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**DAMAGE AND CRACKING OF
EMBANKMENT DAMS BY EARTHQUAKES
AND THE IMPLICATIONS FOR
INTERNAL EROSION AND PIPING**

BY

STEVEN PELLIS AND ROBIN FELL

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**DAMAGE AND CRACKING OF EMBANKMENT DAMS BY EARTHQUAKES
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7. Abstract:

Embankment dams settle and develop longitudinal and transverse cracks when subject to large earthquakes. A database of case studies and the literature have been analysed to study these phenomena. Methods are presented for estimating the damage based on settlement and cracking, and for estimating the maximum crack widths and depths from the dam type and earthquake magnitude and peak-ground acceleration.

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1 INTRODUCTION

When embankment dams are subject to seismic (earthquake) loads, they settle, deform laterally and longitudinally, and may exhibit longitudinal and transverse cracking. This cracking may lead to the initiation of internal erosion and piping failure of the dam. This report gives guidance, based on analysis of the literature, and case studies, on the likelihood and nature of this cracking.

Data was compiled into two forms:

1. gathering basic data on dams, earthquake and damage details;
2. gathering more detailed case studies.

The basic data was compiled in a database comprising over 300 dam incidents, and includes parameters describing the nature of the dam, the earthquake and basic details of earthquake induced damage. This data set was used to examine the performance of dams under seismic loading and investigate relationships that may exist between the damage and the earthquake characteristics, and the dam and foundation conditions. To aid these investigations, a damage classification system is introduced, which allows classification of the extent of damage incurred at the dam, based on cracking and settlement. Relationships that were investigated include:

- damage and seismic intensity
- settlement and seismic intensity
- transverse cracking and dam conditions
- transverse cracking and seismic intensity
- longitudinal cracking and dam conditions
- longitudinal cracking and seismic intensity

The detailed case studies are essentially compilations of information given by other authors, from sources including research papers and consultants and owner's reports. The studies form a more detailed data set that allows closer examination of the nature of damage that may occur and of the influence that the foundation and embankment conditions may have on the type and extent of damage that occurs.

The report considers embankment dams, including earthfill, rockfill and concrete face rockfill dams. Tailings dams been excluded from the study.

Liquefaction is recognised as a determining factor in dam damage. Where liquefaction occurs, it controls the response of the dam to the earthquake . This report mainly analyses dam response for cases that do not incur liquefaction. A brief discussion is given on the nature of liquefaction, and the effects it has on dam response, and some recent information on cracking of dams subject to liquefaction is included.

The main reason for doing work for the report was to characterise cracking in embankment dams under earthquake, as this might lead to internal erosion, and potentially piping failure as reservoir water seeps through the cracks formed by the earthquake.

This report is mostly taken directly from the Master of Engineering Science Thesis of Mr. Steven Pells. He did the thesis under the supervision of Professor Robin Fell, with financial support from the research project "Research into Internal Erosion and Piping Failure of Embankment Dams".

2 LITERATURE REVIEW

2.1 Significant Papers in the Formation of a Database of Damage to Embankment Dams Subject to Earthquake Loading

Ambraseys (1960), On the Seismic Behaviour of Earth Dams

Apparently at this time a common understanding on the performance of earth dams was that a dam with an adequate factor of safety under static conditions also had ample stability under dynamic conditions. This idea, Ambraseys believed, was based on the 'spectacular performance' of earth dams along the San Andreas fault in the 1906 San Francisco earthquake. However, in the introduction to this paper Ambraseys declared that there was need for research into the field of seismic damage to embankment dams saying that, outside of the San Francisco event, 'of the limited number of dams...that have been subject to... earthquakes, a great percentage were severely damaged'.

He argued that a systematic and rational development of earthquake resistant design was required and that it would stem from the knowledge of the actual behaviour of such structures during an earthquake. The reports available on this matter were widely scattered in literature and there were 'several sets of conflicting statements concerning the effects of earthquakes'. In this report Ambraseys wished to make a summary of all available information.

For this report, he compiled information sourced from 'a large number of references and private communications'. He covered 58 dams as affected by over 25 earthquake events. Due to lack of space, the cases were discussed briefly and no conclusions were presented. A summary table including basic dam, earthquake and damage details was given. Sherard *et al*, (1963), referred to this paper as 'the first comprehensive collection of records of earthquake damage'.

Sherard, Woodward, Gizienski and Clevenger (1963), Earth and Earth Rock Dams

This book, which was an important publication on embankment dams in its own right, included a section on earthquake damage. The information discussed is sourced largely from Ambraseys' article, but Sherard *et al* forwarded some theories and observations:

- That two primary forms of damage to embankments result from seismic loading – longitudinal cracking and settlement. The crest settlement is usually not enough to threaten failure by overtopping.
- Only one case of complete failure occurred, and it was 'probably due to liquefaction'.
- That damage appears to be caused most by shaking in the direction transverse to the dam axis.
- That some amplification of shaking occurs such that accelerations at the crest may be in excess to that at the base
- That earthquake shocks have produced remarkably few slides in embankments
- That there is some evidence that greater damage is induced by shaking of lower frequencies than those which cause most damage to buildings – suggesting a reason for cases where greater damage was induced in dams further away from the epicentre
- That due to 'relative vibration', dams with concrete cores sustain more cracking damage.

In another section of the book, Sherard et al (1963) discussed in detail the nature and mechanisms of embankment cracking under static loading. These insights have been important in understanding the nature of cracking that occurs from seismic loading.

Seed, Makdisi & De Alba (1978), Performance of Earth Dams During Earthquakes

This paper set out to investigate the observation that some dams suffer severe damage when subjected to minor earthquake events and others survive severe shaking with little or no damage.

At the time of writing this paper, Seed et al (1978) states that earthquake damage has been the subject of much study, and that major advances have been made – especially in the development of dynamic analyses. However, a re-evaluation was prompted by the near failure of the Lower San Fernando dam in the 1971 San Fernando earthquake. They argue that the successful use of dynamic analysis requires understanding and judgement – and that this can only be gained through intimate knowledge of the analysis techniques and by studying the behaviour of similar embankments under actual loading conditions. Thus, like the Ambraseys (1960) paper, Seed et al (1978) summarise available information on the behaviour of actual case studies. Their examination focussed on 6 major earthquakes (4 of which were also covered by Ambraseys (1960)), including the 1906 San Francisco earthquake, but also briefly mention a further nine events.

Seed et al (1978) examined the excellent performance of dams in the San Francisco earthquake, giving consideration to the major factors of section geometry, construction method, type of embankment and foundation material. No correlation was found between the performance of the dams and their section geometry, and the construction methods used for these dams were considered, if anything, to have had a negative effect on the performance. Seed et al (1978) proposed that the excellent performance could be attributed to the fact that the dams were in most cases constructed from clayey soils of low to medium plasticity.

From the examination of case histories, the following conclusions were reached:

- Hydraulic fill dams are particularly susceptible to earthquake damage.
- Liquefaction is the cause of poorer performance of saturated cohesionless soils

- Virtually any well built dam (including hydraulic fill) can withstand moderate shaking (<0.2 g) without any detrimental affects
- Rockfill dams that are kept dry by some facing should be able to withstand extremely strong shaking.
- Dams constructed of clay soils have withstood shaking of 0.35 to 0.8g without any damage
- A number of dams were reported to fail some 24 hours after the earthquake event which was attributed to piping failure and thus the need to provide an adequate system of filter materials is emphasised.

Serafim and Oliveira (1987), Effects Caused by Earthquakes on Dams

The purpose of the article was to begin the formation of ‘world data bank’ on the performance of dams under earthquake loads. It comprises a review of reports obtained from various countries and the performance of different types of dams. The article produced two tables of data (the beginning of this ‘world bank’); one of earthquake damage to dams and the other of the earthquakes that damaged them. The table of dam damage incidents had over 100 entries, including dams of all types.

In a brief examination of the database, Serafim and Oliveira (1987) offered some interesting conclusions:

- that “all types of dams have resisted earthquakes very well, much better than other structures”,
- that “only low dams, not well built and founded on soft soils are usually affected”. The authors claim that statistics show that “dams above 60m high never suffered anything due to earthquakes”. The reason given for this is that the resonant period of large dams is “beyond the usual important frequencies in seisms”,
- that “younger dams have been more often damaged than older ones”,
- that “damage never occurred in dams subjected to earthquakes with a local intensity of VI or less”,
- that a few rockfill dams have shown a “somewhat defective performance”, withstanding only 0.15g ground shaking before damage. However, they do not distinguish between dams that were constructed with compaction of the rockfill, and those that were not.

They also concurred with the conclusions made by Seed et al (1978). There were plans that the inventory would be continued to some degree by the later ICOLD activities.

Hinks, and Gosschalk, (1993), Dams and Earthquakes – a review

This paper has a similar concept to papers by Seed et al (1978) and Ambraseys (1960) - that dams have been damaged by earthquakes and that case studies should be examined that we may learn from them. The paper essentially is a review of previous case study compilations, and offers little new information. One important observation is that most complete failures have been due to liquefaction.

U.S. Committee on Large Dams (USCOLD) (1992), Observed Performance of Dams During Earthquakes

This publication presented a compilation of details available on the performance of all dam types under seismic loading. The data was presented in two formats: an inventory which included basic details of dams and earthquake, and a one word damage descriptor; and, 11 detailed case studies. The inventory was comprehensive including over 150 entries ranging from the 1906 San Francisco earthquake to the 1989 Loma Prieta earthquake.

No attempt was made by USCOLD to systematically analyse data from the inventory, but some key observations on the status of the study of seismic damage of dams were included from members of the USCOLD committee:

- performance of embankment dams have shown great variability;
- performance of embankment dams is closely related to the materials used for construction;
- embankments built of compacted clayey material on clay or bedrock foundations “have historically withstood extremely strong levels of ground motion, even when obsolete or inefficient means of compaction procedures were used”;
- older embankments built of inadequately compacted sands or silts (e.g. hydraulic fill dams) represent nearly all the known cases of failures – primarily due to liquefaction;
- rockfill and concrete fill dams “are considered to be inherently stable under extreme earthquake loading”.

It was stated that, to the date of publication, an excellent record of performance was enjoyed by dams on a worldwide scale, but that this may be largely due to the fact that few dams have been shaken very severely.

International Commission on Large Dams (ICOLD) (1999), Guidelines on Design Features of Dams to Effectively Resist Seismic Ground Motion

This paper was of similar format to USCOLD (1992), including a large inventory and 15 detailed case studies, but included details from earthquakes from 1990 to 1994. There was also an additional but separate inventory on the performance of Japanese dams.

The purpose of the article, apart from providing data compilations, was to use the data to aid formulation of structural considerations required to effectively resist seismic loading. A summary of performance given at the start was in many ways just a re-iteration of observations made by Seed *et al* (1978).

2.2 Papers Utilising Case Studies to Examine an Aspect of Damage to Dams Under Seismic Loading

Lane (1980), The failure of earth embankments by cracking during earthquake

Lane (1980) stated that the study aimed to provide a theoretical explanation for cracking, based on observation, which could be used in numerical analysis. Lane noted that visible damage usually takes the form of “vertical cracks parallel to the dam axis” (longitudinal cracks). He proposed that this was the same phenomenon as ‘lurching’ – the cracks parallel to a natural slope escarpment forming after earthquake loading.

His theory of longitudinal cracking about the crest is based on understanding of amplification of shaking through the dam such that near the crest, horizontal and vertical forces exceed friction forces. Cracking may or may not occur depending on the shape of soil zone providing support about the shaken area, and the cohesive forces of the soil. The theory is concordant with the location and depth of longitudinal cracking and the observation by Seed *et al* (1978) that clay dams had high resistance to earthquake loading. The paper offered no case studies and was thus more a theory prompted by observation than an examination of dam behaviour.

Vick (1993), Effects of Seismic Shaking on Internal Erosion of Embankment Dams

The study was prompted by a dam safety review of the Terzaghi Dam (also known as Mission Dam), as an investigation into the possible mechanisms of piping following earthquake. To aid understanding of the risks relevant to Terzaghi Dam, Vick (1993) wished to make comparisons with other similar dams. He considered 14 dams, which all had history of internal piping damage prior to being shaken by an earthquake event. Of these dams, only two had failed due to seismic loading and they were both Japanese dams, for which little information was available.

Vick (1993) found that peak ground acceleration (PGA) was the best predictor of such damage, and that a PGA of 0.05 to 0.1g 'discriminated well between dams that experienced some piping and those that did not'. For failure through piping, this figure was taken as 0.3 to 0.4g. No evidence that the threshold for piping reactivation was reduced with repeated earthquake loading was found. Vick (1993) also found that, for dams which had some piping history prior to earthquakes, embankments of clayey soils were just as susceptible to reactivation of piping from seismic loading as those of cohesionless soils.

Fong and Bennett (1995), Transverse Cracking on Embankment Dams Due to Earthquakes

The authors' position in the Department of Water Resources, the Division of Safety of Dams, California required them to inspect a number of dams after earthquake events. The hazard that transverse cracking poses was recognised and their study was performed to evaluate that risk. A number of factors that were perceived to affect the incidence of transverse cracking were identified:

- Height and geometry of the embankment
- Construction and compaction techniques
- Seismic Intensity
- Saturation (degree of saturation of the earthfill)
- Reservoir Level
- Shape of Canyon
- Abutment and Foundation profiles
- Internal structure and embankment zones

- Soil type and plasticity in the embankment
- Other localised items such as buried utilities.

Using data that was available, positive correlations between transverse cracking depth and settlement ('D'), dam height ('H') and crest length ('L') were produced. These are shown below in Figure 1 and Figure 2.

The amount of data gathered on transverse cracking in the paper is small, but is seen to represent the most comprehensive of any articles found to date on the topic.

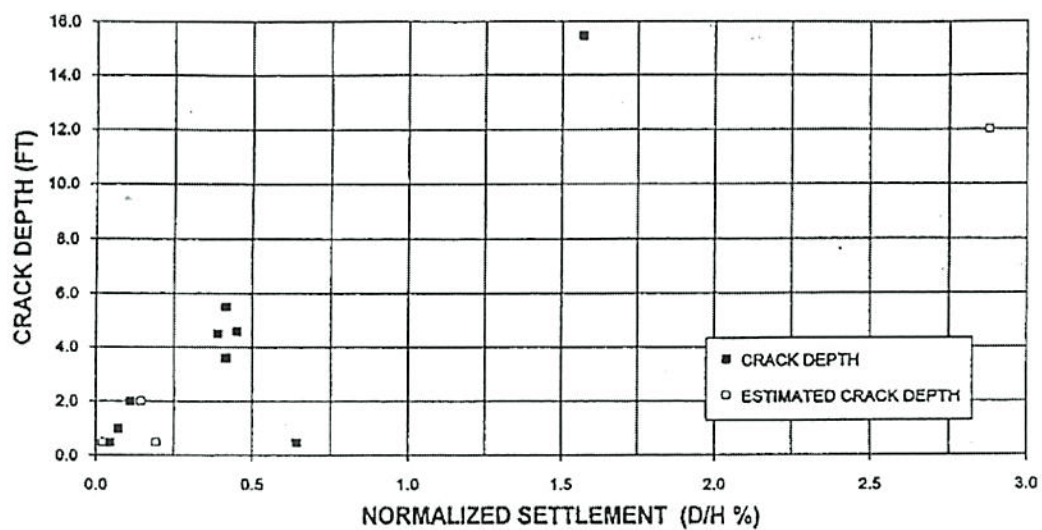


Figure 1 Transverse crack depth vs normalised settlement (Fong and Bennett, 1995)

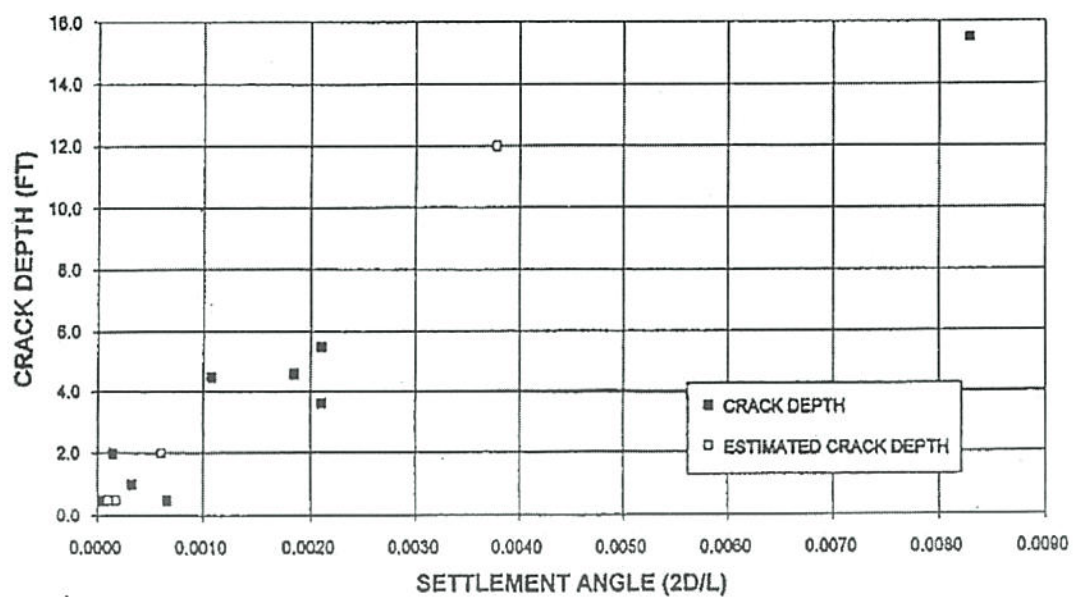


Figure 2 Transverse crack depth vs settlement angle (Fong and Bennett, 1995)

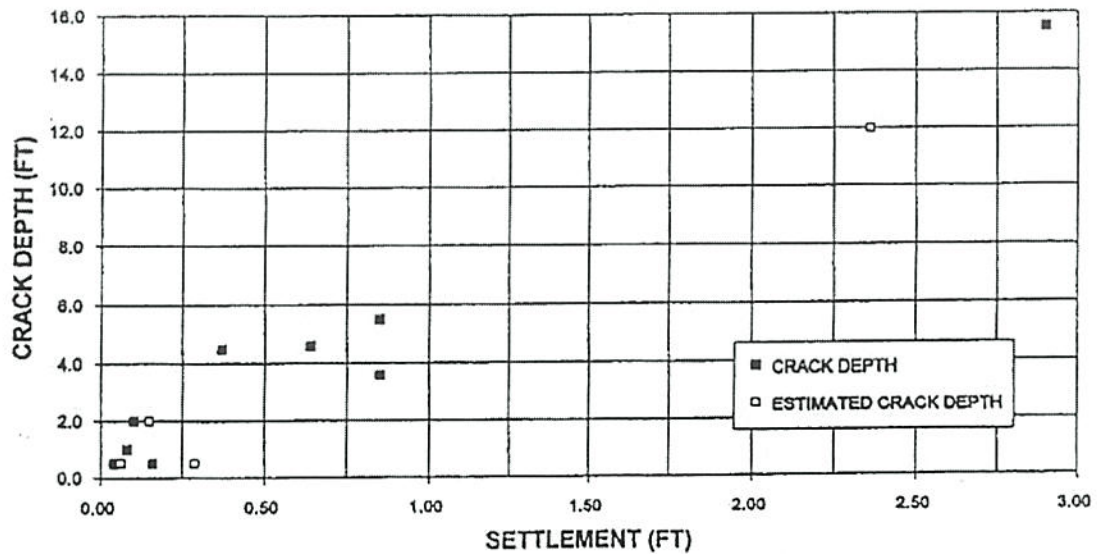


Figure 3 Transverse crack depth vs settlement (Fong and Bennett, 1995)

Swaisgood and Au-Yeung (1991), Behaviour of Dams During the 1990 Philippines Earthquake

Swaisgood (1995), Estimating Deformation of Embankment Dams Caused By Earthquakes

Swaisgood (1998), Seismically-Induced Deformation of Embankment Dams

This series of papers were precipitated by a task involving the evaluation of stability of embankment dams in the Philippines. The investigation of a number of case studies of Philippine dams in 1991 offered the opportunity to examine the important factors involved in the relationship between settlement of an embankment and seismic intensity

A relationship between deformation, peak ground acceleration and earthquake magnitude was developed in the first article, using just the Philippine case studies. In 1995, this relationship was reviewed with the use of a database including 54 cases. The review showed that additional factors were important. In total, the relationship included factors for:

- Earthquake magnitude
- Peak ground acceleration
- Dam type
- Dam height
- Thickness of alluvium in the dam foundation

A basic premise to the study was that settlement could be considered an index of damage.

In 1998, another review was published, using the same database. In this paper, the critical criteria for estimation of settlement were re-examined, and were changed to:

- Earthquake magnitude
- Peak ground acceleration
- Dam type
- Distance from causative fault.

That is, dam height, and thickness of alluvium in the dam foundation were no longer included, but distance from the fault was added. Swaisgood (1998) states that these ‘other factors’ were considered to have only minimal affect. Of the accuracy of the new method he states that estimation of settlement “cannot be improved until estimation of PGA can be improved”.

In the 1998 article, a brief discussion of the various types of seismic induced damage observed is given, including liquefaction, settlement / deformation, sliding / shearing and cracking. Of liquefaction, Swaisgood (1998) makes the important observation that with modern design and construction techniques liquefaction damage to embankments is an increasingly rare event.

Forster & MacDonald (1998) Post-Earthquake Response Procedures for Embankment Dams – Lessons from the Loma Prieta Earthquake

In this paper, embankment dams damaged by the 1989 Loma Prieta earthquake were examined with the aim of formulating an effective post-earthquake response procedure. In doing so, the paper provided detailed documentation of the earthquake damage to eight embankment dams and included important observations on the nature of earthquake damage. The following observations were made of ‘typical’ dam damage:

- **Transverse cracking near the abutments.** Forster and MacDonald (1998) reported that transverse cracking occurred near or at the abutments on most of the embankments that were near to the epicentre. They proposed that such cracking was due to “differential movements between the embankment and the abutments, or the result of out-of phase shaking [between these units]”.
- **Longitudinal cracking near the dam crest.** Was reported to be the most common form of damage found. Forster and MacDonald (1998) found such cracking to occur at the

crest and was usually the result of settlement and downslope movement, but also due to phase differences in shaking between different zones. They also suggested that longitudinal cracks may represent the failure surface of a slide in the dam slope.

- **Settlement and/or upstream/downstream movement of the crest.** Forster and MacDonald (1998) reported that such movements occurred at most of the embankments that were near to the epicentre. Settlements were described to be usually due to consolidation, but they may be an indication of liquefaction in the embankment or the foundation. Changes in alignment were also reported to be common, and were described to only be of concern if such deformations resulted in internal cracking.
- **Increased or new seepage.** Increased seepage was reported at some dams after the earthquake. Increasing seepage and turbid outflows may be an indication of piping or internal erosion. However, in the cases described by Forster and MacDonald (1998), the seepage returned to pre-earthquake conditions within a month after the event.

(**Authors note:** This could be due to sealing of the cracks by swelling, or by filters or transition zones eventually controlling erosion).

- **Differential Movement at Concrete/Embankment Junction.** Forster and MacDonald write “because of the different strength and deformation properties of concrete and embankment material, and junction between these material is susceptible to differential movements and consequent cracks and openings”.
- **Piezometer Level Rises.** An earthquake-induced rise in piezometer levels was observed in two cases. In both cases the high levels subsided over months after the event. The rise in piezometric level was attributed to the effects of consolidation or deformation, and was described as only being of concern if stability of the embankment slopes was marginal.

(**Authors note:** This could be due to the dissipation of earthquake displacement induced positive pore pressures deep in the dam where the soils are nearly normally consolidated, and contractile on shearing).

- **Slope Failures and Deep - Seated Shears.** No cases of sliding were reported in the dams examined in this report. However, the authors acknowledge the potential for earthquake-induced slides.

Tani & Nakashima (1999), Earthquake damage to earth dams in Japan – maximum epicentral distance to cause damage as a function of magnitude.

This study of earthquake damage to several small agricultural dams in Japan highlighted a limiting distance for a given magnitude of earthquake outside of which no damage occurred ('critical epicentral distance'). This distance was found to increase with magnitude. A similar relationship with distance, magnitude and instances of liquefaction was found, leading to the conclusion that 'liquefaction to the foundation ground is the main cause of damage to earth dams'.

Although the 'critical epicentral distance' is a credible relationship for both dam damage and liquefaction, the premise that, because they are similar, dam damage is therefore due to liquefaction, is considered speculative. However, if it is a correct view, it can only be seen as a situation-specific phenomenon. There are many dams that are damaged by earthquakes that show no signs of liquefaction of the foundations – for example, those founded on rock.

2.3 Other Papers

There are numerous articles written on numerical and dynamic analysis of embankment dams under seismic loading which, although offering insights into the same area, are considered to be complementary, but of a different nature to this study.

2.4 Literature Summary

A conviction that is echoed throughout the literature is that developments in understanding of seismic damage of embankments will follow the close examination of case histories. A number of papers have been produced with the aim of gathering together available data on case histories. These papers have each built on the databases of the paper before, and there have been plans to begin a 'world data bank' of case histories on the performance of dams under seismic loading.

Various observations have been made from authors studying these databases. Some of the key observations include:

- That two main forms of cracking are produced on embankments by earthquakes – cracking longitudinal to the dam axis and cracking transverse to the dam axis. Longitudinal cracking is more common, but transverse potentially more dangerous. Both crack types usually form at and near the crest, with transverse cracking usually occurring near the abutments and adjacent to spillway structures.

- Most complete failures of embankments have been due to liquefaction in the foundation, of hydraulic fill, or of upstream-construction tailings dams.
- The performance of dams constructed from clay materials has been described as excellent and it has been stated that any well built dam can withstand shaking less than 0.2g without significant damage. This concept is based largely on the performance of dams in the 1906 San Francisco earthquake.
- Rockfill dams have reported good performance, and especially those with concrete facing.
- Hydraulic fill dams have been found to be particularly susceptible to seismic loading.
- The possibility of internal erosion and piping events following earthquakes has long been recognised, with a few case studies proving the risk to be real. These cases have usually occurred at dams that have shown susceptibility to internal erosion by suffering piping events before the earthquake. There are also other cases where it seems likely that piping did not occur only because the reservoir level was low at the time of the earthquake.
- There are a number of different explanations for the formation of cracks.
- Relationships have been found between cracking, settlement and seismic intensity.
- Amplification of shaking has been found to occur through an embankment such that peak accelerations are often found to be greatest near the crest of the dam.
- In general, dams have reported excellent performance, but it is said to be largely due to the fact that few dams have been shaken very severely.

3 DETAILS OF DATA USED IN THIS STUDY

3.1 Data Sources

Data was drawn from a number of sources including published papers, earthquake reports, dam incidence reports, dam construction reports, owners reports, internet web sites and published databases.

Where available, such papers and reports were taken from journals and books in various Australian libraries, or downloaded from internet based catalogues. Efforts were also made, with limited success, to obtain literature from engineering consultants who had been

involved in investigative or remedial aspects of embankments that had suffered seismic damage.

A number of USCOLD internal newsletters provided much useful information, and the generosity of USCOLD in providing them is acknowledged.

Internet based databases that were used include:

- For information on dam properties - The Resources Agency, Department of Water Resources, Division of Safety of Dams 'Dams Under Our Jurisdiction' – database at <http://damsafety.water.ca.gov/bltn17/bul17.txt>
- For information on earthquakes - USGS National Earthquake Center Database at http://neic.usgs.gov/neis/epic/epic_global.html

The latter website was also used to get latitudinal and longitudinal locations of dams, based on the location of seismic instrumentation within the dam as given in their 'station book' database – see http://www.neic.cr.usgs.gov/neis/station_book/station_book.html .

For general dam information, internet sites run by dam owners / managers were also used where applicable. These sites included:

- The Santa Clara Valley Water District, at <http://www.scvwd.dst.ca.us/>
- The United States Bureau of Reclamation, at <http://dataweb.usbr.gov/>

Online newspapers or other more obscure tourism / interest sites from various countries sometimes offered small contributions of information to the case studies. Where used, these sites are referenced after the appropriate studies.

Where data conflicted, efforts were made to resolve the reason for conflict and ascertain the correct values. Values were preferred from sources that were considered to be more reliable, as were values which enjoyed agreement between authors. Notwithstanding this, there were many instances of data conflicts that were not resolvable and there was no alternative but to take a value on faith of one author over another.

3.2 Database Details

For the database, the data entry process started with using published reports. For each incident that a dam experienced was described by an author, a line was added to the database. Each column in that line was filled out as much as possible using data provided by that author. Where data was incomplete, a “-” symbol was placed. Many of the entries were taken from other inventories provided in the references cited in the literature review.

The data was then sorted, grouping all entries of the same dam together, and a summary line was created using the sum of information available on that dam. Searches were then performed on the internet databases listed above to obtain additional information, or to fill in any gaps that were remaining. The USGS seismic database was particularly useful here in obtaining details of the earthquakes.

3.2.1 Explanation of data

An explanation of the data in each column in the database is given below:

3.2.1.1 Dam Description

Dam Name

The name of the dam or reservoir, whichever is most commonly used by authors to describe the embankment.

Country

The country in which the embankment is located

Long E

The longitudinal coordinates of the dam location, taken as positive east, negative west in degrees from Greenwich. Values were rarely quoted in the literature and were obtained mostly from the databases listed above. The location of dams in the USA was often determined based on the location of seismic instrumentation within the dam – for which there was a listing in the USGS National Earthquake Information Centre database. No effort was made to distinguish between sources.

Lat N

As above, except for latitudinal coordinates, taken as positive north, negative south, in degrees from the equator.

Owner

The owner of the dam / reservoir. This data was taken from literature or from the databases listed above.

Ref No.

Each number represents a reference used to obtain data for that dam and/or event. The references are listed according to these numbers at the end of this report.

Height

The height of the dam (H) in metres. Where possible, structural height (the height from the dam crest to the lowest foundation level) was chosen, but in most cases height was given without specification. The height was usually quoted in literature, but was also available from owners websites and databases of the 'Division of Safety of Dams' (U.S.A.), of the 'Bureau of Reclamation' (U.S.A), and of the ICOLD international listing of dams.

Type

A brief description of type of dam construction. Where section drawings or detailed descriptions of the embankment dam were available, the terms used to classify the dam were taken from Figure 4.

In most instances, drawings or detailed descriptions were not available, so the terms used to describe the embankment type were taken as those used by authors describing that dam. No tailings dams of any form were included.

Crest Length

Crest length (L) in metres as quoted in literature or in the above databases.

L/H

The ratio of crest length to height.

Year Completed

Year in which construction of the dam was completed.

Slope Batter

Slope of the upstream and downstream slopes - horizontal distance per unit rise. Values separated by semi-colons represent various slopes existing over the upstream or downstream slope.

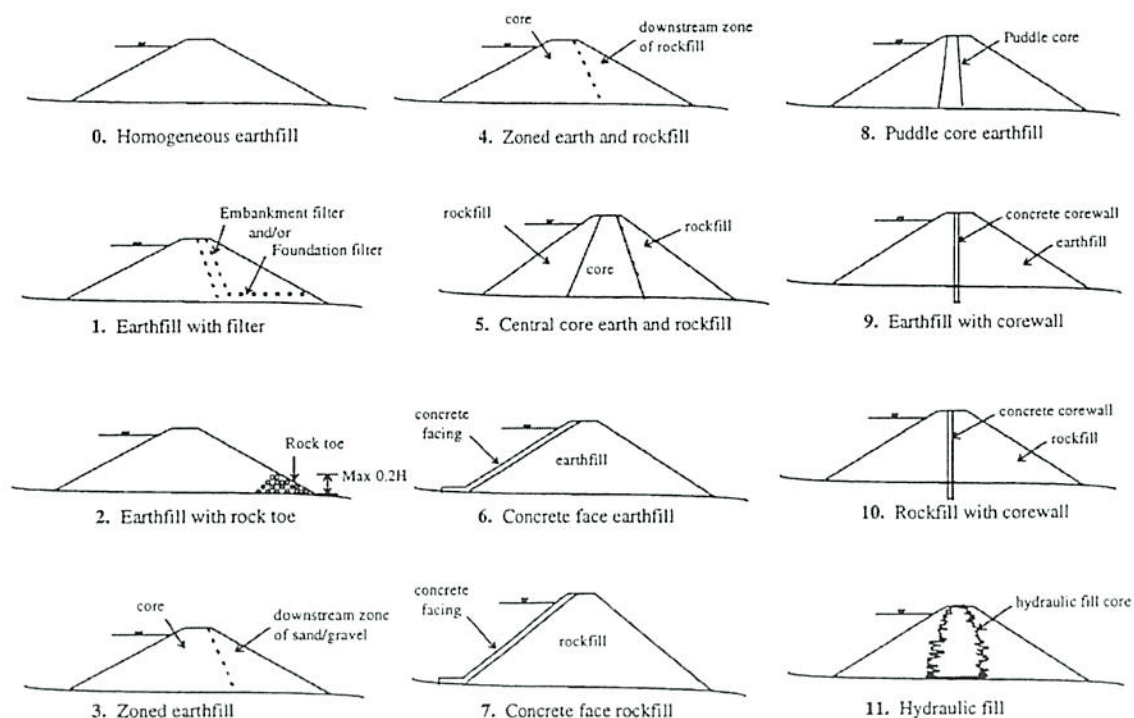


Figure 4 Dam zoning categories (Foster, Fell and Spannagle, 1998)

Freeboard at Quake

Distance, in metres, from the crest of the embankment to the reservoir water level at the time of the earthquake.

Alluvial Thickness

The approximate maximum thickness of alluvium left in the foundation underneath the shoulders of the dam, in metres. This value usually represents numbers quoted in the

literature, but occasionally an estimation of alluvial thickness was made based on drawings available for the dam.

Foundation Geology

A brief description of the geology of the foundation.

3.2.1.2 Earthquake Description

Name

The name given to the earthquake event or existing name of the epicentre location of that event, whichever is most commonly taken by authors to represent the seismic event. Earthquake events typically occur with a number of shocks of varying magnitude, but the name (and the date) of the earthquake is defined here by the largest event. Thus the named event may include a number of lesser quakes that happened soon after or before.

Mag

The magnitude of the earthquake is described on the USGS database as "a logarithmic measure of the "size" of an earthquake ... related to the energy released as seismic waves at the focus of an earthquake". The magnitude is calculated using measurements recorded by instruments which could be any distance from the epicentre, and thus there is a degree of inaccuracy in the process. There are also a number of methods used to determine magnitude, resulting in a number of different forms of magnitude.

The first measure was that introduced in the 1930's by Charles Richter, this 'Richter magnitude' is referred to as M_L , with L standing for local. It was noted that this measure was strictly only valid for limited distance and frequency ranges, and as the number of seismological stations about the world grew, new measures were developed, including:

- M_b (body wave magnitude), and
- M_s (surface wave magnitude).

The USGS database states that each measurement 'is valid for a particular frequency range and type of seismic signal. In its range of validity each is equivalent to the Richter magnitude'.

In response to the limitations of the above measures the ‘moment magnitude’ or M_W was developed, which is more uniformly applicable. It is the most reliable measure of very large earthquake magnitudes.

Values of earthquake magnitude were commonly quoted in the literature, but often without indication of which measure of magnitude was applicable. Values were also available from the USGS earthquake databases, and although the type of magnitude was always quoted here, no one form was always used. For this reason, the type of magnitude was not shown in the database. Values for magnitude used in the database were taken from both the published literature and the databases. There were often inconsistencies between sources in the figure of magnitude quoted, and effort was made to choose the value for which there was most agreement, but values quoted on the USGS database were preferred. This has the result that the magnitudes in the database are not a consistent set, but within the accuracy of the analyses we have done this is not significant. It is noteworthy that the San Francisco earthquake of 1906 is commonly taken as magnitude 8.3, but is given a magnitude of 7.8 on the USGS database.

Date

The date of the main earthquake event as quoted in literature or in the fore mentioned databases. There is no clear definition of a ‘main event’ as compared to an before/after-shock of a main event, but any event magnitude $\sim > 7$ is given its own date.

Location

The location of the earthquake epicentre as measured in coordinates of latitude and longitude.

- Longitude is taken as positive east, negative west in degrees from Greenwich.
- Latitude is taken as positive north, negative south, in degrees from the equator.

Location values were rarely quoted in literature and were obtained mostly from the USGS earthquake database.

Distance to Epicentre

The distance, in kilometres, between the dam location and the epicentre location. The epicentre is taken as the point on the earth surface closest to the earthquake focus. Otherwise referred to as the ‘epicentral distance’.

In response to the limitations of the above measures the ‘moment magnitude’ or M_W was developed, which is more uniformly applicable. It is the most reliable measure of very large earthquake magnitudes.

Values of earthquake magnitude were commonly quoted in the literature, but often without indication of which measure of magnitude was applicable. Values were also available from the USGS earthquake databases, and although the type of magnitude was always quoted here, no one form was always used. For this reason, the type of magnitude was not shown in the database. Values for magnitude used in the database were taken from both the published literature and the databases. There were often inconsistencies between sources in the figure of magnitude quoted, and effort was made to choose the value for which there was most agreement, but values quoted on the USGS database were preferred. This has the result that the magnitudes in the database are not a consistent set, but within the accuracy of the analyses we have done this is not significant. It is noteworthy that the San Francisco earthquake of 1906 is commonly taken as magnitude 8.3, but is given a magnitude of 7.8 on the USGS database.

Date

The date of the main earthquake event as quoted in literature or in the fore mentioned databases. There is no clear definition of a ‘main event’ as compared to an before/after-shock of a main event, but any event magnitude ≥ 7 is given its own date.

Location

The location of the earthquake epicentre as measured in coordinates of latitude and longitude.

- Longitude is taken as positive east, negative west in degrees from Greenwich.
- Latitude is taken as positive north, negative south, in degrees from the equator.

Location values were rarely quoted in literature and were obtained mostly from the USGS earthquake database.

Distance to Epicentre

The distance, in kilometres, between the dam location and the epicentre location. The epicentre is taken as the point on the earth surface closest to the earthquake focus. Otherwise referred to as the ‘epicentral distance’.

“by author” where the epicentral distance was given in literature.

“by coordinate” where the epicentre location as calculated using co-ordinates for the location of dam and the location of the earthquake epicentre. The calculation assumed that the earth is a perfect sphere of radius 6371 km and involved converting the latitudinal and longitudinal co-ordinates to x,y,z co-ordinates and applying Pythagoras theorem in three dimensions. This calculation does not take into account the curvature of the earth surface (a valid assumption for distances we are dealing with).

Focal Depth

The depth, in kilometres, of the focus of the earthquake below the earth surface. This value was taken from the literature where given, but was mostly taken from the USGS earthquake database.

Focal Distance

The shortest distance, in kilometres, between the focus of the earthquake and location of the dam. This was calculated by using Pythagoras' theorem with the focal depth and epicentral distance forming two sides of a right triangle.

“by author” - where the focal distance was calculated using epicentral distance that was given in literature.

“by coordinate” - where the focal distance was calculated using an epicentral distance that was calculated by using the locational coordinates of the dam and of the epicentre.

Crest Peak Accn

The maximum earthquake induced ground accelerations as measured at the crest of the dam. These values were taken from the literature and are measured values, not estimations or calculations. Values are given as a fraction of the acceleration due to gravity.

Foundation peak Accn

The maximum earthquake induced ground accelerations at the dam foundation. These values were taken from the literature, and could represent a measured value or an estimation made or quoted by the author. Values are given as a fraction of the acceleration due to gravity.

Esteva and Rosenblueth PGA

An estimation of the peak ground acceleration at the foundation by using the formula of Esteva and Rosenblueth (1969). The calculation requires knowledge of the magnitude, focal depth and epicentral distance. Values are given as a fraction of the acceleration due to gravity.

The Esteva and Rosenblueth (1969) formula is given by:

$$PGA = \frac{20000}{9810} e^{0.8M} \left(\sqrt{ED^2 + FD^2 + 400} \right)^{-2}$$

Where PGA = peak ground acceleration as a fraction of gravity
 M = magnitude
 ED = epicentral distance in kilometres
 FD = focal depth in kilometres

As taken from ANCOLD (1998)

As the focal distance 'r' is given by $\sqrt{ED^2 + FD^2}$, then the formula reduces to

$$PGA = \frac{20000}{9810} e^{0.8M} \left(\sqrt{r^2 + 400} \right)^{-2}$$

“by author” - where the calculation was performed using an epicentral distance that was given in the literature.

“by coordinate” - where the calculation was performed using an epicentral distance that was calculated by using the locational coordinates of the dam and of the epicentre

3.2.1.3 Damage Description

Settlement

“metres” - maximum recorded total crest settlement in metres as quoted in the literature.

“% dam height” - maximum relative crest settlement. The ratio of total recorded maximum crest settlement to dam height, expressed as a percentage.

“% dam height and alluvium” The ratio of the recorded total maximum crest settlement to the sum of the dam height and the thickness of the alluvium in the foundation, expressed as a percentage.

“Swaigood as % dam height and alluvium (Given PGA)” - The ratio of total maximum crest settlement to the sum of the dam height and the thickness of the alluvium in the foundation, expressed as a percentage, as calculated by the Swaigood (1995) formula, using a PGA given in the literature. This PGA usually represents a value that is measured by instruments or at least is an estimation by a seismologist or knowledgeable person, and is thus considered to be of better reliability. The form of the Swaigood formula that was used was that published in 1995, using criteria of magnitude, PGA, dam type, dam height and alluvial thickness. This was chosen over the 1998 formula because the 1998 version required knowledge of distance between the fault (not epicentre) and the dam, which was not recorded or readily available.

“Swaigood as % dam height and alluvium (calculated PGA)” – As above, but using the PGA calculated using the Esteva and Rosenblueth (1969) Formula, in turn using a epicentral distance which is given by authors.

“Swaigood as % dam height and alluvium (calc PGA with calc epi dist)” - As above, but using the PGA calculated using the Esteva and Rosenblueth (1969) Formula, and using a epicentral distance which is calculated from knowledge of the dam and epicentre coordinates.

Deformation

Often a dam undergoes lateral deformations under earthquake loading. A note of this occurrence was made, but no numerical values recorded.

Longitudinal cracks

Longitudinal cracks are cracks that are oriented parallel to the crest of the dam.

“Max length” - The length in metres of the longest longitudinal crack forming on the dam in response to the earthquake event. This value was obtained from written reports.

“Max width” - The maximum width, in millimetres, of any longitudinal crack forming on the dam in response to the earthquake event. The crack is not necessarily the same one as reported for maximum length. This value was obtained from written reports.

“Max depth” - The maximum depth, in metres, of any longitudinal crack forming on the dam in response to the earthquake event. The crack is not necessarily the same one as

reported for maximum length or maximum width. This value was obtained from written reports.

Transverse cracks

Transverse cracks are cracks that are oriented perpendicular (or near perpendicular) to the crest of the dam.

“Max width” - The maximum width, in millimetres, of any transverse crack forming on the dam in response to the earthquake event. This value ever obtained from written reports.

“Max depth” - The maximum depth, in metres, of any transverse crack forming on the dam in response to the earthquake event. The crack is not necessarily the same one as reported for maximum width. This value was only ever obtained from written reports.

Sliding

A note was made if any sliding occurred on the embankment slopes. No values or descriptions of the sliding are given.

Liquefaction

A note was made if any liquefaction occurred in the dam or its foundation during the earthquake event. No values or descriptions of the liquefaction are given.

Seepage

A note was made if any increased seepage was observed through the dam or its foundation after the earthquake event. No values or descriptions of the seepage are given.

Author's Damage Assessment

Many authors used descriptive terms to report on damage. Terms used were noted for each author, and the most regularly used term for an event was recorded for each dam.

A numerical value was then given to the term most used by authors for an event. This number was determined as per Table 1 (see Section 4.2).

Our Damage Assessment

A one-word descriptor of the damage was given, based on the classification system outlined in Section 4.2. A numerical value was given to classify damage, based on the classification system outlined in Section 4.2.

3.3 Case Studies and Overviews

The choice of dams presented in the case studies has been dictated mostly by availability of data. However, an effort has been made to choose a representative array of dams including various dam types, foundation types and earthquake events.

1. La Villita Dam, Mexico – Central core earth and rockfill
2. El Infiernillo Dam, Mexico – Central core earth and rockfill
3. Guadalupe Dam, USA – Zoned earthfill
4. Leroy Anderson Dam, USA – Central core earth and rockfill
5. Matahina Dam, New Zealand - Central core earth and rockfill
6. Austrian Dam, USA – Homogeneous Earthfill
7. Cogoti Dam, Chile – Concrete Face Rockfill
8. Chabot (San Leandro), U.S.A. – Rolled Earthfill and Hydraulic Fill
9. Coyote Dam, U.S.A - Earthfill
10. Los Angeles Dam, U.S.A. - Asphalt faced Earthfill with Corewall
11. Lexington, U.S.A – Zoned Earthfill
12. Oroville, U.S.A – Zoned Earthfill
13. San Justo, U.S.A. – Zoned Earth and Rockfill

3.4 Data Limitations

To get data on the cracking of dams under seismic loading, requires a seismic event to occur within proximity of a dam, and for someone to compile and make available data on the event. The distribution of large earthquakes around the globe is shown in Figure 5.

For this report, it was also preferable that the data is published in English. These requirements significantly limited available data and meant that most of the data on the topic of this report comes from the USA - they have many earthquakes and they have the resources to fund research, producing reports which are written in English and made

globally available. In compiling the databases used in this report it was clear that, save a handful of cases, adequate information is only found for dams of the Western USA. In short, many of the articles are written about the same dams. It was difficult to find new information. Considering the world distribution of earthquakes, it is likely that there still exist many more dams that have been subject to seismic loading and reported on, that are not readily accessed in the English literature.

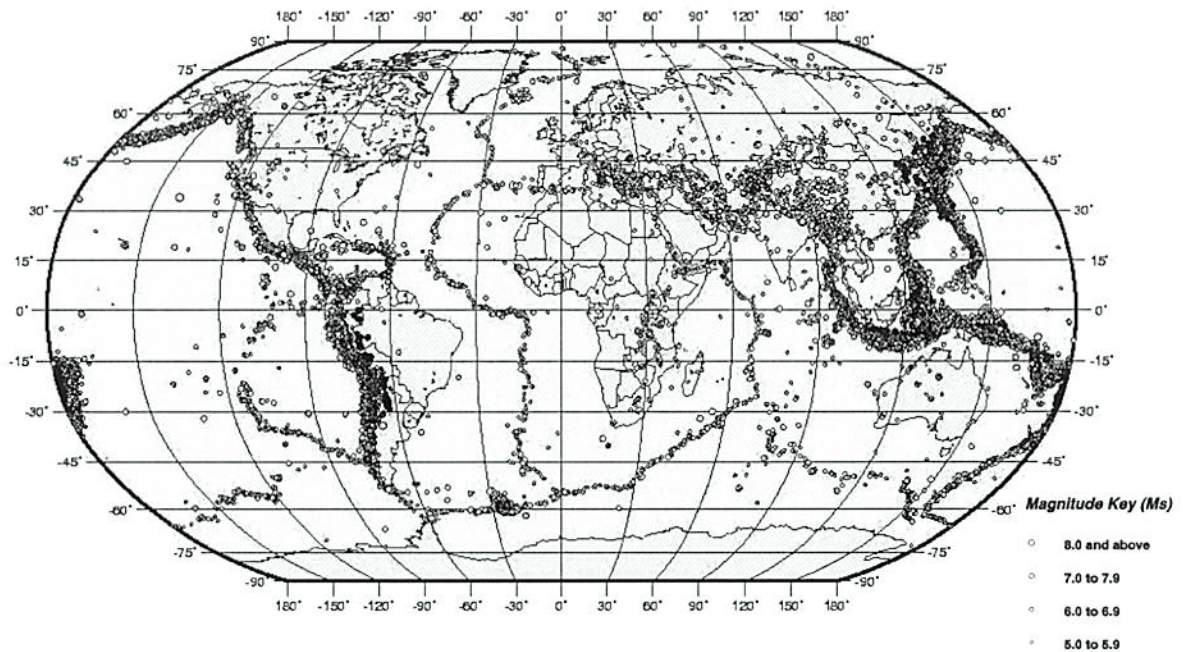


Figure 5 World distribution of large earthquakes (British Geological Survey website)

Many of the dams which are in the database are small or obscure and only covered by a single reference or data source, and often the published details of the dam or earthquake are in table format, offering only basic data. Thus, despite our efforts there are still many entry locations in the database that are blank.

