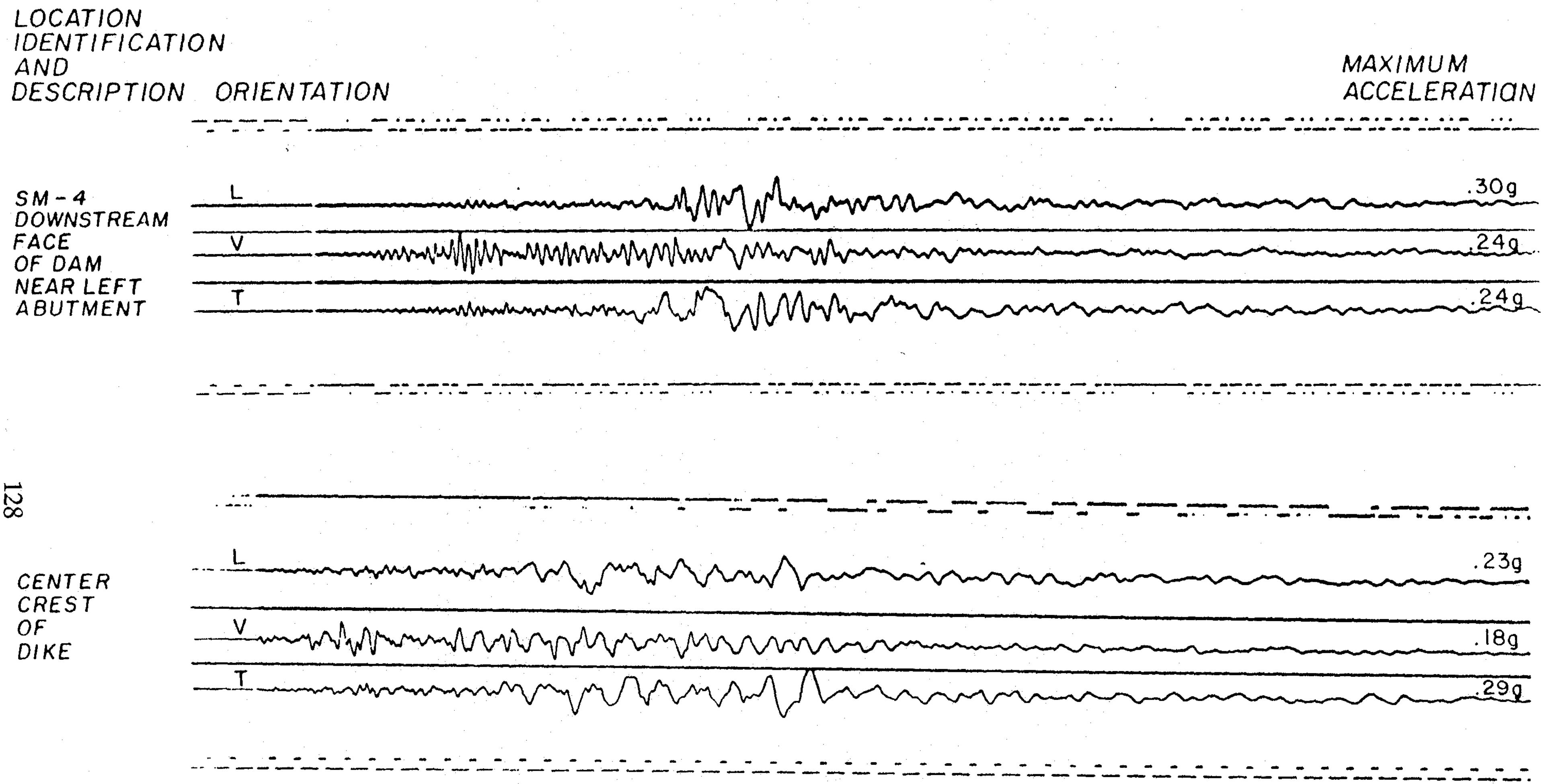
DAM AND EPICENTER LOCATIONS, OCTOBER 17, 1989
LOMA PRIETA EARTHQUAKE

San Justo Dam
Figure 3



San Justo Dam accelerograms from 4 locations. Orthogonal, three-component sets of accelerometers are at each location, and are connected by hard-wire to a central recorder. A minor equipment (galvanometer) failure caused the downstream toe vertical component to be lost.

STRONG MOTION RECORDS OCTOBER 17, 1989
LOMA PRIETA EARTHQUAKE
San Justo Dam
Figure 4



San Justo Dam and Dike accelerograms. Two three-component accelerograms recorded on SMA-1 instruments.

STRONG MOTION RECORDS OCTOBER 17, 1989
LOMA PRIETA EARTHQUAKE
San Justo Dam
Figure 5

amplification factor of response/input motions at San Justo Dam is probably mostly due to the dam's height. However, reservoir volume, dam geometry, construction materials and methods, etc., must also be considered as other factors that may affect amplification of seismic motion from the base to the crest of the dam.

CONCLUSION

San Justo Dam was designed for larger seismic loading than the 0.50g horizontal acceleration it experienced at the crest during the Loma Prieta earthquake. There was no damage and no significant deformation from that earthquake.

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