Techniques in collection of field data are also expected to produce inconsistencies in the data. Fong and Bennett (1995) were involved first hand in inspecting dams after earthquakes and list a number of factors that could compromise data integrity, including:

- The time between earthquake event and time of inspection. This is especially true for measurement of cracks, which often are reported to change significantly over the period of time after an earthquake.
- The weather conditions – washing soil materials and water into cracks and obscuring them and / or changing their dimensions
- Shrink / Swell properties of the soils which may change dimensions of cracks

- Uniformity of investigators. Minor cracks may be overlooked by some, and noted by others. Techniques of measuring cracks may also change, whether it is by estimation or with a scale
- Different levels of investigation and / or reporting, and lack of standardisation in investigation techniques.

It's often stated that damage is closely related to embankment construction materials (ICOLD, 1999, Seed 1978), and to foundation conditions (Swaigood, 1995; Fong and Bennett, 1995). Although the analysis of the case studies to examined this, these details are poorly recorded in the earthquake literature. Basic details of embankment zoning are often given, but details of construction, compaction and actual material parameters are more scarce. With more time, a literature search for construction details of individual dams in the case studies would be desirable.

Another problem encountered with the data is in trying to establish when no damage occurred. People generally have limited interest in writing in specific detail about damage that didn't happen. For example, if no transverse cracking occurs after an event, authors reporting on the event will seldom state explicitly that 'no transverse cracking occurred'. Rather, they will report only on what they did observe. In these cases it may be safe to assume that no 'significant' transverse cracking occurred, but often it remains uncertain. There is also a chance that data sources that do not report comprehensively on the damage, will report no transverse cracking, even in cases where some did occur, because it was not systematically investigated, or the cracking was relatively narrow.

4 EARTHQUAKES AND DAM DAMAGE

4.1 Trends in Dam Behaviour

4.1.1 Settlement

Swaigood (1998) defines settlement (and mass deformation) as "those movements that occur within the body of the dam without distinct signs of shearing displacement". By this it is understood that settlement is to be distinguished from movements with clearly defined sliding surfaces developed. In reality there is seldom data on displacements within the dam, so it is difficult to be sure that no sliding surface develops.

Settlement of this form is the most commonly reported effect of earthquakes on embankments. A point which Swaisgood (1998) makes, and one that is highlighted in the case studies, is that earthquake induced settlements are often very similar in characteristic to settlements under static loading. This was shown clearly in the El Infiernillo and La Villita case studies, as both dams were well instrumented from construction and through numerous earthquake events allowing clear examination of settlements before and after earthquakes. This concept was also highlighted in the case study of the Cogoti concrete face rockfill dam in Chile. In many cases, some believe that settlement from seismic loading is essentially no more than an acceleration of static-loading settlement. (The authors acknowledge this may be true in some cases, but do not agree it is generally true).

The patterns of settlement induced by earthquake common to embankments are covered in numerous texts, and are described by Swaisgood (1998) as a movement 'downward and sideways, towards the deepest centre portion of the valley' coupled with a spreading movement 'upstream and downstream away from the dam axis'.

There are strong correlations between settlement and cracking and these are covered in detail in Section 5. Correlations have also been produced between settlement and seismic intensity (see Sections 4.3, 4.4 and 5.1.1).

4.1.2 Deformation

Deformations are often taken to mean the same thing as settlement, but in this paper are distinguished to represent any changes in shape of the embankment.

Horizontal movement of the crest and/or body of the dam is a regularly observed embankment earthquake response. Many of the case studies report the ends of the dam crest moving laterally – often even in opposite directions (see Figure 4 of the Austrian Dam case study, Appendix B). Some measure of deformation may occur for every case of larger seismic loading, albeit often in magnitudes that are insignificant or too small to measure.

In the database, a record was made where authors had considered deformations to be significant enough to report. Nearly 30 such cases are recorded.

4.1.3 Cracking

Dams often develop cracks of various kinds under seismic loading. Along with settlement, cracking is the most regular feature of seismic loading. There are two main forms - 'longitudinal' cracking and 'transverse' cracking. Swaisgood (1998) indicates that cracking occurs in nearly all cases in his database and is commonly longitudinal and more numerous on the upstream slope. Longitudinal cracking is of course fairly common feature in dams under normal operating loads (Sherard et al 1963).

Of the 305 cases examined in the database (Appendix A), 97 report some kind of cracking. However, this is not necessarily an indication of the proportion of dams that suffer cracking under seismic loading. Many of the data on dams in the database came from lists, tables or other databases which provided only basic details of the embankment, of the earthquake and of the damage. In these sources damage was usually given by a one word description and thus details of cracking were not given. By sorting the database to remove cases where cracking details were not addressed it became more like 97 cases of cracking out of 180 cases, but it is still a somewhat unreliable guide. Even some more detailed articles fail to give adequate detail to cracking, especially ones that were focussed on other aspects of the dam damage, such as settlement. It is unclear whether no report of cracking is an indication that no cracking occurred, or just a failure to notice or to report it.

4.1.3.1 Longitudinal Cracking

Longitudinal cracking of embankments is cracking that is oriented parallel to the crest of the dam. In compiling the database and case studies it became clear that it is the most common form of seismic-induced cracking.

Of the 97 cases of cracking in the database (in Appendix A), 69 report longitudinal cracking, and a further 16 report cracking without description of which type. As longitudinal cracking tends to appear first, it is likely then that 85 of the 97 cases feature longitudinal cracking. 34 of these cases include details of crack width, length or depth. The

reported cases in the database include cracks with widths ranging from 5 to 450 mm, depths of 0.5 to 20 metres and lengths of 3 to 800 metres.

In most cases, longitudinal cracks are positioned close to the crest on both slopes, and may run over the complete length of the crest. They also form, although less commonly, at other locations down the slopes. The location of cracking and the mechanisms of formation are discussed in detail in Section 5.

4.1.3.2 Transverse Cracking

Transverse cracking of embankments is cracking that is oriented perpendicular to the crest of the dam. Cracking in this direction poses a greater threat to the safety of the dam by opening a path for water to begin passing through, washing out fines, initiating erosion which may lead to development of a pipe and uncontrolled release of the reservoir.

In compiling the case studies and databases it appeared that this form of cracking occurs less frequently than longitudinal cracking. It also seems that it is relatively uncommon for transverse cracking to occur without any longitudinal cracking, suggesting either it requires higher seismic intensity to occur, greater magnitudes of (seismic-induced) settlement, or that the embankment conditions that provide the mechanisms for its formation are less common, or that transverse cracking is sometimes as a result of the deformations induced by the longitudinal cracks.

From the 305 recorded incidents in the database, 37 cases of transverse cracking are reported. Again, however, these figures cannot be seen as an accurate representation of the proportion of dams that suffer transverse cracking, because it could be due to a failing to notice or report it, but it does suggest that transverse cracking is relatively rare. Of these reported cases, only 17 give an indication of the transverse crack width and only 11 of transverse crack depth.

The widths of cracking reported range from 1 mm to 250 mm. The depths of transverse cracking range from 100 mm to 5 metres (although a case of 10 metres depth was reported for a 'separation' crack between forming between the spillway and embankment at Austrian

dam (USCOLD, 1992)). Fong and Bennett (1995) showed a relationship exists between transverse crack width and depth (see Figure 6).

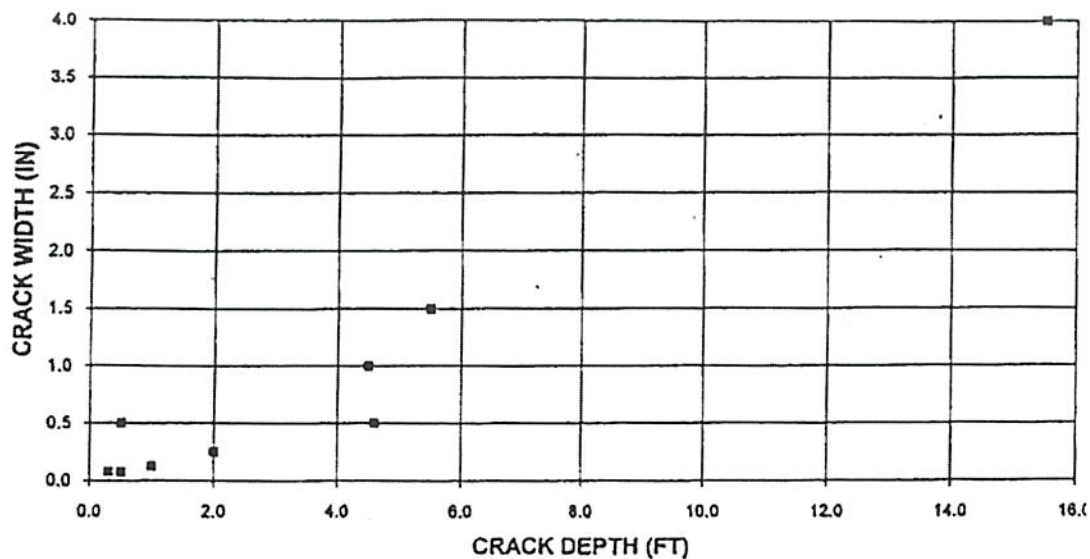


Figure 6 Transverse crack width vs depth (Fong and Bennett, 1995)

Of the 37 cases of transverse cracking reported, 26 also report longitudinal cracking. Assuming that authors who reported longitudinal cracking would also have reported transverse cracking, we can say that to have transverse cracking without longitudinal cracking is reasonably uncommon.

In the case studies, transverse cracking that is usually found on the crest adjacent to the abutments. After examination of many dams in the California region, Fong and Bennett (1995) report that “almost all transverse cracking has been observed at or near the abutment contacts”. Swaisgood (1998) adds that they tend to occur where there are steep slopes in the underlying foundation bedrock.

4.1.4 Sliding

Sliding of embankment dams and their foundations is a well recognised phenomenon, leading to the failure of a number of dams through history, even when not affected by seismic loading. Sliding events have also been observed as a result of earthquakes, although Sherard *et al*, (1963), reported there were surprisingly few. Of earthquake induced slides, Swaisgood (1998) writes ‘most appear to be shallow, sloughing type slides located in the

outer portions of the slopes' and that the majority of these have been found on upstream slopes.

The mechanisms of sliding, settlement and cracking are viewed in many cases to be intertwined. Some authors believe that often longitudinal cracking represent the top of a slide scarp. The various mechanisms of cracking are discussed further in Section 5.

Sliding with large displacements, where it occurs, is commonly the result of liquefaction. Of the 18 cases of earthquake induced embankment sliding events reported in the database, 6 mention the occurrence of liquefaction in the embankment of the foundation.

4.1.5 Liquefaction

Hinks and Gosschalk (1991) made the observation that the most complete failures have been due to liquefaction. Some well documented cases of liquefaction based failures of embankment dams are the San Fernando dams in California and the La Marquesa and La Palma dams of Chile. It also seems likely that the failure of the Sheffield dam in Santa Barbara was at least partly due to liquefaction, but this is still a topic of debate.

The almost complete loss of strength of a soil unit under liquefaction means that where liquefaction occurs, it usually controls the dam response, and usually results in severe damage. Because of this, much research has been directed towards the study of liquefaction. However, Swaisgood (1998) points out that it has become a relatively rare event. There are 16 cases in the database for which liquefaction is mentioned. These have been excluded when analysing the data. Liquefaction induced deformation and cracking are discussed separately in Section 5.4.

4.1.6 Post earthquake internal erosion

The possibility of activation or reactivation of internal erosion by seismic loading has been a long recognised phenomenon (Vick, 1993). It is understood that shaking could destabilise the internal structure of the embankment, or by cracking, create pathways for water to begin flowing.

Seed *et al* (1978) examined a number of Japanese case studies in which failures of embankments occurred some 24 hours after the earthquake event. There was not much further information available on these dams and Seed *et al* (1978) surmised that the delayed failure may have occurred either by slope instability resulting from pore pressure redistribution, or piping 'through cracks induced by earthquake motions'.

Vick (1993) produced a study of the re-activation of piping after earthquakes and found that PGA was the best predictor of such an occurrence - that a PGA of 0.05 to 0.1 was the limit that 'discriminated well between dams that experienced some piping and those that did not'. For failure through piping, this figure was taken as 0.3 to 0.4. Vick also found that, for dams which had some piping history prior to earthquakes, embankments of clayey soils were just as susceptible to reactivation of piping from seismic loading. No evidence that the threshold for piping reactivation was reduced with repeated earthquake loading was found by Vick. ICOLD (1999), citing the case of La Villita dam, note that increased settlements under recent earthquake events could highlight a progressive weakening of embankment materials – a phenomenon that could affect susceptibility to piping.

In many of the case studies (e.g. see Austrian Dam, and Matahina Dam case studies) increased seepage was noted after an earthquake event, and many of these events were followed by increased seepage discharges that were also cloudy. In most of those cases the seepage and pore pressure distributions within the embankments returned to pre-earthquake conditions by one week after the earthquake event, a phenomenon often attributed to the seal-healing nature of zoned earthfill embankments. (see Harder *et al* 1991).

4.2 A Damage Classification System

4.2.1 The classification system

Most authors use descriptive terms to report damage to a dam. Words such as 'minor', 'major', and 'significant' appear in the damage descriptions. However, in most cases these terms are only subjective representations. For this report, a damage classification system was used, based upon two criteria: maximum relative crest settlement and maximum longitudinal crack width. Table 1 shows the classification damage system.

Table 1 Damage classification system

Damage Class		Maximum Longitudinal Crack Width ⁽¹⁾ mm	Maximum Relative Crest Settlement ⁽²⁾ %
Number	Description		
0	No or Slight	< 10 mm	<0.03
1	Minor	10 - 30	0.03 - 0.2
2	Moderate	30 - 80	0.2 - 0.5
3	Major	80 - 150	0.5 - 1.5
4	Severe	150 - 500	1.5 - 5
5	Collapse	> 500	> 5

(1) Maximum crack width is taken as the maximum width, in millimetres, of any longitudinal cracking that occurs.

(2) Maximum relative crest settlement is expressed as a percentage of the structural dam height.

With this system, the damage to a dam is given numerically or descriptively, and is determined by choosing the worst case of the above criteria. For example, if a dam records a maximum crack width of 50 mm and a maximum crest settlement of 0.05% it is described as 'moderate' damage, class 2. In the case where major discrepancies are found between the crack width and settlement criterion, some averaging is permitted. To use the system it is preferable to have data on both cracking and settlement. However, where data is limited, classification was done with consideration of only one of these criteria.

4.2.2 Rationale behind the damage classification system

It is preferable to use quantitative data to classify damage, to avoid subjectivity. However, quantitative damage data on many dams is quite scarce, and so a complex system will be unusable for all but the most well documented events. A useable system needs to include effects that are common to all earthquake damage events, and that are commonly recorded. A meaningful system needs to include effects that can be considered representative of the other forms of damage that may occur.

Settlement is used because it is regularly measured and reported and because it is seen as fit to act as an indicator of total dam damage. Degree of settlement is observed to increase concomitantly with other forms of damage including longitudinal and transverse cracking.

It is considered that settlement alone is not an adequate damage descriptor, and an indication of cracking should also be included. Although settlement may infer cracking, it is

possible in the case of dams with cores which have dried out or were compacted dry, that cracking may be great despite small settlement. Conversely, dams which have cores that are well wet of optimum water content may not crack even under large settlement. Longitudinal cracking was chosen over transverse cracking as a damage index because relatively few dams have data on transverse cracks. Longitudinal cracking often occurs and the width of this cracking is often measured and reported. It is noted, however, that the measurement of crack width on site may be subject to a degree of inaccuracy, as outlined by Fong and Bennett (1995).

The choice of these two descriptors is not to infer that only settlement and longitudinal cracking are important, but rather that the amount of overall damage, of any form, may be represented by values in settlement and cracking - i.e. they each are an 'index' of damage - for example, where settlement is great, the likelihood of others forms of damage such as loosening and transverse cracking are expected to be greater.

Other authors have developed forms of damage classification. Swaisgood (1995, 1998) uses crest settlement alone as 'an index to deformation and damage'. His system is shown in Table 2. It does not allow unique classification of a dam - e.g. with a settlement of 0.1%, we can get damage criteria of none, minor and moderate. Furthermore, as explained above, it was felt that settlement alone was not always a reliable index of damage.

Table 2 Swaisgood (1995, 1998) damage descriptor parameters

Crest settlement, % of dam height + alluvial thickness	Degree of damage
Less than 0.02	None
0.02 to 0.1	None-Minor
0.1 to 0.5	Minor-Moderate
0.5 to 1.0	Moderate-serious
Greater than 1.0	Serious

A damage classification system was also presented by ICOLD - as shown in Table 3. This system was only applicable to dams between 4 and 30 metres height and included 4 criteria. With only 4 levels of damage it is considered constrictive. Retemal *et al* (1989) addressed this problem in proposing a modified version of the above system, offering 5 damage levels. However, many of these criteria, although meaningful, are not readily available from many

However, many of these criteria, although meaningful, are not readily available from many dam records and so, combined with the restrictive dam height, the system is in many cases unusable.

Table 3 ICOLD (1989) damage descriptor parameters

Damage Level	Crack thickness	Differential settlements	Landslides	Dislocations in large blocks
No Damage	0.01m, only lengthwise	No	No	No
Minor Damage	<0.05m	No	No	No
Major Damage	<0.05m	Yes	Yes	No
Collapse	<0.1m	Yes	Yes	Yes

Consideration was given to these other classification systems when choosing values for the system used in this report. Values of settlement and crack width that define a damage classification were selected by two methods: by a subjective consideration of what was considered reasonable, and by comparison with what was used by other authors.

The database shown in Appendix A includes a documentation of each authors descriptive terms of the damage e.g. ‘minor’, ‘moderate’ etc. By giving these descriptions a numerical value, as per Table 1, a data set of 83 points was provided where an authors terms could be compared against the ‘damage class’ used in this report. The results are shown in **Error! Reference source not found..**

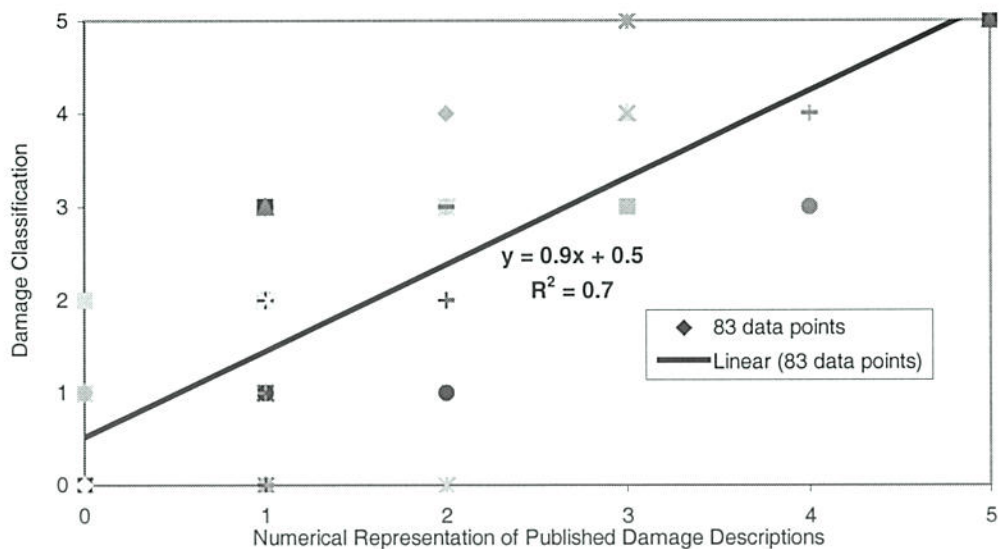


Figure 7 Relationship between ‘Damage Classification’ and published description

Considering the simplicity of the approach, a reasonable correlation between this system and the subjective descriptions of many authors is found (note, a perfect correlation would produce a trendline fit of formula $y = 1x$ and $R^2 = 1$).

4.3 A Damage–Seismic Intensity Relationship

4.3.1 Development of the relationship

The database was analysed to develop a relationship between damage class and seismic intensity. The ‘seismic intensity’ of an earthquake event has been characterised by both magnitude and peak ground acceleration (PGA). Earthquakes of larger magnitudes give longer durations and a greater number of cycles of shaking, and are therefore likely to induce more damage.

Attenuation of the ground motion induced by the earthquake occurs over distance, and thus the intensity of the shaking at a dam site is a function of distance from the earthquake epicentre. For this reason it is necessary to include a figure PGA measured, or estimated at the dam site.

The database of Appendix A was sorted for each of four dam types:

- Earthfill dams, including zoned earthfill, puddle core, homogeneous earthfill, earthfill with filter or rock toe, and what are described as "rolled" earthfill by some authors
- Hydraulic Fill Dams including fully hydraulic fill and partly hydraulic fill
- Earth and Rockfill Dams – central and sloping upstream core earth and rockfill
- Concrete Face Rockfill Dams

Each of these data sets was culled so that only data sets that included PGA, Magnitude and data sufficient to give a damage classification remained. However, for these data there were various methods (each with various reliability) of obtaining PGA and a damage classification.

For PGA, values are sometimes quoted in literature. These values sometimes represent readings taken from instrumentation at or near the dam, or are estimations by the author or

by seismologists for the dam location. In the absence of such data, the PGA was calculated using the formula by Esteva and Rosenblueth (1969). This formula requires knowledge of the focal depth and epicentral distance. Epicentral distance for each event is often quoted in literature, but where absent, was calculated by using global co-ordinates given for the dam and earthquake epicentre, as provided in the USGS earthquake database. The sources of each of these forms of data are outlined in Section 3.1. It is recognised that more recent attenuation equations may give better results in some situations, but the estimates using Esteva and Rosenblueth (1969) are considered adequate for this purpose.

For damage, classification required knowledge of relative crest settlement and maximum longitudinal crack width. Different sources often quote different settlement values, crest heights or crack widths for dams. Values with the most agreement were chosen. A damage classification was still performed where only crack width data was available or where only settlement data was available. Where settlement values were not provided by authors, an option existed to apply Swaisgood (1995, 1998) relationship to determine an estimation of settlement. This estimation was performed, but it is noted that classification of damage based on this estimation alone results in the damage-seismic intensity relationship becoming only an exercise checking in Swaisgood method, and is not an examination of case data.

With these different methods for obtaining PGA and damage, a total of fifteen data sets were possible for any dam type. These are shown in matrix form in Table 4.

Table 4 Matrix of sources of available damage data

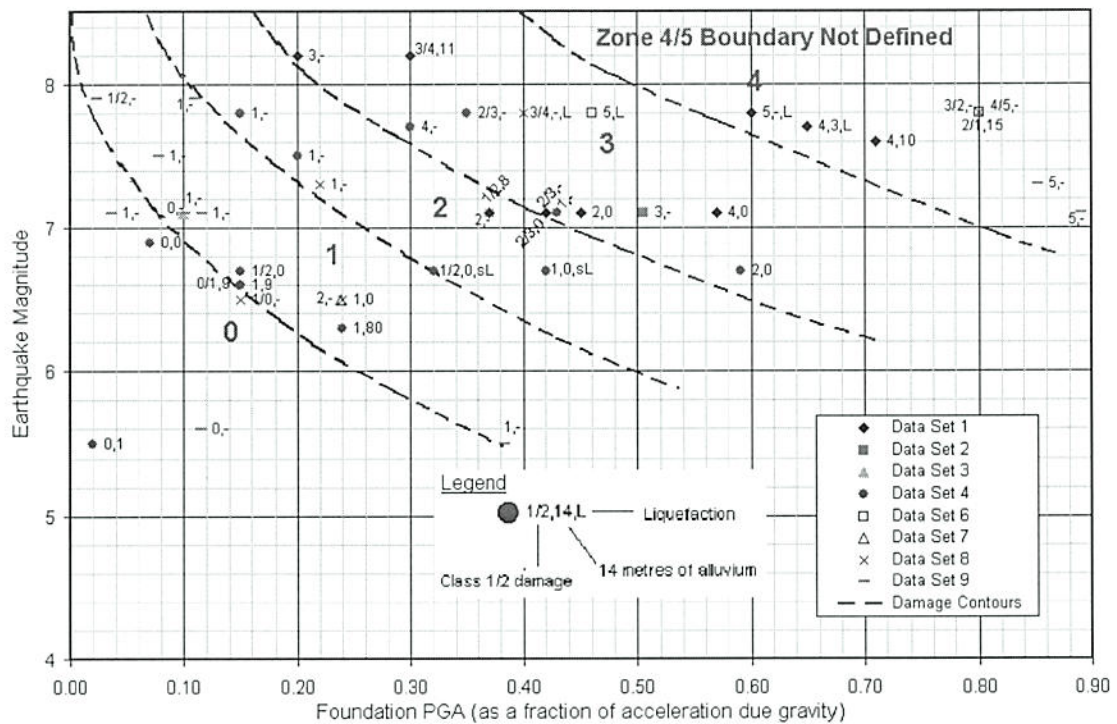
			PGA		
			PGA given	PGA calculated using Rosenblueth	PGA calculated using Rosenblueth but using co-ordinates for epicentral distance
Damage Definable	Cracking Data Given	Settlement given	x	x	x
		Settlement calculated using Swaisgood with given PGA	x	-	-
		Settlement calculated using Swaisgood with Rosenblueth PGA	-	x	-
		Settlement calculated using Swaisgood with Rosenblueth PGA based on co-ordinates	-	-	x
	Cracking Data Not Given	Settlement given	x	x	x
		Settlement calculated using Swaisgood with given PGA	x	-	-
		Settlement calculated using Swaisgood with Rosenblueth PGA	-	x	-
		Settlement calculated using Swaisgood with Rosenblueth PGA based on co-ordinates	-	-	x
	Cracking Data Given	Settlement not obtainable	x	x	x

After sorting some of the data sets became redundant. Also, it was assumed that PGA calculated with a given epicentral distance was of negligible difference and equal reliability to that using a calculated epicentral distance (by co-ordinates). Thus only nine of these data sets became relevant. These shown in Table 5 in approximate order of decreasing reliability:

Table 5 Description of damage classification data sets

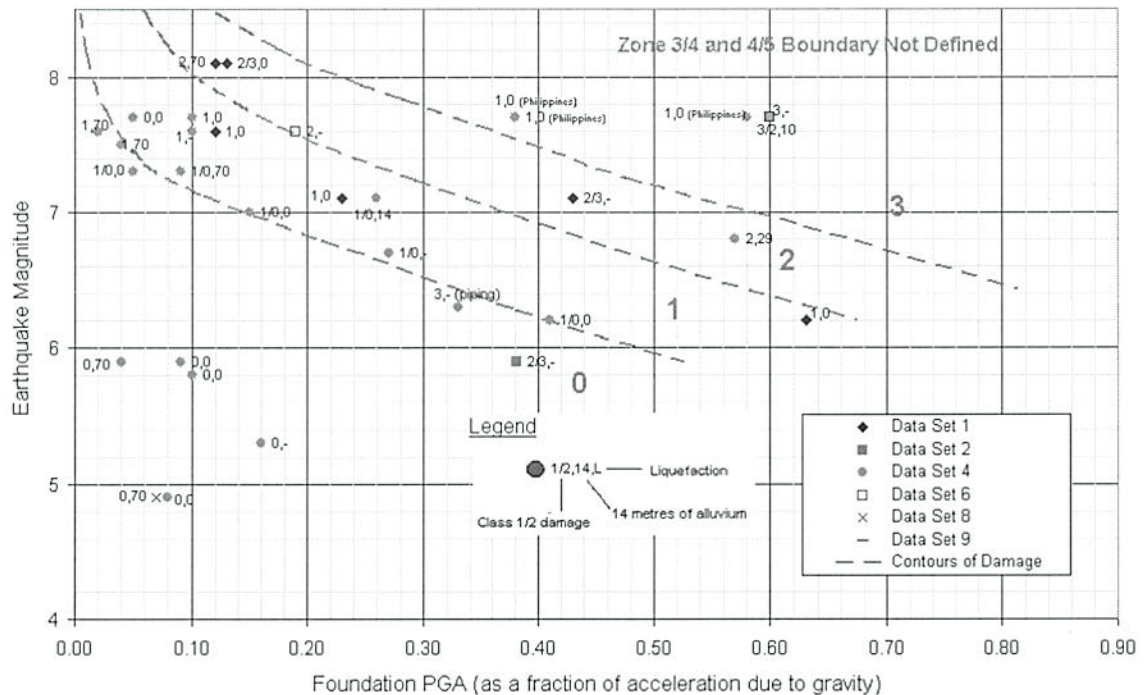
Data Set	Description
1	Damage classification based on cracking data and given settlement. PGA given or recorded
2	As for Data Set 1, but PGA calculated
3	Damage classification based on cracking data and calculated settlement. PGA given or recorded
4	Damage classification based on given settlement data only. PGA given or recorded
5	Damage classification based on given settlement data only. PGA calculated.
6	Damage classification based on cracking data only. PGA given.
7	As for Data Set 6, but PGA calculated
8	Damage based on calculated (Swaisgood) settlement only. PGA given. Alluvial thickness assumed = 0 where not given
9	Damage based on calculated (Swaisgood) settlement only. This calculation relies on the PGA which was also calculated. Alluvial thickness assumed = 0 where not given.

Data recovered for each of the four dam types are shown in Appendix C. These data were used to plot figures of damage contours as shown in Figure 8, Figure 9 Figure 10 and Figure 11. Data points were used from data sets 1 to 7 where available, but were only used from data sets 8 and 9 where agreement between predicted damage description and the author's damage description was found.



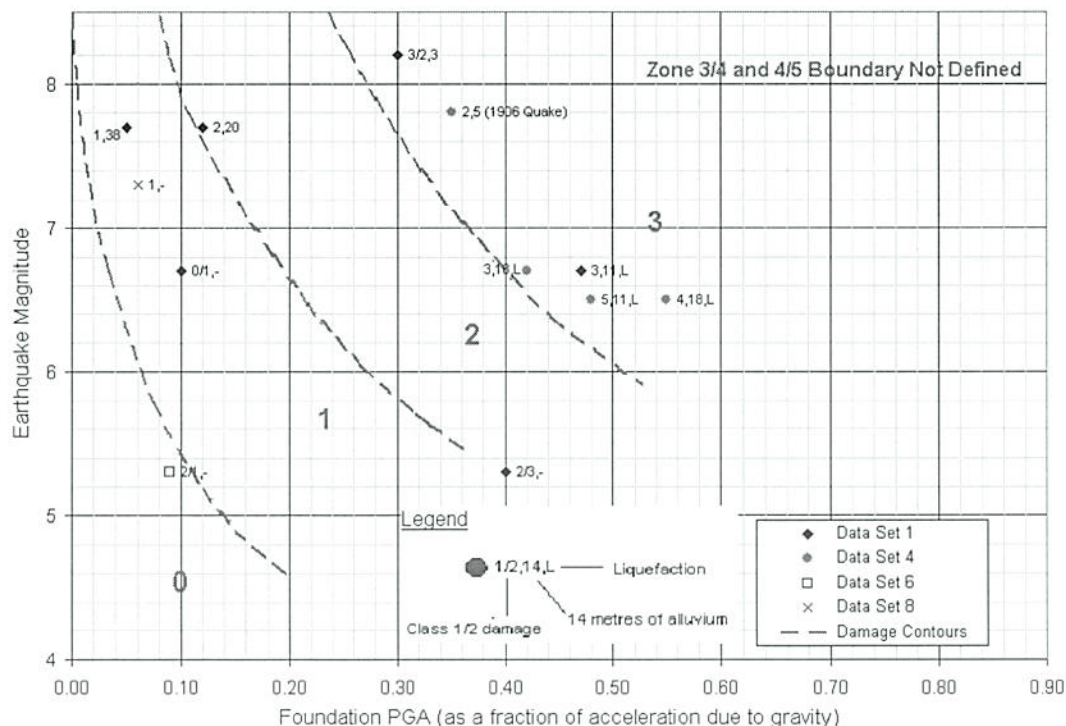
Note: Contours drawn without consideration for cases that had liquefaction

Figure 8 Earthfill Dams – Contours of damage class versus earthquake magnitude and peak ground acceleration



Note: 1. Contours drawn without consideration for cases that had liquefaction
2. Boundaries shown assume rockfill is well compacted. For dumped or poorly compacted rockfill use Figure 8 to estimate the damage

Figure 9 Earth and Rockfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration



Note: Contours drawn without consideration of cases that had liquefaction events

Figure 10 Hydraulic Earthfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration

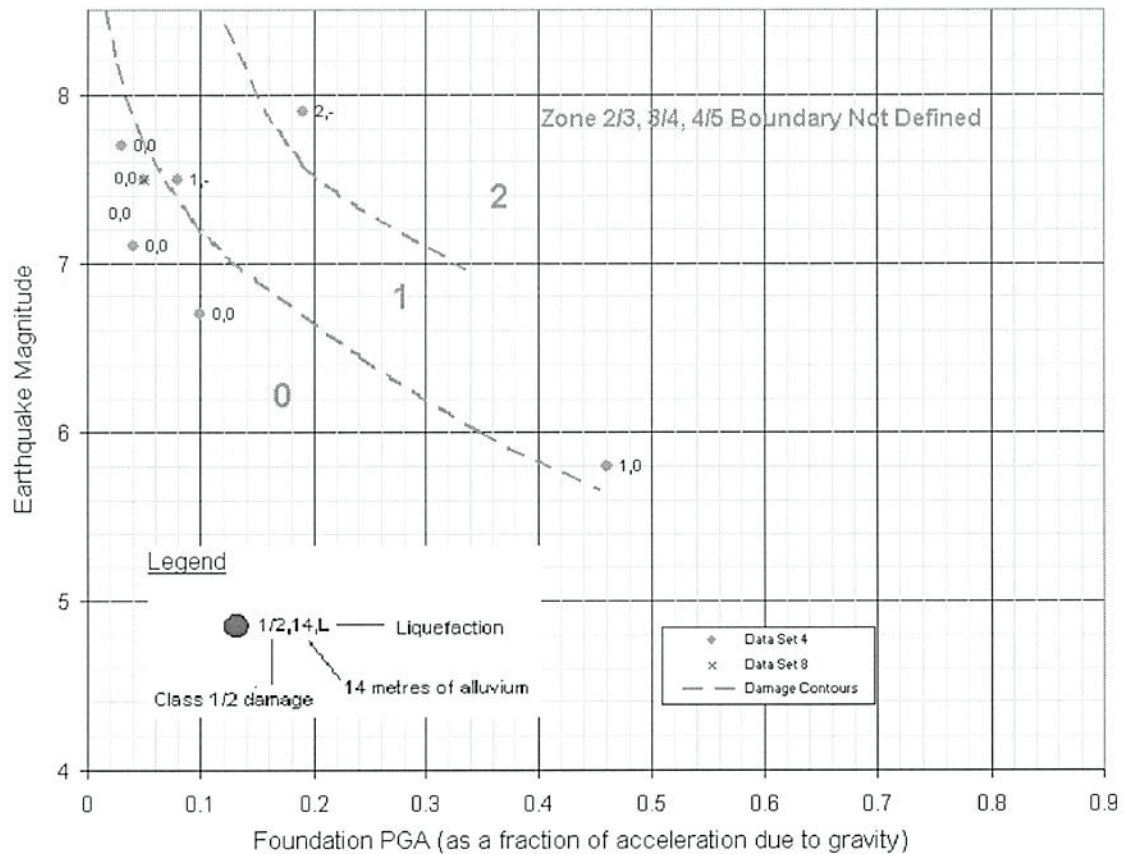


Figure 11 Concrete Face Rockfill Dams – contours of damage class versus earthquake magnitude and peak ground acceleration

4.3.2 Comments on the damage-seismic intensity relationship

Considering the errors inherent in measuring peak ground acceleration and earthquake magnitude, and errors in characterising the damage, the charts show a reasonable trend with damage increasing with increasing PGA, and with increasing magnitude. Damage to the different types of dams also follows an expected pattern with greatest damage, for a given event, for hydraulic fill dams, and least for concrete face rockfill dams.

Data for the earthfill and earth and rockfill cases was adequate, but was limited for the hydraulic fill dams, and scarce for the concrete face rockfill dams.

An effort was made to understand and adjust (if justifiable) any outliers. It is interesting to note that the dams of the San Francisco (1906) earthquake are in most cases strong outliers. This is not a new observation and has been noted by authors over the last century (Ambraseys 1960; Sherard *et al* 1963, Seed 1978, USCOLD 1992). Seed 1978 publication

was in fact prompted by this observation, and after his investigations he suggested the main reason of the dams' outstanding performance was the fact that they were largely constructed from clays of low to medium plasticity. Notwithstanding this, in comparison to the trend, many cases simply performed too well to be believable. For example, the San Andreas dam was said to lie virtually right on the fault of this 8.3 magnitude earthquake, receiving an estimated PGA of 0.8, but it sustained only "minor longitudinal and transverse cracks" (Seed *et al* 1978). Lake Ranch was the same, but for it "no damage" was reported (*ibid*).

There is some differences of view of the magnitude of this earthquake (estimates vary from 7.8 to 8.3), and the focal depth is not available, but these would not explain the difference in behaviour. With this uncertainty, and the fact that most the cases from the San Francisco quake provided insufficient data to characterise damage, most of these points were left off the plots.

A degree of interpretation was required to establish the contours. On a basic level, the relationship was clear, with good separation between extreme values. However, it was often more difficult to differentiate between neighbouring values – for example, to differentiate between class 2 and class 3 damage. The plots were initially drawn by hand and the full details of each data point was examined, with consideration of the amount of information available, the damage descriptions given by authors, and the accuracy and / or range of the PGA or magnitude values being used.

The boundaries for earth and rockfill dams assume that the rockfill is well compacted. Dumped or poorly compacted rockfill provides less support for the dam core, and it is recommended that the damage for these cases be estimated using the earthfill dam figures.

As a check, the data was also fed to the contour plotting package "Surfer". This was done to see if a computer algorithm, that has no mind of the meaning of the data, would still produce a relationship. The results showed good trends, albeit with many inflexions, that are similar to the contours drawn by hand.

An examination of the goodness of fit of data suggests that the damage description system is accurate to approximately plus/minus a class. For example, an earth dam subject to a

PGA of 0.3g and magnitude of 6.5, could reasonably be expected to suffer somewhere between class 0 and class 2 damage.

4.3.3 The Effects of alluvium in the foundation

It is well recognised that amplification of shaking can occur for dams resting on soil or alluvial deposits. Newmark (1965) wrote that structures founded on “relatively soft sedimental deposit of fairly great depth and wide extent will generally have a larger response than it would have if it were subjected to the motions of the bedrock”. In practise, earthquakes with smaller magnitude and PGA are amplified, whereas large earthquake giving large PGA may be de-amplified. Furthermore, the alluvium sometimes represents a region that could settle significantly leading to differential settlement and cracking.

For this reason, Swaisgood (1995) when developing a relationship between bedrock PGA, magnitude, dam type and settlement, included a factor based on alluvium thickness.

For this study, the depth of alluvium under a dam structure was noted (if the information was available) for all the data used in the damage contour plots (Appendix D). Alluvium thickness was known for 63 of the 230 cases for all dam types, including 15 of 40 data points for earthfill dams, 24 out of 31 for rockfill, 8 out of 17 for hydraulic fill, and 7 out of 9 for concrete face rockfill. However, some of these cases represent different earthquakes at the same dam.

It was expected that for a given magnitude and bedrock PGA, cases with larger alluvial deposits would have more damage. However, it was difficult to identify such a trend. In some cases the trend was followed, but in almost as many it was violated. This inability to verify a trend was felt to be more a reflection on the inadequacy of the database than on the phenomenon of resonance.

There are many variables that can affect dam performance including (Vick 1993):

- Height and geometry of the embankment
- Construction techniques
- Seismic intensity
- Degree of saturation of the embankment and foundation soils

- Reservoir level at the time of the earthquake
- Shape of the Canyon (valley) and abutment and foundation profiles
- Embankment zoning and properties
- Embankment soil type and plasticity
- Other localised items such as buried utilities (eg. outlet conduits).

Swaigood (1995, 1998) indicated that dam height and distance from causative fault were important; and the ratio of crest length to dam height, embankment slope angles, and reservoir water levels to have minimal effect.

With so many factors, the performance of each dam becomes somewhat unique. Although a trend was found between seismic intensity, damage and depth of alluvium, trying to establish the effects of one factor requires a large database of case studies to choose from, to address the many 'degrees of freedom'. For example, a particular dam may have been underlain by a 50 metre deposit of alluvium, and for a given earthquake event it may show more damage than dams with no alluvium. However, this dam may also be a particularly old, poorly zoned dam, or may be built from poorly compacted sandy soils, etc, so to say that the reason it suffered more damage is due to the alluvial deposits is to potentially overlook the effects of the other factors. In addition to this, the phenomenon of amplification itself is not related only to alluvium depth, also being subject to other site specific and earthquake specific factors.

On the other hand, if a factor has a large enough affect, its affects will be manifest, regardless of the many other more subtle affects that attempt to obscure it. An example of this is dam type – there is an obvious trend between datasets of dam types, with hydraulic fill dams being most easily damaged, and concrete face rockfill dams least easily.

In summary, the affect that alluvium has is neither discounted nor established. It is felt that its affect is obscured by the many other factors that make a dam unique. However, that the affect is obscured suggests that it is of limited impact, or at least no greater affect than some of the other factors.

It is interesting to note that Swaigood (1998), in a revision of the 1995 seismic intensity – settlement relationship, has dropped the alluvial thickness factor.

4.3.4 The effects of liquefaction on damage

This report focuses on seismic damage of earth dams that have not undergone liquefaction,, but there are some cases in the database where liquefaction occurred these are marked with a small “L”. There are 14 cases each with differing degrees of the extent of liquefied area. Six of these data are in the earth dam dataset, and 4 in the hydraulic fill dataset (for the others insufficient data exists to feature on the plots).

The identification of liquefaction provides an explanation for some outliers in individual cases, especially for those where liquefaction was the controlling feature of earthquake damage (e.g. La Palma dam). However, without distinction of ‘extent’ of liquefaction there is no trend observed.

Of the 5 points that define the damage 5 ‘collapse’ contour on Figure 8, 2 are cases of liquefaction. The other 3 “collapse” points (they are Lower Howell dam, 1906 quake; Volcano Lake dam, 1940 quake; Hosorogi dam, 1948 quake) are from the least reliable data sets and might also have experienced liquefaction, but this is speculative - information available for them does not indicate liquefaction occurred. If these points did experience liquefaction, then it is possible that the location of the collapse contour on Figure 8 is conservative for non liquefaction situations.

This is also true of the hydraulic fill dam contours in Figure 10, for which the four largest damage events all involve liquefaction (these points represent the Upper and Lower San Fernando dams in the San Fernando (1971) and Northridge (1994) earthquakes). It is interesting to note that the Lower San Fernando dam in the 1971 earthquake (PGA 0.48, Mag 6.5) suffered damage of class 5 but, after being rebuilt with consideration to liquefaction, in the Northridge earthquake (PGA 0.47, Mag 6.7) suffered damage of class 2/3.

4.3.5 Maximum seismic intensity without causing greater than slight damage

The question is posed as to what is the maximum seismic intensity a particular dam may experience without suffering more than slight damage – i.e. to remain in the classification of “class 0, none or slight”.

The damage classification of zero includes no damage or 'slight' damage, so we can use Figures 8 to 11 to establish these limiting combination of magnitude and PGA. There were plenty of data points available for the damage contour plots to do this – in many cases it was the best defined contour.

Users should however realise that if there is an incipient condition of near hydraulic fracture, or tensile stress nearing the strength of the dam core under static loading, even small seismic intensities less than those defined by the 0-1 boundary could lead to cracking in the dam. This may not be apparent as significant settlement, or even surface cracking.

4.4 Investigation of The Swaisgood Settlement Relationship

Swaisgood (1991, 1995, 1998) used regression techniques to derive a relationship between relative crest settlement, magnitude, peak ground acceleration, dam type and some other factors. In his 1995 publication these 'other factors' included dam height and alluvial thickness, but in a revised paper (1998) only the distance between seismic energy source and the dam was included.

With the database produced for this report it was realised that extra data points were available to allow an examination of the Swaisgood relationship.

The database was sorted for cases for which sufficient data existed to solve the Swaisgood formula and compare to measured values of settlement. Sixty-eight such cases were found.

However, 54 of these cases were already used by Swaisgood, and so there is just an additional 14 points. These are shown in Table 6.

Table 6 Datapoints to add to Swaisgood's analysis

Data Point	Dam Name	Country	Height	Dam Type	Alluvium Thickness metres	Earthquake			Focal Distance km	PGA g	Measured Settlement			Swaisgood (1998) Settlement		
						Name	Mag	Date			Metres	% dam height	% dam and alluvium	SEF	RF	% dam height and alluvium
a	Almaden	USA	33	earthfill	-	Loma Prieta	7.1	17-Oct-89	24.1	0.43	0.025	0.076	-	0.276	0.656	0.181
b	Ambuklao	Philippines	129	earthcore rockfill	10	Philippine / Luzon	7.7	16-Jul-90	32.0	0.6	1.100	0.853	0.791	1.236	0.994	1.229
c	Cogoti	Chile	85	concrete face rockfill	0	Papudo-Zapallar	7.5	08-Jul-71	172.7	0.05	0.000	0.000	0.000	0.034	2.779	0.094
d	Cogoti	Chile	85	concrete face rockfill	0	La Ligua	7.1	28-Mar-65	112.9	0.04	0.000	0.000	0.000	0.024	2.144	0.051
e	Cogoti	Chile	85	concrete face rockfill	0	Llollea-Algarrobo	7.7	03-Mar-85	174.2	0.03	0.000	0.000	0.000	0.034	2.794	0.096
f	Murayama Lower (Shimo)	Japan	16	earthfill	-	Kwanto	8.2	01-Sep-23	65.9	0.12	0.240	1.500	-	0.087	0.462	0.040
g	Newell Creek	USA	55	earthcore rockfill	-	Loma Prieta	7.1	17-Oct-89	26.1	0.43	0.030	0.055	-	0.276	0.878	0.242
h	Piedmont	USA	16	homogeneous embankment	-	San Francisco	7.8	18-Apr-06	36.9	0.35	0.150	0.938	-	0.276	0.566	0.156
i	Porter Estates	USA	12	rolled fill	0	Northridge	6.7	17-Jan-94	19.0	0.59	0.020	0.167	0.167	0.565	0.716	0.405
j	San Leandro	USA	38	semi-hydraulic fill	5	San Francisco	7.8	18-Apr-06	36.9	0.35	0.120	0.316	0.279	0.276	2.263	0.625
k	Shui Shui	Taiwan	30.3	earthfill concrete core	-	Chi Chi	7.6	21-Sep-99	35.5	0.45	0.130	0.429	-	0.448	0.574	0.257
l	Surgu	Turkey	55	rockfill	-	Malatya	5.9	05-May-86	14.1	0.38	0.150	0.273	-	0.086	0.604	0.052
m	Vermilion	USA	50	zoned earthfill	80	Mammoth Lakes	6.3	27-May-80	25.6	0.24	0.053	0.106	0.041	0.047	0.643	0.030
n	Yuvacik	Turkey	108	earthcore embankment	-	Kocaeli	7.8	17-Aug-99	18.4	0.15	0.100	0.093	-	0.079	0.722	0.057

The distance from the dam to the causative fault was unknown, and so the epicentral distance (which was known) was assumed to do the analysis. This would generally tend to over-estimate the distance, particularly for earthquakes with deep focal points. The values in Table 2 were added to the plots produced by Swaisgood as shown in Figure 12 and Figure 13.

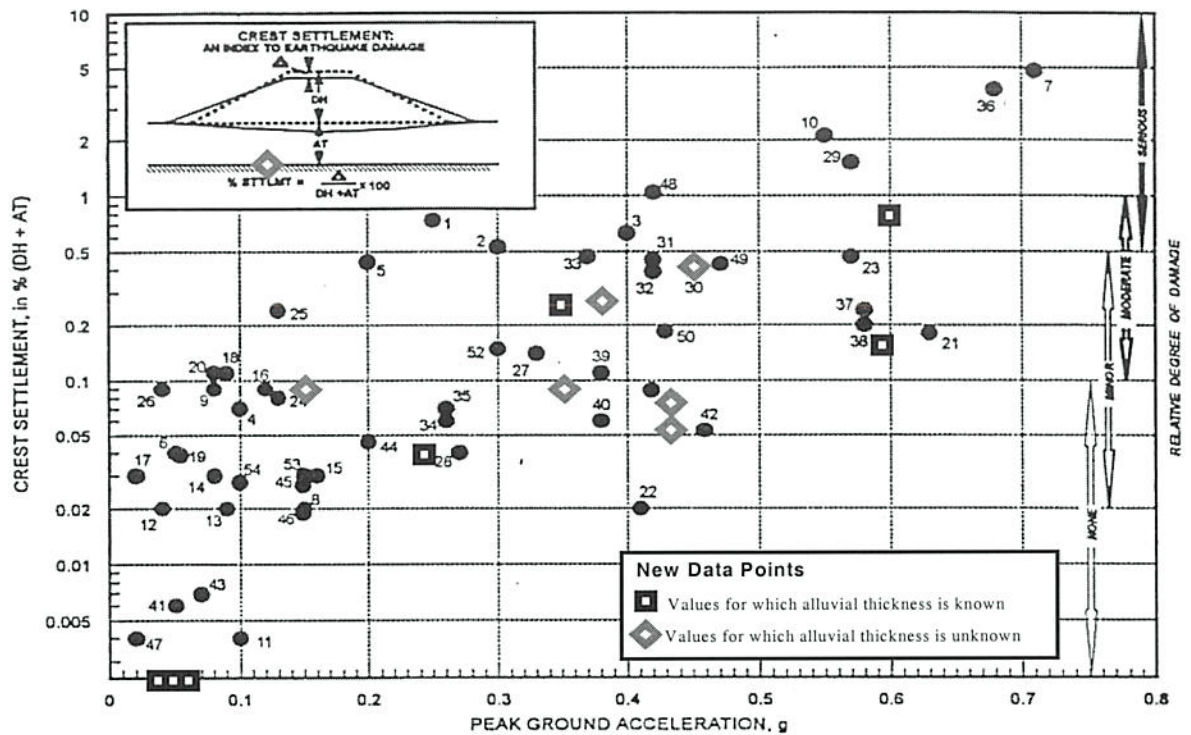


Figure 12 Swaisgood (1998) Plot of crest settlement percentage vs PGA, showing new datapoints

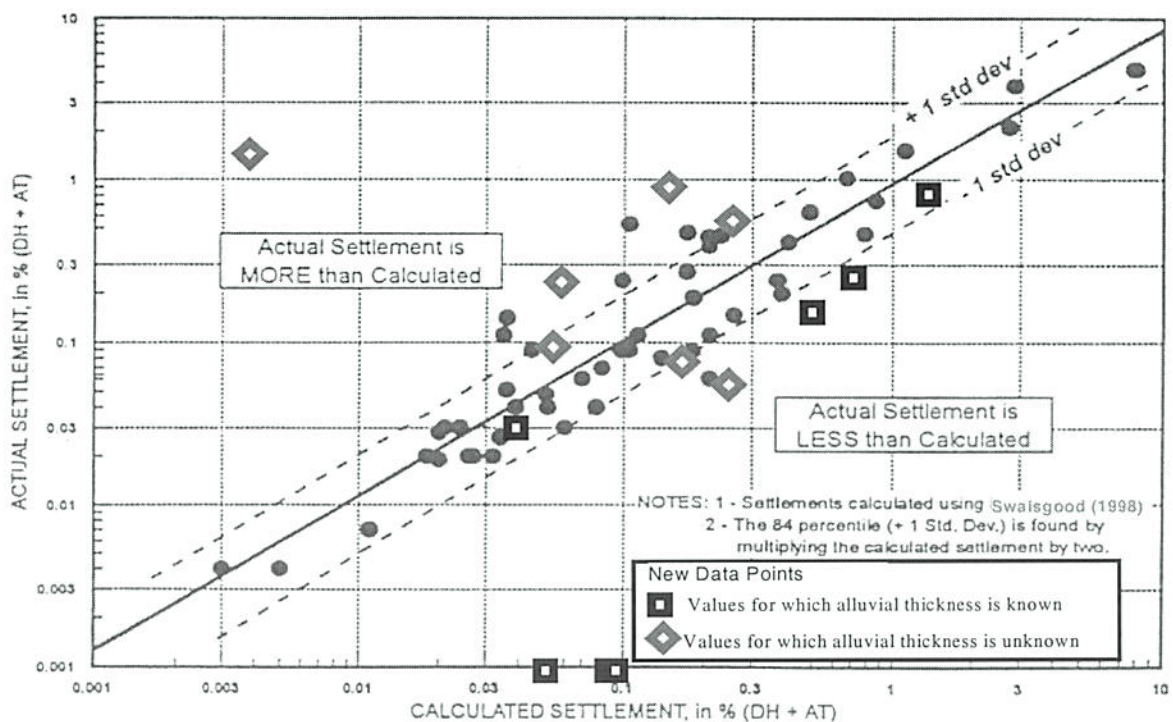


Figure 13 Swaisgood (1998) Plot of predicted vs observed settlement percentages, showing new datapoints

Aside from a few outliers, the new data is in general centred but shows more scatter than existing data.

A number of outliers are immediately obvious. The biggest outlier results from data from the Murayama Shimo dam. This dam was under construction at the time of the earthquake and was only 16 metres high. It was later to be finished in 1927 at 30 metres high (Tani, 2000; USCOLD, 1992 and ICOLD, 1974). The earthquake induced settlement used here was a value of 240 mm, as reported by Ambraseys (1960). Sherard *et al* (1963) reported a similar but slightly smaller settlement of 200mm. Ambraseys was the only source to report a foundation P.G.A for this M8.2 event, which he said was about 0.12g. However, an Esteva and Rosenblueth estimation gives a value of 0.3 P.G.A. Using the larger P.G.A gives a predicted settlement of 0.42 %, but even with only 200mm settlement (1.25%), it remains a strong outlier. It seems likely the dam or its foundation experience large strength loss due to liquefaction or strain weakening of high clay content soils, but nothing is known of the foundations of this dam, and how much alluvium may be present under the foundation.

There are three outliers located along the bottom abscissa. These points are all from various earthquakes at the Cogoti dam (Chile), where zero settlement was reported. This value of zero was reported by the same author for each earthquake – Arrau *et al* (1985). It is unknown whether this means no settlement occurred, whether no settlement was observed, or whether the author considered that only negligible settlement occurred. This is a concrete face rockfill dam, which is not strictly addressed with this Swaisgood technique – the formula for rockfill dams was used, and this may have contributed to the an overestimate of the settlement.

5 EARTHQUAKES AND DAM CRACKING

Having established a relationship for the extent of ‘damage’ that may be expected at a dam for a given seismic event, we now turn to examine longitudinal and transverse cracking. This examination draws on more detailed case studies of dams that have been cracked as presented in Appendix B, as well as the broad-base data from Appendix A. To assist in understand the cracking that may be expected in a dam, a brief examination of the mechanisms of cracking is presented.

5.1 Mechanisms of Embankment Cracking

There are a number of mechanisms that can cause to embankment cracking. Some of these mechanisms are common to those that cause cracking under static loading but some are unique to seismic loading. The information presented below is both a summary of observations made in examining the case studies, and a compilation of material presented in some key publications on the subject of embankment cracking – for both seismic and static loading.

5.1.1 Settlement and cracking

Settlement of embankments is almost always reported after earthquake events, and is described in Section 4.1.1. In most of the case studies, settlements go hand in hand with horizontal spreading and deformation, which involves shear displacements. Under these movements, strains are set-up in the embankment that manifest as cracks. Figure 14 (Resendiz *et al* 1982) depicts these movements and the location of cracks that can occur. Resendiz (*et al*) write “the distortion pattern shown [in the figure] produces transversal extensions in the proximity of the crest, thus longitudinal cracks are likely to develop”.

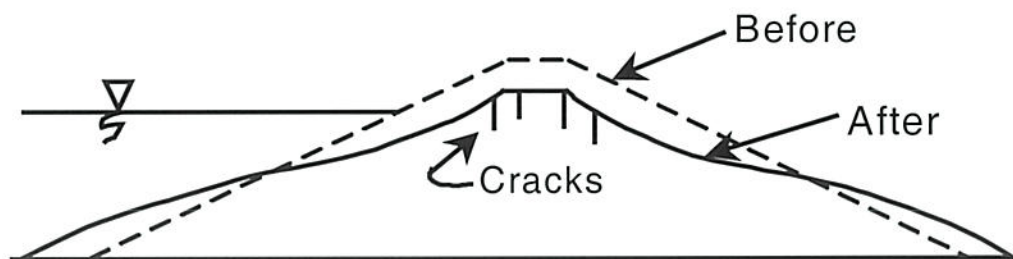


Figure 14 Settlement and longitudinal cracking (Resendiz *et al* 1982)

They suggest that “significant decrease in volume [in modern embankments] during shaking is unlikely and attributes the loss of freeboard in Figure 14 to lateral displacement. with the difference between lateral spreading of this type and sliding under seismic loading being only one of degree. Figure 25 shows typical longitudinal cracking related to settlement.

Fong and Bennett (1995) examined the relationship between settlement and depth of transverse cracking in a number of dams in the western U.S.A. The results of their

investigations are shown in Figures 1 and 2. They show a positive correlation between crack depth and settlement with a ratio of settlement to crack depth of 1 to 5 or 6.

5.1.2 Differential settlement and cracking

Differential settlement will set up such strains that, if large enough, will produce cracking. This is a common cause for cracking in embankments from both static and seismic loading. Differential settlements may occur for a number of reasons, but is usually related to changes in geometry or strength and compressibility of material in the embankment or the foundations. This includes:

- sudden changes in depths of embankment material the foundation,
- the existence of highly stressed zones within the embankment
- differences in strength and modulus of material, in the embankment and the foundation
- difference in degree of compaction of materials used in the embankment.

Sherard et al (1963) produced a number of figures explaining the occurrence of cracking from differential settlements. They are reproduced in Figure 15, Figure 17, Figure 19 and Figure 20.

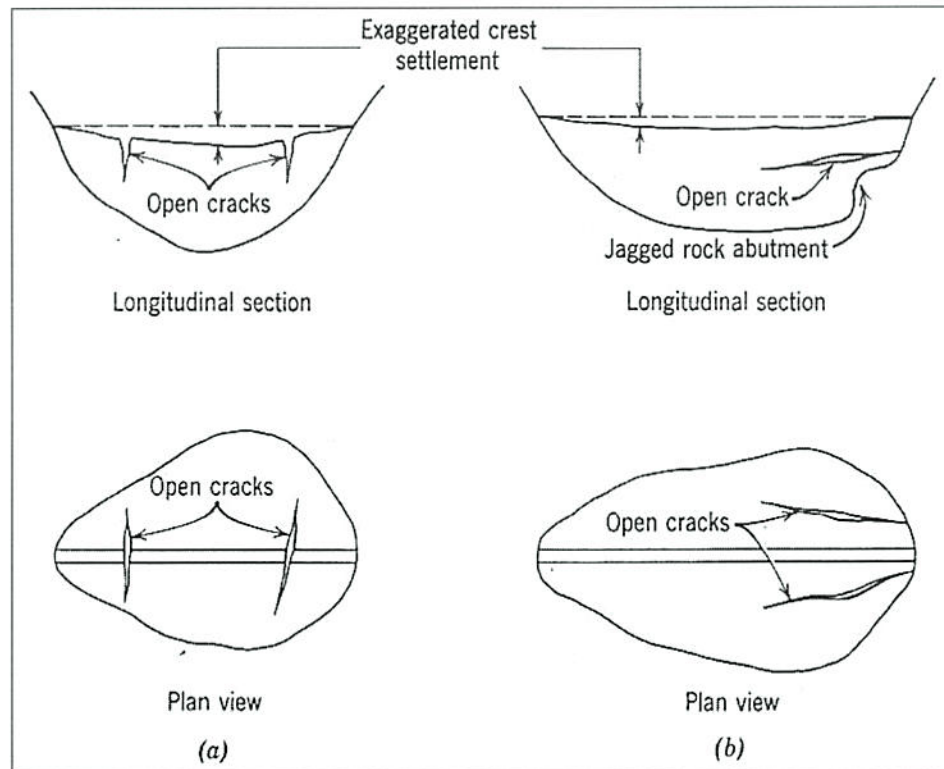


Figure 15 Typical transverse differential settlement cracks (Sherard *et al* 1963)

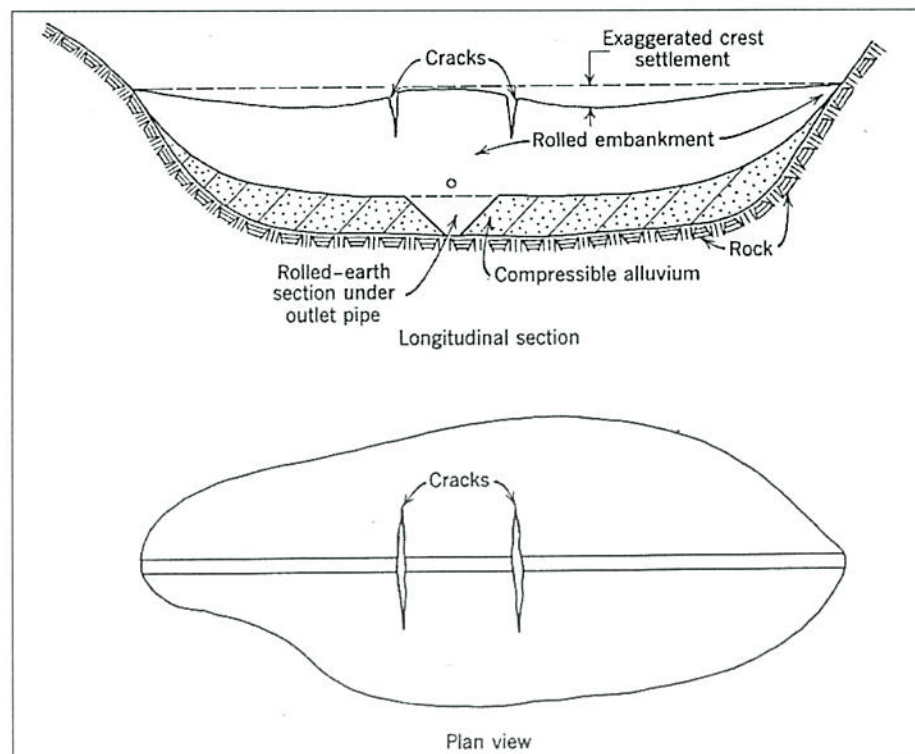


Figure 16 Transverse cracking due to differential settlements over embankment discontinuities (Sherard *et al* 1963)

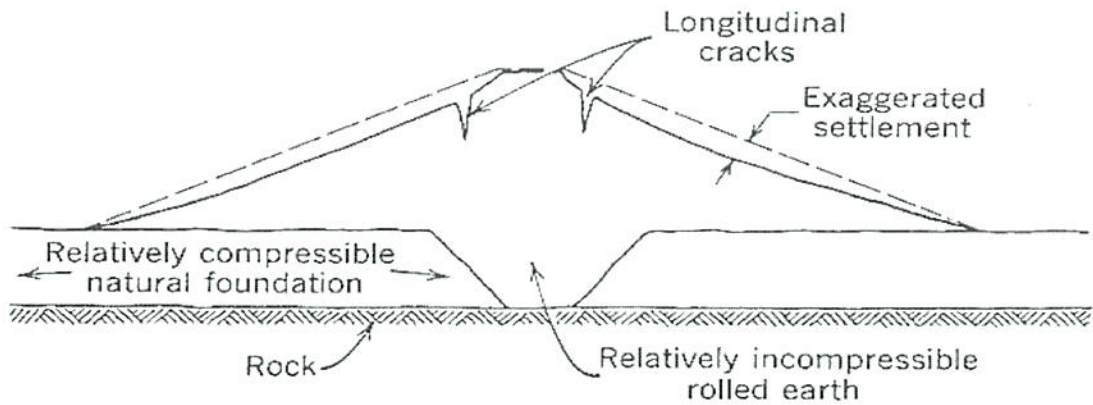


Figure 17 Longitudinal cracking due to differential settlements over foundation discontinuities (Sherard *et al* 1963)

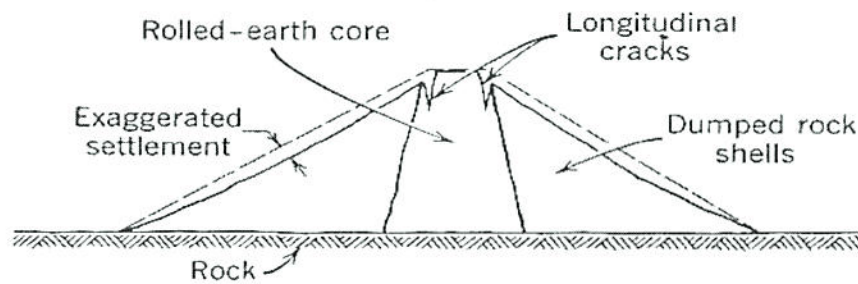


Figure 18 Longitudinal cracking due to differential settlements between embankment zones Sherard *et al* 1963)

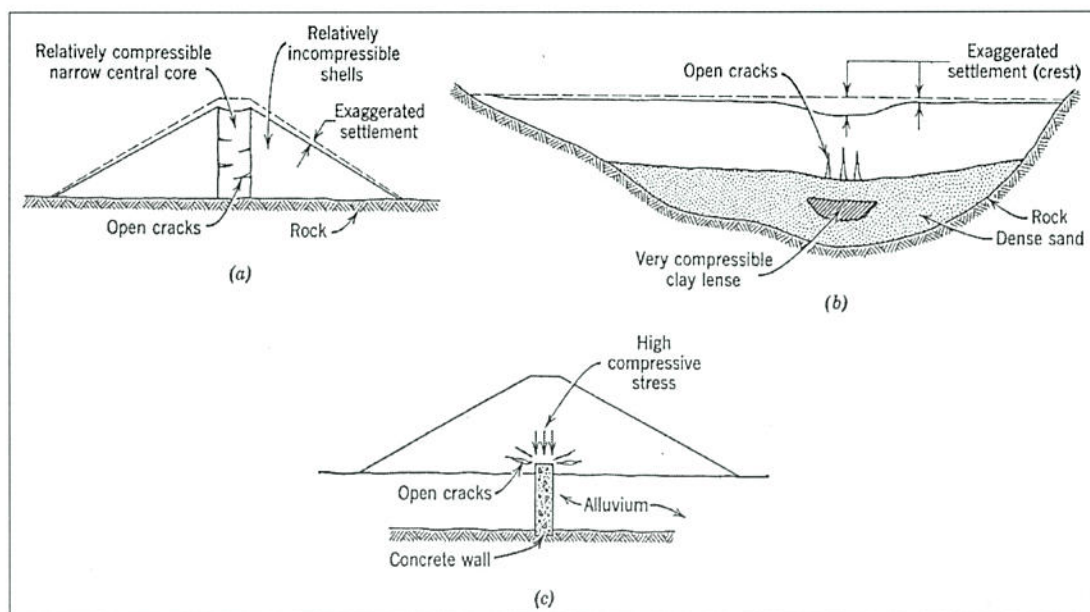


Figure 19 Internal cracking in embankments due to differential settlements (Sherard *et al* 1963)

An important insight made by Sherard *et al* (1963) is that compressive stresses from the overburden of material will work against the development of cracks inside the dam. These forces would be more significant in larger dams.

Figure 20 depicts the same phenomenon, but is taken from Resendiz *et al* 1982 for cracking induced by settlement under seismic loading.

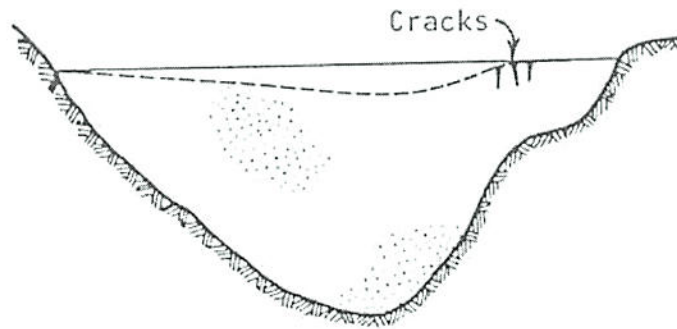
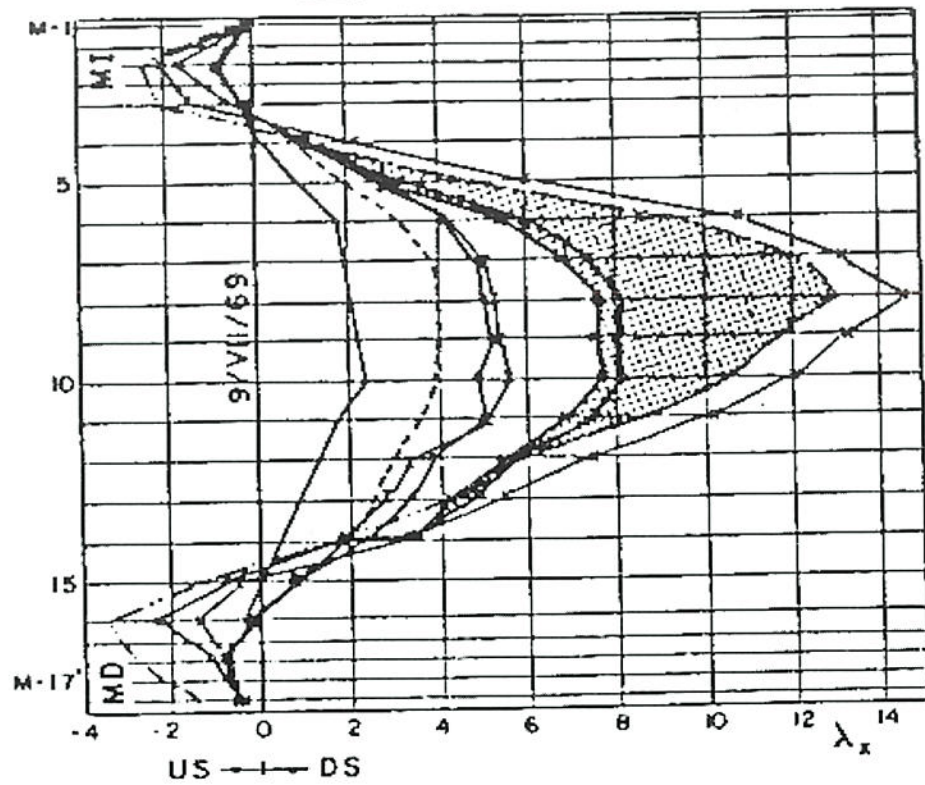


Figure 20 Transverse Cracking Due to Differential Settlements Over Foundation Discontinuities (Resendiz *et al* 1982)

5.1.3 Cracking due to overall horizontal displacement and twisting

What is considered here is horizontal movement and/or 'twisting' of the whole dam alignment. This was found to be a relatively common form of earthquake damage, and 26 such cases were recorded in the database. With the embankment crest being displaced in an upstream or downstream direction – often with each end moving a different direction.

Figure 21 shows measurement of horizontal displacements taken from the crest of El Infiernillo dam, featuring movements after 1975 and 1979 (shaded area) earthquakes. Transverse cracking was reported at the latter earthquake.



(shaded region represents the 1979 Guerrero Earthquake)

—	3/VII/70	M-i Bench marks at crest (row L-1)
- - -	9/IX/71	MD Right bank MI Left bank
- · - ·	22/VIII/73	US Upstream direction; DS Downstream direction
Z — Z	15/VII/75	λ_x Horizontal displacement of the crest (river direction), in cm
○ — ○	8/XI/77	
N — N	24/VIII/78	
X — X	16/III/79	
· · · ·	9/VIII/79	

Figure 21 Upstream-downstream displacements in El Infiernillo Dam (Resendiz *et al* 1982)

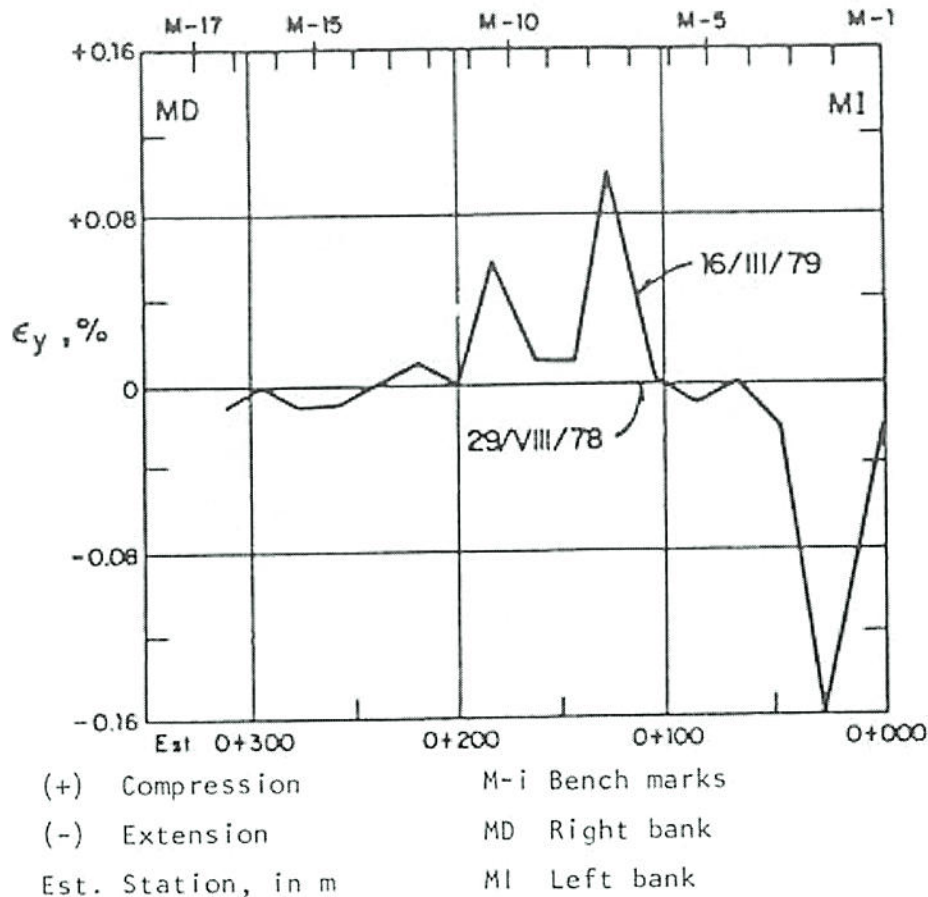


Figure 22 Longitudinal strains along the crest of El Infiernillo Dam (Resendiz *et al* 1982)

Measurements of vertical strains of the crest are shown in Figure 22. According to Resendiz *et al* (1982), “a perfect match exists between the zone of maximum tensile strain [on this figure] and that of intensive transverse cracking”. It is noted that the strains would be from a combination of vertical and horizontal displacements.

5.1.4 Slope failure and cracking

Slope failure on a slide surface can be activated by seismic loading. This is often, but not always, associated with loss of embankment and/or foundation strength through liquefaction. The slide surface is likely to dilate, and in effect becomes a “crack”. The early stages of formation of a slip surface may result in scarps at the surface that are not dissimilar from longitudinal cracking due to settlement or lateral spreading. Although this is a credible cause of longitudinal cracking, it is evidently not widely acknowledged, as only one reference used in the case studies gives mention to it – and only to state that it was not

the cause for longitudinal cracking in that particular case (Austrian dam – Harder *et al* 1996).

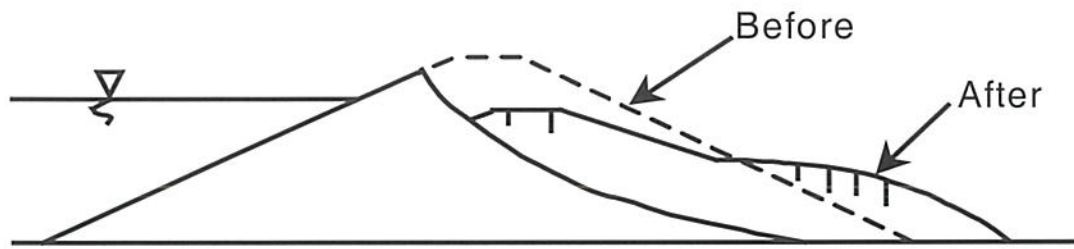


Figure 23 Sliding and longitudinal cracking (Resendiz *et al* 1982)

A full examination of the types and causes of slope failure / sliding is considered to be beyond the scope of the report. It is considered suffice to say here that longitudinal cracking in some case studies is thought likely to represent, at least in some cases, the top of a slide surface.

5.1.5 Separation cracks

‘Separations’ are taken to mean cracks that occur between two different units eg. between zones in a dam, or between a dam and a wall which the dam abuts.

An example of this was seen in the Austrian dam, where a separation crack over 8 metres deep and up to 500 mm wide at the surface was formed between the left abutment and the embankment (see case study in Appendix B). Also at Austrian dam, rocking of the spillway chute under earthquake loading resulted in separation cracks of 7 metres depth and 250mm width (as well as extensive damage to the spillway) (see Figure 24 and Figure 25). Rodda *et al* (1990) reported that some of the longitudinal cracking at Austrian dam aligned with the location of a line of buried grouting pipes left after construction.

At the Hebgen dam, which is an embankment with a concrete core, Sherard *et al* (1963) report ‘gaps’ opening on both sides of the core wall. Other case studies (El Infiernillo; Leroy Anderson) reported that longitudinal cracks were located at the interface between the zones of the embankments.



Figure 24 Photograph of transverse cracking adjacent to the spillway chute at Austrian Dam (U.S.G.S)

Sherard *et al* (1963) explain that the cracking about the concrete core at Hebgen dam was due to a 'relative vibration' between the concrete core and the embankment materials. They report that case studies of St Mary's dam (also concrete core) also support this theory. They suggest the same mechanism to be partly the cause for cracking of the clay core of Ono dam, and Upper and Lower Murayama dams.

This concept of 'relative vibration' (i.e. that different units respond differently to frequency of vibration) could be the explanation for a number of the incidences reported above. The separations between concrete spillways and dam materials are almost certainly partly due to this, but separation between units is also probably due to differential settlements too.

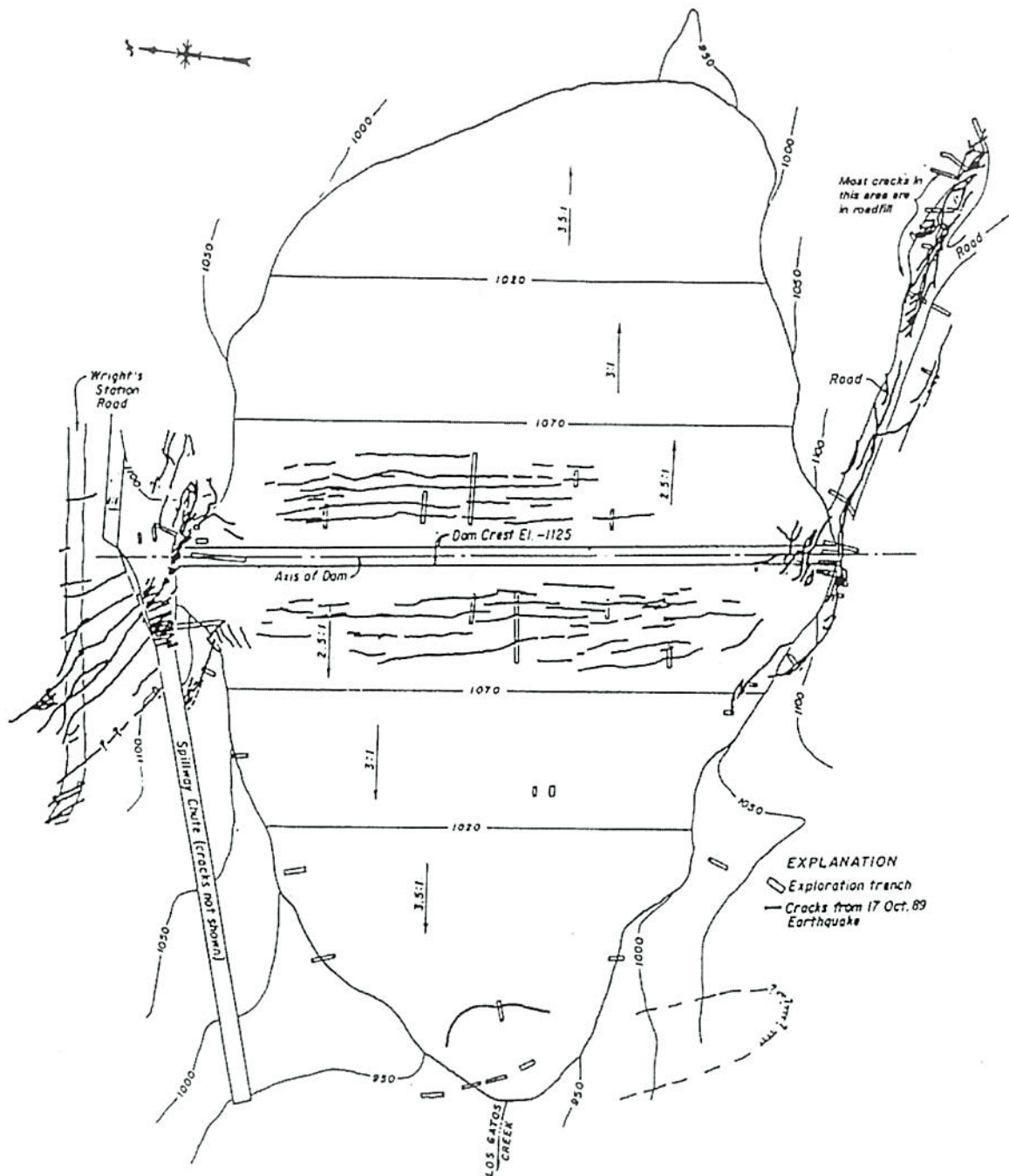


Figure 25 A plan of transverse and longitudinal cracking at Austrian Dam (Forster and MacDonald, 1998)

5.1.6 Dynamic cracking

Newmark (1965) in the paper “effects of earthquakes on dams and embankments” spoke of cracking being caused through shear forces being set-up by “relative shearing motions”. By this he explained as relative motions “between the dam and foundation” or from “differential movements arising from the wave characteristics of the surface motion”, or

“from stresses arising when parts of the mass of the dam and foundation are accelerated in one direction and other parts on other directions”. He said that these cracks were “usually vertical”, and indicated that they could be in either the longitudinal or transverse directions.

Figure 26 depicts the ‘measured resonant configurations’ of El Infiernillo rockfill dam (from Elgamal & Gunturi 1992). The figure is included to depict the types of movements described by Newmark (1965). It is clear that under these movements, shearing stresses would result transverse to the crest, and tensile stresses longitudinally.

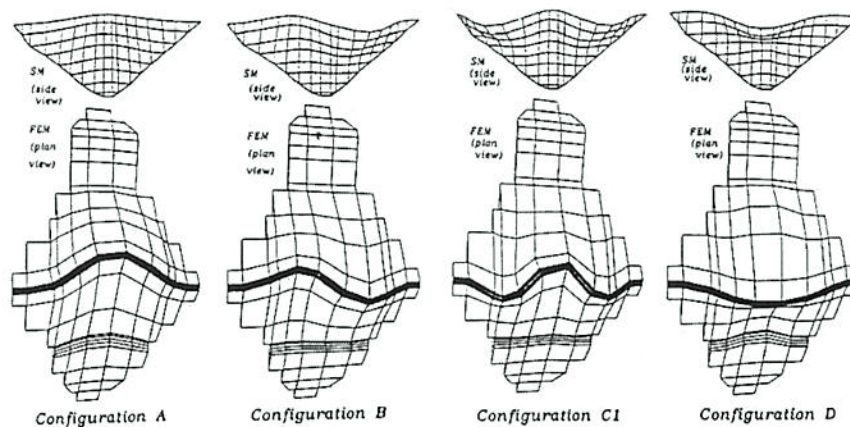


Figure 26 Measured resonant configurations of El Infiernillo rockfill dam (Elgamal and Gunturi, 1992)

Lane, (1980) in his paper “the failure of earth embankments by cracking during earthquake” postulated that longitudinal cracks formed from the phenomenon of ‘lurching’. His theory is based on understanding of amplification of shaking through the dam such that near the crest, horizontal and vertical forces exceed friction forces. Cracking may or may not occur depending on the shape of the dam zoning providing support about the shaken area, and the cohesive forces of the soil.

Sherard *et al* (1963) observed that “damage to the typical dam appears to have been caused primarily by the horizontal component of the earthquake movement in the upstream-downstream direction ... and this results in a ‘whipping’ action of the thinner top of the dam and causes longitudinal cracking”.

It is arguable that cracks forming under this such mechanism should close after the earthquake shaking ceases, unless other movements (e.g. settlements) cause the faces of the crack to misalign, so that a perfect closure cannot be made.

5.1.7 A Summary of cracking mechanics

Longitudinal cracking could be due to: settlements, lateral spreading and deformation, differential settlement activated for a variety of different reasons, lateral twisting of the embankment, slope instability, separations between units of the dam, either from differential settlement or relative vibrations, dynamic shearing or tensile forces, and ‘lurching’

Transverse cracking can occur due to: differential settlements, lateral twisting of the embankment, separations between units (e.g. between spillway walls and embankment materials) either from differential settlement or relative vibrations, and dynamic shearing or tensile forces

There are seen to be fewer mechanisms that may cause transverse cracking than there are that cause longitudinal cracking. The main reason is that there is a greater degree of freedom for lateral movements in the upstream-downstream direction.

5.2 Factors Influencing the Likelihood of Cracking or Hydraulic Fracture

Foster (1999) and Foster and Fell (1999, 2000) have summarized the factors which influence the likelihood of transverse cracking or hydraulic fracture of embankment dams under normal operating conditions. These are summarised Table 7. Given the incidence of transverse cracking at low seismic intensities, (see section 5.3.1) and the likely mechanisms leading to cracking, it is considered reasonable to assume that these factors will also apply to the seismic loading case.

Table 7 Influence of factors in the likelihood of cracking or hydraulic fracturing – features giving low stress conditions (Foster and Fell 2000)

Factor	Influence on likelihood of cracking or hydraulic fracture		
	More Likely	Neutral	Less Likely
Overall abutment profile	Deep and narrow valley. Abrupt changes in abutment profile, continuous across core. Near vertical abutment slopes	Reasonably uniform slopes and moderate steepness, eg. 0.25H:1V to 0.5H:1V	Uniform abutment profile, or large scale slope modification. Flat abutment slopes (>0.5H:1V)
Small scale irregularities in abutment profile	Steps, benches, depressions in rock foundation, particularly if continuous across width of core (examples: haul road, grouting platforms during construction, river channel)	Irregularities present, but not continuous across width of the core	Careful slope modification or smooth profile
Differential foundation settlement	Deep soil foundation adjacent to rock abutments. Variable depth of foundation soils. Variation in compressibility of foundation soils	Soil foundation, gradual variation in depth	Low compressibility soil foundation. No soil in foundation
Core characteristics	Narrow core, $H/W > 2$, particularly core with vertical sides	Average core width, $2 < H/W < 1$	Wide core $H/W < 1$
	Core material less stiff than shell material	Core and shell materials equivalent stiffness	Core material stiffer than shell material
	Central core		Upstream sloping core
Closure section (during construction)	River diversion through closure section in dam, or new fill placed a long time after original construction		No closure section (river diversion through outlet conduit or tunnel)

Foster (1999) shows that for normal operating conditions, the most likely location of cracking (and internal erosion and piping) is:

- Associated with small scale irregularities in the foundation profile near the foundation, and usually in the lower half of the core
- Associated with narrow cores and arching of the core between the shell zones – in the $\frac{1}{3}$ to height section of the core
- Associated with large scale changes in abutment profile – in the upper $\frac{1}{3}^{\text{rd}}$ of the dam core.

Factors influencing the likelihood of a concentrated leak associated with a conduit through the dam, and contacts between the core and spillway walls are summarized in Table 8 and 9.

Seismic loading is likely to increase the likelihood of a concentrated leak due to:

- Cracking and opening of joints in conduits due to lateral spreading deformations
- Creating stress conditions conducive to cracking or hydraulic fracture due to settlement caused by the earthquake
- Settlement caused by the earthquake leading to separation of the core and spillway walls.

Table 8 Influence of factors in the likelihood of a concentrated leak associated with a conduit (Foster and Fell 2000)

Factor	Influence on Likelihood of a Concentrated Leak			
	Much More Likely	More Likely	Neutral	Less Likely
Conduit Type (1)	Masonry, brick Corrugated steel	Steel, cast iron, not encased	Cast iron, concrete encased Concrete precast	Concrete encased steel Concrete cast in- situ
Conduit Joints (1)	Open joints, or cracks signs of erosion	Open joints	High quality joints, "open" up to 5mm but with waterstops	High quality joints, no openings, waterstops
Pipe Corrosion (1)	Old, corroded cast iron or steel	Old cast iron, steel		New steel with corrosion protection
Conduit Details (1)	Significant settlement or deep compressible foundation soils. Junction with shaft in embankment	Some settlement, shallow compressible foundation soils		Little or no settlement or rock foundation
Conduit Trench Details (2)	Narrow, deep, near vertical sides. Vertical sides, trench in soil (backfilled with concrete)	Medium depth, width, slope. Excavated through dam	Wide, side slopes flatter than 1H:1V	Trench totally in rock, backfilled with concrete

(1) Conduit type, joints, corrosion and details mostly affect piping into the conduit

(2) Conduit trench details mostly affect piping along the conduit

Table 9 Influence of factors in the likelihood of a concentrated leak associated with a spillway wall (Foster and Fell 2000)

Factor	Influence on Likelihood of a Concentrated Leak		
	More Likely	Neutral	Less Likely
Slope of wall	Overhanging	Vertical	Sloping $\geq 0.1H:1V$
Founding material	Soil, wall subject to settlement and rotation	Soil or weathered rock, no rotation	Rock
Finish on wall	Rough, irregular		Smooth, planar
Concrete collars, buttresses, overhangs	Present, particularly if shape makes compaction of core difficult		Not present
Special compaction	No special compaction adjacent to wall		Careful compaction adjacent to wall

5.3 Causes for, the Type of and the Extent of Cracking

To examine the causes for the type and incidence of cracking that a dam suffers, the database was sorted for the cases for which cracking occurred and for which details of PGA and magnitude was available. This was done for both transverse and longitudinal cracking. The sorted data sets are shown in Appendix D. These data points were then plotted in Figures 27 to 32 according to their seismic parameters in a manner similar to the damage contour plots shown above. These figures form the basis of the following discussion.

5.3.1 Seismic intensity and the incidence of cracking

It has been shown that there exists a relationship between seismic intensity and settlement (section 4.3 and 4.4) and there is a relationship between settlement and cracking (section 5.1), so it follows there should be a relationship between cracking and seismic intensity. Because the absence of reporting of cracking is not a reliable guide of 'no cracking', it is difficult to use the database to determine precisely a cut-off of when cracking doesn't occur. However, we can use the knowledge on events that did crack to examine trends. Figure 27 to Figure 32 show a trend of increasing transverse and longitudinal crack width with increasing seismic intensity (i.e. increasing magnitude and peak ground acceleration).. The trends, although reasonable, are not clearly defined enough to support the drawing of crack-width contours on the figures. This may be partly due to the variability in measuring and reporting of crack dimensions as discussed by Fong and Bennett (1995), but more likely due to many other variables which are not reported or analysed in this report. The dams which feature both longitudinal and transverse cracking tend to be found at higher seismic intensities (i.e. toward the top right corner). It suggests that for high intensity events, both types of cracking can be expected.

The data does not provide a clear 'cut-off' of what magnitude and PGA is required to cause cracking. Longitudinal and transverse cracking has been reported for PGA's as low as 0.02g, for a M7.4 earthquake. In general, however, a magnitude of greater than 6.5 is required to produce visible cracking.

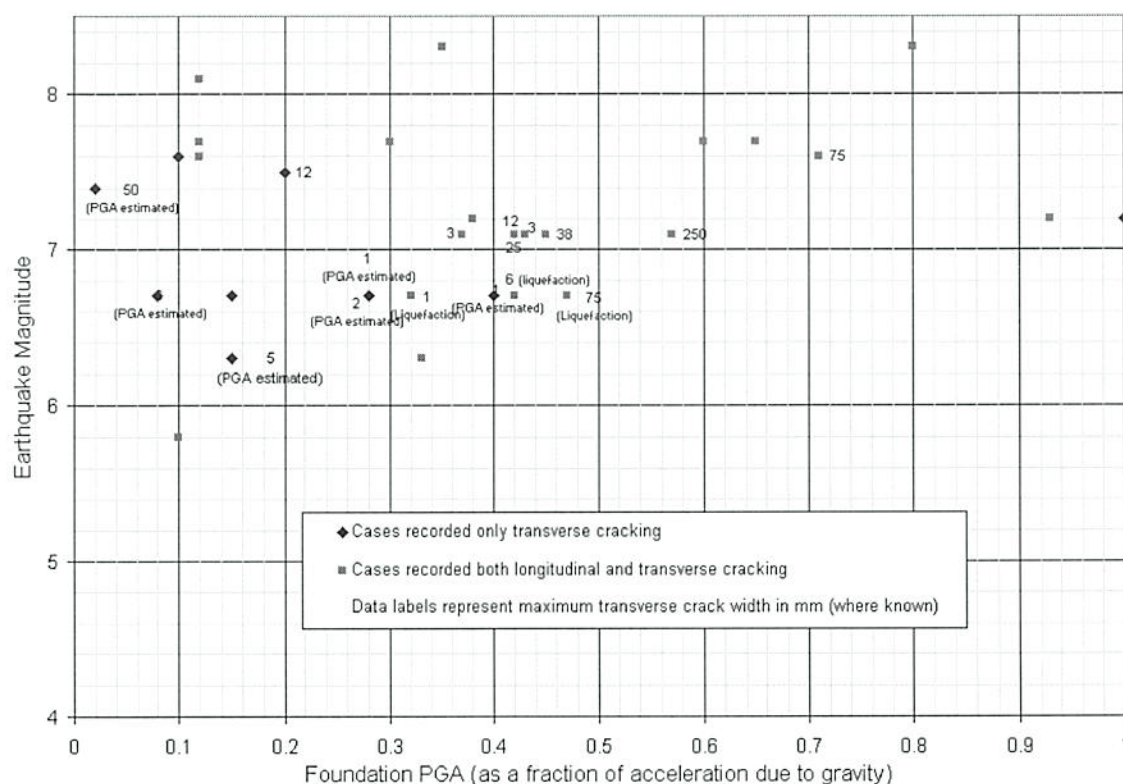


Figure 27 Incidence of transverse cracking vs seismic intensity for all dam types

It is possible that cracking may occur in the body of the dam at lower seismic intensities, if the dam is susceptible to cracking under normal operating loads.

For visible longitudinal cracks to occur, the dam needs to experience a magnitude 6.5 or greater earthquake, and a peak ground acceleration greater than about 0.15g for earthfill dams and 0.3g for earth and rockfill dams or an earthquake magnitude 7.0 or greater, and a PGA of 0.05g for earthfill and say 0.15g for earth and rockfill dams. For hydraulic fill dams, visible cracking may occur for magnitude 6 earthquakes, and 0.05 PGA

Dams which experience damage of class 2 or greater are highly likely to experience transverse cracking. However, transverse cracking has been observed at under M6.0 to M6.5 earthquakes, at PGA as low as 0.1 to 0.15g. There were 10 such cases.

Transverse cracking and longitudinal cracking occur together for M6.3, PGA >0.3g and M7.5, PGA >0.12g. This would seem to indicate there are two mechanisms acting for transverse cracking, the former for cases which were susceptible to cracking under static

loading (due to the cases outlined in Section 5.2), the latter where earthquake induced settlements and deformations alone are enough to cause transverse cracking.

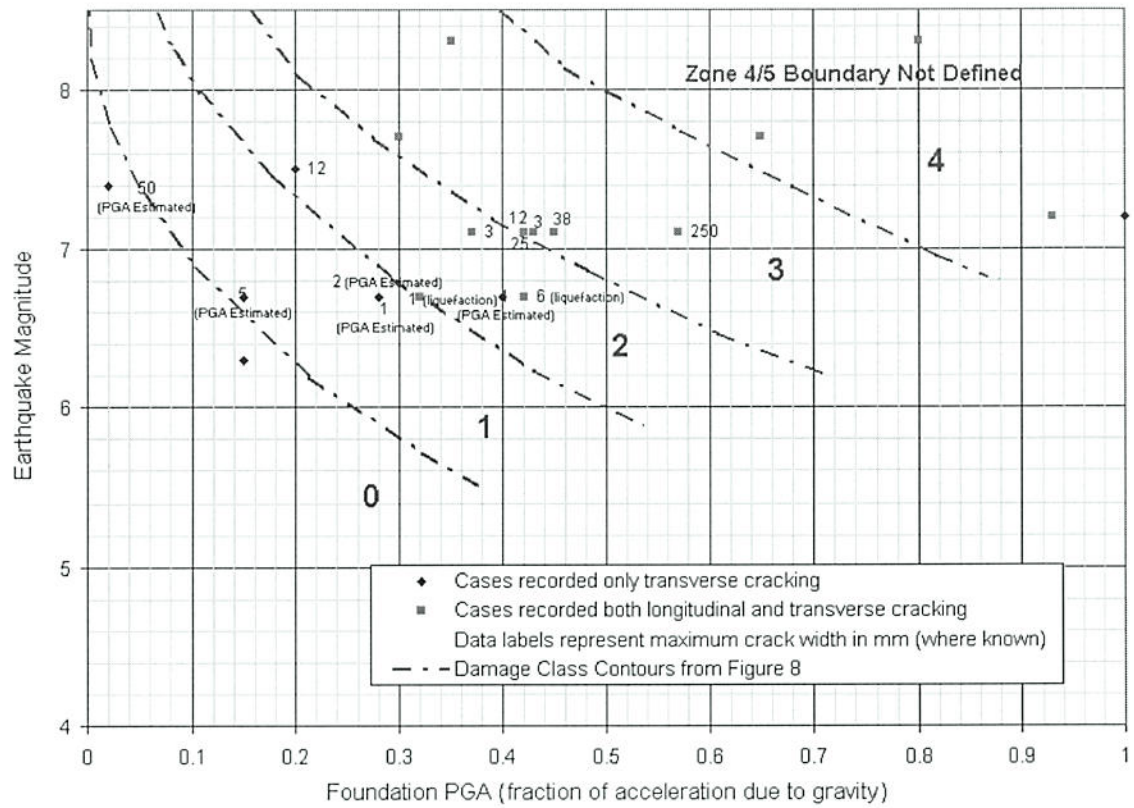


Figure 28 Incidence of transverse cracking vs seismic intensity and damage class contours for earthfill dams

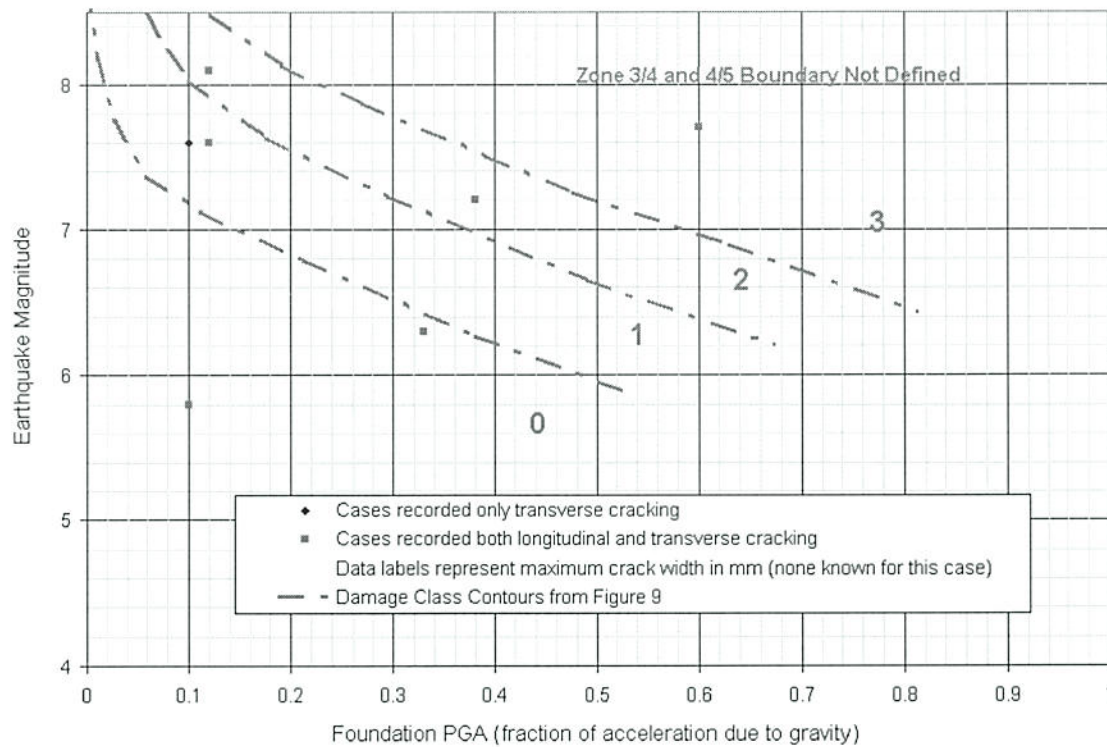


Figure 29 Incidence of transverse cracking vs seismic intensity and damage class contours for earthfill and rockfill dams

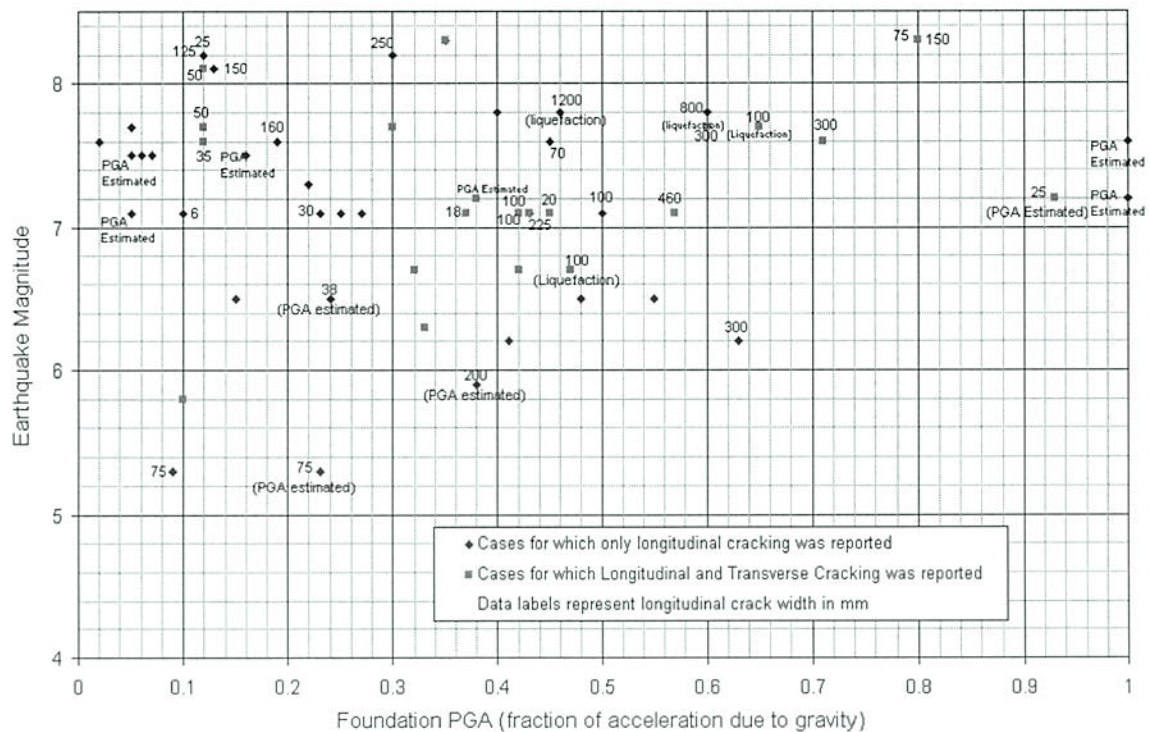


Figure 30 Incidence of longitudinal cracking vs seismic intensity and damage class contours for all dam types

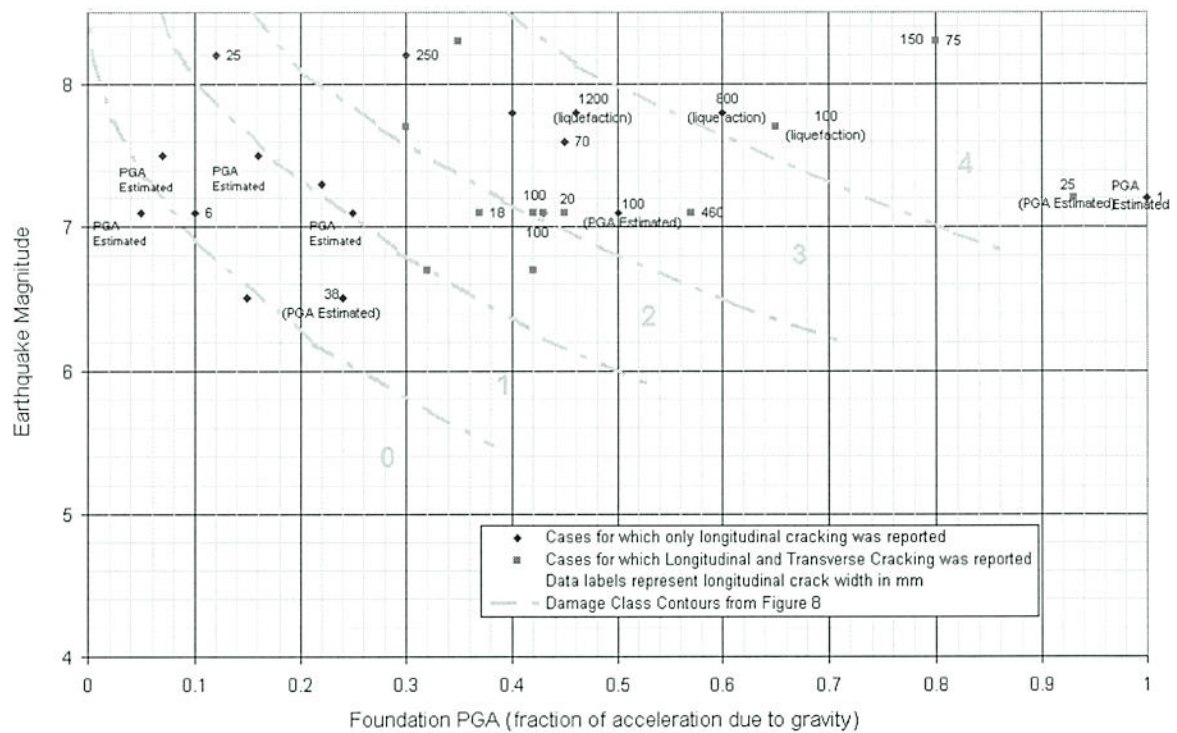


Figure 31 Incidence of longitudinal cracking vs seismic intensity and damage class contours for earth dams

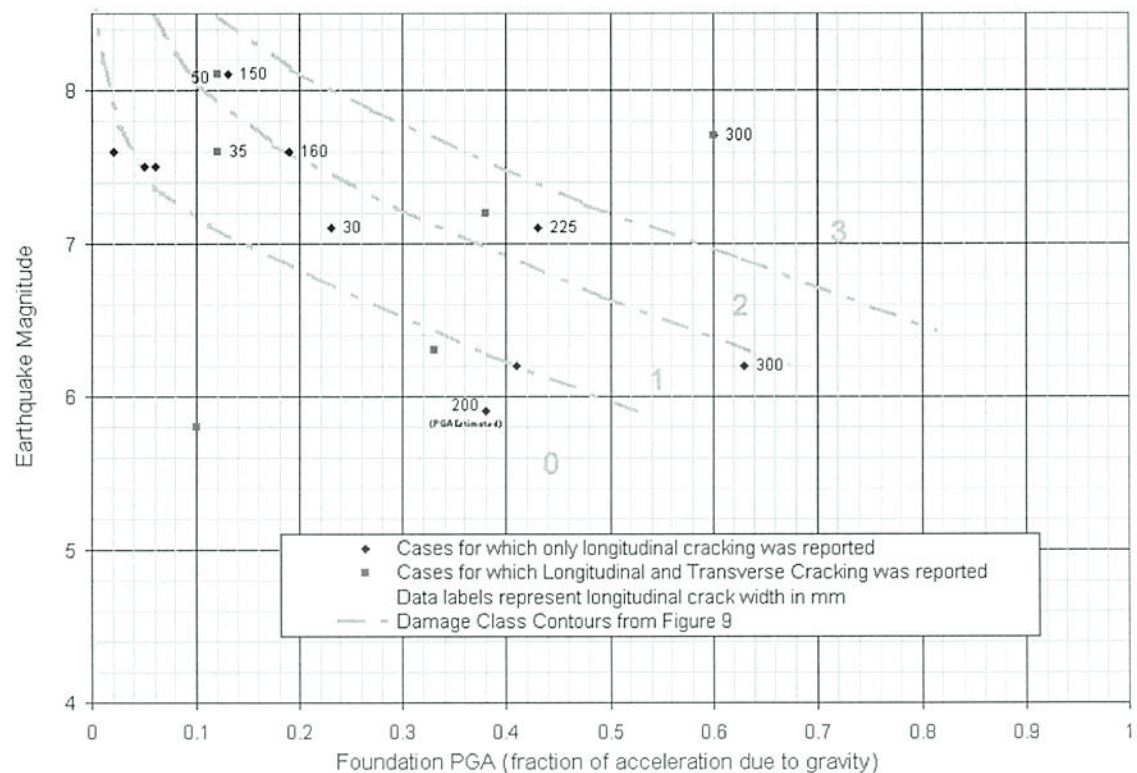


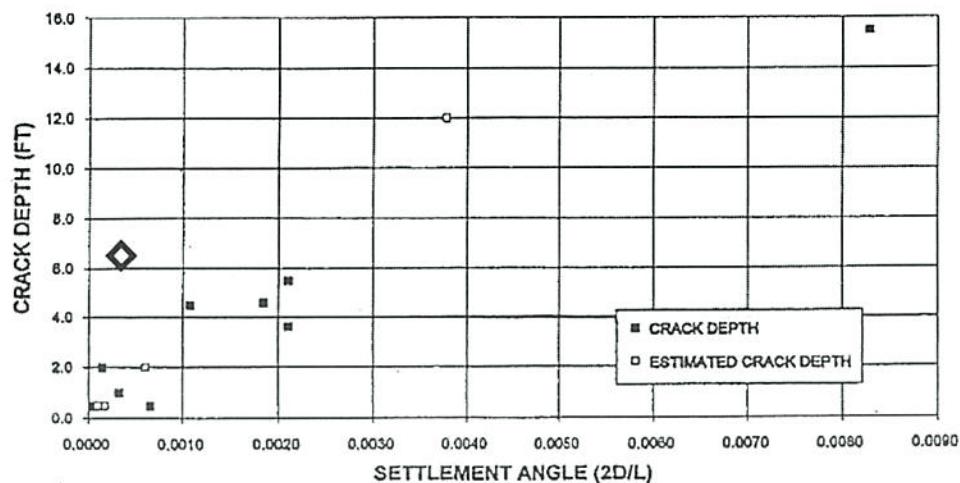
Figure 32 Incidence of longitudinal cracking vs seismic intensity and damage class contours for earth and rockfill Dams

5.3.2 The Causes of, location, and depth of transverse cracking

The data available for the dams showing transverse cracking at low seismic intensities are shown in Appendix D. Each case was examined, but for many of these dams only basic data was available. So it was not possible to show why these dams were particularly susceptible to transverse cracking.

Where transverse cracking occurs at higher earthquake intensities, the cracking can be explained by the differential settlements which occur across the valley under the earthquake loading.

It has been noted that most transverse cracking “has been observed at or near the abutment contacts” (Fong and Bennett, 1995). Swaisgood (1998) stated that transverse cracks ‘they tend to occur where there are steep slopes in the underlying foundation bedrock’. Fong and Bennett (1995) examined the relationship of transverse cracking with dam geometry by devising an index of ‘settlement angle’ – which is the ratio of the total maximum crest settlement to half the crest length. The relationship found is shown in Figure 33.



Note: D = total crest settlement, L = dam crest length

Figure 33 Fong and Bennett (1995) Plot of transverse crack depth vs settlement angle, showing one new data point

The data set was examined to see if this trend could be found, or if contributions could be made to this relationship, but the database offered only one point in addition to those shown. Its approximate location is shown on the plot.

The final point made in Fong and Bennett's paper (1995) was that "geometry, special conditions and other circumstances at individual embankments may strongly influence transverse cracking at those structures". The authors agree and suggest that any assessment of susceptibility to seismically induced transverse cracking should begin with assessing the likelihood of transverse cracking under normal loads as detailed in Section 5.2. Table 10 summarizes data on the depth of cracking:

Table 10 Data on the width and depth of transverse and longitudinal cracking

Dam	Longitudinal Cracking		Transverse Cracking		Comments
	Width(mm)	Depth(m)	Width(mm)	Depth(m)	
Austrian	25-100	1.5-4.5			Upstream slope (Harder et al)(1991))
	50-150	1.0-2.5			Downstream slope (Harder et al)(1991))
	460 max	4.5 max	250 max	3.1 max	(Forster and MacDonald 1998)
			500	8.2	Separation crack at left abutment (F&M)
			250	7	Separation crack adjacent spillway (F&M)
Leroy Anderson	20	0.6			
La Villita	50 max	1.5 max	"some"		
Guadalupe	100 max	1.4	"minor"		(Harder et al (1991))
Lexington	10	0.6 to 2	38	1.7	Left abutment, (Fong and Bennett (1995)
			10	1.1	Right abutment, (Fong and Bennett 1995)

It can be seen that there is at best a crude relationship between crack width and depth, probably reflecting poor quality data on crack depth, and the different mechanisms causing the cracking. For transverse cracking it is suggested this data be used with Figures 1, 2, 3 and 6, and an understanding of the mechanism causing the cracking to estimate the depth. Figures 6, 23 and 30 could be used as a crude guide to crack width. It seems that the ratio of transverse crack depth to crack width at the surface is about 15 to 100, averaging about 40.

5.4 Cracking of Dams Susceptible to Liquefaction

This report concentrates on dams which are not susceptible to liquefaction in the dam or its foundations.

Where there are liquefiable materials present, modern practice (eg. ANCOLD 1998, Fell et al 2000) mainly involves assessing the likelihood of liquefaction based on earthquake magnitude and PGA, and Standard Penetration Test (SPT) or Cone Penetration Test (CPT) and particle size distribution of the liquefiable soils. If flow liquefaction is possible, the extent of flow liquefaction and the post liquefaction strength (residual undrained strength) is used to calculate post liquefaction factor of safety. In many cases, if this is less than 1.0 it is assumed collapse may occur. In other studies a deformation analysis may be carried out to assess the loss of freeboard. Some (eg. Fell et al 2000) will make a subjective judgement of the likelihood of cracking, its depth, and susceptibility to leading to internal erosion and piping. The second authors experience is that where there are thin layers of liquefiable soil, the deformations are often calculated to be small, so freeboard may not be lost. However, during the Bhuj earthquake of January 2001 in Gujarat Province in India, several earth dams experienced extensive cracking and lateral spreading due it seems to liquefaction in the foundation. On the available report (from <http://geoinfo/usc.edu/gees>) which is attached in Appendix E, the amount of crest settlement was generally not extensive, but the extent of cracking (although mostly longitudinal) is such that the damage would be class 3 or 4, so transverse cracking would be likely. We are not able to quantify this but suggest some consideration be given to this phenomenon when assessing the likelihood of cracking under earthquake.

6 SUMMARY AND ASSESSMENT OF LIKELIHOOD OF INTERNAL EROSION AND PIPING

6.1 Damage and Cracking by Earthquakes

The study has shown that:

- Earthquakes cause settlement, lateral spreading and cracking of embankment dams. Slope instability may occur but it is not common.
- There are two main types of earthquake induced cracking found in dams -longitudinal cracks and transverse cracks. Longitudinal cracks are more common than transverse cracks, and are mostly in the upper part of the dam, more likely on the upstream face than the downstream.

- There are a number of mechanisms that can lead to formation of either of these types of cracks. Many of these mechanisms are common to both seismic loading and normal operating loading. There are seen to be more mechanisms that may lead to the formation of longitudinal cracks than transverse cracks, largely because lateral displacement is more readily achieved in the upstream-downstream directions.
- A method for predicting the likely extent of “damage” to an embankment dam under seismic loading has been developed. The damage is measured by the relative settlement and the width of longitudinal cracks, and accounts for four different dam types, and the intensity of seismic loading, as measured by the earthquake magnitude and peak ground acceleration.
- For visible longitudinal cracks to occur the dam needs to experience a magnitude 6.5 or greater earthquake, and a peak ground acceleration greater than about 0.15g for earthfill dams and 0.3g for earth and rockfill dams or an earthquake magnitude 7.0 or greater, and a PGA of 0.05g for earthfill and say 0.15g for earth and rockfill dams.. For hydraulic fill dams, visible cracking may occur for magnitude 6 earthquakes, and 0.05 PGA.
- Dams which experience damage of class 2 (relative settlement of 0.2% to 0.5% and/or longitudinal cracks 30-80mm wide) or greater are highly likely to experience transverse cracking. However, transverse cracking has been observed at under M6.0 to M6.5 earthquakes, at PGA as low as 0.1 to 0.15g.
- There is evidence to show that higher seismic loads will result in the formation of larger and deeper cracks, and is more likely to result in transverse cracking due to the greater differential settlements across the valley.
- At low seismic loading, there is evidence to show that only one type of cracking is likely to develop. This may be either longitudinal (more common) or transverse, depending on factors within the embankment that make it predisposed to a particular form. It is the nature of the embankment, zoning and the foundation geometry and presence of compressible materials and not the seismic loading, that differentiates between which form of cracking develops, and in particular determines whether transverse cracking occurs at low seismic intensities.
- The susceptibility of an embankment to a particular type of crack could not be related to dam type, but a weak relationship has been developed with dam shape (Fong and

Bennett, 1995) – dams in steeper valleys tend to show be more susceptible to transverse cracking near the abutments.

- The only way to predict which form of cracking may develop at low seismic loads is to examine the embankment and its foundations in detail, and identify and assess the crack mechanisms that exist. Dams which are susceptible to transverse cracking under static loading will be likely to experience transverse cracking under seismic loads.
- Crack depths may be estimated from Figures 1, 2 and 6.
- There is evidence that earthquakes have led to cracking followed by internal erosion, and led to failures of dams. It seems likely that more incidents would have occurred if, in a number of cases, reservoir levels had been higher when the earthquake occurred.
-

6.2 Assessment of the Likelihood of Internal Erosion and Piping

Cracking caused by earthquakes may lead to the initiation of internal erosion and piping of embankment dams if the reservoir level is above the base of the cracks at the time of the earthquake.

It is recommended that the procedure outlined in Foster and Fell (1999), Fell et al (2000), and Foster et al (2001), be used. This considers the internal erosion and piping process in four phases - initiation and continuation of erosion, progression to form a pipe, and formation of a breach. This is shown in Figure 34 for piping through the embankment initiated by a concentrated leak. In this case the concentrated leak would be due to cracking by the earthquake. Similar processes apply for piping through the foundations, and from the embankment to the foundations.

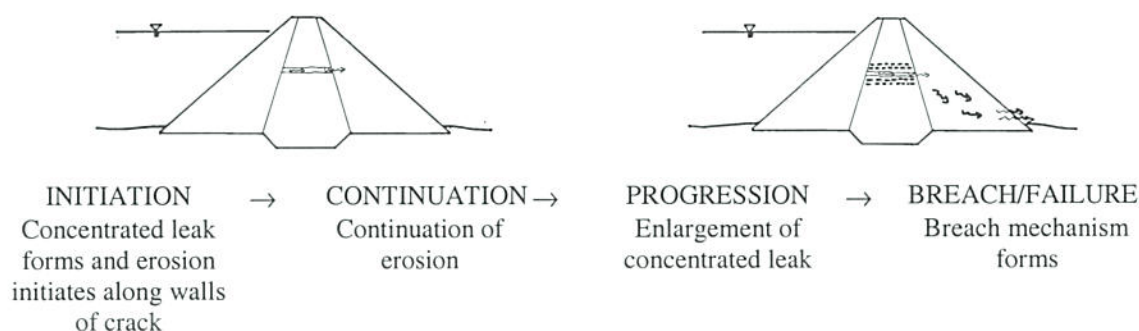


Figure 34 Model for the development of failure by internal erosion and piping - piping in the embankment initiated by a concentrated leak (Foster and Fell 1999(a), 2000).

The steps which should be followed include:

- (a) Assess the seismic loading for the dam, including relationships between the AEP of the peak ground acceleration and earthquake magnitude. This data should be obtained from seismologists.
- (b) Either select an operating basis earthquake (OBE), and maximum design earthquake (MDE), or, if a risk based approach is being used, partition the earthquake loading into say 6 or 7 load ranges, with matching AEP. The procedures for doing this are given in ANCOLD (1998).
- (c) Determine the zoning of the dam, the foundation conditions, and material properties needed to assess settlements which may occur under earthquake loading and the potential for liquefaction of the dam and the foundation.
- (d) Assess the likelihood of liquefaction of the dam and the foundation using methods set out in ANCOLD (1998) or other appropriate methods. If liquefaction is likely, assess the potential for loss of freeboard using post liquefaction strengths. If freeboard is lost for the assumed reservoir level it is probable the dam would breach. If freeboard is not lost, assess the likely settlement lateral spreading and cracking which may occur using the procedures given in (f), plus judgemental allowance for the effects of liquefaction.
- (e) Calculate the maximum settlement which is likely to occur for each earthquake load conditions. This may be done using simplified methods such as Swaisgood (1998), Makdisi and Seed (1978), or more sophisticated methods if the project warrants. For cases where liquefaction occurs the simplified methods do not apply, so a numerical analysis using post liquefaction strengths to judge the deformations will be needed.
- (f) Estimate the likely damage class for each earthquake load condition using Figures 8 to 11, and the estimated maximum settlement. From the damage class, estimate the maximum width of longitudinal cracking using Table 1 and Figures 30 to 32.
- (g) Estimate the incidence of, and maximum transverse crack width using Figures 27 to 29, but also taking account of the factors which lead to transverse cracking under normal operating loads as detailed in Section 5.2. Use this to assess the likely location of the cracking.
- (h) Estimate the likely depth of transverse cracking using Figures 1, 2 and 6 and Table 10, using judgement to account for foundation geometry and the embankment abutting spillway walls, buried conduits through the dam etc.

- (i) Use these depths, coupled with the dam zoning, filters, downstream zone properties and the likely reservoir level at the time of the earthquake, to assess the likelihood of internal erosion and piping using the procedures outlined in Foster et al (2001).

In doing this considerable care should be taken to ascertain the actual zoning and particle size distribution of the materials in the upper part of the dam, where cracking is most likely. There should not be over-reliance on design drawings and specifications as recent experience is that some dams were not constructed according to the drawings and specifications particularly during the 1960s.

It should also be remembered that not all cracking will be at the crest of the dam. As is evident in Figures 15 to 20, cracking may also occur lower down, and possibly even in the foundation soils.

This is best put into a risk based framework, because the likelihood of cracking and the depth of cracking, are related to be AEP of the earthquake, and if cracking occurs, internal erosion and piping are not certain, but dependent on many factors. However it may be sufficient to assess an OBE and MDE condition, and use qualitative terms to assess the likelihood of internal erosion and piping as was done in Bell et al (2001).

It will be clear that there is a large uncertainty in these calculations, with a considerable amount of sound dam engineering judgement needed.

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APPENDIX A – A Database of Dam Performance During Earthquakes

Dam Description

Dam Name	Location			Owner	Ref No.	Height m	Type	Crest Length m	L/H	Year Completed	Slope Batter		Freeboard at Quake m	Alluvium Thickness m	Foundation Geology
	Country	Long E	Lat N								Upstream	Down- stream			
Amoroto	Japan	-	-	-	50	41.0	earthfill	-	-	1962	-	-	-	-	-
Almaden	USA	-121.828	37.165	-	14,22,55	33.0	earthfill	152	5	1936	-	-	-	-	-
Almaden Valley	USA	-121.840	37.198	-	22	11.5	earthfill	-	-	1965	-	-	-	-	-
Ambuklao	Philippines	-	-	Philippines National Power Corporation	24,31	129.0	earthcore rockfill	-	-	1956	2: 1.75	1.75; 2	6	10	river gravel
Arcata	USA	-124.060	40.878	-	1,53	17.0	earthfill	49	3	-	2	2.5	-	-	rock
Asot	USA	-	-	-	40,53	220	rolled fill	-	-	-	-	-	-	-	-
Awana High	Egypt	32.888	24.078	-	38	111.0	earthcore rockfill	3600	32	-	-	-	-	130	-
Austrian	USA	-121.940	37.132	-	5,14,15,22,23,36,37,48,53	56.0	homogeneous earthfill	213	4	1950	3.5; 3.1; 2.5	2.5; 3.1; 3.5	30	0	rock
Aya	Philippines	-	-	-	48	102.0	earthcore rockfill	-	-	-	-	-	-	0	0m alluv
Bai River	China	-	-	-	43,53	65.0	compacted earthcore earthfill	-	-	-	-	-	-	-	-
Bay Street	USA	-122.047	36.075	-	22	8.2	earthfill	-	-	1924	-	-	-	-	-
Bear Gulch	USA	-122.339	37.342	-	1,40,53	14.0	earthfill	222	16	1896	3	3	-	-	weathered st / shale
Belvedere	USA	-	-	-	40,53	14.0	earthfill	-	-	1905	-	-	-	-	-
Berryman	USA	-122.262	37.885	-	40,53	12.0	earthfill	-	-	1905	-	-	-	-	stiff silty clay w/ gr
Binga	Philippines	-	-	Philippines National Power Corporation	24	102.0	earthcore rockfill	-	-	1960	-	-	10	-	-
Birusawa	Japan	-	-	-	43,50	24.0	compacted earthcore earthfill	-	-	1948	-	-	-	-	-
Birusawa	Japan	-	-	-	43,50	24.0	compacted earthcore earthfill	-	-	1948	-	-	-	-	-
Bouquet Canyon	USA	-118.383	34.540	-	1,40,53	58.0	earthfill	-	-	-	3	3	-	-	-
Bouquet Canyon	USA	-118.383	34.540	-	1,40,53	58.0	earthfill	-	-	-	3	3	-	-	-
Brand DB	USA	-118.280	34.185	-	14	14.0	embankment ...	122	9	-	-	-	-	-	-
Brand Park	USA	-118.273	34.185	-	40	29.0	rolled fill	-	-	-	-	-	-	-	-
Buena Vista	USA	-119.292	35.217	-	1,40,43,44,53,54	6.0	earthfill - some puddle core	10032	1672	1890	3, 4	3, 4	1.5	-	v. soft alluv w/ gypsum
Burlingame	USA	-	-	-	40,53	7.0	earthfill	-	-	1876	-	-	-	-	sandy clay
Calaveras	USA	-121.820	37.492	-	22,24	64.0	hydraulic earthfill / rockfill	365	6	1925	3	2.5	21	-	rock
Calero	USA	-121.792	37.183	-	22	27.0	earthfill	-	-	1935	-	-	-	-	-
Canil	Philippines	-	-	-	48	70.0	earthcore rockfill	351	-	1976	-	-	-	-	0m alluv
Canillo	Chile	-71.312	-32.572	-	43	14.0	earthfill	-	-	1853	-	-	-	-	rock
Canipulo	Chile	-71.312	-32.572	-	43	14.0	earthfill	-	-	1853	-	-	-	-	rock
Chatsworth	USA	-118.630	34.227	-	1,23,40,43,53,54	13.4	hydraulic earthfill	821	61	1918	2.5	2.5	-	-	-
Chatsworth	USA	-118.630	34.227	-	1,23,40,43,53,54	13.4	hydraulic earthfill	821	61	1918	2.5	2.5	13	-	-
Chatsworth #2	USA	-	-	-	44,48,53	10.5	hydraulic earthfill	610	58	1918	2.5	2.5	-	-	-
Chesbro	USA	-121.689	37.112	Santa Clara Valley Water District	5	29.0	earthfill	220	8	1955	-	-	-	-	-
Cherry Chase	USA	-118.203	34.175	-	40,53	11.0	rolled fill	-	-	-	-	-	-	-	-
Cipresses	Chile	-70.421	-34.320	-	43	28.0	earthfill	-	-	1958	-	-	-	-	basalt
Coastways	USA	-	-	Coastways Ranch Operations	5	14.0	earthfill	-	-	-	-	-	-	-	-
Cogon	Chile	-	-	-	1,2,23,24,25,43,38,51,53	85.0	concrete face rockfill	160	2	1939	1.4	1.5	7.7	0	rock
Cogon	Chile	-	-	-	1,2,23,24,25,43,38,51,53	85.0	concrete face rockfill	160	2	1938	1.4	1.5	-	0	bacteria
Cogon	Chile	-	-	-	1,2,23,24,25,43,38,51,53	85.0	concrete face rockfill	160	2	1939	1.4	1.5	-	0	rock
Cogon	Chile	-	-	-	1,2,23,24,25,43,38,51,53	85.0	concrete face rockfill	160	2	1938	1.4	1.5	-	0	bacteria
Cogswell	USA	-117.965	37.245	-	14,24,48	80.6	concrete face rockfill	180	2	1934	-	-	-	0	rock
Cogswell	USA	-117.965	34.245	-	14,24,48	80.6	concrete face rockfill	180	2	1934	-	-	-	0	rock
Colban	Chile	-	-	-	43	116.0	earthfill	-	-	1984	-	-	-	-	soil and rock
Cokman	USA	-	-	-	1,43	-	earth and concrete	-	-	-	-	-	-	-	glacial sand / gravel / boulders
Como	USA	-114.233	46.063	Bitter Root Irrigation District	52,54	21.0	semi-hydraulic fill	-	-	1910	-	-	-	-	-
Cowell	USA	-	-	-	22,40,53	15.2	earthfill	-	-	1890	-	-	-	-	-
Cowell	USA	-	-	-	22,40,53	15.2	earthfill	-	-	1890	-	-	15	-	-
Coyote	USA	-121.548	37.118	-	22,31,48,53	43.0	earthcore rockfill	299	7	1936	3	3	1.6	0	-
Coyote	USA	-121.548	37.118	-	22,31,48,53	42.5	earthcore rockfill	299	7	1936	3	3	-	0	-
Crocker	USA	-122.373	37.565	-	40,53	14.0	earthfill	-	-	1890	-	-	-	-	sandy clay
Crystal Springs Upper	USA	-	-	-	1,23,40,44,53	23.0	earthfill puddle core	213	9	1878	2, 3, 25	2, 3, 5	causeway	-	spans black and alluv
Culmo	Chile	-	-	-	43	36.0	earthfill	-	-	1933	-	-	-	-	soil
Culmo	Chile	-	-	-	43	36.0	earthfill	-	-	1933	-	-	-	-	soil
Damouke	Japan	-	-	-	50	15.0	earthfill	-	-	-	-	-	-	-	-
Dchell	USA	-121.633	37.003	-	22	16.0	earthfill	-	-	1952	-	-	-	-	-

Dam Description

Dam Name	Location		Owner	Ref No.	Height m	Type	Crest Length m	L/H	Year Completed	Slope Batter		Freeboard at Quake m	Alluvium Thickness m	Foundation Geology
	Country	Long E.	Lat N							Upstream	Down- stream			
Deer Creek	USA	-	-	43	72.0	centrally impermeable earthfill	-	-	1941	-	-	-	-	soil
Dayo	Philippines	-	-	48	60.0	carthoite rockfill	-	-	-	-	-	-	0	0m alluv
Declench	USA	-118.248	34.168	40.53	18.0	rolled fill	-	-	-	-	-	-	-	-
Dooche	China	-	-	23.39	22.0	homogeneous earthfill	6000	273	-	-	-	10	-	soft clay/silt
Drinkwater	USA	-118.522	34.530	40.53	40.53	earthfill	-	-	1923	-	-	-	-	-
Drinkwater	USA	-118.522	34.530	40.53	32.0	earthfill	-	-	1923	-	-	-	-	-
Dry Canyon	USA	-118.527	34.482	1,40,43,44,53,54	20.0	semi-hydraulic fill	180	9	1912	2.5	2.25	-	20	alt soft sat base 20m alluv
Dry Canyon	USA	-118.527	34.482	1,40,43,44,53,54	20.0	semi-hydraulic fill	180	9	1912	2.5	2.25	-	20	alt soft sat base 20m alluv
Eagle Rock	USA	-118.188	34.138	40.53	34.0	rolled fill	-	-	-	-	-	-	-	-
Earldrum	USA	-	-	23	6.0	earthfill	-	-	-	-	-	-	-	-
Eaton Wash	Scotland	-118.090	34.168	40.53	19.0	rolled fill	-	-	-	-	-	-	-	-
El Carpintero	Chile	-	-	35	-	homogeneous embankment	-	-	-	-	-	-	-	alt resid soil, find silt / sand / clay
El Infernillo	Mexico	-	-	6,12,23,27,28,29,30,34,43,48,53	147.0	carthoite rockfill	344	2	1964	1.75	1.75	-	0	rock, some alluv
El Infernillo	Mexico	-	-	6,12,23,27,28,29,30,34,43,48,53	147.0	carthoite rockfill	344	2	1964	1.75	1.75	-	0	rock, some alluv
El Infernillo	Mexico	-	-	6,12,23,27,28,29,30,34,43,48,53	147.0	carthoite rockfill	344	2	1964	1.75	1.75	-	0	rock, some alluv
El Infernillo	Mexico	-	-	6,12,23,27,28,29,30,34,43,48,53	147.0	carthoite rockfill	344	2	1964	1.75	1.75	-	0	rock, some alluv
El Infernillo	Mexico	-	-	6,12,23,27,28,29,30,34,43,48,53	147.0	carthoite rockfill	344	2	1964	1.75	1.75	-	0	rock, some alluv
Elmer J Chesbro	USA	-121.693	37.167	40.53	29.0	zoned embankment	210	7	1955	-	-	21	0	-
Elystan	USA	-118.228	34.078	40.53	22.0	rolled fill	-	-	-	-	-	-	-	-
Emerald Lake #1	USA	-122.387	37.467	40.53	17.0	earthfill	-	-	1855	1.3	-	-	-	-
Encina	USA	-	-	40.53	51.0	rolled fill	-	-	-	-	-	-	-	v stiff sand / clay
Estates	USA	-122.215	37.827	40.53	28.0	earthfill	-	-	1903	-	-	-	-	rock
Eureka	USA	-120.712	39.763	43	-	earthfill	-	-	-	-	-	-	-	-
Fairmont	USA	-118.427	34.687	40.53	37.0	hydraulic earthfill	-	-	1912	-	-	-	-	-
Fairmont	USA	-118.427	34.687	40.53	37.0	hydraulic earthfill	-	-	1912	-	-	-	-	-
Fontenelle	USA	-	-	24,52,54	42.0	rolled fill	1652	39	1965	3	2	27	-	weathered sat, some alluv
Forest Lake	USA	-121.942	36.592	40.53	18.0	earthfill	-	-	1892	-	-	-	-	-
Franklin Lower	USA	-118.410	34.095	14,40,53	31.0	hydraulic earthfill	152	5	-	-	-	-	-	-
Franklin Lower	USA	-118.410	34.950	14,40,53	31.0	hydraulic earthfill	152	5	-	-	-	-	-	-
Franklin Upper	USA	-	-	40.53	17.0	rolled fill	-	-	-	-	-	-	-	-
Fukutani	Japan	-	-	40.50	41.0	carthoite rockfill	-	-	1971	-	-	17	-	-
Fukutani	Japan	-	-	40.50	21.0	earthfill	-	-	1926	-	-	-	-	-
Gamanosawa	Japan	-	-	53	10.0	earthfill	-	-	-	-	-	-	-	-
Garvey Reservoir	USA	-118.115	34.050	53	48.0	earthfill	-	-	-	-	-	-	-	-
Glenooks	USA	-118.212	34.155	40.53	19.0	rolled fill	-	-	1989	-	-	-	-	-
Golce	Turkey	-	-	38	50.0	embankment	-	-	-	-	-	-	-	-
Green Verdugo	USA	-118.335	34.257	40.53	35.0	rolled fill	-	-	-	-	-	-	-	-
Guadalupe	USA	-121.878	37.198	5,14,15,22,48,53	43.0	earthfill	211	5	1935	5; 2.5	2.5	24	0	0m alluv
Hachi	Japan	-	-	43	-	earthfill	-	-	-	-	-	-	-	-
Hawee South	USA	-117.948	36.137	1,40,43,44,48,53	27.0	hydraulic earthfill	450	17	1913	2.5	2.5	-	38	lit alt rock, rt alt allv, base slope
Hansen	USA	-	-	40.53	29.0	rolled fill	-	-	-	-	-	-	-	-
Harold	USA	-118.115	34.527	40.53	9.0	rolled fill	-	-	-	-	-	-	-	-
Hascie	Japan	-	-	50	16.0	earthfill	-	-	1953	-	-	-	-	-
Haskins	USA	-121.310	36.957	22	22.0	hydraulic earthfill	-	-	1931	-	-	22	-	-
Hayakunuma	Japan	-	-	53	12.0	earthfill	-	-	-	-	-	-	-	-
Helgen	USA	-	-	1,23,24,43,44,48,53,55	37.2	earth / rock / concrete core	218	6	1915	3	2.5	"full"	10	rock, alluv rt alluv
Helena Valley	USA	-111.885	46.642	54	23.0	zoned earthfill	-	-	1959	-	-	-	-	alluvial
Higashidaira	Japan	-	-	50	17.0	earthfill	-	-	1940	-	-	-	-	-
Highland	USA	-	-	40	4.0	rolled fill	-	-	-	-	-	-	-	-
Hollywood Upper	USA	-118.335	34.125	40.53	26.0	rolled fill	-	-	1956	-	-	-	-	-
Hongou	Japan	-	-	50	17.0	earthfill	-	-	-	-	-	-	-	-
Hosogaki	Japan	-	-	53	8.5	earthfill	-	-	-	-	-	-	-	-
Howell Lower	USA	-122.013	37.190	1,22,40,43,53,54	12.0	earthfill	146	12	1877	2	2.5	-	-	-
Howell Lower	USA	-122.013	37.190	1,22,40,43,53,54	12.0	earthfill	146	12	1877	2	2.5	-	-	-

Dam Description

Dam Name	Location		Owner	Ref No.	Height	Type	Crest Length	L/H	Year Completed	Slope Batter		Freeboard at Quake	Alluvium Thickness	Foundation Geology
	Country	Long E.	Lat N							Upstream	Downstream			
Howell Upper	USA	-122.015	37.190	-	12.0	earthfill	192	17	1878	2	2	-	-	-
Howell Upper	USA	-122.015	37.190	-	11.0	earthfill	-	-	1878	-	-	-	-	-
Huechun	Chile	-	-	-	15.0	earthfill	-	-	1932	-	-	-	-	soil
Ichinogaya	Japan	-	-	-	8.0	earthfill	-	-	-	-	-	-	-	-
Idonoshin	Japan	-	-	-	6.0	earthfill	-	-	-	-	-	-	-	sand/gravel
Iroka Ike	Japan	-	-	-	29.0	earthfill	-	-	1633	-	-	-	-	-
Isabella	USA	-	-	-	56.0	earth / rockfill	502	9	1950s	-	-	-	-	-
Ivanhoe	USA	-	-	-	6.0	rolled fill	-	-	-	-	-	-	-	-
Iwakura Tameike	Japan	-	-	-	17.0	earthfill	-	-	1931	-	-	-	-	-
Iwanake	Japan	-	-	-	17.0	earthfill	-	-	-	-	-	-	-	-
Izumi	Japan	-	-	-	6.0	earthfill	-	-	-	-	-	-	-	-
Izumaka	Japan	-	-	-	7.8	earthfill	-	-	-	-	-	-	-	-
J.W.Winda	USA	-	-	-	15.0	rolled fill	-	-	-	-	-	-	-	-
Jen-I-Tan	Taiwan	-118.592	34.093	-	20.0	earthcore rockfill	1500	75	-	-	-	-	-	-
Kasai	Japan	-	-	-	-	earthfill	-	-	-	-	-	-	-	-
Kelly Cabin Canyon	USA	-121.428	37.122	-	9.7	earthfill	-	-	1955	-	-	-	-	-
Kiseniyama	Japan	-	-	-	95.0	earthcore rockfill	-	-	-	-	-	-	-	-
Kinayama	Japan	-	-	-	24.5	homogeneous earthfill	-	-	1968	3.2, 2.5; 2.0	2.0, 2.5	-	-	weathered granite
Koganezawa	Japan	-	-	-	21.0	earthfill	-	-	1938	-	-	-	-	-
Kotaya	Japan	-	-	-	44.0	earthcore rockfill	-	-	1991	-	-	-	-	-
Konaka Tameike	Japan	-	-	-	21.0	earthfill	-	-	1936	-	-	-	-	-
Kremasta	Greece	-	-	-	165.0	earthfill	-	-	1965	-	-	-	-	rock
Kushikino	Japan	-	-	-	32.0	rockfill	-	-	1971	-	-	-	-	-
Kuzuryu	Japan	-	-	-	127.0	earthcore rockfill	-	-	-	-	-	-	-	-
La Calera	Mexico	-	-	-	30.0	earthcore rockfill	-	-	1962	-	-	-	-	earth
La Marquesa	Chile	-	-	-	10.0	earthfill	220	22	1943 (1928)	-	-	3.5	0	alt residual soil, find silt / sand / clay
La Palma	Chile	-	-	-	10.0	earthfill	140	14	1935	-	-	4.5	0	alt residual soil, find silt / sand / clay
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
La Villita	Mexico	-	-	-	60.0	earthcore rockfill	427	7	1968	3; 2.5	3; 2.5	-	70	70m alluv. Rock abutments
Laguna	USA	-	-	-	15.0	earthfill	176	12	-	2.5	2.5	-	-	sandy clay / silty sand
Lagunita	USA	-	-	-	4.5	earthfill	760	169	1900	3	3	-	-	sandy gr clay
Lagunitas	USA	-	-	-	14.5	earthfill	-	-	1872	-	-	-	-	mudstone / stiff clay
Lahontan	USA	-119.067	39.463	-	38.0	earthfill	517	14	-	3	2	-	-	-
Lake Camille	USA	-122.253	38.277	-	9.0	earthfill	-	-	1800	-	-	-	-	-
Lake Chabot	USA	-122.238	38.140	-	41.0	earthfill	-	-	1870	-	-	-	-	slly gr clay
Lake Frey	USA	-122.187	38.288	-	25.0	earthfill	-	-	1894	-	-	-	-	rock
Lake Herman	USA	-122.150	38.097	-	15.0	earthfill	-	-	1905	-	-	-	-	clayey alluv
Lake Mary Upper	USA	-	-	-	48	earthfill	-	-	-	-	-	-	1	1m alluv
Lake Ralphine	USA	-122.668	38.455	-	11.0	earthfill	-	-	1882	-	-	-	-	-
Lake Ranch	USA	-122.052	37.220	-	11.6	earthfill	49	4	1877	2.75	2.5	-	-	weathered sst
Lake Ranch	USA	-122.052	37.220	-	11.6	earthfill	49	4	1877	2.75	2.5	-	-	weathered sst
Lake Temescal	USA	-122.290	37.848	-	32.0	semi-hydraulic fill	-	-	1869	3	5	-	-	rock
Lake Wohlford	USA	-117.003	33.167	-	35.0	rockfill / timber / masonry	-	-	1895	-	-	-	-	weathered granite
Lan Lajay	USA	-118.690	34.302	-	29.0	embankment	176	6	-	-	-	-	-	-
Leroy Anderson	USA	-121.628	37.161	-	73.2	earthcore rockfill	436	6	1950	2.5	2.5	-	0	-
Leroy Anderson	USA	-121.628	37.161	-	73.2	earthcore rockfill	436	6	1950	2.5	2.5	-	0	-

Dam Description

Dam Name	Location		Owner	Ref No.	Height m	Type	Crest Length m	L/H	Year Completed	Slope Batter		Freeboard at Quake m	Alluvium Thickness m	Foundation Geology
	Country	Long E	Lat N							Upstream	Down- stream			
Lexington	USA	-121.986	37.202	5,14,15,22,48,53	62.0	zoned embankment	246	4	1953	5.5	3	30	0	-
Lian Tan	Taiwan	-	-	16	28.0	earthfill concrete core	-	-	1600's	-	-	-	-	-
Liyutan	Taiwan	-	-	16	96.0	earthcore rockfill	235	2	1992	-	-	-	-	interbed slsten
Liu Jiu	Chile	-	-	43	20.0	earthfill	-	-	1912	-	-	-	-	-
Liu Jiu	Chile	-	-	43	20.0	earthfill	-	-	1912	-	-	-	-	-
Lo Ovalle	Chile	-	-	35,43	13.0	homogeneous embankment	-	-	1932	-	-	-	-	alt resid soil, find silt / sand / clay
Lo Ovalle	Chile	-	-	35,43	13.0	homogeneous embankment	-	-	1932	-	-	-	-	alt resid soil, find silt / sand / clay
Long Valley	USA	-118.707	37.590	38,39,53	55.0	earthfill	182	3	1941	3	3	-	0	-
Lopez	USA	-120.487	35.188	40,53	50.0	rolled fill	-	-	-	-	-	-	-	-
Los Angeles	USA	-118.483	34.295	3,7,10,14,24,48	47.0	asphalt face embankment	1018	22	1979	3.5	3	8	0	nonmarine dep & str
Los Angeles North Dyke	USA	-	-	3,10,14,48	36.0	embankment	426	12	1977	3	3	8	0	rock
Los Quilayes (La Marquesa?)	Chile	-	-	35	-	homogeneous embankment	-	-	-	-	-	-	-	alt resid soil, find silt / sand / clay
Mackay	USA	-	-	54	23.0	embankment	-	-	1918	-	1.5	-	-	sand / gravel alluv, limestone rbt
Magat	Philippines	-	-	48	100.0	earthcore rockfill	-	-	-	-	-	-	0	0m alt m
Makio	Japan	-	-	31,48,50	79.0	earthcore rockfill	-	-	1961	3	2.25	-	29	29m alt m
Makio	Japan	-	-	31,48,50	106.0	earthcore rockfill	260	2	1961	3	2.25	-	29	29m alt m
Makomanai	Japan	-	-	50	67.0	rockfill	-	-	1985	-	-	-	-	-
Malpasso	Peru	-	-	1,23,48	78.0	rockfill	152	2	1936	0.5	1.3, 1.5	"full"	30	glacial drift - bldt, gravel, sand, silt
Malpasso	Peru	-	-	1,23,48	78.0	rockfill	152	2	1936	0.5	1.3, 1.5	-	30	glacial drift - bldt, gravel, sand, silt
Manno Ike	Japan	-	-	50	23.0	earthfill	-	-	Around 700	-	-	-	-	grst, sand, silt
Masaway	Philippines	-	-	23,24,48	25.0	zoned earthfill	425	17	1981	2.3	2	2	3	-
Marabina	NZ	-	-	17,18,19,20,21,33,45,48,53,54	86.0	earthcore rockfill	400	5	1967	3.25, 2.5	2.3	-	-	rock abts, dense sed
Miloto	Japan	-	-	23,48,53	130.0	earthcore rockfill	-	-	-	-	-	-	0	base
Mill Creek	USA	-122.210	37.117	22,24,52,53	23/44	hydraulic earthfill	975/915	-	1889/1939/1942	4	3	4.9	-	<6m loose base + abts
Minase	Japan	-	-	23,43,48,50,53	67.0	concrete faced rockfill	-	-	1963	-	-	-	-	-
Minase	Japan	-	-	23,43,48,50,53	67.0	concrete faced rockfill	-	-	1963	-	-	-	-	-
Minogawa	Japan	-	-	49	47.0	earthcore rockfill	-	-	1983	-	-	-	-	-
Nitarai	Japan	-	-	50	43.0	rockfill	-	-	1979	-	-	-	-	-
Mount N. Basin Upper	USA	-	-	53	5.0	earthfill	-	-	-	-	-	-	-	-
Mukamoto	Japan	-	-	50	5.8	earthfill	-	-	-	-	-	-	-	-
Murayama Lower (Shimo)	Japan	-	-	1,24,43,44,50,53	16.0	earthfill	587	37	1927	3	2.4	9.2	-	dense gravel, stiff clayey sand
Murayama Upper (Kano)	Japan	-	-	1,24,43,50,53	24.0	semi-hydraulic fill	322	13	1923	3	2.4	-	3	dense sand gravel w/ soft lenses
Nabetsunike	Japan	-	-	50	19.0	earthfill	-	-	before 1953	-	-	-	-	-
Nagara	Japan	-	-	48,50	52.0	earthcore rockfill	-	-	1985	-	-	-	-	-
Narawa	Japan	-	-	48,50	52.0	earthcore rockfill	-	-	1982	-	-	-	0	-
Newell Creek	USA	-122.072	37.103	5,15,22,53	55.0	earthcore rockfill	-	-	1960	3	-	15	-	-
Nimai-Bashi	Japan	-	-	53	-	earthfill	-	-	-	-	-	-	-	-
Niwakumine	Japan	-	-	50	16.0	earthfill	-	-	1927	-	-	-	-	-
Niomichi	Japan	-	-	50	16.0	earthfill	-	-	1957	-	-	-	-	-
Notre Dame	USA	-122.307	37.508	40,53	15.0	earthfill	-	-	-	-	-	-	-	rock
Oak Site	USA	-122.170	37.128	22	13.0	earthfill	-	-	1969	-	-	-	-	-
Oharakike	Japan	-	-	50	19.0	earthfill	-	-	-	-	-	-	-	-
Ohale	Japan	-	-	50	16.0	earthfill	-	-	1927	-	-	-	-	-
Ohrutsumi	Japan	-	-	50	15.0	earthfill	-	-	1940	-	-	-	-	-
O'Neill	USA	-121.050	37.117	22	21.0	earthfill	14300 ft	-	1967	-	-	4	-	rt alt rock, rest alluvial
Oro	Japan	-	-	1,23,24,43,44,48,53	37.0	puddle clay core	328	9	1914	2.5, 3	2.5, 3	14.8	11	-
Orange County Reservoir	USA	-117.883	33.937	50,53	35.0	earthfill	-	-	-	-	-	-	0	rock
Oroville	USA	-121.493	39.545	23,39,43,46,48,53	230.0	zoned earthfill	2104	9	1968	-	-	7.5	-	-
Orant	Japan	-	-	49,50	16.5	zoned earthfill	-	-	1968	-	-	-	-	-

Dam Description

Dam Name	Location			Owner	Ref No.	Height m	Type	Crest Length m	L/H	Year Completed	Slope Batter		Freeboard at Quake m	Alluvium Thickness m	Foundation Geology
	Country	Long E.	Lat N								Upstream	Down- stream			
Otanike	Japan	-	-	-	24.53	27.0	zoned earthfill earthfill	-	-	1920	4	3	-	-	rock
Otanike	Japan	-	-	-	24.53	27.0	homogeneous embankment	-	-	1920	4	3	-	-	rock
Pablo	USA	-	-	U.S. Bureau of Indian Affairs	54	13.0	earthfill	-	-	1912	-	-	-	-	silt/gravel
Pacific Grove	USA	-121.917	36.612	-	40.53	6.0	earthfill	-	-	1882	-	-	-	-	-
Pailao Man	China	-	-	-	23,39,53	66.0	earthfill	-	-	1882	3	-	-	-	-
Panabangan	Philippines	-	-	Philippines National Power Corporation	24.48	107.0	zoned earthfill	730	7	1977	2.5	2.2	14	0	-
Peboddy	USA	-121.507	37.042	-	22	19.0	earthfill	-	-	1950	-	-	-	-	alt resid soil, find silt / sand / clay
Peralta de Taphue	Chile	-	-	-	35	-	homogeneous embankment	-	-	-	-	-	-	-	alt resid soil, find silt / sand / clay
Piedmont	USA	-122.223	37.827	-	1,24,40,43	16.0	homogeneous embankment	76	5	1905	2	2	-	-	sst
Placintos	USA	-122.423	37.548	-	1,40,44,53	31.0	puddle core, rolled fill	158	5	1866	2, 2.5	2	-	16	sst / shale / limestone / basalt
Pinzanes	Mexico	-	-	-	1,53	67.0	concrete faced rockfill	249	4	-	1.5	1.3	-	-	rock
Pizama	Chile	-	-	-	43	17.0	earthfill	-	-	1931	-	-	-	-	soil
Pizama	Chile	-	-	-	43	17.0	earthfill	-	-	1931	-	-	-	-	soil
Poggio Cancelli	Italy	-	-	-	1,53	17.0	earthfill	456	27	1943	2.5	2, 2.5	-	-	soil
Port Costa	USA	-	-	-	40.53	14.0	earthfill	-	-	1890	-	-	-	-	sandy clay / silt
Porter Estates	USA	-118.562	34.263	-	14,40,48,53	12.0	rolled fill	94	8	-	-	-	-	0	-
Porter Estates	USA	-118.562	34.263	-	14,40,48,53	12.0	rolled fill	94	8	-	-	-	-	0	-
Recoleta	Chile	-	-	-	43	47.0	earthfill	-	-	1934	-	-	-	-	gneiss, schists
Reservoir #1	USA	-118.298	34.195	-	40.53	11.0	earthfill	-	-	1969	-	-	-	-	0m alluv
Rinconada	USA	-121.983	37.257	-	22.53	12.0	earthfill	-	-	-	-	-	-	-	-
Robert Matthews	USA	-123.432	40.368	-	48	46.0	earthfill	-	-	-	-	-	-	0	-
Rogers	USA	-	-	-	43	-	earthfill	-	-	-	-	-	-	-	-
Rotanike	Japan	-	-	-	50	15.0	earthfill	-	-	-	-	-	-	-	-
Rosnera	USA	-	-	-	40.53	9.0	rolled fill	-	-	-	-	-	-	-	-
Rubio	USA	-118.117	34.180	-	14	19.0	embankment ...	237	12	-	-	-	-	-	-
Rubio Div. Delors	USA	-118.117	34.180	-	53	17.0	earthfill	-	-	-	-	-	-	-	-
Rungue	Chile	-	-	-	43	21.0	earthfill	-	-	1962	-	-	-	-	gneiss, schists
Runkle	USA	-118.730	34.243	-	40.53	12.0	rolled fill	-	-	-	-	-	-	-	gravelly sand / clay up to 12m
San Andreas	USA	-122.412	37.580	-	1,23,40,44	29.0	puddle core, rolled fill	271	9	1870	3.5	3	-	15	-
San Andreas Old	USA	-	-	-	53	9.0	earthfill	-	-	-	-	-	-	-	-
San Antonio	USA	-120.883	35.798	-	53	48.0	earthfill	-	-	-	-	-	-	-	-
San Fernando Dyke	USA	-	-	-	40.53	11.0	rolled fill	-	-	-	-	-	-	-	-
San Fernando Lower	USA	-118.475	34.287	Los Angeles Department of Water and Power	3,4,8,9,23,40,41,43,48,53	43.0	hydraulic earthfill	-	-	1911-1940	3	4.5	38	11	11m stiff clay alluv, shale /sst abts
San Fernando Lower	USA	-118.475	34.287	Los Angeles Department of Water and Power	3,4,8,9,23,40,41,43,48,53	43.0	hydraulic earthfill	-	-	1911-1940	3	4.5	38	11	11m stiff clay alluv, shale /sst abts
San Fernando Upper	USA	-118.492	34.303	Los Angeles Department of Water and Power	3,4,14,23,40,41,43,48,53	24.0	hydraulic earthfill	380	16	1921	-	-	-	18	11m stiff clay alluv, shale /sst abts
San Fernando Upper	USA	-118.492	34.303	Los Angeles Department of Water and Power	3,4,14,23,40,41,43,48,53	24.0	hydraulic earthfill	380	16	1921	-	-	-	18	11m stiff clay alluv, shale /sst abts
San Justo	USA	-121.446	33.817	-	22,48,53	41.0	earthcore rockfill	330	7	1986	2.5	2	-	14	clay / sand gr sandy clay / bedrock
San Leandro	USA	-	-	-	1	38.0	semi-hydraulic fill	152	4	1892	3, 4	7	3.30	5	gr sandy clay / bedrock
San Leandro (Chabot)	USA	-	-	-	22,24,40,53	43.0	semi-hydraulic fill	152	4	1892	3, 4	7	-	5	gr sandy clay / bedrock
San Luis	USA	-	-	-	22	95.0	homogeneous earthfill	-	-	1967	-	-	52	-	-
Sandaling	China	-	-	-	43	17.0	earthfill	-	-	-	-	-	-	-	-
Santa Felicia	USA	-118.752	34.462	-	38,40,48,53	61.0	zoned earthfill	389	6	-	-	-	-	0	-
Santa Felicia	USA	-118.752	34.462	-	38,40,48,53	61.0	zoned earthfill	389	6	-	-	-	-	0	-
Saratoga	USA	-	-	-	53	434	earthfill	-	-	1948	-	-	-	-	soil
Sausal	Chile	-	-	-	434	32.0	earthfill	-	-	-	-	-	-	-	-
Sawtell	USA	-	-	-	40.53	10.0	rolled fill	-	-	1958	-	-	-	-	-
Schokohatamske	Japan	-	-	-	50	30.0	earthfill	-	-	1948	-	-	-	-	-
Schlage No. 2	USA	-121.623	37.048	-	22	13.0	earthfill	-	-	-	-	-	-	-	-

Dam Description

Dam Name	Location		Owner	Ref No.	Height m	Type	Crest Length m	L/H	Year Completed	Slope Batter		Freeboard at Quake m	Alluvium Thickness m	Foundation Geology
	Country	Long E Lat N								Upstream	Down- stream			
Temperovrens	USA	-122.207	37.190	22	13.0	earthfill	-	-	1931	-	-	-	-	-
Sepulveda	USA	-	-	40.53	17.0	rolled fill	-	-	-	-	-	-	-	-
Sheffield	USA	-119.692	34.445	1,2,3,24,42,43,44,53	7.6	concrete face earthfill	219	29	1917	2.5	2.5	3.2	6	1.3m sand/silt allm over st
Shonosen	Japan	-	-	50	25.0	earthfill	-	-	1936	-	-	-	-	-
Showa-ike	Japan	-	-	49	16.0	earthfill	-	-	1942	-	-	-	-	-
Shui Shui	Taiwan	-	-	16,38	30.3	earthfill concrete core	363	12	1934	4	4	-	-	rock
Silver Lake	USA	-	-	40.53	16.0	hydraulic earthfill	-	-	-	-	-	-	-	-
St. Helena Lower	USA	-	-	40.53	15.0	earthfill	-	-	1878	-	-	-	-	-
St. Helena Upper	USA	-	-	40.53	15.0	earthfill	-	-	1900	-	-	-	-	-
St. Mary's	USA	-	-	1,43,44	17.0	concrete core earthfill	87	5	1928	2.5	2	-	-	soil
Steinaker	USA	-109.533	40.506	54	49.0	zoned earthfill	-	-	1962	-	-	-	-	alluvial
Stone Canyon	USA	-118.452	34.107	40	56.0	rolled fill	-	-	-	-	-	-	-	-
Stone Canyon Upper	USA	-118.455	34.122	40.53	33.0	rolled fill	-	-	-	-	-	-	-	-
Summit	USA	-122.270	37.905	40,43,53	6.0	embankment	-	-	1891	-	-	-	-	stiff silty sandy clay gness abtments
Surge	Turkey	-	-	32	55.0	rockfill	736	13	1972	2.5; 3; 1.75	2; 2.5; 6	-	-	-
Sycamore Canyon	USA	-118.797	34.255	14	12.0	embankment ...	462	39	-	-	-	-	-	-
Takamatukanchiroke	Japan	-	-	50	15.0	earthfill	-	-	-	-	-	-	-	-
Takinosawa	Japan	-	-	50	15.0	earthfill	-	-	1954	-	-	-	-	-
Tanjima	Japan	-	-	49,50	28.0	zoned earthfill	-	-	1974	-	-	-	-	-
Tanosaawa	Japan	-	-	43,50	23.0	compacted homogeneous earthcore rockfill	-	-	1956	-	-	-	-	-
Tarumizu	Japan	-	-	53	43.0	earthfill	-	-	-	-	-	-	-	-
Tatenuchikamuke	Japan	-	-	50	21.0	earthfill	-	-	-	-	-	-	-	-
Tejon Ranch	USA	-	-	1,40,53	10.0	earthfill	176	18	-	-	-	-	-	-
Tokawa	Japan	-	-	49,50	33.5	zoned embankment	-	-	1974	3	2	-	-	-
Tokyo W.S.	Japan	-	-	53	16.0	earthfill	-	-	-	-	-	-	-	-
Tocho	China	-	-	53	22.0	earthfill	-	-	-	-	-	-	-	-
Touho	Taiwan	-	-	38	-	-	-	-	-	-	-	-	-	-
Touho	Taiwan	-	-	48	131.0	earthcore rockfill	-	-	-	-	-	-	-	sandy gr. Clay / silty sand sod and rock
Tsengwen	Taiwan	-	-	-	-	-	-	-	-	-	-	-	-	-
Univ. Mount N Basin	USA	-122.412	37.727	40	5.0	embankment	-	-	1885	-	-	-	-	-
Ushino	Japan	-	-	43,50	23.0	rockfill	-	-	1965	-	-	-	-	-
Uvas	USA	-121.690	37.067	22	36.0	earthfill	-	-	1957	-	-	-	-	-
Van Norman Bypass	USA	-	-	3	-	-	-	-	-	-	-	-	-	-
Vascona	USA	-121.963	37.247	5,14,15,22,48,53	10.0	earthfill	149	19	1935	-	-	3.6	8	-
Vermilion	USA	-118.983	37.370	24,53	50.0	zoned earthfill	1287	26	1954	2.25; 3	2; 2.5	-	80	silt / sands / gravel <80m rock
Vir	India	-	-	24,53	24.0	earthfill	-	-	1961	2	-	-	-	-
Volvano Lake	Mexico	-	-	53	4.0	earthfill	-	-	-	-	-	-	-	-
Volvano Lake	Mexico	-	-	53	4.0	earthfill	-	-	-	-	-	-	-	-
Wadake	USA	-	-	54	19.0	zoned earthfill	-	-	1935	-	-	-	-	silt/gravel
Watsuka	Japan	-	-	50	27.0	earthfill	-	-	1923	-	-	-	-	-
Whittier Narrows	USA	-	-	53	28.0	earthfill	-	-	-	-	-	-	-	-
Wide Canyon	USA	-116.393	33.967	14,48	26.0	earthfill	676	26	-	-	-	-	-	-
Wood Ranch	USA	-118.817	34.238	40,53	44.0	rolled fill	-	-	-	-	-	-	-	-
Yamashinae	Japan	-	-	43	18.0	compacted homogeneous rolled fill	-	-	1914	-	-	-	-	-
Yarnell Delns	USA	-118.483	34.305	40,53	15.0	rolled fill	-	-	1909	-	-	-	-	clay
Yuba	USA	-	-	43	-	earthfill	-	-	-	-	-	-	-	-
Yucapa No. 1	USA	-117.053	34.050	48	13.0	earthfill	-	-	-	-	-	-	9	9m allm
Yucapa No. 2	USA	-117.052	34.053	48	15.0	earthfill	-	-	-	-	-	-	9	9m allm
Yunosawa	Japan	-	-	50	27.0	earthfill	-	-	1930	-	-	-	-	-
Yuzucik	Turkey	-	-	38	108.0	earthcore embankment	-	-	-	-	-	15	-	-

Dam Name	Earthquake Description																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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Alonso Almaden Almaden Valley Ambuklao Arcata Ascot Assuan High Austrian Aya Bai River Bay Street Bear Gulch Belvedere Berryman Binga Birusawa Birusawa Bouquet Canyon Bouquet Canyon Brand DB Brand Park Buena Vista Burlingame Calaveras Calero Canil Caripico Caripico Chatsworth Chatsworth #2 Cheshro Chevy Chase Cipreses Coastways Cogon Cogon Cogon Cogon Cogswell Cogswell Coburn Coleman Como Cowell Cowell Coyote Coyote Crocker Crystal Springs Upper Culmo Culmo Damonike Debell	Akita-Nantou	6.5	26-Oct-70	-	-	15.00	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Dam Name	Earthquake Description									
	Earthquake Name	Mag	Date	Location		Distance to Epicentre (km)		Focal Depth		Focal Distance (km)
				Long E	Lat N	by Author	by co-ordinate	(km)	by co-ordinate	
Deer Creek	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	-	-	-	-	-
Diary	Philippine / Luzon	7.7	16-Jul-90	121.170	15.680	-	-	25.0	-	-
Disdorch	San Fernando	6.5	9-Feb-71	-118.400	34.410	27.20	27.96	8.0	0.38	-
Doube	Tangshan	7.8	28-Jul-76	117.900	39.500	-	-	25.0	0.24	0.30
Drinkwater	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	67.20	70.15	-	0.4	-
Drinkwater	San Fernando	6.5	9-Feb-71	-118.400	34.410	14.40	17.48	8.0	0.13	-
Dry Canyon	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	70.00	72.50	-	0.43	0.48
Dry Canyon	San Fernando	6.5	9-Feb-71	-118.400	34.410	10.40	15.61	8.0	0.12	-
Eagle Rock	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	34.39	8.0	0.55	0.52
Earthburn	-	4.8	1839	-	-	-	-	-	0.20	0.22
Eaton Wash	San Fernando	6.5	9-Feb-71	-118.400	34.410	33.60	41.06	8.0	-	-
El Capitanero	Chilean	7.8	3-Mar-85	-71.700	-33.100	-	-	36.0	0.19	0.23
El Infernillo	Mexico	4.9	11-Oct-75	-102.030	18.330	79.20	-	63.0	-	-
El Infernillo	Mexico	5.9	15-Nov-75	-102.180	18.250	22.90	-	33.0	0.08	0.01
El Infernillo	Guerrero	7.6	14-Mar-79	-101.280	17.810	125.40	-	49.0	0.28	0.11
El Infernillo	Playa Azul	7.3	25-Oct-81	-102.080	18.050	55.30	-	28.0	0.12	-
El Infernillo	Michoacan Guerrero	8.1	19-Sep-85	-102.530	18.190	68.00	-	29.0	0.05	0.17
El Infernillo	Michoacan aft. shk	7.5	21-Sep-85	-101.650	17.800	75.00	-	42.0	0.13	0.23
Elmer J Chesbro	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	12.80	23.67	18.5	0.06	0.11
Elysian	San Fernando	6.5	9-Feb-71	-118.400	34.410	35.20	36.05	8.0	0.42	0.46
Emerald Lake #1	San Francisco	8.3	18-Apr-06	-122.480	37.670	2.20	20.69	20.0	0.18	0.21
Encina	San Fernando	6.5	9-Feb-71	-118.400	34.410	26.20	-	8.0	0.75	1.27
Estates	San Francisco	8.3	18-Apr-06	-122.480	37.670	28.80	32.58	20.0	0.24	0.32
Eureka	Eureka	6.6	21-Dec-54	-123.870	40.780	-	361.95	0.0	0.35	0.96
Farmont	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	57.60	71.94	-	-	-
Farmont	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	25.57	8.0	0.18	-
Fontenelle	-	-	-	-	-	-	-	-	0.20	0.33
Forest Lake	San Francisco	8.3	18-Apr-06	-122.480	37.670	44.80	112.87	20.0	-	-
Franklin Lower	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	29.00	8.0	0.25	0.56
Franklin Lower	Northridge	6.7	17-Jun-94	-118.540	34.210	-	69.35	18.0	0.20	0.28
Franklin Upper	San Fernando	6.5	9-Feb-71	-118.400	34.410	28.80	-	8.0	0.30	-
Fukutani	Hirogoken Nambu	7.2	17-Jun-95	-	-	33.00	-	14.0	0.23	0.29
Fukutani	Yamaguchi-hokubu	5.5	15-Jun-97	-	-	3.00	-	-	-	0.38
Gamanosawa	Tokachi-Oki	7.9	16-May-68	143.400	40.900	-	-	9.0	-	-
Garvey Reservoir	Whittier	6.1	1-Oct-87	-118.080	34.070	3.00	4.31	10.0	-	-
Glenoaks	San Fernando	6.5	9-Feb-71	-118.400	34.410	30.40	31.43	8.0	0.53	0.52
Gokee	Kosachi	7.8	17-Aug-99	29.860	40.750	55.00	-	17.0	0.22	0.25
Green Verdugo	San Fernando	6.5	9-Feb-71	-118.400	34.410	14.40	15.82	8.0	0.28	-
Gundulope	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	18.00	14.03	18.5	0.43	0.52
Hachi	-	-	1964	-	-	-	-	-	0.42	0.64
Hawee South	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	152.00	-	-	-	-
Hansen	San Fernando	6.5	9-Feb-71	-118.400	34.410	14.40	-	8.0	0.05	-
Harold	San Fernando	6.5	9-Feb-71	-118.400	34.410	26.40	33.49	8.0	0.43	0.55
Haselke	Nigata	7.5	16-Jun-64	139.100	38.300	-	-	57.0	0.25	0.23
Hawkins	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	34.00	63.88	18.5	-	0.08
Hayakakenuma	Tokachi-Oki	7.9	16-May-68	143.400	40.900	-	-	9.0	0.25	0.31
Helgen	Helgen Lake / Yellowstone	7.6	17-Aug-59	-111.210	44.710	16.00	-	5.0	-	-
Helen Valley	Various	7.7	26-May-83	139.100	40.600	230.00	-	24.0	0.71	1.31
Higashidaira	Nihookai-Chubu	7.7	26-May-83	139.100	40.600	230.00	-	24.0	-	-
Higland	San Fernando	6.5	9-Feb-71	-118.400	34.410	30.00	-	8.0	-	0.02
Hollywood Upper	San Fernando	6.5	9-Feb-71	-118.400	34.410	28.80	27.20	8.0	0.20	0.27
Hongou	Nihookai-Chubu	7.7	26-May-83	139.100	40.600	137.00	-	24.0	0.23	0.31
Hosotogi	Fukui	7.3	28-Jun-48	136.200	36.100	4.80	-	20.0	-	0.05
Howell Lower	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.20	67.10	20.0	0.80	0.85
Howell Lower	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	2.40	19.91	18.5	-	1.95
									0.43	0.52

Dam Name	Earthquake Name	Mag	Date	Location		Distance to Epicentre (km)		Focal Depth (km)	Focal Distance (km)		Crest Peak Accn	Foundn peak Accn	Esteva and Rosenbluth PGA (fraction of g)				
				Long E	Lat N	by Author	by co-ordinate		(km)	by Author			by co-ordinate	Fraction of g	(fraction of g)	by Author	by co-ordinate
Howell Upper	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.20	66.93	20.0	20	70	-	0.80	1.95	0.30			
Howell Upper	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	2.40	20.08	18.5	19	27	-	0.43	0.80	0.52			
Huechun	La Laguna	7.1	28-Mar-65	-71.100	-32.420	87.00	-	68.0	110	-	-	-	0.05	-			
Ichijoya	Tokachi-Oki	7.9	16-May-68	143.400	40.900	-	-	9.0	-	-	-	-	-	-			
Idenoshin	Hyogoken Nambu	7.2	17-Jan-95	-	-	-	-	14.0	-	-	-	-	-	-			
Iroka Ike	Nobi	8.4	10/28/1891	136.800	35.400	46.00	-	-	-	-	-	-	-	-			
Isabella	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	86.00	-	-	-	-	-	-	-	-			
Ivanhoe	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	-	8.0	33	-	0.20	0.25	-	-			
Iwakura Taneike	Oga	7	1-May-39	139.800	40.000	40.00	-	-	-	-	-	-	-	-			
Iwancke	Hyogoken Nambu	7.2	17-Jan-95	-	-	11.00	-	14.0	18	-	-	-	0.90	-			
Izumi	Hyogoken Nambu	7.2	17-Jan-95	-	-	-	-	14.0	-	-	-	-	-	-			
Izumakura	Hyogoken Nambu	7.2	17-Jan-95	-	-	-	-	14.0	-	-	-	-	-	-			
J.W.Wisla	San Fernando	6.5	9-Feb-71	-118.400	34.410	33.60	36.16	8.0	35	37	0.19	0.23	0.21	-			
Jen-I-Tan	Chi Chi	7.6	21-Sep-99	120.980	23.770	50.00	-	33.0	60	-	0.19	0.22	-	-			
Katari	Tokachi-Oki	7.9	16-May-68	143.400	40.900	-	-	9.0	-	-	-	-	-	-			
Kelly Cabin Canyon	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	29.00	50.84	18.5	34	54	0.20	0.38	0.18	-			
Kiseniyama	Gifu	6.7	9-Sep-69	-	-	-	-	-	-	-	-	-	-	-			
Kitayama	Hyogoken Nambu	7.2	17-Jan-95	-	-	31.00	-	14.0	34	-	-	-	0.42	-			
Koganezawa	Tokachi-Oki	7.9	16-May-68	142.000	41.500	234.00	-	33.0	236	-	-	-	0.02	-			
Kojoya	Hyogoken Nambu	7.2	17-Jan-95	-	-	48.00	-	14.0	50	-	-	-	0.22	-			
Konaka Taneike	Chiba-Tsushima	6.7	17-Dec-87	-	-	40.00	-	-	-	-	-	-	-	-			
Kremasta	-	6.3	5-Feb-66	-	-	-	-	-	-	-	-	-	-	-			
Kushikino	Kagoshima-Satsuma	6.2	3-May-97	-	-	28.00	-	-	-	-	-	-	-	-			
Kuzuryu	Gifu	6.7	9-Sep-69	-	-	40.00	-	-	-	-	-	-	-	-			
La Calera	Mexico	-	1-Jul-64	-	-	-	-	-	-	-	-	-	-	-			
La Marquesa	Chilean	7.8	3-Mar-85	-71.700	-33.100	45.00	-	36.0	58	-	0.60	0.28	-	-			
La Palma	Chilean	7.8	3-Mar-85	-71.700	-33.100	80.00	-	36.0	87	-	-	0.46	0.13	-			
La Villita	Mexico	4.9	11-Oct-75	-102.030	18.330	52.00	-	63.0	82	-	0.35	0.07	0.01	-			
La Villita	Mexico	5.9	15-Nov-75	-102.180	18.230	10.00	-	33.0	34	-	0.21	0.04	0.14	-			
La Villita	Guerrero	7.6	14-Mar-79	-101.280	17.810	121.00	-	49.0	131	-	0.40	0.02	0.05	-			
La Villita	Playa Azul	7.3	25-Oct-81	-102.080	18.050	31.00	-	28.0	42	-	0.43	0.09	0.33	-			
La Villita	Michoacan Guerrero	8.1	19-Sep-85	-102.530	18.190	58.00	-	29.0	65	-	0.79	0.12	0.29	-			
La Villita	Michoacan afc.skk	7.5	21-Sep-85	-101.650	17.800	61.00	-	42.0	74	-	0.21	0.04	0.14	-			
Laguna	El Centro / Imperial Valley	7.1	18-May-40	-115.500	32.700	67.00	-	16.0	69	-	-	-	0.12	-			
Lagunita	San Francisco	8.3	18-Apr-06	-122.480	37.670	6.40	-	20.0	21	-	-	0.65	1.85	-			
Lagunitas	San Francisco	8.3	18-Apr-06	-122.480	37.670	8.00	-	20.0	22	-	-	0.60	1.81	-			
Lahontan	Fallon	6.7	23-Aug-54	-118.450	39.580	48.00	69.37	-	-	-	-	-	-	-			
Lake Camille	San Francisco	8.3	18-Apr-06	-122.480	37.670	52.80	58.95	20.0	56	62	-	0.25	0.43	0.36			
Lake Chabot	San Francisco	8.3	18-Apr-06	-122.480	37.670	46.40	49.29	20.0	51	53	-	0.25	0.53	0.48			
Lake Frey	San Francisco	8.3	18-Apr-06	-122.480	37.670	50.20	63.28	20.0	62	66	-	0.20	0.36	0.32			
Lake Herman	San Francisco	8.3	18-Apr-06	-122.480	37.670	51.20	52.51	20.0	55	56	-	0.25	0.46	0.44			
Lake Mary Upper	Grand Canyon	5.5	29-Apr-93	-112.110	35.610	-	-	10.0	-	-	-	0.02	-	-			
Lake Raphael	San Francisco	8.3	18-Apr-06	-122.480	37.670	35.20	71.91	20.0	40	75	-	0.35	0.76	0.26			
Lake Ranch	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.10	62.06	20.0	20	65	-	0.80	1.95	0.34			
Lake Ranch	San Francisco	7.1	17-Oct-89	-121.880	37.040	6.40	24.94	18.5	20	31	-	0.43	0.76	0.44			
Lake Temescal	Loma Prieta	8.3	18-Apr-06	-122.480	37.670	29.00	31.94	20.0	35	38	-	0.35	0.95	0.86			
Lake Wohlford	Various	-	-	-	-	-	-	-	-	-	-	-	-	-			
Las Lajas	Northridge	6.7	17-Jan-94	-118.540	34.210	-	18.72	18.0	-	26	-	-	-	0.40			
Leroy Anderson	Morgan Hill	6.2	24-Apr-84	-121.700	37.320	16.00	16.21	8.0	18	18	0.63	0.41	0.40	0.40			
Leroy Anderson	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	26.90	30.04	18.5	33	35	0.43	0.23	0.41	0.36			

Dam Name	Earthquake Description													
	Earthquake Name	Mag	Date	Location		Distance to Epicentre (km)		Focal Depth (km)	Focal Distance (km)		Crest Peak Acren	Foundn peak Acren (fraction of g)	Esteve and Rosenbluth PGA (fraction of g)	
				Long E	Lat N	by Author	by co-ordinate		by Author	by co-ordinate			by Author	by co-ordinate
Lexington	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	21.00	18.60	18.5	28	26	0.45	0.45	0.50	0.55
Lan Tan	Chi Chi	7.6	21-Sep-99	120.980	23.770	-	-	33.0	-	-	-	-	-	-
Layan	Chi Chi	7.6	21-Sep-99	120.980	23.770	-	-	33.0	-	-	0.24	0.10	-	-
Liu-Liu	La Ligua	7.1	28-Mar-65	-71.100	-32.420	86.00	-	68.0	110	-	-	-	0.05	-
Liu-Liu	Papudo-Zapallar	7.5	8-Jul-71	-71.210	-32.510	87.00	-	40.0	96	-	-	-	0.09	-
Lo Ovalle	Papudo-Zapallar	7.5	8-Jul-71	-71.210	-32.510	96.00	-	40.0	104	-	-	-	0.07	-
Lo Ovalle	Chilean	7.8	3-Mar-85	-71.700	-33.100	-	-	36.0	-	-	-	-	-	-
Long Valley	Various	-	-	-	-	-	-	-	-	-	-	-	-	-
Lopez	San Fernando	6.5	9-Feb-71	-118.400	34.410	8.00	242.95	8.0	11	243	-	0.60	0.70	0.01
Los Angeles	Northridge	6.7	17-Jan-94	-118.540	34.210	10.00	10.07	18.0	21	21	0.56	0.32	0.53	0.53
Los Angeles North Dyke	Northridge	6.7	17-Jan-94	-118.540	34.210	10.00	-	18.0	21	-	0.65	0.42	0.53	-
Los Quilayes (La Marquesa?)	Chilean	7.8	3-Mar-85	-71.700	-33.100	-	-	36.0	-	-	-	-	-	-
Mackay	Borah Peak	7.3	28-Oct-83	-113.860	44.060	16.00	-	10.0	19	-	-	0.22	0.93	-
Magat	Philippine / Luzon	7.7	16-Jul-90	121.170	15.680	-	-	25.0	-	-	-	0.05	-	-
Makio	Nagano-Seibu	6.9	26-May-84	-	-	5.00	-	-	-	-	-	0.55	-	-
Makio	Nagasaki	6.8	14-Sep-84	141.500	34.010	5.00	-	47.0	47	-	-	0.57	0.18	-
Makomanai	Hokkaido Nanseioki	7.8	12-Jul-73	-	-	34.00	-	-	-	-	-	0.18	-	-
Malpasso	Peru	-	10-Oct-38	-	-	-	-	-	-	-	-	0.10	-	-
Malpasso	-	-	*1938*	-	-	-	-	-	-	-	-	-	-	-
Manno Ike	Ansei-Nankan	8.4	7/24/1854	135.600	33.200	166.00	-	-	-	-	-	-	-	-
Massey	Philippine / Luzon	7.7	16-Jul-90	121.170	15.680	19.00	-	25.0	31	-	-	0.65	0.70	-
Matania	Bay of Plenty	6.3	2-Mar-87	-	-	23.00	-	12.0	26	-	0.42	0.33	0.29	-
Miboto	Kitamino	7	19-Aug-61	136.700	36.000	17.00	-	-	-	-	-	0.15	-	-
Mill Creek	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	19.00	37.37	18.5	27	42	-	0.27	0.54	0.28
Minase	Oga	7	1-May-39	139.800	40.000	198.00	-	-	-	-	-	-	-	-
Minase	Nigata	7.5	16-Jun-64	139.100	38.300	147.00	-	57.0	158	-	-	0.08	0.03	-
Minogawa	Hyogoken Nambu	7.2	17-Jan-95	-	-	49.00	-	14.0	51	-	-	-	0.22	-
Mirairi	Kagoshima-Satsuna	6.3	6-Mar-97	-	-	7.00	-	-	-	-	-	-	-	-
Mount N. Basin Upper	San Francisco	8.3	18-Apr-66	-122.480	37.670	8.00	-	20.0	22	-	-	-	1.81	-
Mukamoto	Hyogoken Nambu	7.2	17-Jan-95	-	-	-	-	14.0	-	-	-	-	-	-
Murayama Lower (Shimo)	Kwanto	8.2	1-Sep-23	139.500	35.300	61.00	-	25.0	66	-	-	0.12	0.30	-
Murayama Upper (Kano)	Kwanto	8.2	1-Sep-23	139.500	35.300	62.00	-	25.0	67	-	-	0.12	0.30	-
Nabersanike	Hyogoken Nambu	7.2	17-Jan-95	-	-	12.00	-	14.0	18	-	-	-	0.87	-
Nagara	Chiba-Touhooski	6.7	17-Dec-87	-	-	29.00	-	-	-	-	0.38	0.27	-	-
Namoka	Nhonkar-Chubu	7.7	26-May-83	139.100	40.600	141.00	-	24.0	143	-	0.23	0.10	0.05	-
Newell Creek	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	18.40	22.09	18.5	26	29	-	0.43	0.55	0.49
Nimai-Bashi	Tokachi-Oki	7.9	16-May-68	143.400	40.900	-	-	9.0	-	-	-	-	-	-
Nirewakumine	Hokkaido Nansetsuki	7.8	2-Jul-73	-	-	71.00	-	-	-	-	-	-	-	-
Nomichi	Yamaguchi-Iokubu	6.1	15-Jun-97	-	-	4.00	-	-	-	-	-	-	-	-
Notre Dame	San Francisco	8.3	18-Apr-66	-122.480	37.670	3.20	23.98	20.0	20	31	-	0.75	1.93	1.13
Oak Site	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	15.00	33.21	18.5	24	38	-	0.35	0.62	0.32
Obarakke	Hyogoken Nambu	7.2	17-Jan-95	-	-	15.00	-	14.0	21	-	-	-	0.79	-
Ohike	Matsushiro	5.4	1965-1968	-	-	-	-	-	-	-	-	-	-	-
Obuzurumi	Nhonkar-Chubu	7.7	26-May-83	139.100	40.600	73.00	-	-	-	-	-	-	-	-
O'Neill	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	59.00	92.65	18.5	62	94	0.14	0.10	0.14	0.06
Oso	Kwanto	8.2	1-Sep-23	139.500	35.300	60.00	-	25.0	65	-	-	0.30	0.31	-
Orange County Reservoir	Whittier	6.1	1-Oct-87	-118.080	34.070	23.00	25.13	10.0	25	27	-	-	0.26	0.24
Oroville	Oroville	5.8	1-Aug-75	-121.530	39.440	12.00	9.92	8.8	15	13	0.20	0.10	0.34	0.37
Otani	Hyogoken Nambu	7.2	17-Jan-95	-	-	7.00	-	14.0	16	-	-	-	1.00	-

Dam Name	Earthquake Name	Mag	Date	Location		Distance to Epicentre (km)			Focal Depth		Focal Distance (km)		Crest Peak Accn	Founda peak Accn (fraction of g)	Esteva and Rosenbluth PG:A (fraction of g)	
				Long E	Lat N	by Author	by co-ordinate	(km)	by Author	by co-ordinate	Fraction of g	(fraction of g)			by Author	by co-ordinate
Oranike	Kwanto	-	1-Sep-23	139.500	35.300	-	-	25.0	-	-	-	-	-	-	-	-
Oranike	Nankai	7.2	21-Dec-46	135.600	33.000	80.00	-	30.0	85	-	-	-	-	-	0.08	-
Pablo	various	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Pacific Grove	San Francisco	8.3	18-Apr-06	-122.480	37.670	41.60	112.88	20.0	46	115	-	-	0.30	0.62	0.12	-
Pauho Man	Tangshan	7.8	28-Jul-76	117.900	39.500	150.00	-	23.0	152	-	0.14	0.05	0.04	-	-	-
Panabangan	Philippine / Luzon	7.7	16-Jul-90	121.170	15.680	20.00	-	25.0	32	-	-	-	0.65	0.68	-	-
Pebodly	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	19.00	41.52	18.5	27	45	-	-	0.25	0.54	0.24	-
Perales de Taphue	Chilean	7.8	3-Mar-85	-71.700	-33.100	-	-	36.0	-	-	-	-	-	-	-	-
Piedmont	San Francisco	8.3	18-Apr-06	-122.480	37.670	31.00	31.77	20.0	37	38	-	-	0.35	0.89	0.86	-
Pilarcitos	San Francisco	8.3	18-Apr-06	-122.480	37.670	3.00	12.49	20.0	20	24	-	-	0.75	1.93	1.63	-
Pirizates	Mexico	7.5	28-Jul-57	-99.100	17.110	-	-	-	-	-	-	-	0.05	-	-	-
Pitama	La Lajeta	7.1	28-Mar-65	-71.100	-32.420	107.00	-	68.0	127	-	-	-	-	0.04	-	-
Pitama	Papado-Zapallar	7.5	8-Jul-71	-71.210	-32.510	93.00	-	40.0	101	-	-	-	-	0.08	-	-
Poggio Cancelli	Gran Sasso	5.5	9-May-50	-	-	6.40	-	-	-	-	-	-	-	-	-	-
Port Costa	San Francisco	8.3	18-Apr-06	-122.480	37.670	44.80	-	20.0	49	-	-	-	0.25	0.56	-	-
Porter Estates	San Fernando	6.5	9-Feb-71	-118.400	34.410	16.00	22.53	8.0	18	24	-	-	0.40	0.51	0.38	-
Porter Estates	Northridge	6.7	17-Jan-94	-118.540	34.210	-	5.46	18.0	-	19	-	-	0.59	-	0.58	-
Recoleta	Illapel	7.9	6-Apr-43	-72.000	-30.750	83.00	-	55.0	100	-	-	-	-	0.11	-	-
Reservoir #1	San Fernando	6.5	9-Feb-71	-118.400	34.410	22.40	22.80	8.0	24	24	-	-	0.29	0.38	0.38	-
Rinconada	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	26.20	22.41	18.5	32	29	-	-	0.37	0.42	0.48	-
Robert Matthews	Perola	6.9	25-Apr-92	-124.320	40.370	-	98.85	15.0	-	100	-	-	0.07	-	0.05	-
Rogers	-	6.6	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Rotanike	Hyogo-ken Nambu	7.2	17-Jan-95	-	-	10.00	-	14.0	17	-	-	-	-	0.93	-	-
Rosenta	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	-	8.0	33	-	-	-	0.20	0.25	-	-
Rubio	Northridge	6.7	17-Jan-94	-118.540	34.210	-	47.17	18.0	-	50	-	-	-	0.15	-	-
Rubio Div. Debris	San Fernando	6.5	9-Feb-71	-118.400	34.410	32.00	37.94	8.0	33	39	-	-	-	0.25	0.19	-
Rungsee	Papado-Zapallar	7.5	8-Jul-71	-71.210	-32.510	94.00	-	40.0	102	-	-	-	-	0.08	-	-
Runkle	San Fernando	6.5	9-Feb-71	-118.400	34.410	28.80	39.81	8.0	30	41	-	-	0.23	0.29	0.18	-
San Andreas	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.00	10.97	20.0	20	23	-	-	0.80	1.95	1.69	-
San Andreas Old	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.00	-	20.0	20	-	-	-	-	1.95	-	-
San Antonio	Pomona Valley	5.5	28-Feb-90	-117.700	34.140	3.00	385.21	5.0	6	385	-	-	-	0.38	0.00	-
San Fernando Dyke	San Fernando	6.5	9-Feb-71	-118.400	34.410	10.40	-	8.0	13	-	-	-	0.55	0.65	-	-
San Fernando Lower	San Fernando	6.5	9-Feb-71	-118.400	34.410	11.20	14.05	13.0	17	19	0.55	0.48	0.53	0.48	-	-
San Fernando Lower	Northridge	6.7	17-Jan-94	-118.540	34.210	-	10.13	18.0	-	21	-	-	0.47	-	0.52	-
San Fernando Upper	San Fernando	6.5	9-Feb-71	-118.400	34.410	10.00	14.20	13.0	16	19	-	-	0.55	0.55	0.48	-
San Fernando Upper	Northridge	6.7	17-Jan-94	-118.540	34.210	-	10.09	18.0	-	21	-	-	0.42	-	0.53	-
San Justo	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	27.00	296.27	18.5	33	297	0.40	0.26	0.41	0.01	-	-
San Leandro	San Francisco	8.3	18-Apr-06	-122.480	37.670	31.00	-	20.0	37	-	-	-	0.35	0.89	-	-
San Leandro (Chabon)	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	59.00	-	18.5	62	-	-	-	0.10	0.14	-	-
San Luis	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	54.00	-	18.5	57	-	0.25	0.10	0.16	-	-	-
Sandalong	-	-	1975	-	-	-	-	-	-	-	-	-	-	-	-	-
Santa Felicia	San Fernando	6.5	9-Feb-71	-118.400	34.410	26.20	39.47	8.0	27	40	-	-	0.24	0.32	0.18	-
Santa Felicia	Northridge	6.7	17-Jan-94	-118.540	34.210	-	33.07	18.0	-	38	-	-	0.15	-	0.24	-
Saratoga	San Francisco	8.3	18-Apr-06	-122.480	37.670	0.10	-	20.0	20	-	-	-	-	1.95	-	-
Sausal	Chilean	7.8	3-Mar-85	-71.700	-33.100	-	-	36.0	-	-	-	-	-	-	-	-
Sawell	San Fernando	6.5	9-Feb-71	-118.400	34.410	33.60	-	8.0	35	-	-	-	0.19	0.23	-	-
Schubertanische	Nigata	7.5	16-Jun-64	139.100	38.300	99.00	-	57.0	114	-	-	-	-	0.66	-	-
Schwege No. 2	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	11.00	28.62	18.5	22	34	-	-	0.40	0.69	0.38	-

Dam Name	Earthquake Description													
	Earthquake Name	Mag	Date	Location		Distance to Epicentre (km)		Focal Depth (km)	Focal Distance (km)		Crest Peak Accn	Foundn peak Accn	Esteva and Rosenbluth PGA (fraction of g)	
				Long E	Lat N	by Author	by co-ordinate		by Author	by co-ordinate			by Author	by co-ordinate
Sempervires Sepulveda Sheffield	Loma Prieta San Fernando	7.1 6.5	17-Oct-89 9-Feb-71	-121.880 -118.400	37.040 34.410	19.00 24.80	38.76 -	18.5 8.0	27 26	43 -	- -	0.30 0.27	0.54 0.34	0.27 -
	Santa Barbara	6.3	29-Jun-25	-119.800	34.300	11.00	17.95	-	-	-	-	0.15	-	-
	Matsumoto	5.4	1965-1968	-	-	-	-	-	-	-	-	-	-	-
	Hyogoken Nambu Chi Chi	7.2 7.6	17-Jan-95 21-Sep-99	- 120.980	- 23.770	- 13.00	4.00 -	- -	14.0 33.0	15 35	-	-	-	1.06 0.54
Silver Lake St. Helena Lower St. Helena Upper St. Mary's	San Fernando	6.5	9-Feb-71	-118.400	34.410	33.60	-	8.0	35	-	-	0.19	0.23	-
	San Francisco	8.3	18-Apr-06	-122.480	37.670	51.20	-	20.0	55	-	-	0.25	0.46	-
	San Francisco	8.3	18-Apr-06	-122.480	37.670	51.20	-	20.0	55	-	-	0.25	0.46	-
	San Francisco Bay East	5.4	23-Oct-55	-122.050	37.970	-	-	-	-	-	-	-	-	-
Stemmer Stone Canyon Stone Canyon Upper Summit	Various	-	-	-	-	-	-	-	-	-	-	-	-	-
	San Fernando	6.5	9-Feb-71	-118.400	34.410	30.00	28.47	8.0	31	30	-	0.22	0.27	0.29
	San Fernando	6.5	9-Feb-71	-118.400	34.410	29.00	27.19	8.0	30	28	-	0.23	0.28	0.31
	San Francisco	8.3	18-Apr-06	-122.480	37.670	30.40	31.21	20.0	36	37	-	0.35	0.90	0.88
Surgo Sycamore Canyon Takamatsukushiroake Takinosawa	Malaya	5.9	5-May-86	37.810	37.990	10.00	-	10.0	14	-	-	-	0.38	-
	Northridge	6.7	17-Jan-94	-118.540	34.210	-	28.91	18.0	-	34	-	-	-	0.28
	Hyogoken Nambu	7.2	17-Jan-95	-	-	39.00	-	14.0	41	-	-	-	0.31	-
	Nigata	7.5	16-Jun-64	139.100	38.300	86.00	-	57.0	103	-	-	-	0.07	-
Taniyama Tanosawa Tanunzu Tatemichikamake	Hyogoken Nambu	7.2	17-Jan-95	-	-	7.00	-	14.0	16	-	-	1.00	-	-
	Tokachi-Oki	7.9	16-May-68	142.000	41.500	219.00	-	33.0	221	-	-	-	0.02	-
	Myagkan Oki	7.4	12-Jun-78	142.000	38.200	100.00	-	44.0	109	-	-	-	0.06	-
	Hyogoken Nambu	7.2	17-Jan-95	-	-	13.00	-	14.0	19	-	-	-	0.85	-
Tejon Ranch Tokwa Tokyo W.S. Toudou	Kern County / Arvin-Tehachapi	7.7	21-Jul-52	-119.020	35.000	-	-	18.00	-	-	-	0.60	-	-
	Hyogoken Nambu	7.2	17-Jan-95	-	-	10.00	-	14.0	17	-	-	-	0.93	-
	Kwanto	8.2	1-Sep-23	139.500	35.300	24.00	-	25.0	35	-	-	-	0.90	-
	Tangshan	7.8	28-Jul-76	117.900	39.500	-	-	23.0	-	-	-	-	-	-
Tsengsen Univ. Mound N Basin Ushino Uvas	Toudou	7.6	21-Sep-99	120.980	23.770	-	-	33.0	-	-	-	1.00	-	-
	Chi-Chi	5.3	14-Apr-76	120.750	23.220	-	-	33.0	-	-	-	0.16	-	-
	San Francisco	8.3	18-Apr-06	-122.480	37.670	-	9.08	20.0	-	22	-	0.60	-	1.77
	Ushino	7.4	12-Jun-78	142.000	38.200	125.00	-	44.0	133	-	-	-	0.04	-
Van Norman Bypass Vascona Vermilion Vir	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	9.00	21.29	18.5	21	28	-	0.45	0.73	0.50
	Northridge	6.7	17-Jan-94	-118.540	34.210	-	-	18.0	-	-	-	-	-	-
	Loma Prieta	7.1	17-Oct-89	-121.880	37.040	24.00	20.56	18.5	30	28	-	0.37	0.45	0.51
	Mammoth Lakes	6.3	27-May-80	-118.810	37.470	22.00	21.19	13.0	26	25	-	0.24	0.30	0.31
Volvano Lake Volvano Lake Washakie Wasuka	Mammoth Lakes	6.3	27-May-80	-118.810	37.470	22.00	21.19	13.0	26	25	-	0.24	0.30	0.31
	Koyna	6.5	11-Dec-67	73.900	17.700	8.00	-	33.0	34	-	-	-	0.24	-
	Mexico	5.3	22-Jun-15	-	-	0.00	-	-	-	-	-	-	-	-
	El Centro	7.1	18-May-40	-115.500	32.700	0.00	-	16.0	16	-	-	-	0.91	-
Whitner-Narrows Wide Canyon Wood Ranch Yamashiroake	Various	-	-	-	-	-	-	-	-	-	-	-	-	-
	Sorachichibou	5.6	23-May-96	-	-	9.00	-	33.0	34	-	-	-	0.11	-
	Whittier	6.1	1-Oct-87	-118.080	34.070	4.00	-	10.0	11	-	-	-	0.52	-
	Landers	7.5	28-Jun-92	-116.440	34.200	-	22.11	1.0	-	22	-	0.20	-	0.92
Yucapa No. 1 Yucapa No. 2 Yunoyawa Yuzack	San Fernando	6.5	9-Feb-71	-118.400	34.410	35.20	49.04	8.0	36	50	-	0.17	0.22	0.13
	Yamashiroake	6.4	1-Apr-75	-	-	-	-	-	-	-	-	-	-	-
	Yarnell Debris	6.5	9-Feb-71	-118.400	34.410	8.80	13.36	8.0	12	16	-	0.68	0.68	0.58
	Yuba	-	1-Jan-51	-	-	-	-	-	-	-	-	-	-	-
Yucapa No. 1 Yucapa No. 2 Yunoyawa Yuzack	Big Bear	6.6	28-Jun-92	-116.830	34.200	-	28.41	5.0	-	29	-	0.15	-	0.32
	Big Bear	6.6	28-Jun-92	-116.830	34.200	-	28.18	5.0	-	29	-	0.15	-	0.33
	Akita-Nantou	6.5	26-Oct-70	-	-	39.00	-	-	-	-	-	-	-	-
	Kocaeli	7.8	17-Aug-99	29.860	40.750	7.00	-	17.0	18	-	-	0.15	1.42	-

Dam Name	Damage Description												Our Damage Assessment	
	settlement						deformation			transverse cracks			Author's Damage Assessment	
	% dam height	% dam height and alluvial	Swagwood as % Dam and Alluvial (Given PGA)	Swagwood as % Dam and Alluvial (Calculated PGA)	Swagwood as % Dam and Alluvial (Calc PGA with Calc Epi Dist)		max length (m)	max width (mm)	max depth (m)	sliding	liquefaction	scarping		
Anisno	0.025	-	-	-	-	-	-	light cracks	-	-	-	-	0	not defined
Almuden	0.08	-	-	-	-	-	-	light cracks	-	-	-	-	1	minor
Almuden Valley	-	-	-	-	-	-	-	light cracks	-	-	-	-	0	not defined
Amhuklao	1.100	0.85	0.96	1.58	-	-	-	light cracks	-	-	-	scarping	2	major
Arcata	-	-	-	-	-	-	-	-	-	-	-	-	2	not defined
Ascor	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Aswan High	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined
Austrian	0.850	1.52	0.76	1.56	1.87	deformation	-	460	4.5	4.5 / 10	-	-	3	severe
Aya	0.200	0.20	0.83	-	-	-	-	-	-	-	-	-	1	minor
Bar River	-	-	-	-	-	-	-	-	-	sliding	-	-	4	serious
Bay Street	-	-	-	-	-	-	-	light cracks	-	-	-	-	1	not defined
Bear Gulch	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Belvedere	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Berryman	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Binga	-	-	-	-	-	deformation	100	300	-	trans cracks	-	-	2	moderate
Birusawa	-	-	-	-	-	-	-	-	-	-	-	-	0	slight
Birusawa	-	-	-	-	-	-	-	-	-	5	-	-	0	slight
Bouquet Canyon	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Bouquet Canyon	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Brand DB	-	-	-	-	-	-	-	-	1.25	-	-	-	-	not defined
Brand Park	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Buena Vista	0.750	12.50	-	-	-	-	60.00	light cracks	-	trans cracks	sliding	scarping	3	collapse
Burlingame	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Calaveras	-	-	-	-	-	-	-	-	-	-	-	scarping	0	slight
Calero	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Canli	0.040	0.06	-	-	-	-	-	"cracks"	-	"cracks"	-	-	1	minor
Capalco	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	not defined
Capalco	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	4	severe
Chatsworth	-	-	-	-	-	-	>800	75	3.5	-	-	scarping	1	minor
Chatsworth #2	0.070	0.67	-	-	-	deformation	-	75	3.5	-	-	scarping	0	not defined
Chesbro	0.110	0.38	-	-	-	-	73	100	-	-	sliding	scarping	2	moderate/major
Ches's Chase	-	-	-	-	-	-	-	-	-	-	-	-	-	major/moderate
Cipreses	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Coastways	-	-	-	-	-	-	-	minor	-	-	-	-	-	not defined
Cogoni	0.400	0.47	0.09	0.03	-	-	-	-	-	-	sliding	none	1	not defined
Cogoni	0.000	0.00	0.02	0.02	-	-	-	-	-	-	-	-	0	no or slight
Cogoni	0.000	0.00	0.03	0.03	-	-	-	light cracks	-	-	-	-	1	no or slight
Cogoni	0.000	0.00	0.03	0.05	-	-	-	-	-	-	-	-	0	no or slight
Cogwell	0.040	0.05	0.11	-	0.01	-	-	-	-	-	-	-	1	minor
Cogwell	0.020	0.02	0.02	0.02	0.02	-	-	-	-	-	-	-	1	no or slight
Colban	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	not defined
Coleman	-	-	-	-	-	deformation	-	-	-	-	-	scarping	5	failure
Como	-	-	-	-	-	-	-	-	-	-	-	-	1	minor
Cowell	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Cowell	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Coyote	0.080	0.19	0.66	0.07	0.07	deformation	3	300	2	-	-	-	1	minor
Coyote	-	-	0.07	0.44	0.11	-	-	-	-	-	-	-	0	not defined
Crocker	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Crystal Springs Upper	-	-	-	-	-	deformation	-	150	1.2	trans cracks	-	-	2	mod / min
Culmo	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	1	small
Culmo	-	-	-	-	-	-	-	light cracks	-	-	-	-	-	not defined
Damouke	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Debell	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined

Dam Name	Damage Description															Author's Damage Assessment		Our Damage Assessment
	settlement					deformation	longitudinal cracks			transverse cracks		sliding	liquefaction	seepage	No.			
	metres	% dam height	% dam height and alluvial	Swatgood as % Dam and Alluvial (Given PGA)	Swatgood as % Dam and Alluvial (Calculated PGA)		Swatgood as % Dam and Alluvial (Calc PGA with Cyclic Epist Dam)	max length (m)	max width (mm)	max depth (m)	max width (mm)					max depth (mm)		
Deer Creek	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	-	-	-	not defined	
Dayo	0.070	0.12	0.12	0.29	-	-	-	-	-	-	-	-	-	-	-	minor	1	
Desckerch	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Doube	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	serious	4	
Drinkwater	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Drinkwater	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Dry Canyon	0.080	0.40	0.20	0.44	-	-	-	50	5	trns cracks	-	sliding	-	-	seepage	mod / min	2	
Dry Canyon	-	-	-	2.93	5.42	2.46	-	-	-	-	-	-	-	-	-	none	0	
Eagle Rock	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Earlburn	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	failure	5	
Eaton Wash	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
El Capintero	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	major	3	
El Infernillo	0.040	0.03	0.03	0.00	0.02	-	-	-	-	-	-	-	-	-	-	none	0	
El Infernillo	0.020	0.01	0.01	0.01	0.04	-	-	-	-	-	-	-	-	-	-	none	0	
El Infernillo	0.130	0.09	0.09	0.03	0.10	-	-	35	-	trns cracks	-	-	-	-	-	moderate/minor	2	
El Infernillo	0.060	0.04	0.04	0.02	0.00	-	-	-	-	-	-	-	-	-	-	none	0	
El Infernillo	0.110	0.07	0.07	0.05	0.03	-	330	150	-	-	-	-	-	-	-	mod / min	2	
El Infernillo	-	-	-	0.02	0.01	-	-	long cracks	-	-	-	-	-	-	-	mod / min	0	
Elmer J Chesbro	0.110	0.38	-	-	-	-	100	100	2.4	25	1.4	-	-	-	-	moderate	2	
Elysian	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Emerald Lake #1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Encina	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Estates	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Eureka	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Farmont	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Farmont	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	seepage	-	
Farmont	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Fontenelle	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Forest Lake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	
Franklin Lower	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Franklin Lower	0.050	0.16	-	-	-	-	-	-	-	6	-	-	-	-	-	none	0	
Franklin Upper	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	1	
Fukarani	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	-	
Fukarani	-	-	-	-	-	-	-	-	-	trns cracks	-	-	-	-	-	moderate	1	
Fukarani	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	collapse	2	
Gamanosawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	5	
Garvey Reservoir	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Glenoaks	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0	
Gokee	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Green Verdugo	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Guadalupe	0.200	0.47	0.47	0.33	0.80	1.30	-	100	1.4	12	1.4	-	-	-	-	mod / min	2	
Hachi	-	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	-	-	major/moderate	3	
Hawee South	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1	
Hansen	0.020	0.07	0.03	0.31	-	-	210.00	25	2	"cracks"	-	-	-	-	-	none	0	
Harold	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Haskie	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	
Hawkins	-	-	-	-	-	-	-	-	-	minor	-	-	-	-	-	minor	1	
Hayakenuma	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	collapse	5	
Helgen	0.250	0.67	0.53	-	-	-	-	300	-	75	-	sliding	-	-	seepage	serious	4	
Helena Valley	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Higashidawawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	
Highland	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	-	
Hollywood Upper	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	
Hongou	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	
Hosotogi	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	collapse	5	
Howell Lower	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	set / mod	4	
Howell Lower	-	-	-	-	-	-	-	long cracks	-	-	-	-	-	-	-	minor	1	

Dam Name	Damage Description													Our Damage Assessment		
														Author's Damage Assessment		
	metres	*s dam height	*s dam height and alluvial	settlement	deformation	longitudinal cracks			transverse cracks			sliding	liquefaction	seepage	No.	No.
						max length (m)	max width (mm)	max depth (m)	max width (mm)	max depth (m)						
Howell Upper	-	-	-	-	-	-	long cracks	-	trans cracks	-	-	-	-	-	moderate	2
Howell Upper	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	-	-	none	0
Huehuen	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ichingoya	-	-	-	-	-	-	-	-	-	-	-	-	-	-	collapse	5
Idenoshin	-	-	-	-	-	-	-	-	-	-	-	-	liquefaction	-	failure	5
Iruka Ike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Isabella	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Ivanhoe	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Iwakura Tameike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Iwancake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Izumi	-	-	-	-	-	-	-	-	-	-	-	-	-	-	medium	2
Izumitaka	-	-	-	-	-	-	-	-	-	-	-	-	-	-	moderate	2
J.W.Wida	-	-	-	-	-	-	-	-	-	-	-	-	seepage	-	serious	4
Jen I-Tan	-	-	-	-	-	113	160	-	-	-	-	-	-	-	none	0
Katani	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1
Kelly Cabin Canyon	-	-	-	-	-	-	-	-	-	-	-	-	-	-	severe	4
Kiseniyama	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Kitayama	-	-	-	-	-	-	-	-	-	-	-	sliding	-	-	not defined	1
Koganezawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0
Kotija	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Konaka Tameike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Kremasta	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Kushikino	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
Kuzuryu	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	0
La Calera	-	-	-	-	-	-	-	-	-	-	-	sliding	-	-	not defined	-
La Marquesa	2.000	20.00	-	-	-	-	800	2	-	-	-	sliding	liquefaction	-	collapse	5
La Palma	-	-	-	-	-	60	1200	-	-	-	-	sliding	liquefaction	-	collapse	5
La Villita	-	-	0.01	0.36	-	-	-	-	-	-	-	-	-	-	none	0
La Villita	0.020	0.03	0.02	0.02	0.04	-	-	-	-	-	-	-	-	-	none	0
La Villita	0.045	0.08	0.03	0.06	0.01	-	long cracks	-	"cracks"	-	-	-	-	-	minor	1
La Villita	0.130	0.22	0.10	0.08	0.50	-	-	-	-	-	-	-	-	-	none	0
La Villita	0.300	0.50	0.23	0.17	0.13	80	50	1.5	trans cracks	-	-	-	-	-	minor	1
La Villita	0.120	0.20	0.09	0.07	0.08	-	-	-	-	-	-	-	-	-	minor	1
Laguna	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Minor	1
Lagunita	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lagunitas	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lahontan	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Camille	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Chabot	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Frey	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Herman	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Mary Upper	0.000	0.00	0.00	0.01	-	-	-	-	-	-	-	-	-	-	none	0
Lake Rapphine	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Ranch	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Ranch	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0
Lake Temescal	-	-	-	-	-	-	long cracks	-	trans cracks	-	-	-	-	-	minor	1
Lake Wohlford	-	-	-	-	-	-	long cracks	-	"cracks"	-	-	-	-	-	Minor	1
Las Lajas	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Leroy Anderson	0.020	0.03	0.03	0.11	0.10	-	long cracks	-	-	-	-	-	-	-	minor	1
Leroy Anderson	0.040	0.05	0.05	0.08	0.18	-	30	0.6	-	-	-	-	-	-	Minor	1

Dam Name	Damage Description														Our Damage Assessment	
	metres	settlement			deformation	longitudinal cracks			transverse cracks		sliding	liquefaction	seepage	Author's Damage Assessment		
		% dam height	Swagwood as % Dam and Alluvial (Given PGA)	Swagwood as % Dam and Alluvial (Calculated PGA)		Swagwood as % Dam and Alluvial (Calc PGA with Calc Epa Dss)	max length (m)	max width (mm)	max depth (m)	max width (mm)				max depth (m)	No.	No.
Lexington	0.260	0.42	0.34	0.48	0.63	-	20	2.3	38	2	-	-	-	minor	1	minor/moderate
Lian Tan	0.220	0.79	-	-	-	-	"cracking"	0.65	-	-	-	-	-	minor	1	major
Liyutan	0.090	0.09	-	-	-	-	-	-	trns cracks	2.1	-	-	seepage	minor	1	minor
Liau Liu	-	-	-	-	-	-	lngr cracks	-	-	-	-	-	-	-	-	not defined
Liu Liu	-	-	-	-	-	-	"cracks"	-	"cracks"	-	-	-	-	severe	4	not defined
Lo Ovalle	-	-	-	-	-	-	lngr cracks	-	-	-	-	-	-	-	-	not defined
Lo Ovalle	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined
Long Valley	-	-	-	-	-	-	-	-	-	-	-	-	-	major	3	not defined
Lopez	-	-	-	-	-	-	-	-	trns cracks	-	-	-	-	none - minor	1	not defined
Los Angeles	0.090	0.19	0.12	0.46	0.46	deformation	-	lngr cracks	-	1	-	-	-	none	0	not defined
Los Angeles North Dyke	0.030	0.08	-	-	-	-	-	lngr cracks	-	6	1.2	-	liquefaction	moderate	2	minor
Los Quillayes (La Marquesa?)	-	-	-	-	-	-	-	-	-	-	-	-	liquefaction	-	-	minor
Mackay	-	-	0.14	12.71	-	-	-	-	-	-	-	-	-	major	3	not defined
Magat	0.010	0.01	0.03	-	-	-	-	lngr cracks	-	-	-	-	seepage	minor	1	minor
Makio	-	0.47	0.62	-	-	-	-	-	-	-	-	-	-	none	0	no or slight
Makio	0.500	0.37	0.57	0.05	-	deformation	-	-	-	-	-	-	-	slight	0	major
Makomanai	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1	moderate
Malpasso	0.080	0.10	0.07	-	-	deformation	-	-	-	-	-	-	-	slight	0	not defined
Malpasso	0.032	0.04	0.03	-	-	deformation	-	-	-	-	-	-	-	minor	1	minor
Manno Ike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor
Masray	1.000	4.00	3.57	3.86	-	-	-	100	3.3	trns cracks	-	-	liquefaction	failure	5	not defined
Matuhina	0.120	0.14	-	-	-	deformation	-	lngr cracks	-	trns cracks	-	-	seepage	sev / mod / min	4	major / severe
Maboto	0.030	0.02	0.03	-	-	-	-	-	-	-	-	-	seepage	sev / mod / min	4	minor
Mill Creek	-	-	-	-	-	-	-	lngr cracks	-	-	-	-	-	Minor	1	no or slight
Minase	-	-	-	-	-	-	-	-	-	-	-	-	-	min / none	1	not defined
Minase	0.060	0.09	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Minogawa	-	-	-	-	-	-	-	-	-	-	-	-	-	Minor	1	minor
Mitazai	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Mount N. Basin Upper	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
Makumoto	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Murayama Lower (Shimo)	0.240	1.50	-	-	-	deformation	68	25	10	"cracks"	-	-	-	failure	5	not defined
Murayama Upper (Kuni)	0.180	0.75	0.48	1.47	-	deformation	18	125	-	"cracks"	-	-	none	mod / slight	2	minor/moder
Nabctanike	-	-	-	-	-	-	-	-	-	-	-	-	-	mod / slight	1	major
Nagata	0.020	0.04	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
Namoka	0.060	0.12	0.05	0.04	-	-	cracks	-	cracks	-	-	-	-	slight	0	minor
Newell Creek	0.030	0.05	-	-	-	-	-	225	3	-	-	-	seepage	slight / none	0	minor
Nimai Bashi	-	-	-	-	-	-	-	-	-	-	-	-	-	moderate	2	severe/minor
Niwakumne	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined
Nomichi	-	-	-	-	-	-	-	-	-	-	-	-	-	serious	4	not defined
Notre Dame	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
Oak Site	-	-	-	-	-	-	-	-	-	-	-	-	seepage	none	0	not defined
Obarakake	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Ohike	-	-	-	-	-	-	-	-	-	-	-	-	-	medium	2	not defined
Ohzushima	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
O'Neill	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
Ono	0.270	0.73	0.56	-	-	-	250	23.00	"cracks"	-	-	-	none	none	0	not defined
Orange County Reservoir	-	-	-	-	-	-	-	-	-	-	-	-	sliding	sev / maj / med	3	severe/major
Oroville	0.010	0.00	0.01	0.04	0.04	-	-	-	-	-	-	-	-	none	0	not defined
Orani	-	-	-	-	-	-	lngr cracks	-	-	-	-	-	-	min / none	0/1	no or slight
														minor / slight	1	not defined

Dam Name	Damage Description														Author's Damage Assessment		Our Damage Assessment	
	settlement				deformation	longitudinal cracks			transverse cracks		sliding	liquefaction	seepage			No.	No.	
	*s dam height	*s dam height and alluvial	Swanggood as *s Dam and Alluvial (Given PGA)	Swanggood as *s Dam and Alluvial (Calculated PGA)		Swanggood as *s Dam and Alluvial (Calc Epi Dst)	max length (m)	max width (mm)	max depth (mm)	max width (mm)								max depth (m)
Oranike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	moderate	2	not defined
O'ranike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	not defined
Pablo	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Pacific Grove	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Pailio Man	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	scr / mod	4	not defined
Pantabangan	0.270	0.25	-	-	-	-	-	-	-	-	-	-	-	-	-	mod / min	2	moderate
Peabody	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Pedrales de Tapihue	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	major	3	not defined
Piedmont	0.150	0.94	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1	major
Pilarcitos	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Pinzones	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Piutana	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	small	1	not defined
Piutana	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	small	1	not defined
Poggio Cancelli	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Port Costa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Porter Estates	-	-	0.33	0.67	0.29	-	-	-	-	-	-	-	-	-	-	none	0	moderate
Porter Estates	0.020	0.17	1.27	-	1.15	-	-	-	-	-	-	-	-	-	-	serious	4	minor
Recoleta	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1	not defined
Reservoir #1	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Rinconada	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Minor	1	not defined
Robert Matthews	0.003	0.01	0.03	-	0.03	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Rogers	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	failure	5	not defined
Roranike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	medium	2	not defined
Rowena	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Rubio	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5	0	not defined
Rubio Div. Debris	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	cracks"	0	not defined
Rumque	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	cracks"	0	not defined
Runkle	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
San Andreas	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	minor	1	moderate
San Andreas Old	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Minor	1	not defined
San Antonio	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	Minor	1	not defined
San Fernando Dyke	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	mod / min	2	not defined
San Fernando Lower	8.500	19.77	15.74	1.68	1.22	-	-	-	-	-	-	-	-	-	-	coll / scr / maj	5	collapse
San Fernando Lower	0.200	0.47	0.37	-	1.85	-	-	-	-	-	-	-	-	-	-	scr / maj / mod	4	major/moderate
San Fernando Upper	0.920	3.83	2.19	2.69	1.69	-	-	-	-	-	-	-	-	-	-	serious	4	severe
San Fernando Upper	0.440	1.83	1.05	-	2.61	-	-	-	-	-	-	-	-	-	-	scr / mod	3	major
San Justo	0.038	0.09	0.07	0.31	0.02	-	-	-	-	-	-	-	-	-	-	none	0	minor
San Leandro	0.120	0.32	0.28	6.22	-	-	-	-	-	-	-	-	-	-	-	min / none	1	moderate
San Leandro (Chabot)	-	-	-	0.20	-	-	-	-	-	-	-	-	-	-	-	none	0	minor
San Luis	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	no or slight
Sandaling	0.220	1.29	-	-	-	-	-	-	-	-	-	-	-	-	-	-	major	3
Santa Felicia	-	-	0.06	0.10	0.04	-	-	-	-	-	-	-	-	-	-	none	0	minor
Santa Felicia	0.200	0.33	0.33	0.07	0.07	-	-	-	-	-	-	-	-	-	-	minor	1	moderate
Saratoga	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	moderate	2	not defined
Sauzal	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined
Sawell	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined
Sekshibanancie	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	slight	0	not defined
Schvage No. 2	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	0	not defined

Dam Name	Damage Description															Author's Damage Assessment		Our Damage Assessment	
	metres	settlement				deformation	longitudinal cracks			transverse cracks		sliding	liquefaction	seepage	No.	No.			
		*s dam height	*s dam height and alluvial	Swaggood as *s Dam and Alluvial (Given PGA)	Swaggood as *s Dam and Alluvial (Calculated PGA)		Swaggood as *s Dam and Alluvial (Calc PGA with Calc Epi Den)	max length (m)	max width (mm)	max depth (m)	max width (mm)						max depth (m)		
Sempervirens	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Nepulveda	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Sheffield	-	-	-	-	-	-	-	-	-	-	-	-	liquefaction	-	-	failure	-		
Shionoiri	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	slight	-		
Shoshaike	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	-		
Shui Shui	0.130	0.43	-	-	-	-	-	-	-	-	-	-	-	seepage	1	not defined	-		
Silver Lake	-	-	-	-	-	-	-	-	0.5	-	-	-	-	-	1	moderate/major	3		
St. Helena Lower	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
St. Helena Upper	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
St. Mary's	-	-	-	-	-	-	-	-	1.5	-	-	-	-	-	-	moderate	2		
Steinaker	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	-		
Stone Canyon	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Stone Canyon Upper	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Summit	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	none	-		
Surge	0.150	0.27	-	-	-	-	150	200	3	-	-	-	-	-	0	not defined	-		
Sycamore Canyon	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	severe/moderate	3		
Takamatavahandiroke	-	-	-	-	-	-	-	-	-	2	0.1	-	-	-	-	not defined	-		
Takinosawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	slight	-		
Taniyama	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	slight	-		
Tanosawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	-		
Tarunizui	-	-	-	-	-	-	-	-	-	-	-	sliding	-	-	2	medium	-		
Tatemuchikamake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Teton Ranch	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	medium	-		
Tokawa	-	-	-	-	-	-	5	25	-	-	-	-	-	-	1	Min / none mod / slight	1		
Tokyo W.S.	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	-		
Tocho	-	-	-	-	-	-	-	-	-	-	-	-	-	-	4	not defined	-		
Toohio	0.300	-	-	-	-	-	-	-	-	-	-	-	-	-	1	serious	-		
Tsodih	0.040	0.03	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	-		
Tsengsen	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	-		
Univ. Mound N Basin	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	no or slight	0		
Ushino	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Uyas	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	slight	-		
Van Norman Bypass	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	-		
Vascona	0.050	0.50	0.29	0.49	0.83	-	-	18	2	3	0.15	-	-	-	1	minor/moderate	2		
Vermilion	0.053	0.11	0.04	0.14	0.21	-	-	-	-	-	-	-	-	-	-	not defined	-		
Vir	-	-	-	-	-	-	46	38	1.5	-	-	-	-	-	1	minor	1		
Volcano Lake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	moderate	3		
Volcano Lake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5	not defined	-		
Wahake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	5	collapse	-		
Wasuka	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Whitner Narrows	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Wilde Canyon	0.012	0.05	-	-	-	-	-	-	-	12	0.15	-	-	-	0	not defined	-		
Wood Ranch	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	minor	1		
Yamashirake	-	-	-	-	-	-	-	-	-	-	-	-	-	-	0	not defined	-		
Yarnell Dohrs	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	not defined	-		
Yuba	-	-	-	-	-	-	-	-	-	-	-	sliding	-	-	0	not defined	-		
Yucapa No. 1	0.010	0.08	0.05	0.08	0.23	-	-	-	-	-	-	-	-	-	1	minor	1		
Yucapa No. 2	0.005	0.03	0.02	0.07	0.22	-	-	-	-	-	-	-	-	-	1	minor	0		
Yunocawa	-	-	-	-	-	-	-	-	-	-	-	-	-	-	2	not defined	-		
Yuvack	0.100	0.09	-	-	-	-	-	-	-	-	-	-	-	-	0	medium	1		

APPENDIX B – Case Studies of Dam Cracking Under Seismic Loading

Austrian Dam

(Loma Prieta Quake, 17 October 1989)

Dam Details

Location	Lake Elsman, California, U.S.A.
Type	Homogenous Embankment
Height (m)	56 (184ft)
Crest Length (m)	213 (700ft)
Year Built	1951
Owner	San Jose Water Company
Foundation Geology	highly fractured, sandstone, conglomerate, shale.
Freeboard at Time of Earthquake (m)	~31
Design Freeboard (m)	4.6

Special Features of Dam

A concrete spillway with walls 8 metres in height runs to the right of the embankment, forming the right abutment at the crest for about 25 metres. A reinforced concrete outlet conduit of 1.2 metre diameter runs under the mid-left of the embankment, constructed in a trench excavated into the bedrock..

Earthquake Details

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	11.5	18.5	21.8
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.57g [*]	
Dam Damage Descriptor	Severe, class 4		

Special Features of Earthquake

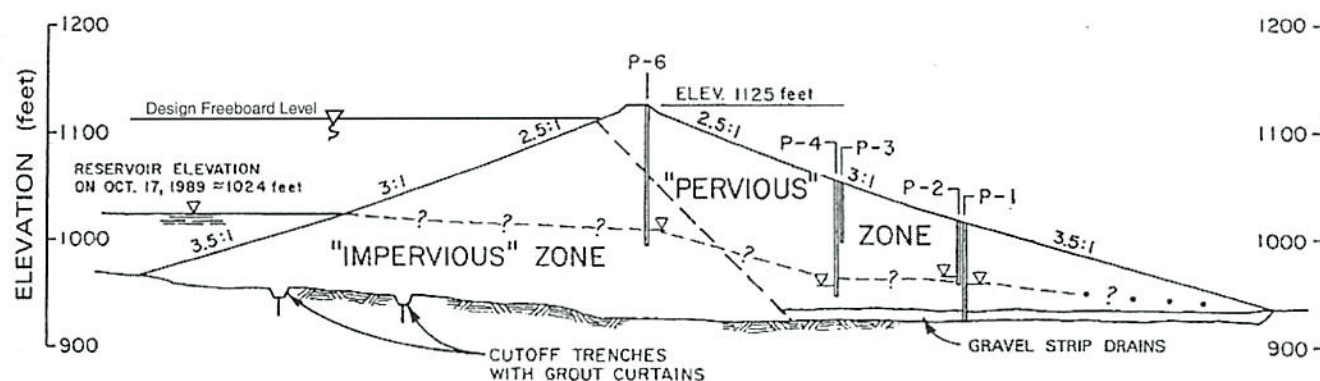
The duration of shaking was generally between 7 and 10 seconds, which is about half of that expected for an event of this magnitude (Harder et al 1996).

^{*} Swaisgood, 1995

Embankment Details

According to USCOLD (1992), the design of Austrian dam incorporated an upstream impervious section, downstream pervious section and gravel strip drains. However, during the excavation, placement and compaction, the materials broke down so that there was little appreciable difference between the zones. Furthermore, later investigations revealed that the strip drains had been rendered ineffective by misplacement of some waste materials. Therefore, the embankment can be considered to be a nearly homogeneous gravelly, clayey sand embankment.

Figure 1
Maximum Section of Austrian Dam
(Harder *et al*, 1991)



The properties of this material as they were at construction (from Harder *et al* 1991) are summarized below in Table A.

Table 1
Embankment Material Properties

Zone	Name	Description	Properties			Compaction and Placement			Wc %
			γ_d (kN/m ³)	c'	ϕ'	Thick- ness	Compaction	Density Ratio	
1	Homogeneous Embankment	Gravelly clayey sand (SC, GC, CL)	19.0- 20.5	0	44		'Rolled'	93% (of 20000 lb/ft ³)*	9.5- 19.5

* from Harder *et al* (1991). *Standard proctor* is based on 12375 ft.lb/ft³, *modified Proctor* is based on 56250 ft.lb/ft³.

As seen, the water level was more than 30 metres below the crest at the time of the earthquake. According to Harder *et al* (1991) the reservoir level had been low for 3 to 4 years prior to the earthquake, and therefore they consider that the upper embankment materials were not saturated at the time of the earthquake.

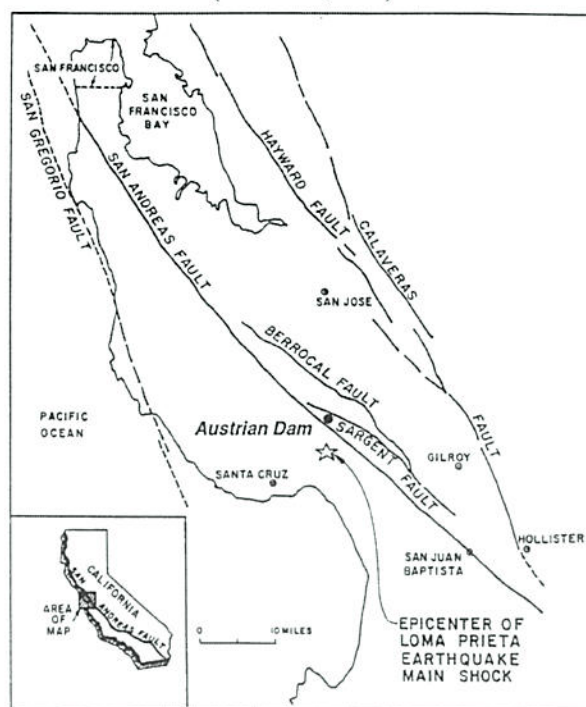
Foundation Geology / Materials

The dam site is situated at Latitude 37 7 9 and Longitude 121 55 8 between the Sargent and San Andreas Faults.

All soils and weathered rock were removed from embankment footprint before construction (ibid) and there is no alluvial layer present under the dam. Harder *et al* (1991) describes the founding rock as “typical of the Franciscan complex, including highly fractured sandstone, graywacke, cobble conglomerate, shale and serpentine”

No longitudinal section was found, but according to USCOLD, the dam is in a “V-shaped Canyon”. The quality of rock is said to be better on the right abutment than on the left abutment, where the material is described as ‘weathered, highly fractured rock’ (Babbitt, 1993).

Figure 2
Location of Austrian Dam
(USCOLD, 1992)



Effects From Earthquake

The Austrian Dam suffered the most damage out of all dams affected by the Loma Prieta earthquake. Damage included settlements (figure 3), deformations in the form of upstream / downstream movement (figure 4) and longitudinal and transverse cracking (figures 5 and 6). The concrete spillway was also severely damaged.

Monitoring of crest monuments showed that **settlements** of up to 850mm (2.7 feet) occurred over crest length (700ft) as a result of the earthquake (fig 3), with slightly greater movements towards the right end.

Figure 3
Vertical Movements of Crest Monuments

(Harder *et al* 1991)

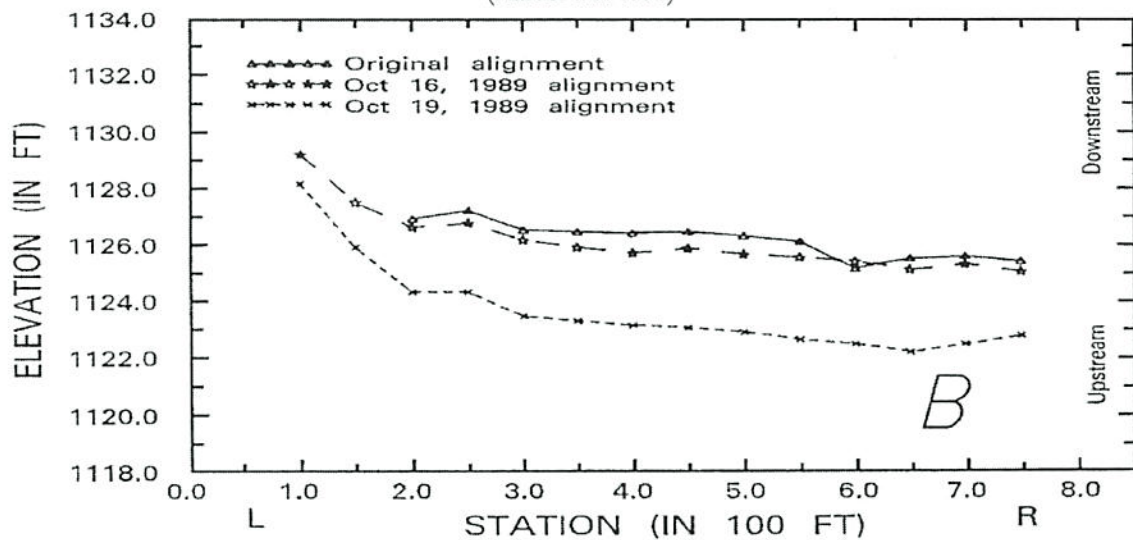
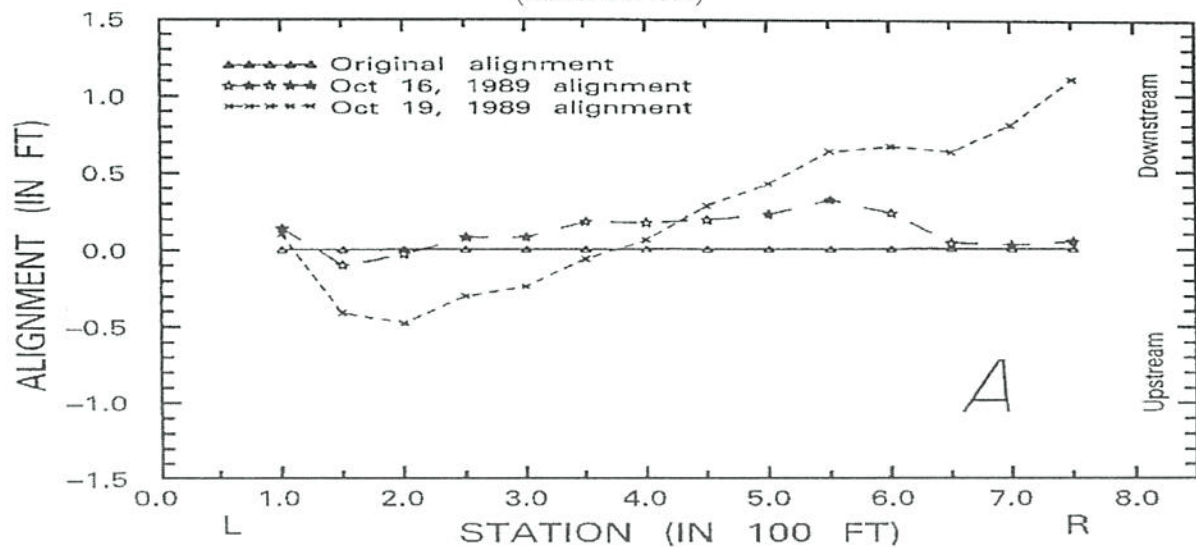


Figure 4
Horizontal Movements of Crest Monuments

(Harder *et al* 1991)

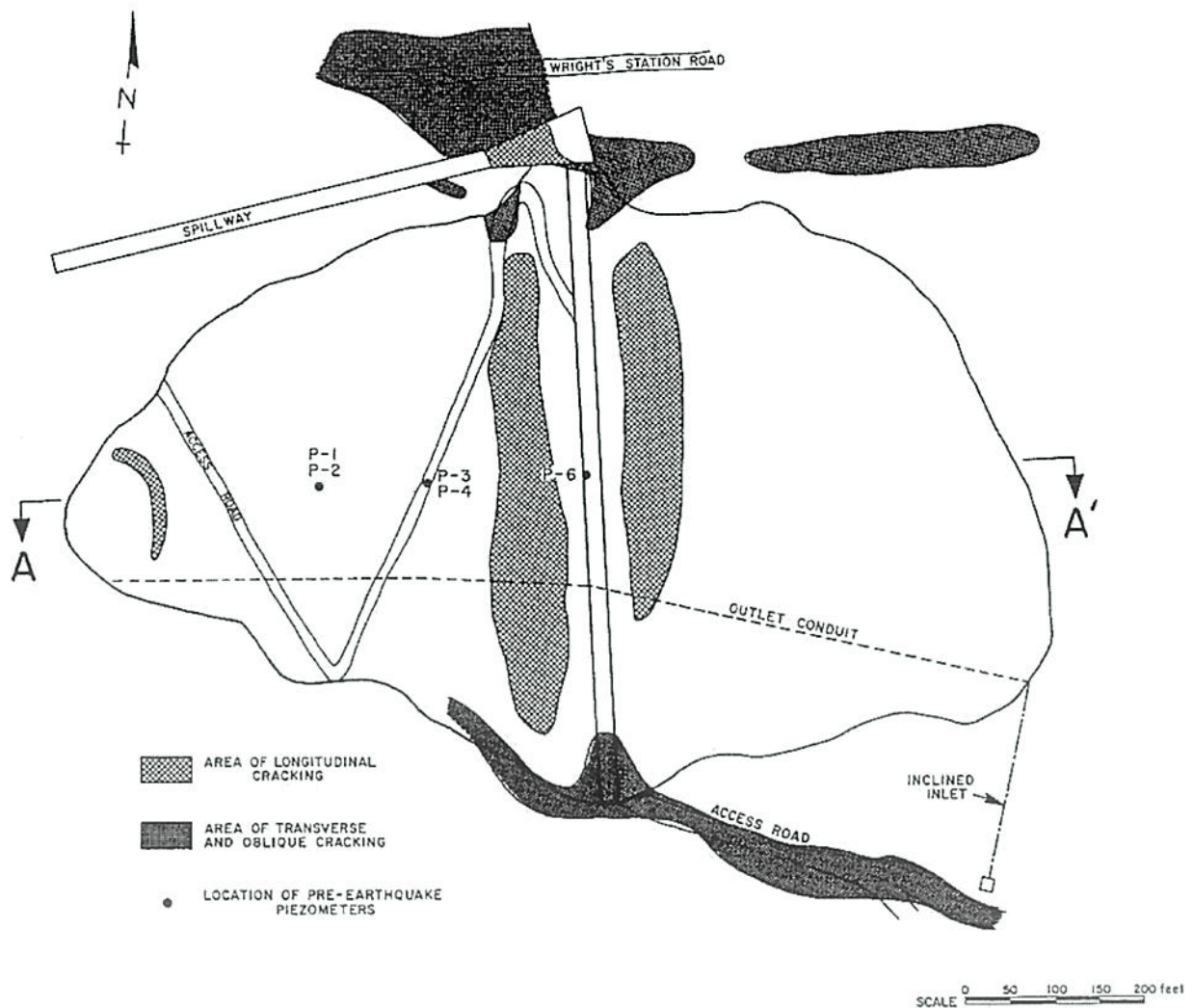


The embankment was **deformed laterally** under the quake loadings with the right end of the embankment

moving 370mm (1.2 ft) downstream, and the left moving 150mm (0.5 ft) upstream (See Figure 4). However, apparently the monuments were not tied to a stationary benchmark and so absolute displacement vectors could not be determined. Analysis of the dam suggests that horizontal movement was 'generally downstream'.

The locations of **transverse** and **longitudinal cracking** on the dam after the earthquake and shown on figure 5 (from Forster & MacDonald 1998) and figure 6 (from Harder et al 1991)

Figure 5
Plan of Austrian Dam Showing Regions of Cracking
(Harder *et al* 1991)



Harder et al (1991) describe a series of **longitudinal cracks** on the upstream and downstream faces on the embankment, with lengths up to 120 metres (labelled "1" on figure 6). They state that initially these

cracks were 'relatively small', but widened over the following 'few weeks' to 'as much as a foot'. Explanations cited for this growth in width include: the consequence of aftershocks, consolidation, rainfall events after the earthquakes, soil creep, and pore pressure dissipation. Longitudinal cracks were described as being more pronounced within the upper 25% of both upstream and downstream faces. According to Harder *et al* (1991) at seven days after the earthquake, four main longitudinal cracks existed on the upstream face within 15 metres of the crest that were 25 to 100mm wide and 1.5 to 4.5 m deep. On the downstream face, assorted longitudinal cracks ranged from 1 to 2.5 metres deep, and 50 to 150 mm wide. Forster & MacDonald (1998) describe longitudinal cracks up to 4.5 metres deep and 460 mm wide in these upper regions, and the lower portions of these slopes as having 'widespread' but 'shallow' and 'hairline' longitudinal cracks. Some cracking occurred near the toe of downstream face which Forster and MacDonald (1998) recorded as being up to 50mm wide (labelled "2" on figure 6).

Explorations indicated that these upper longitudinal cracks dipped steeply toward the crest - which, to Harder *et al* (1991), suggested that cracking was the result of 'settlement and re-arrangement of the earth embankment' rather than from slope instability. Harder *et al* (1991) also noted that the locations of deformations in the standpipes themselves suggested internal movement from lateral spreading, and in summary proposed that embankment movements were 'due to general settlement and spreading of the fill during shaking, followed by downstream creep'. Rodda *et al* (1990) also reported longitudinal cracking at the left abutment related to a line of buried grouting pipes left after construction.

Around the abutments, there was extensive **transverse cracking** through the crest. At the left abutment (labelled '3' on figure 6) most transverse cracks were up to 250mm wide and 3.1 metres deep (Forster & MacDonald, 1998). However, at the contact with the abutment, a vertical scarp of 400mm was accompanied by a separation crack which explorations traced to depths of 8.2 metres, ranging from 3mm to 500mm wide. The explorations suggested the cracks were within old landslide deposits on the upper part of the abutment (*ibid*). It was considered possible that separations existed at even greater depths, and so further grouting / examination programs were planned (Harder *et al* 1991). The papers do not give further mention to these explorations.

At the right abutment, there was also extensive transverse cracking (labelled "4" on figure 6). According to most reports, rocking of the 8 metre spillway walls under earthquake effects resulted in the formation of separation cracks between the concrete walls and the earthfill / right abutment. These cracks were as wide as 250mm, tapering to a depth of 7 metres. These separations would have been of great concern if the dam was full, as the design freeboard is only 4.6 metres (Harder *et al* 1991). Photo 1, from the U.S. Geological Survey is of cracking at the right abutment. The caption for the photo revealed that the cracks shown were considered to be "extensional", in that the fill "settled and pulled away from the concrete spillway". This explanation is different from the above, but it is unclear if they are referring to the same crack system. Nonetheless, the latter explanation is consistent with measurements of embankment settlements (figure 3), and also with incidence of transverse cracking in other case studies. It is likely that both mechanisms could have been contributors to the cracking. Indeed, of this region Babbitt (1993) writes "the separation was apparently due to a soil structure interaction, embankment settlement ... and permanent wall deflection".

Figure 6
Plan of Austrian Dam Showing Cracking
(Forster and MacDonald, 1998)

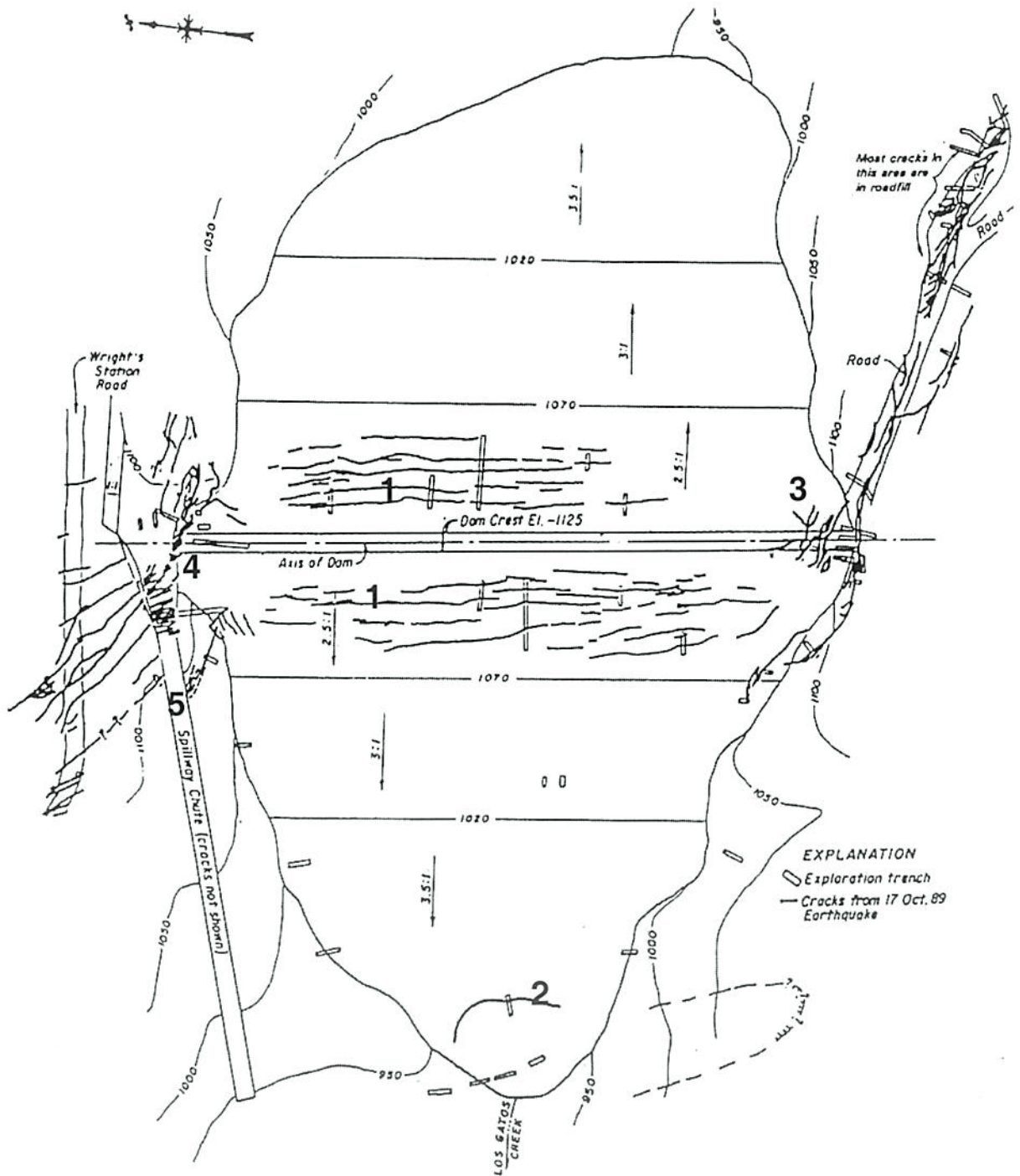
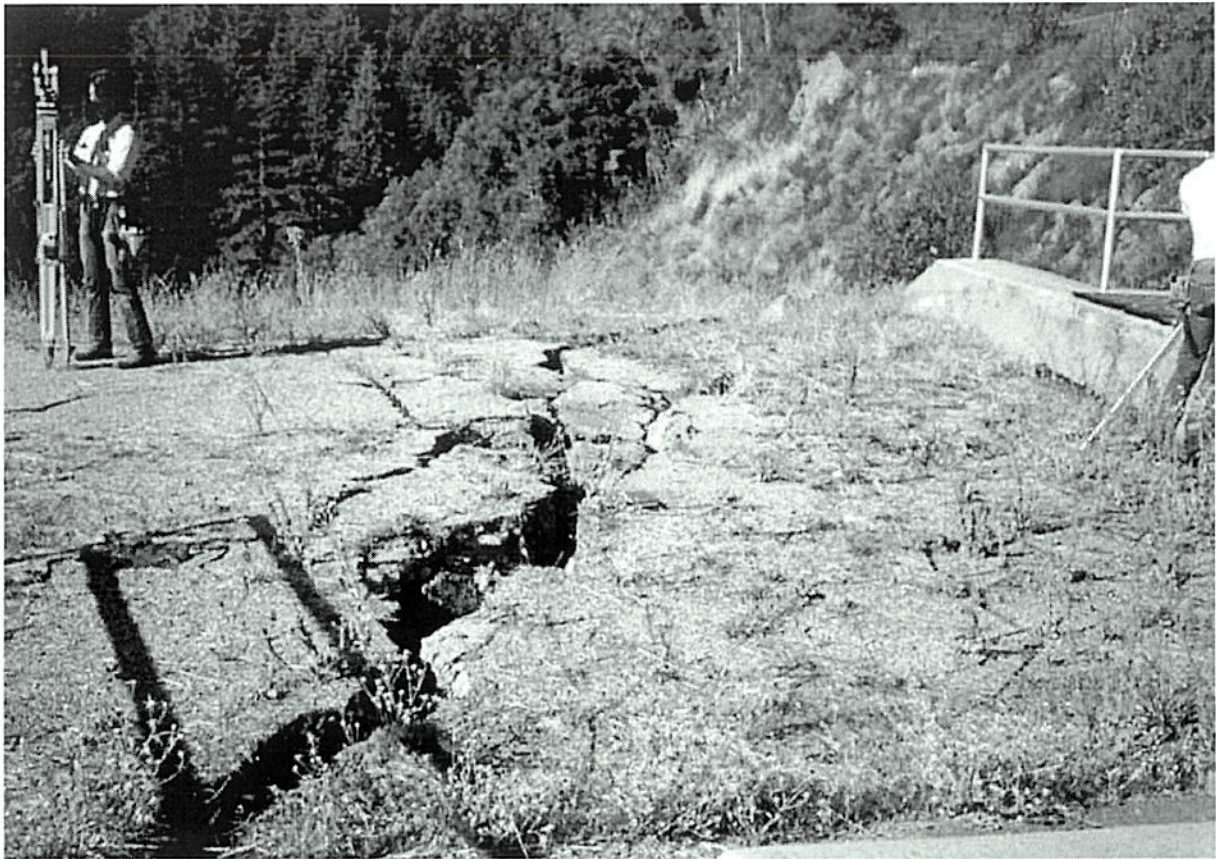


Photo 1
Transverse Cracking of Austrian Dam at the Right Abutment
(U.S.G.S)



Other contributions to the cracking could also be considered. Firstly, in a general sense, cracking in the embankment may have been exacerbated by the ‘unsaturated’ condition of the embankment, considering reservoir levels had been low for the prior 3-4 years (Harder *et al* 1991). Secondly, with the right abutment (spillway side) moving nearly 400 mm downstream, and the left abutment about 150 mm upstream, transverse deformations of over 500 mm need to be accommodated by the embankment. Thirdly, it is interesting to note how well aligned the transverse cracking pattern in the embankment is with cracking over the regions outside of the embankment (see figure 6). These areas correspond with areas considered by Harder *et al* (1991) when they write of significant ground movements in the vicinity of the embankment including “the downstream right abutment above the spillway chute” and “the upstream left abutment between the dam crest and the inclined inlet”. These areas correspond with areas of cracking on figures 5 and 6, which align themselves somewhat with the cracking on the embankment abutments. Of the right abutment, Harder *et al* (1991) describe cracking through bedrock materials and movements ‘coincident with topographic features associated with landsliding’. Of the left abutment, some 120 metres of cracking adjacent to the embankment, and parallel to the slope, was interpreted, after exploratory trenching, ‘to be the expression of a shallow landslide in loose material ...’. Lastly, Babbitt

(1993) mentions that regions of low density embankment were placed over poor rock at the abutments in a hurried effort to increase the dams protection against overtopping during a 1950 rainy season. He states that settlement and cracking at these locations are partially due to this.

The spillway suffered severe damages (labelled “5” on figure 6). Tensions cracks formed perpendicular to the spillway axis through out the length of the spillway, such that it was elongated by over 300 mm (Forster and MacDonald, 1998; and Harder, 1991). Photo (2) shows as cracked section of the spillway – with cracks up to 120 mm wide (U.S. Geological Survey). According to Forster and MacDonald (1998) it was damage to and around the spillway, which gave evidence that the spillway experienced side to side rocking under the earthquake loading. Explorations found areas under the left wall where rocking had produced separations of up to 25 mm between the base of the spillway and the embankment material – also of great concern had the reservoir been full. The movement of the spillway also resulted in several cutoff collars being sheared off.

Photo 2
Cracking of Spillway Walls at Austrian Dam
(U.S.G.S)



There was a general rise in water level in the embankment after the earthquake. A piezometers in the centre of the embankment (P-6 on figure 1) showed a water level rise of over 15 metres, and another tapping into the downstream toe foundation (P-1 on figure 1) showed a rise of over 17 metres (Forster and MacDonald, 1998). (These piezometers were both of the ‘Casagrande’ type, having their influence zones confined at depths). The other piezometers showed little or no increase. Trench excavations around the cracked regions reportedly found ‘considerable free water, typically occurring within the more

granular, gravelly layers of the fill ..” (from Rodda and Harlan, 1990, as quoted in Forster and MacDonald, 1998). Harder *et al* mention this as ‘possibly related’. The elevated water levels were reported to tend toward ‘normal’ levels by less than two weeks after the quake (Forster and MacDonald, 1998) (or by 2 months Rodda *et al* 1990).

The cause of water rise is debatable. Foster and MacDonald (1998), while acknowledging speculation, suggest it could be due to high pore pressures under earthquake induced consolidation of the embankment, dispersing over the following weeks. That the water is residual, from previous high reservoir levels seems unlikely as it does not explain the 17 metre ‘raise’. Furthermore, as said above, Harder *et al* (1991) state that the reservoir level had been low for 3 to 4 years prior to the earthquake.

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Chabot (Lower San Leandro) Dam

(San Francisco Quake, 19 April 1906, Loma Prieta Quake, 17 October, 1989)

Dam Details

Location	California, U.S.A
Type	Rolled Earthfill and Hydraulic Fill
Height (m)	38 / 43
Crest Length (m)	152
Year Built	1892
Owner	-
Foundation Geology	5 metres gravelly sandy clay over bedrock
Design Freeboard (m)	-

Earthquake Details

Earthquake No. 1

Name	San Francisco		
Date	19 April 1906		
Magnitude (M_s)	7.8 M_s (USGS Database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	31 [*]	20 [♥]	37
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.35-0.4g [*]	
Freeboard at Time of Earthquake (m)	3.3		
Dam Damage Descriptor	Minor / Moderate, Class 1/2		

Earthquake No. 2

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1 M_s (USGS Database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	59 [^]	18.5	62
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.1g [^]	
Freeboard at Time of Earthquake (m)	3.3		
Dam Damage Descriptor	Undefined, but authors say no damage		

^{*} as presented in Ambraseys, 1960. Note, USCOLD 1992 gives 37 and Harder *et al* 1992 give 27

[♥] from USGS database

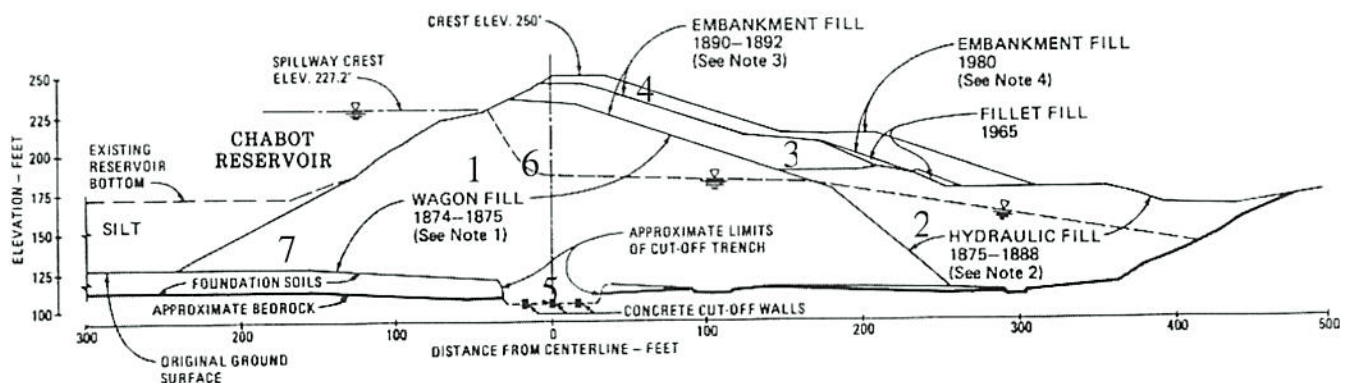
^{*} estimations presented in Seed *et al* 1978 (0.35) and ICOLD 1999 (0.4)

[^] presented in Harder *et al* 1992

Embankment Details

Construction began in 1874 and underwent continuing modifications until its completion in 1892 (ICOLD, 1999). The embankment has had one further modification since then (1984), when a spillway was relocated and the embankment raised. The figure below (from USCOLD, 1988) shows a section of the embankment.

Figure 1
Maximum Section of Chabot Dam
(Fong and Bennett, 1995)



Item	Unit	Description
1	Wagon Fill	Clayey sand with fine gravel size rock fragments and pockets of silty sandy clay.
2	Hydraulic Fill	Lean Sandy Silty Clay with Gravel and Zones of Gravelly Silty Sands and Gravelly Clayey Sands
3	Embankment Fill	Consists of Hydraulic Fill below elevation 222' and wagon fill above
4	Embankment Fill	Consists of material selected from the replacement of spillway excavation
5	Cutoff Trench	3 to 9 metres deep.
6	Phreatic Surface	From E.B.M.U.D records from 1964 to 1972

Construction began with stripping the area under the embankments footprint to a depth of 1 meter. A cutoff / core trench was excavated into bedrock to a depth of 3 to 9 meters (5). The main body of 'wagon fill' (1) was constructed between 1874 and 1875 as a rolled fill structure using horse drawn equipment for transporting and compacting fill material. Although designs called for select material to be placed in the core region, subsequent explorations have showed that the fill can be characterised as an homogeneous mass of predominantly silty sandy clay with clayey sand and gravel.

The wagon fill was reinforced between 1875 and 1888 by placing an hydraulic fill buttress (2) against the downstream slope. Between 1891 and 1892, further modifications included raising the downstream buttress (3), and creating a berm against the upstream face at the location where a slide had occurred during construction (6). In 1984, the embankment was raised (4), using material from excavations for a new spillway.

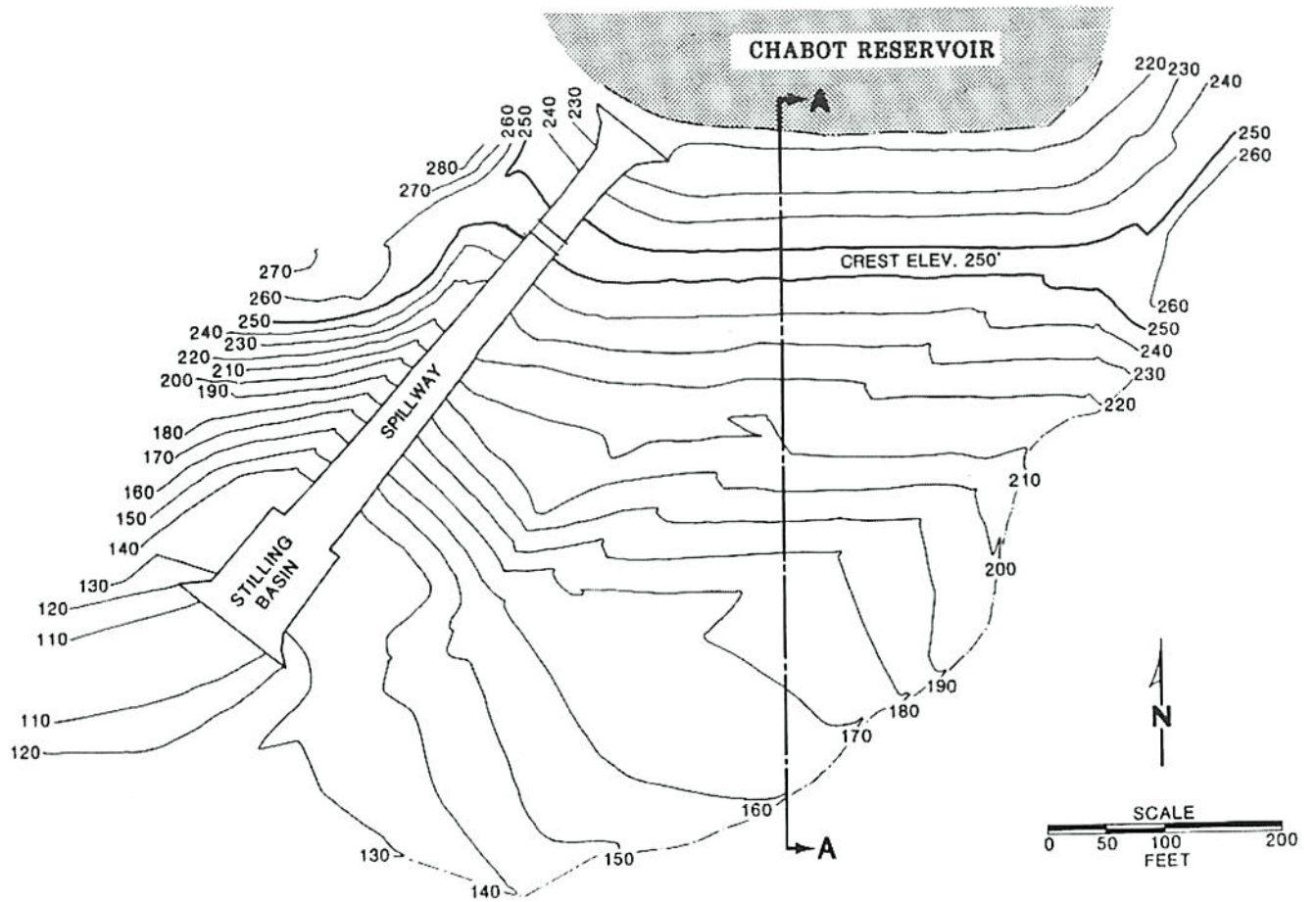


Figure 1 a and b Chabot Dam, Plan and Section from USCOLD 1988



Foundation Geology / Materials

Figure XXX shows the dam is founded upon rock, which Ambraseys described as “tertiary formations of the Leona rhyolite type” (1960). However, the upstream toe of the dam is also founded upon alluvial and residual soils to a depth of 3 to 10 meters (ICOLD, 1999).

Effects from Earthquake

The 1906 San Francisco Earthquake

The earthquake is commonly described as having a magnitude of 8.1 (Seed *et al* 1978; Ambraseys, 1960). However the USGS database gives a value of 7.8 M_s .

Of the 1906 earthquake, Ambraseys said ‘the earthquake did not cause any damage, the intensity of the shock at the site being very low’. However, authors estimate between 0.35 and 0.4 g ground acceleration (USCOLD, 1992; Seed *et al* 1978), and ICOLD (1999) states that the duration of shaking may have been as long as 50 seconds. These loadings have been proved to be an adequate to cause significant damage to other dams. For example, the figures 8 and 10 in the main report show that under these seismic loadings, damage of class 3 to 4 should be expected. Furthermore, an Esteva and Rosenblueth (1969) estimation based on a focal distance of 37 km and magnitude of 7.8 predicts a peak ground acceleration that are even greater - 0.59g.

Seed, Makdisi and De Alba (1978) also state that no damage was reported. However, ICOLD (1999) reports that subsequent studies by Makdisi, Seed and Idriss (1978) found that the Chabot dam may have settled by 100mm. Despite this, no mention of cracking is given.

It is concluded that the performance of the Chabot dam during the 1906 earthquake was particularly good when compared to other case studies. Reasons for this excellent performance was examined by Seed *et al* (1978). This study proposed that the excellent performance could be attributed to the fact that the dams were in most cases constructed from clayey soils of low to medium plasticity.

The 1989 Loma Prieta Earthquake

Harder *et al* (1991) estimate that Chabot dam was subjected to peak ground accelerations of 0.1g from the Loma Prieta earthquake, and suffered ‘no significant damage’. They state that the period of shaking was much less for this earthquake than for the 1906 event, thus it is not surprising that it suffered no significant damage. However, it is useful to confirm the performance of these dams, seeing the performance during the reasonably remote 1906 event was extremely good, when compared to other case studies.

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Cogoti Dam

(Illapel, 6th April 1943, La Ligua, 28th March 1965; Papudo-Zapallar, 8th July 1971; Llollea-Algarrobo, 3rd March 1985)

Dam Details

Location	Chile
Type	Concrete Face Rockfill
Height (m)	85 (280ft)
Crest Length (m)	160 (526ft)
Year Built	1939
Owner	
Foundation Geology	andesitic breccia
Design Freeboard (m)	7.7 (normal), 1.7 (max)

Special Features of Dam

The dam is used primarily for irrigation. An ungated spillway is excavated into the left abutment rock.

Earthquake Details

Earthquake No. 1

Name	Illapel		
Date	6 April 1943		
Magnitude (M_s)	7.9 (8.2 Mw)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	89	55	105
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.55 [‡]	0.19 [*]	
Freeboard at Time of Earthquake (m)	7.7		
Dam Damage Descriptor	Moderate, class 2		

Special Features of Earthquake

Apparently few records are available which describe the earthquake, and none of these are of a technical nature (ICOLD, 1999).

Earthquake No. 2

Name	La Ligua		
Date	28 March 1965		
Magnitude (M_s)	7.4 [♦] (M_s)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	95	61	113
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.04 [*]	
Freeboard at Time of Earthquake (m)	-		
Dam Damage Descriptor	No or slight, class 0		

[‡] Estimation given in Jansen Performance of Cogoti Dam Under Seismic Loading

^{*} An estimation given in Arrau et al 1985.

^{*} from USGS database

Earthquake No. 3

Name	Papudo-Zapallar		
Date	8 July 1971		
Magnitude (M_s)	7.5 (M_s)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	168	40	173
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.05 [*]	
Freeboard at Time of Earthquake (m)	-		
Dam Damage Descriptor	- No or slight to minor, class 0/1		

Earthquake No. 4

Name	Llollea-Algarrobo		
Date	3 March 1985		
Magnitude (M_s)	7.8 [♦] (M_s)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	174	9	174
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.03 [*]	
Freeboard at Time of Earthquake (m)	-		
Dam Damage Descriptor	- No or slight, class 0		

^{*} An estimation given in Arrau et al 1985.

[♦] from USGS database

Embankment Details

To most, the fated voyage of the ‘unsinkable’ Titanic (1912) resides as an uneasy reminder of the necessity for engineering humility. Notwithstanding this, details of the Cogoti Dam embankment are given in a short paper titled “Earthquake-Proof Dam in Chile” presented in the Engineering News Record of 1931 (Vol 107, pg 725). The design of Cogoti Dam was described as ‘earthquake resisting’ on the basis of two design principles: that the rockfill ‘would not flatten further under shock’, and that the concrete face would be constructed such that it “will flex without rupture”. Figure 1 shows the sections of Cogoti Dam.

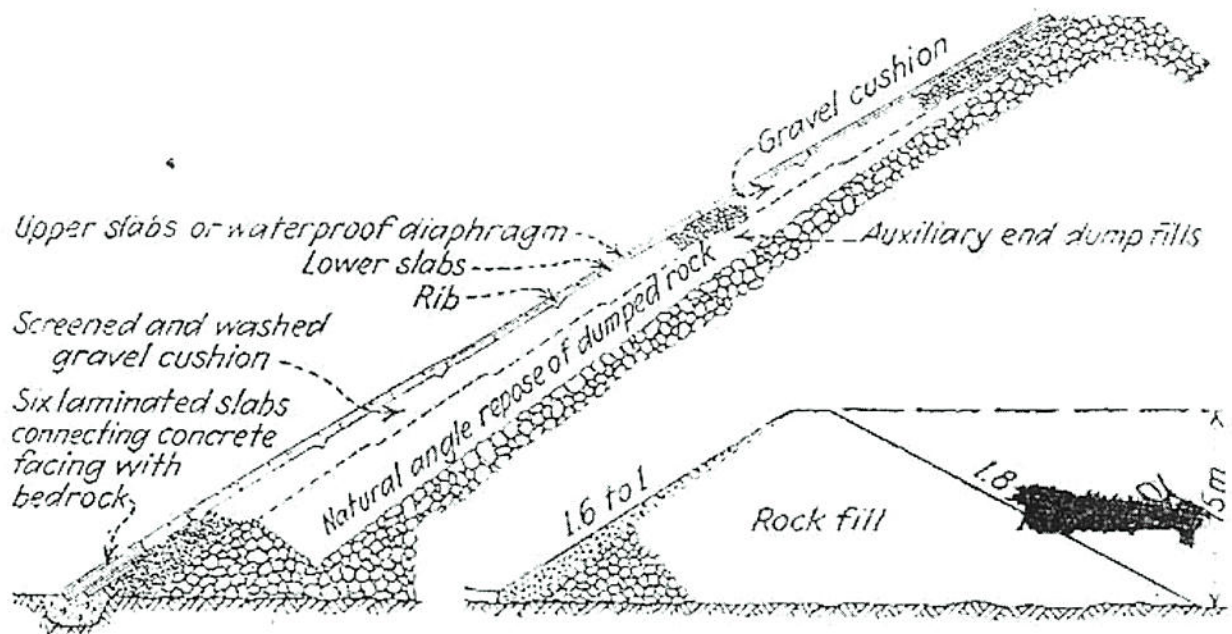


Figure 1

The dam fill was constructed using local rock, which is described as being ‘andesitic breccia’. The construction was started with blasting of the abutments. By allowing the blast spoil to fall on the foundation, the first ‘lifts’ of the main foundation were formed. Once the abutment excavations were completed, the rest of the rockfill was dumped in lifts as thick as possible (presumably from the same material). No mechanical compaction or sluicing was used (ICOLD, 1999). Despite the claims of being ‘earthquake proof’, this type of construction is no longer used as it results in poor compaction of the materials and thus large settlements take place post construction (and under earthquake loading).

A summary of known data on the embankment materials is given in Figure 2 and Table A.

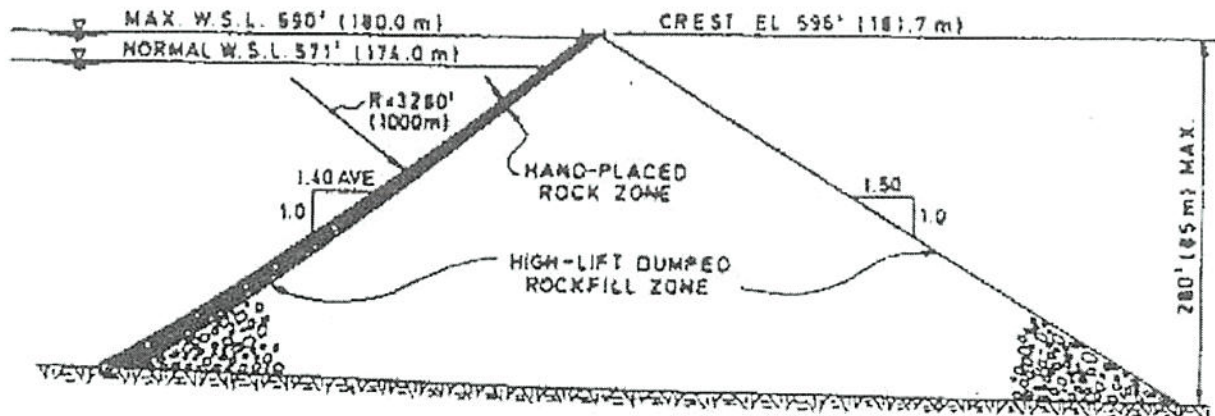


Figure 2

Zone	Name	Description	Properties			Compaction and Placement			Wc %
			γ_d (kN/m ³)	c'	ϕ'	Thick ness	Compaction	Density Ratio	
1	Main Rockfill	Dumped blast spoil from andestic breccia	n/a	0	41	n/a	Dumped	n/a	0
2	Gravel Cushion	2m zone of hand placed gravel	n/a	0	n/a	2m	None	n/a	0

Table A

The concrete face was designed to act as a 'flexible' system (ibid) and employed a segmented system utilizing slabs of average size 10 metres by 10 metres. The thickness of the slabs at the toe was about 800 mm, tapering to 200 mm at the crest. Reinforcement specifications varied with slab width, from 2 layers of 25mm bars at 300mm centres at the toe to a single layer of 19mm bars at 200mm centres at the crest. Joints between slabs were constructed using 600mm wide riveted copper waterstops.

Rockfill under the slab comprised a 2 metre thick zone of small-sized' rock, which according to Arrau et al (1985) was hand placed (the Engineering News Record (Vol107, pg 725) article states that there is 'no hand-placed material'). The Engineering News Record article states that 'if there is a disturbance of the rockfill by settlement or earthquake shock, the thick bed of free running gravel is expected to fill the depression with very slight change of surface next to the pavement'.

Under joint locations, concrete footings 1.2m wide by 0.7m deep were formed in the hand placed gravels to act as bearing support for the panels (Arrau et al 1985). The perimeter joint detail at the toe is as shown in Figure 3.

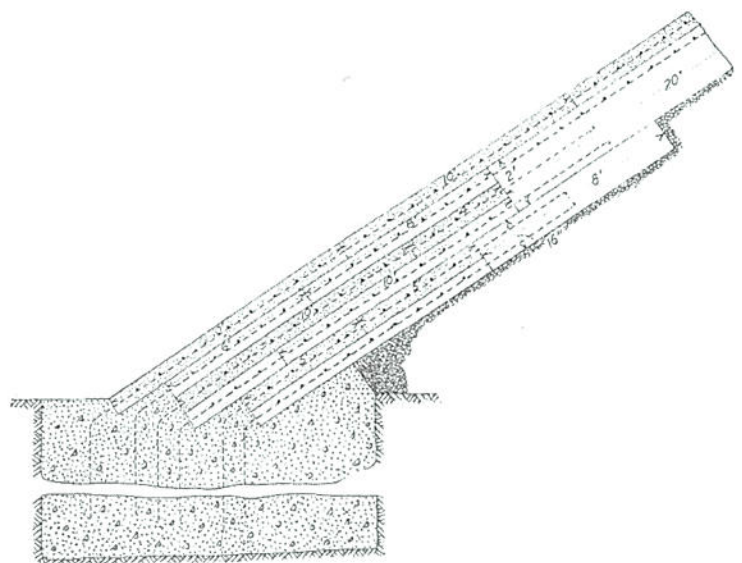


Figure 3 Toe Details

A plan of the dam area is given in Figure 4. Following the blasting, the abutments formed are described in the ICOLD document (1999) as being 'very steep'.

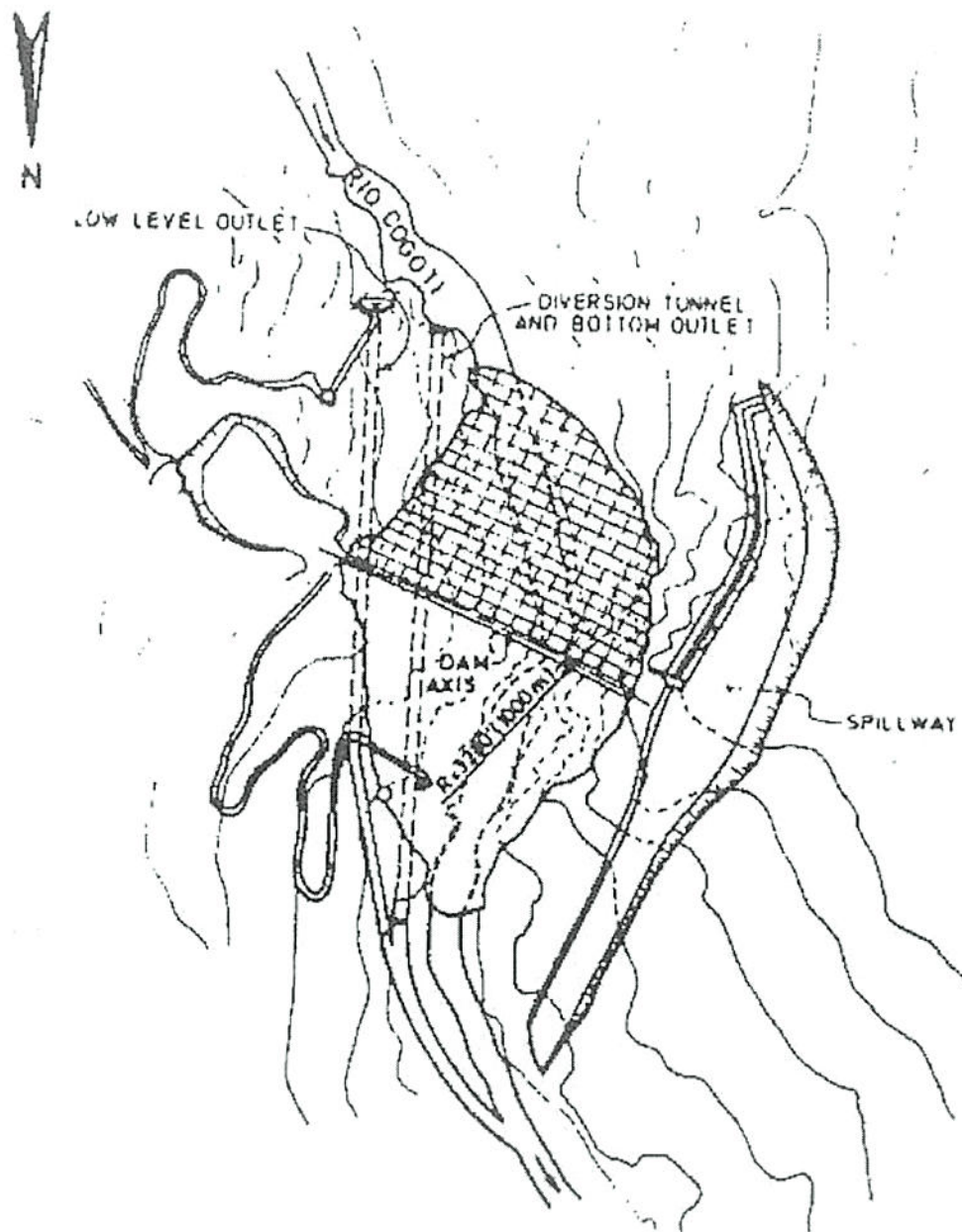


Figure 4

Foundation Geology / Materials

The dam is situated in the foothills of the Andes Mountains in Chile, in a deep gorge carved out by the Cogoti river.

The dam is founded on rock, which is described by ICOLD as 'andesitic breccia'. According to Arrau et al (1985), impervious soils are not found in the area.

Effects From Earthquakes

Illapel Quake, 6 April 1943

The area around the dam is mountainous and sparsely populated. Consequently, few records are available which describe the quake. However, many aftershocks were reported over the week following the quake (ICOLD, 1999).

The principal effect of the Illapel quake on Cogoti dam was **settlement**. As a consequence of the quake, the dam was reported to experience immediate settlements on the crest of 410mm. This is about 0.48% of the dam maximum section height. However, the point of maximum settlement was measured near the centre of the crest, which, interestingly, is not the point of maximum dam height. At the centre, the dam height is about 63 metres, whereas near the right abutment, the maximum section is about 85 metres. It is said that the settlement was less at the maximum height 'because of a restraining effect due to the nearby presence of the very steep abutment' (ibid). The measured earthquake induced settlement of 410mm is said to be equivalent to that of 'natural' settlement observed over a period of 4.5 years after completion of the dam. Figure 3 shows settlements of the dam since construction – although the units are unclear, the significance of settlements as a consequence of the Illapel quake is evident.

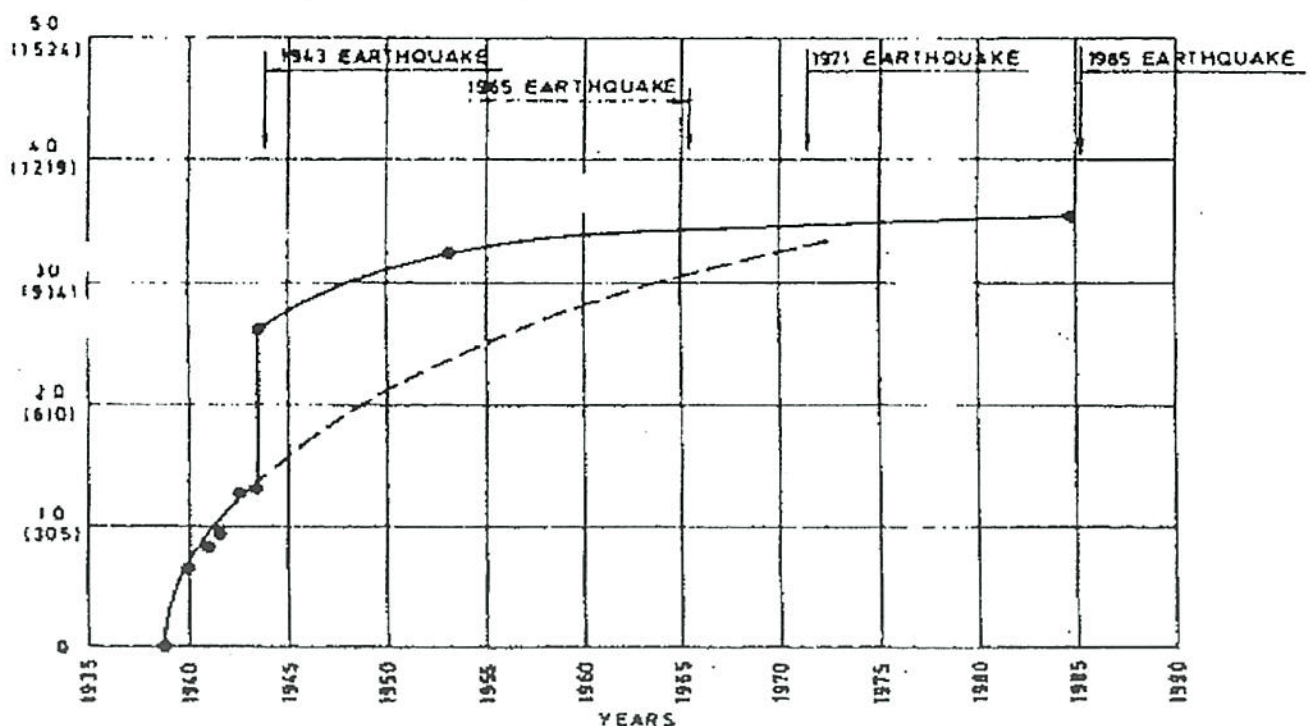


Figure 3

The dam continued to settle after the quake, but at a rate slightly less than what was predicted had the quake not occurred (see Figure 3). The Illapel quake showed a much greater response in settlement compared to other earthquake events – which, according to ICOLD, can be attributed to the closer proximity of the Illapel quake (other quakes were over 165 km away). Other quakes are estimated to have produced peak ground accelerations at the Cogoti site of not greater than 0.05g.

The apparently large magnitude of the settlements can be attributed, according to ICOLD (1999), to the method of rockfill construction used.

The fact that settlements were not greatest over the maximum section, but, due to 'restraining effects' of the abutment, were over a section that is over 20 metres smaller, implies the development of significant internal

stresses. Despite this, no **cracking** or increased seepage is reported as a consequence of this quake (the reservoir was at 'normal' levels, having a freeboard of about 7.7 metres). Apparently the concrete face remained intact throughout the earthquake, as designed, meaning that any cracks that may have occurred in the fill would not be evident by either visual examination or by increased seepage.

Figure 4 shows seepage at Cogoti dam since construction. **Leakage** has been observed since construction, but as shown, is related only to reservoir height, and is attributed in ICOLD (1999) mainly to flow through the foundation. The increase over the years has been attributed to aging (ICOLD, 1999), and not to any earthquake events. Arrau et al (1985) believe the majority of seepage is through joints in the slab and through perimeter joints.

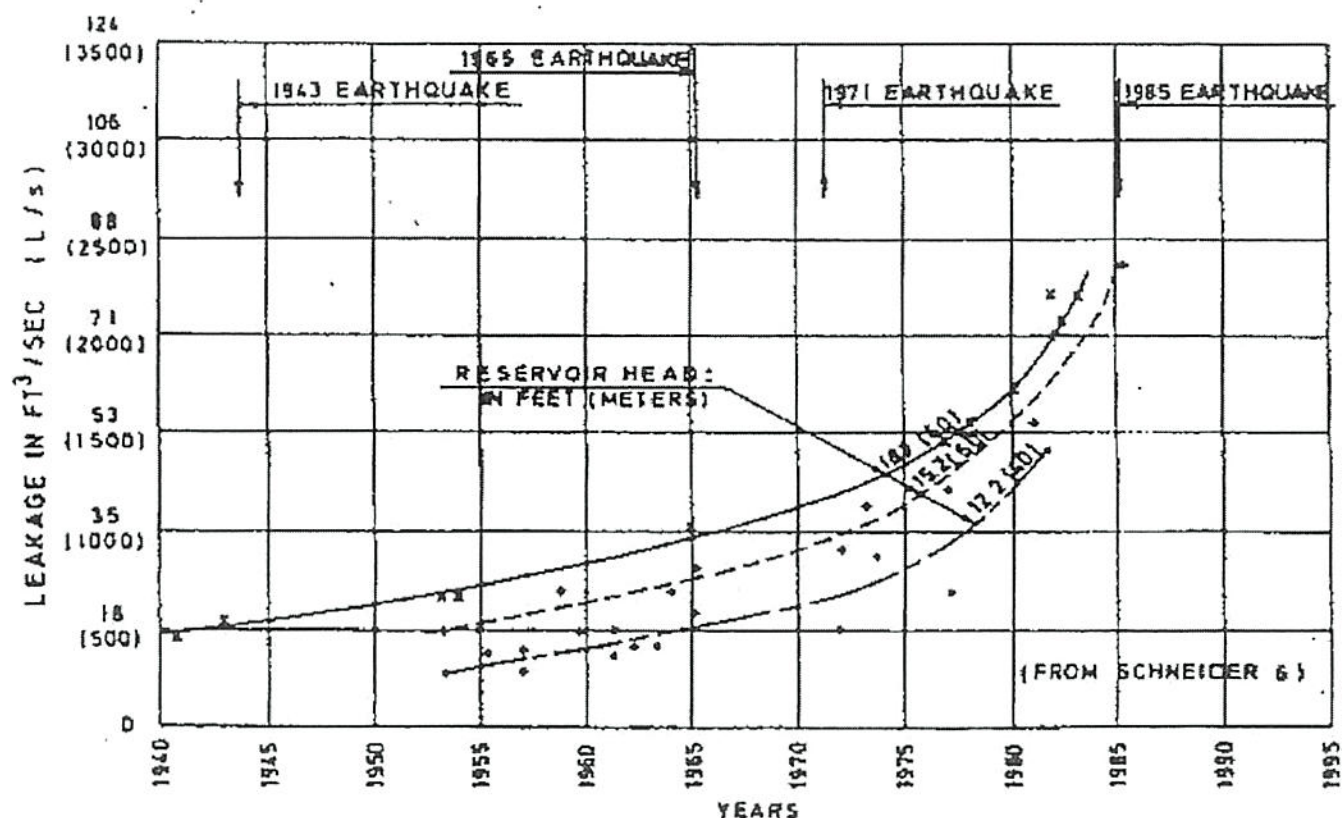


Figure 4 Leakage at Cogoti Dam

Papudo – Zapallar Quake, 8 July 1971

This quake was of lower magnitude and greater distance from the dam than the Illapel and was thought to have induced peak ground accelerations less than 0.05g at the dam site. As shown above (figure 3), for this quake there were no noticeable earthquake induced settlements recorded. However, according to Serafim (1987) and ICOLD (1999), some earthquake damage occurred in the form of longitudinal cracking at the crest, and the dislodging of some rocks. Such damage is interesting considering its absence during the stronger shaking of the Illapel quake. Furthermore, other significant quakes in 1965 (La Ligua M7.1) and 1985 (Llolleo-Algarrobo M7.7), which induced similar shaking failed to produce any noticeable damage.

According to ICOLD (1999) the Cogoti Dam was empty at the time of the Papudo-Zapallar quake.

No further information is available on this earthquake.

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Coyote Dam

(Morgan Hill Quake, 24 April 1984; Loma Prieta Quake, 17 October 1989)

Dam Details

Location	California, U.S.A.
Type	Earthfill
Height (m)	43 (140 ft)
Crest Length (m)	299 (980 ft)
Year Built	1936
Owner	Santa Clara Valley Water District
Foundation Geology	Rock, no alluvium
Design Freeboard (m)	7.6 (total) 13.4 (operating)

Special Features of Dam

Coyote Dam lies on the Calaveras fault and thus has a high seismic risk. Leroy Anderson reservoir lies downstream (less than 10 km away) and acts as a backup for Coyote Dam – it must always maintain sufficient storage for the Coyote water, were the Coyote Dam to fail. This requirement came into action in 1963 (USCOLD 1988).

Earthquake Details

Earthquake No. 1

Name	Morgan Hill		
Date	24 April 1984		
Magnitude (M_L)	6.2 M_L (USGS database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	24	8*	25
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-	0.63g*	
Freeboard at Time of Earthquake (m)	1.6		
Dam Damage Descriptor	Minor, class 1		

Special Features of Earthquake

The peak ground acceleration at the dam is greater for the Morgan Hill quake than for the Loma Prieta quake. However, the duration of strong shaking in the Morgan Hill quake was less than in the Loma Prieta quake (Harder et al 1996).

* from USGS database

* an estimation presented in Swaisgood 1995. Note that a Esteva and Rosenblueth estimation gives a value of only 0.28g.

Earthquake No. 2

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1 M_s (USGS database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	22 [^]	18.5	29
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-	0.19g [^]	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	No or Slight / Minor, class 0/1		

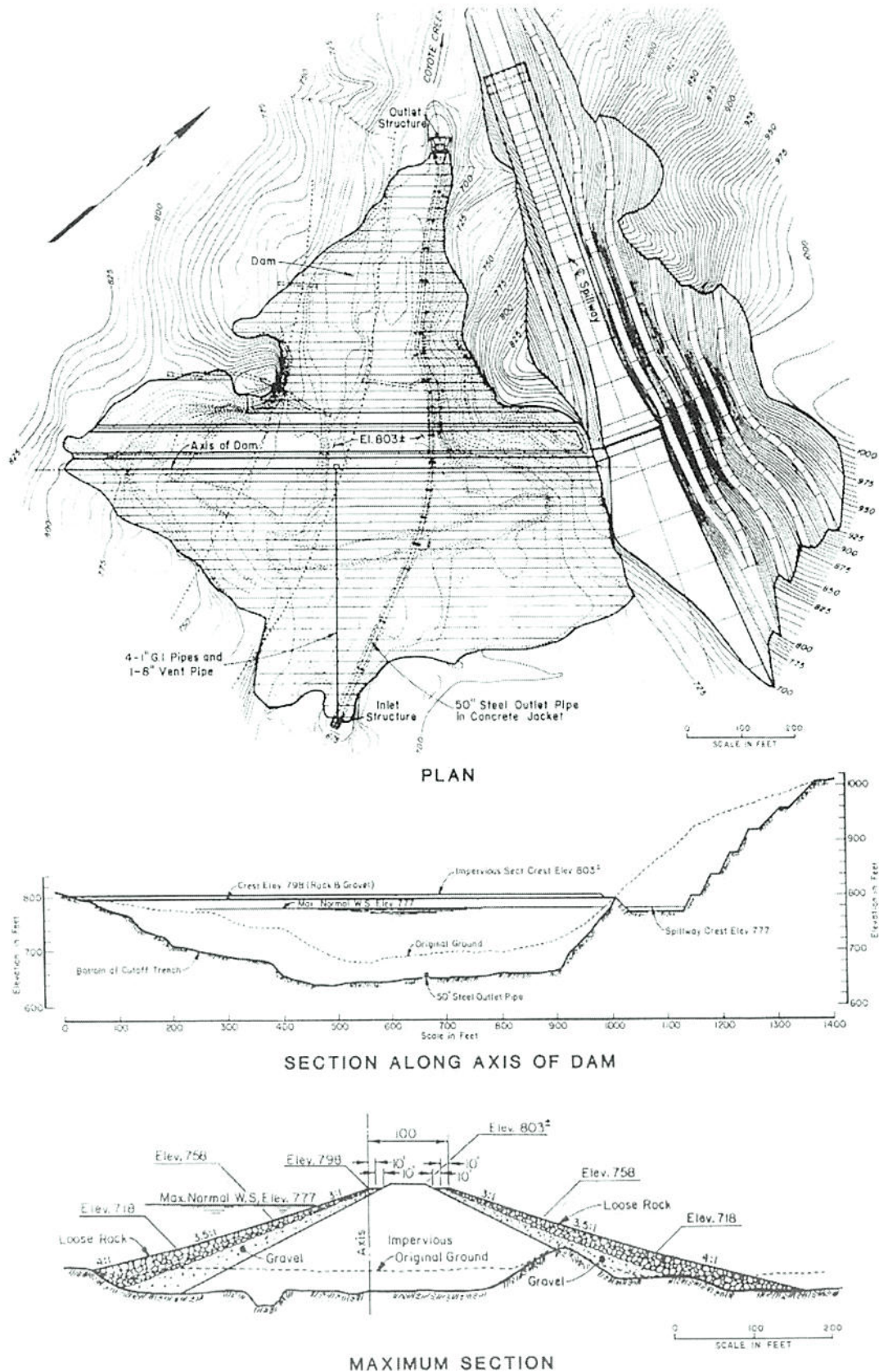
Special Features of Earthquake

The duration of shaking was generally between 7 and 10 seconds (12 seconds according to Foster) which is about half of that expected for an event of this magnitude (Harder *et al* 1996). Despite this, the duration of shaking was still longer than for the Morgan Hill quake.

[^] as presented in Harder *et al* 1992

Embankment Details

Figure XXX shows details of the embankment. No information, further than what is shown in this Figure XXX, was available.



Foundation Geology / Materials

There was no information available on the foundation of this dam, other than, according to Swaisgood (1998), there is no alluvium left under the embankment.

However, the dam lies on the same creek as, and less than 10 km from, the Leroy Anderson dam, for which some knowledge of the foundation material was available. From this we know that the dam is set in a long deep natural gorge on Coyote creek (see photo 1 below) which runs through the Santa Clara Valley District (USCOLD 1988). The Santa Clara Valley is composed mostly of igneous Franciscan rocks. This formation has largely been altered to serpentine, and it extends from the upstream toe of the dam and to the west.

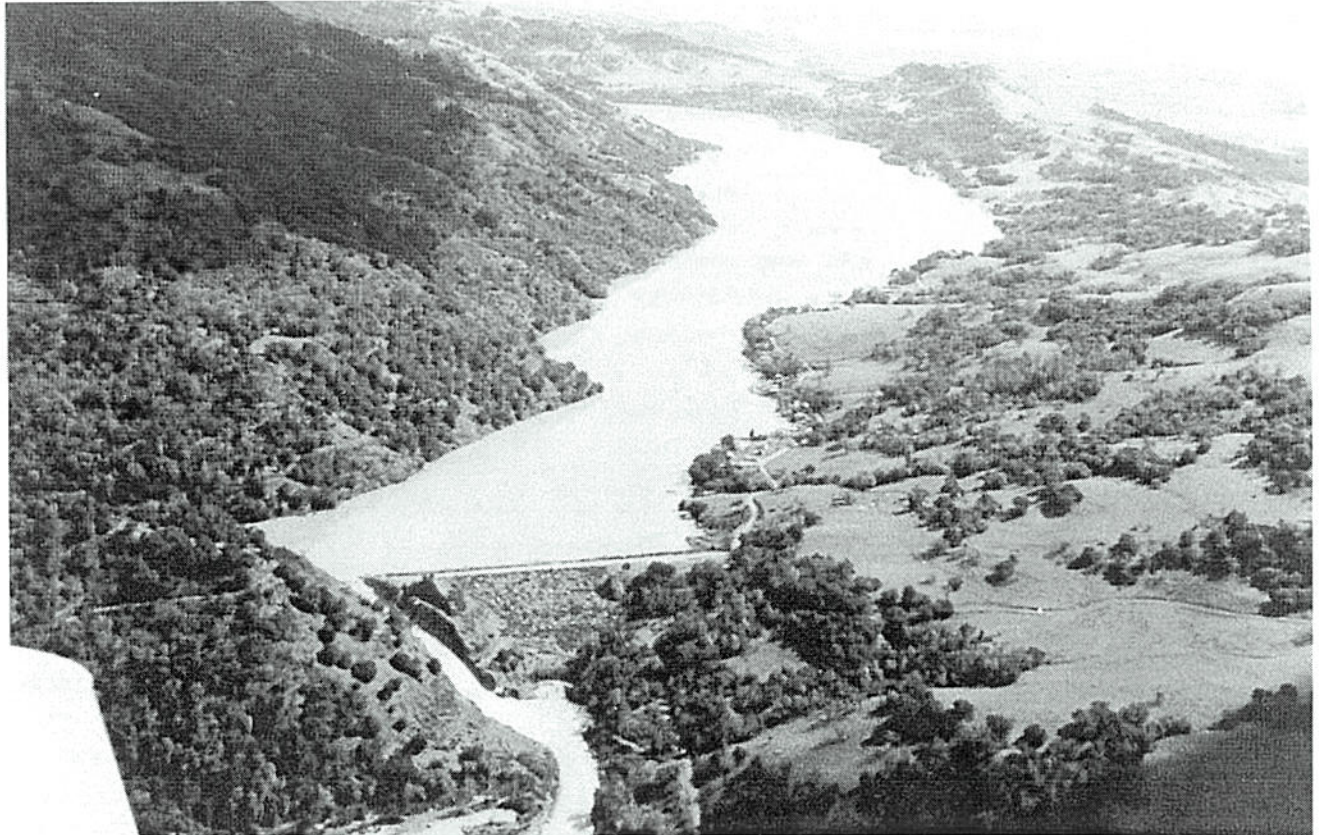


Photo 1 Coyote Dam and Surrounds

Cracking From Earthquake

There is little reported on the effects that these earthquakes had on the Coyote dam. This is probably because there was very little damage to the dam from either earthquake. However, the case study was included because of the opportunity for comparisons with Leroy Anderson dam, which is less than 10km from Coyote dam, but was damaged from both earthquakes.

USCOLD (1992) reports that there was no damage to the Coyote dam from this earthquake. However, Swaisgood reports a crest settlement of 80mm. This corresponds to 0.19 % of dam height, or class 1 'minor' damage in the damage classification system used in this report. No cracking was reported. In comparison, Leroy Anderson dam (which is also founded on rock, with no alluvium) settled only 0.02% of its height, but suffered minor longitudinal cracking.

Similarly, Harder *et al* (1991) report that 'no significant damage' occurred to the Coyote dam after the Loma Prieta earthquake but settlements of up to 40 mm were recorded (0.06% of the dam height) at the Leroy

Anderson site. Also, minor longitudinal cracking occurred on the Leroy Anderson embankment to a depth of 0.6m, with maximum width less than 20mm.

This difference in response could be attributed to a number of things, including: the inherent spatial variability of earthquake loading; inconsistencies in reporting (i.e. cracking may have occurred but been unnoticed or not reported at the Coyote dam); or it could be due to embankment characteristics such as zoning, shape, compaction and amplification.

The two relatively shallow parallel systems of longitudinal cracks (about 25 mm wide) that formed along the crest of the Leroy Anderson dam were located approximately above the interface between rockfill shells and earth core, and the crack formation in both cases was attributed to differential settlement between the well compacted clay core and loosely placed rockfill shell zones (Harder *et al* 1996).

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El Infiernillo Dam

(Various Earthquakes)

Dam Details

Location	Mexico
Type	Central Core Earth and Rockfill
Height (m)	147 (484 ft)
Crest length	344 (1132 ft)
Year Built	1964
Owner	Comision Federal de Electricidad
Foundation Geology	Breccias and Conglomerates, intersected by several dykes.
Design Freeboard (m)	6.5

Special Features of Dam

It is principally to serve an hydro-electric development, but also serves as flood control. The structures for electricity generation and the spillway are both located on the left bank, but are detached from the embankment itself.

Earthquake Details

The El Infiernillo Dam has been subject to eight significant earthquakes since 1966. Of these, only two have produced any significant deformations and cracking. This case study focuses on those since 1975.

Earthquake No. 1

Name	Mexico		
Date	11 October 1975		
Magnitude (M_s)	4.9 [*]		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	79.2 [*]	63 [⊥]	101
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	----	0.09 [*]	
Dam Damage Descriptor	No or Slight, class 0		
Freeboard at Time of Earthquake (m)			

Earthquake No. 2

Name	Mexico		
Date	15 November 1975		
Magnitude (M_s)	5.9 [*]		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	22.9 [*]	33 [⊥]	40
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.28 [*]	0.09 [*]	
Dam Damage Descriptor	No or Slight, class 0		

^{*} Recorded value, as presented in Elgamal, 1992

[⊥] from USGS seismic database

Freeboard at Time of Earthquake (m)

Earthquake No. 3

Name	Guerrero		
Date	14 March 1979		
Magnitude (M_s)	7.6 [*]		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
.....	125.4 [*]	49 [⊥]	134
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.36 [*]	0.12 [⋄]	
Dam Damage Descriptor	Moderate / Minor, class 2/1		
Freeboard at Time of Earthquake (m)			

Earthquake No. 4

Name	Playa Azul		
Date	25 October 1981		
Magnitude (M_s)	7.3 [*]		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
.....	55.3 [*]	28 [⊥]	62
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.05 [⊗]	
Dam Damage Descriptor	No or Slight / Minor, class 0/1		
Freeboard at Time of Earthquake (m)			

Earthquake No. 5a, b (incl. significant aftershock)

Name	Michoacan		
Date	19, 21 September 1985		
Magnitude (M_s)	8.1 [*] , 7.5 [*]		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
.....	68 [*] , 79 [*]	29 [⊥] , 42 [⊥]	74, 86
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.5 [⋄] ,	0.13 [*] , 0.06 [⋄]	
Dam Damage Descriptor	Moderate, class 2		
Freeboard at Time of Earthquake (m)			

[⊥] from USGS seismic database

^{*} as presented in Resendiz et al 1982

[⊗] as estimated in Swaisgood 1995

[⋄] as estimated in USCOLD 1992

^{*} recorded value, as presented in Elgamal, 1992

Embankment Details

El Infiernillo Dam is an earthcore rockfill dam comprising a thin vertical core of clay, upstream and downstream sand filters, transition zones, and external rockfill shells. The total volume of the embankment is 5.5 million cubic metres, of which 5.0 million cubic metres is granular (USCOLD, 1992). The maximum dam section is shown below in Figure 1.

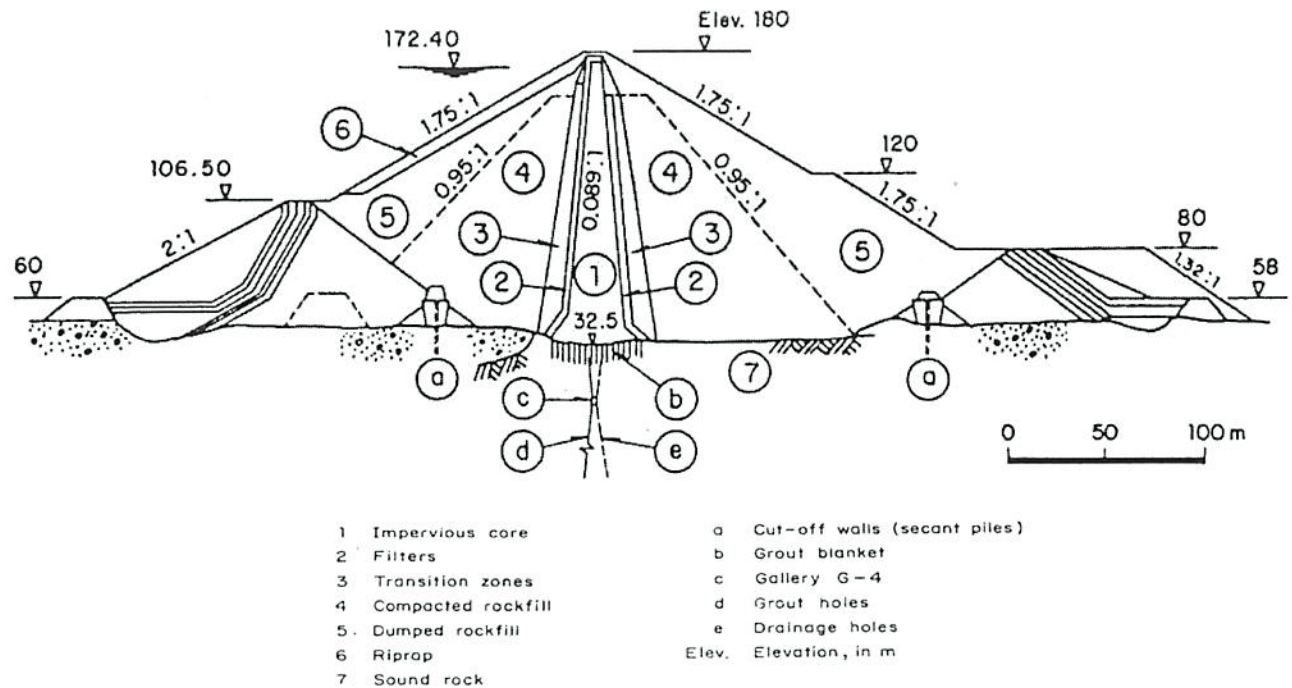


Figure 1 – El Infiernillo Maximum Embankment Section

Basic properties of the embankment materials shown in Figure 1 are given in Table 1 below.

Zone	Name	Description	Properties			Compaction and Placement			Wc %
			γ_d (kN/m ³)	c' (kPa)	ϕ'	Thickness	Compaction	Density Ratio	
1	Earth core	Residual clayey soil PL 24 LL 49	15.5	~50 (undrd)	0	150mm pre comp	15t 'special' sheepsfoot	94% standard	19.8 (3.7 % above optimum)
2	Filter	Well graded, washed, screened sand < 6mm	18.3	-	35	300mm pre comp	4 passes 3t vibratory		10
3	Transition	Well graded <76mm downstream, <150mm upstream	19.6	-	35	200mm pre comp	3t vibratory roller		10
4	Compacted Rockfill	Diorite and silicified conglomerate. <0.6 m	18.1	-	45	0.6-1 m	Four passes of D-8		Placed without water
5	Dumped Rockfill	Diorite and silicified conglomerate >0.6m	17.3	-	-	2 – 2.5 m	Dumped		Placed without water

The core (1), filters (2), transition zones (3) and inner parts of the rockfill shells (4) were compacted. The outer rockfill shells (5) were dumped (USCOLD, 1992).

The impervious core (labeled “1” in Figure 1) used clayey materials that were excavated from a pit some 18 km from the dam site. After excavation the materials were transported to a storage pit where it was ‘sprinkled’ and allowed to rest for 15 days. The material was reported to have a liquid limit of 49, a plastic limit of 24 and mean void ratio of 0.58

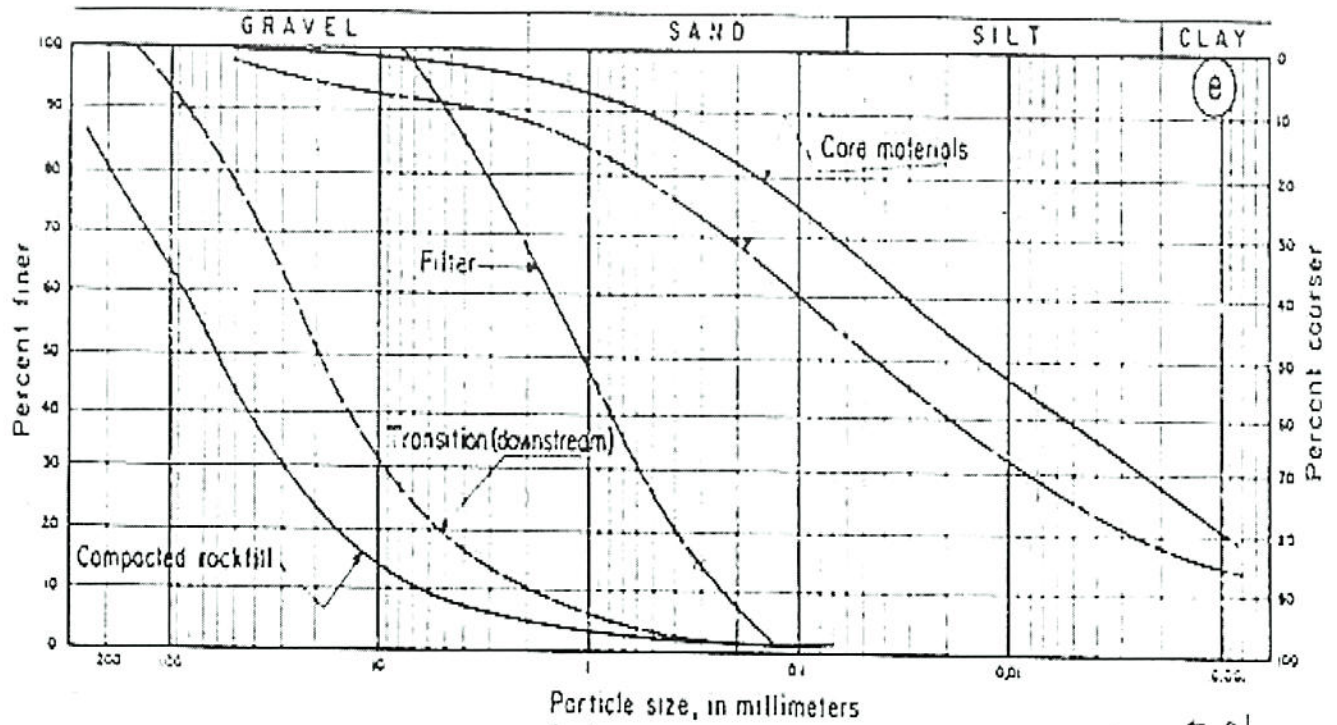
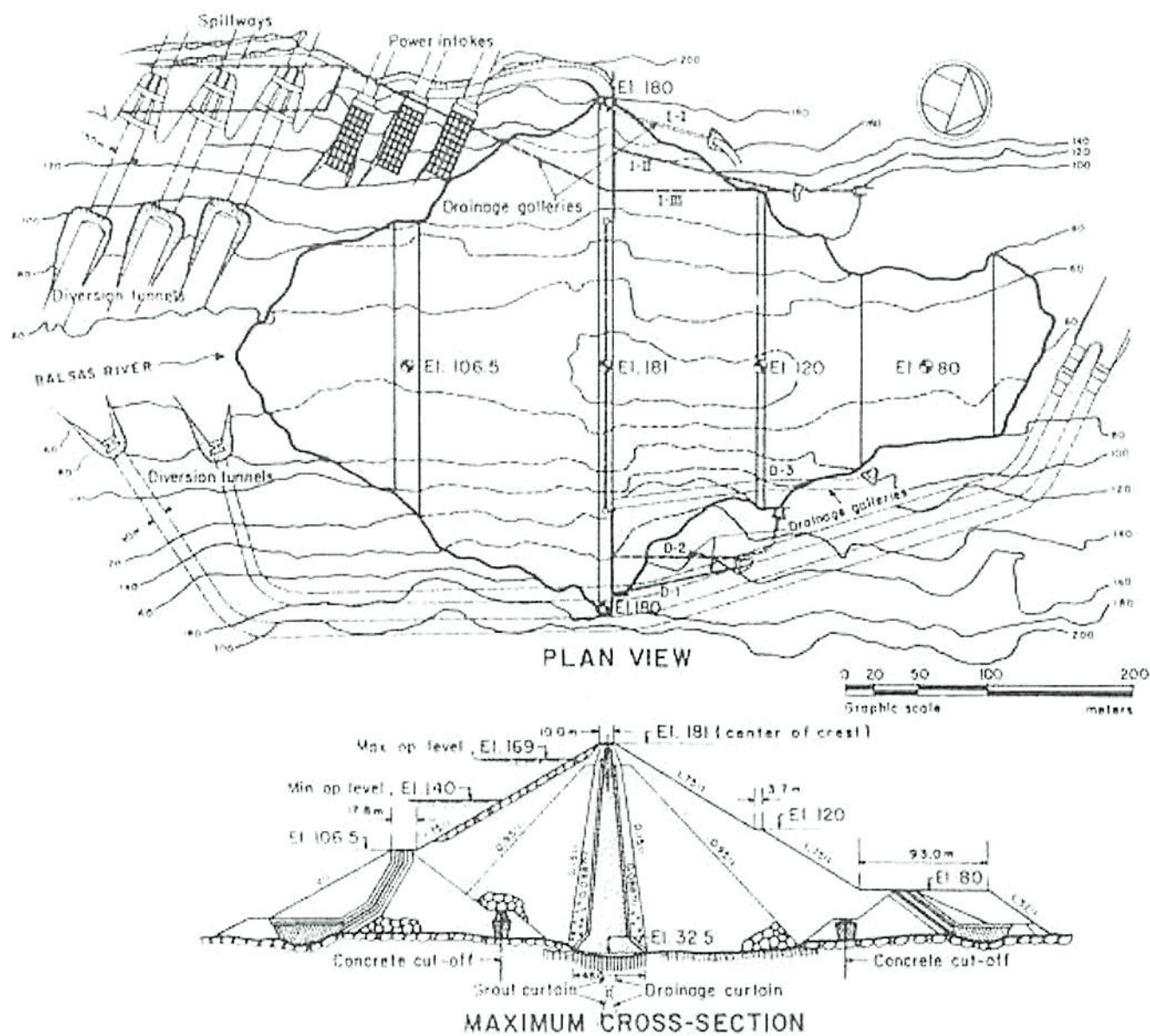


Figure 2 – Particle Size Distributions of El Infiernillo Embankment Materials

The filters (labeled “2” in Figure 1), adjacent to the transition zones, are 2.5 metres wide and used material described as a ‘well-graded, washed and screened sand’

The transition zones (3) used materials that were the product of underground excavations and quarry muck. These materials were placed in layers 300mm thick and remixed in place with a grader before being compacted with a 3 tonne vibratory roller (Marsal and Ramirez de Arellano (1967) say a 2-tonne roller).

Rockfill was processed from three quarries excavated in silicified conglomerate and one quarry in sound diorite (USCOLD, 1992).



Figures 3 and 4 – Plan and Section of El Infiernillo Dam

Foundation Geology / Materials

El Infiernillo dam is founded on breccias and conglomerates, intersected by some basalt dykes and including a number of shear zones (USCOLD, 1992). Figure 5 is a section through the dam site, showing major geological features.

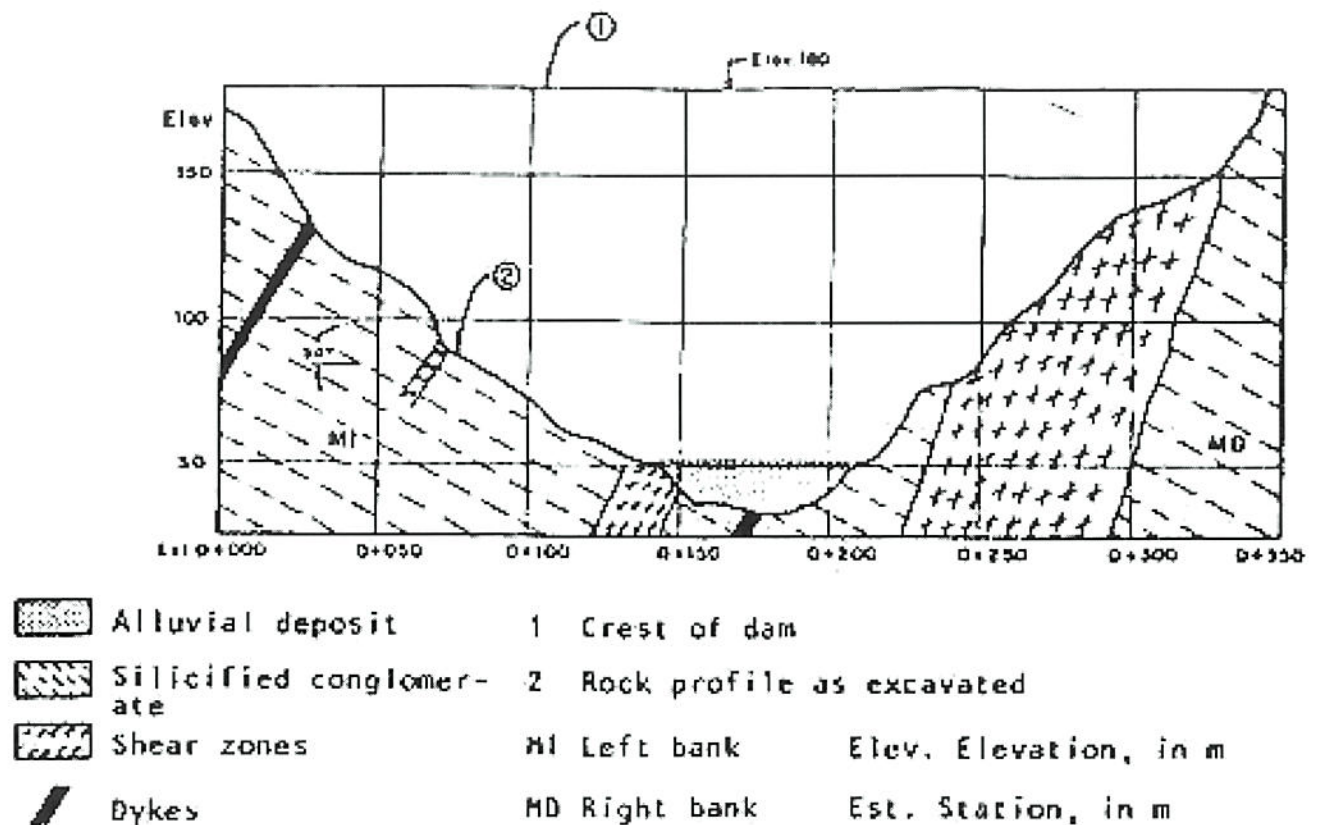


Figure 5 Foundation Geology – Looking Downstream

The canyon bedrock is composed mainly of the silicified conglomerate, which dips 30 degrees to the NE, and has a predominant north-south fracture system (river points NNW at embankment), inclined 70 degrees towards the west. Occasional clay infilling of fractures is reported. The rock is described as ‘competent’ but is brittle and sensitive to blasting. (Marsal and Ramirez de Arellano 1967).

Several basaltic dykes cross the mass of conglomerate, parallel to the fractures. According to Marsal and Ramirez de Arellano (1964) there was no decomposed material in the contacts between the dykes and the bedrock.

Marsal and Ramirez de Arellano (1964) also record the presence of a nearby dioritic intrusion, from which the bedrock around the dam was ‘almost melted’. As a result of this, they describe that the ‘rock layers were cemented together obliterating the existing bedding planes’. The region of bedrock which was affected by the intrusion is unclear.

The rock mass in the shear zones is described by Marsal and Ramirez de Arellano (1964) as ‘sound’, even immediately after blasting. However, they noted that the rock would start to crumble and decay into small fragments following a few days of weathering after being exposed by blasting.

Despite the presence of intrusive dykes and shear zones, Marsal and Ramirez de Arellano (1964) report that “in general, the rock was found to have a rather low permeability in both banks” quoting Lugeon units of

between 1 and 4. However, they record localized surface regions with permeability values of over 20 Lugeon.

Underneath the core, all loose materials and the shallow, high permeability weathered rock was removed. This area was cleaned with water jets and a grout curtain was constructed to a depth of 8 metres. Localized mortaring was applied in areas of significant fracturing (ibid).

Residual soils were stripped from under the embankment footprint for a distance of 130m downstream of the dam axis, but were left in place upstream of the core. The residual materials were mostly boulders packed with fine sand, and containing some tree stumps. Single-line grout curtains were also constructed both upstream and downstream to depths between a 50 and 100 metres (ibid) (see figures 1 and 4).

Effects From Earthquakes

Of the six earthquake events listed above, only two caused any significant damage. These two events will be discussed in detail.

Effects from the 1979 Guerrero Earthquake

Most authors describe the damage to El Infiernillo Dam from the Guerrero earthquake to be 'minor', and included minor settlement, longitudinal and transverse cracking. Locations of cracking are shown on figure 6 below.

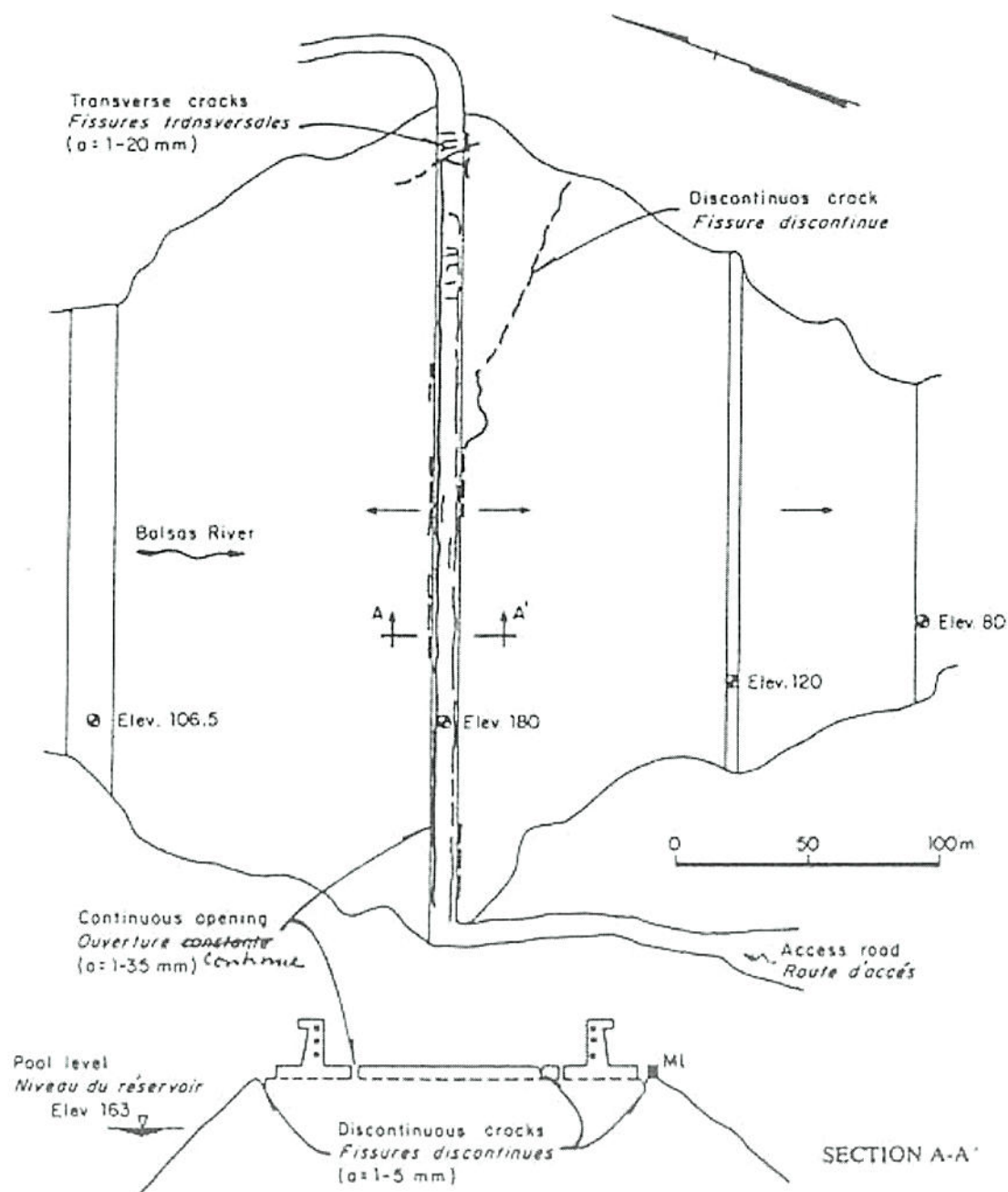


Figure 6 – Cracking Pattern of El Infiernillo Dam After the March 14, 1979 Earthquake

Longitudinal cracking included one significant continuous crack 1 to 35 mm in width just upstream of crest and various discontinuous cracks 1 to 5 mm width. Some **transverse cracking** of 1 to 20 mm width occurred adjacent to left abutment. **Settlements** of up to 120mm were recorded on the crest and are shown in figure 7 below

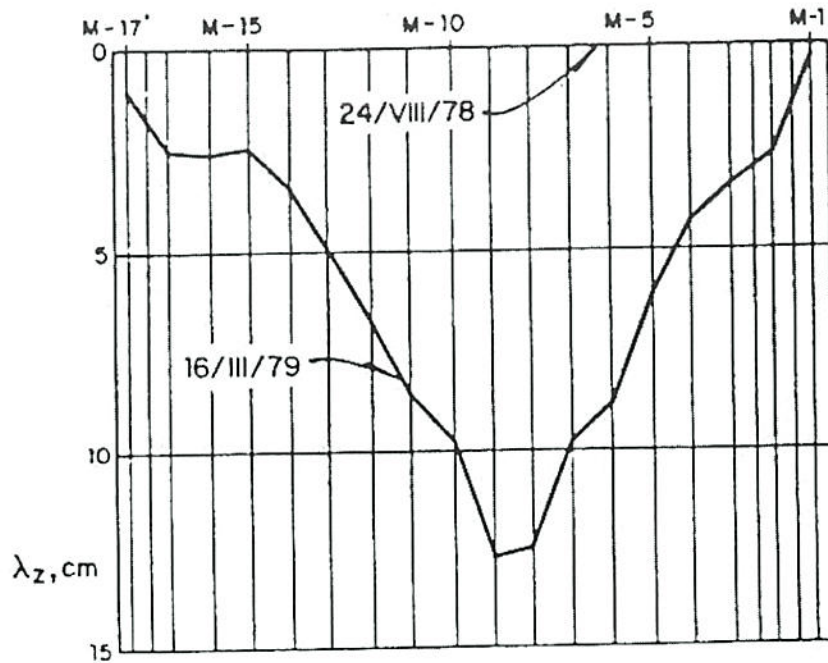
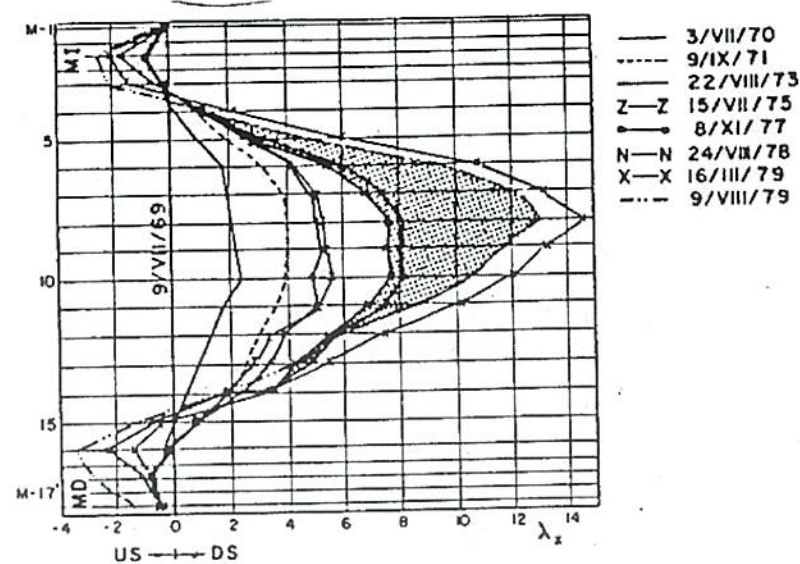


Figure 7

A history of settlements at El Infiernillo, including the Guerrero earthquake, is shown in Figure 8 below.



M-i Bench marks at crest (row L-1)
MD Right bank MI Left bank
US Upstream direction; DS Downstream direction
 λ_x Horizontal displacement of the crest (river direction), in cm

Figure 8 – Shaded Area Represents Effects of the Guerrero Earthquake

Because of the extensive monitoring installed at El Infiernillo, a plot of the displacements of the core and downstream slope is available as is Figure 9 below. Resendiz et al (1982) estimate that approximately 27% of the crest settlement in El Infiernillo dam is due to lateral spreading.

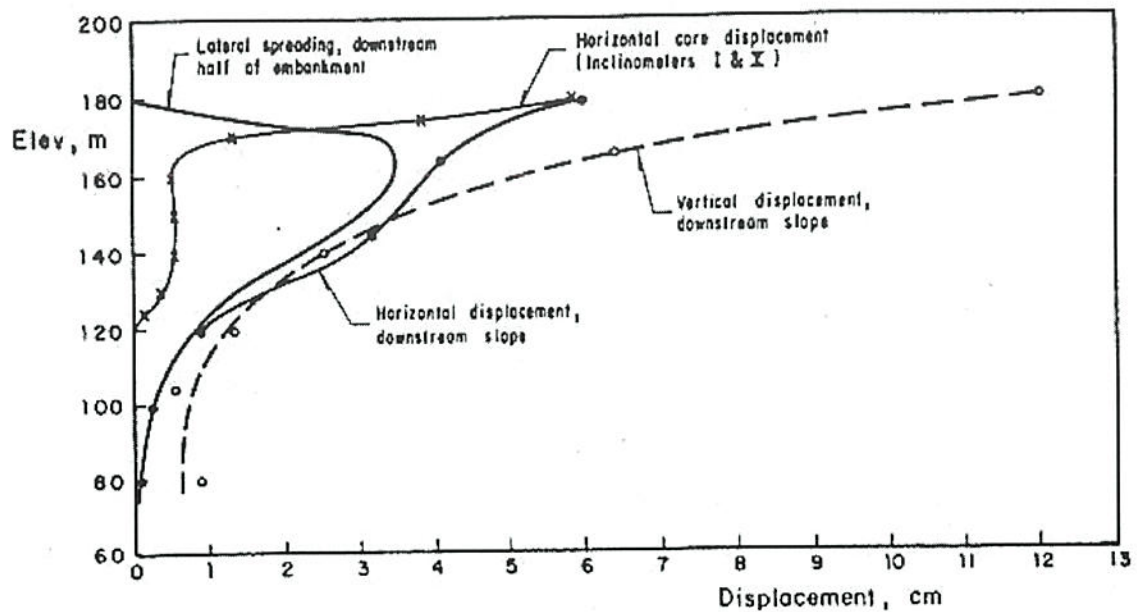


Figure 9

Length measurements between crest benchmarks provided the information to produce a plot of variations of longitudinal strain along the crest as shown in Figure 10

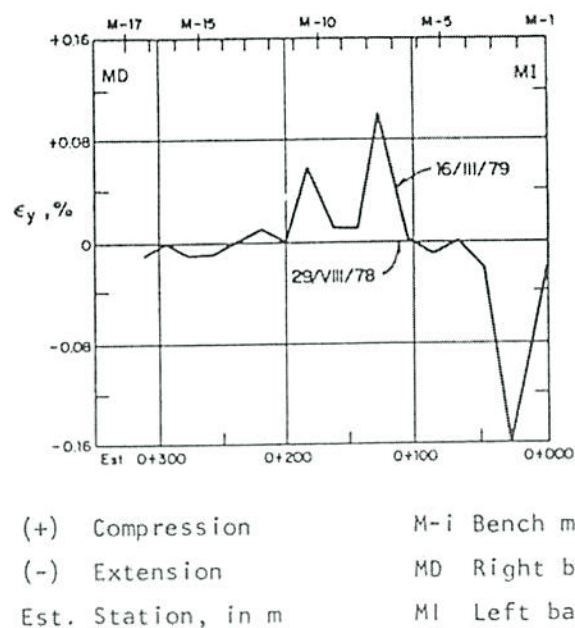


Figure 10 – Variations of Longitudinal Strains Along the Crest of El Infiernillo Dam

The effects of the Guerrero earthquake are significant on all the above plots. Resendiz et al (1982) point out that this is was the first time El Infiernillo suffered permanent seismic deformations, despite being struck by several earlier quakes. The maximum crest acceleration of prior events was 0.23g (compared to the Guerrero acceleration of 0.36g) and produced at most only a temporary minor increase in settlement rates.

Resendiz et al (1982) also point out that the settlement of 0.1% of the dam height under the Guerrero event is equivalent to 22% of the cumulative settlement of the dam since first filling (a time of over 15 years)

The extent of settlement and deformation data made available through the extensive instrumentation at El Infiernillo allows some examination of the mechanics of the cracking. Resendiz et al (1982) write of the data “the positive correlation between the intensity of longitudinal cracking and magnitude of settlements is beyond doubt”. They point out that five lines of longitudinal cracking (in the center of figure 6) correspond to the location of maximum settlement in figure 7, where only one to three lines occur in zones of less settlement. They also comment on the location of transverse cracking and its relationship to maximum tensile strains in figure 10.

Effects from the 1985 Michoacan Earthquake (M8.1) and Major Aftershock (M7.2)

The Michoacan earthquake occurred under subduction processes between the Cocos and North American plates some 12km offshore of the coast. The main rupture occurred in two distinct events separated by about 25 seconds. The major aftershock of Richter magnitude 7.5 occurred 2 days later (USCOLD, 1992).

From the main rupture, El Infiernillo dam was subject to about 60 seconds of strong shaking with crest accelerations estimated to be up to 0.5g (USCOLD, 1992). Despite this, the embankment only suffered damage that is described by authors to be ‘minor’ (USCOLD, 1992; Swaisgood, 1995).

After the main earthquake, a maximum crest **settlement** of 90 mm was recorded (USCOLD, 1992). Measurements revealed that most deformation occurred within the top third of the embankment.

Longitudinal cracks from 0.5 to 150mm wide formed on either side of the dam crest along its entire length. Additional minor longitudinal cracks of 30-40 mm width, 9 metres long formed on the crest adjacent to abutments (USCOLD, 1992). Investigations immediately after the event confirmed the cracks to be only superficial (Bureau and Campos-Pina, 1986). No **transverse cracking** or damage to the spillway or tunnels was recorded. Piezometre data did not record any significant increase in pore pressures.

No further damage was recorded from the aftershocks, despite their significant magnitude.

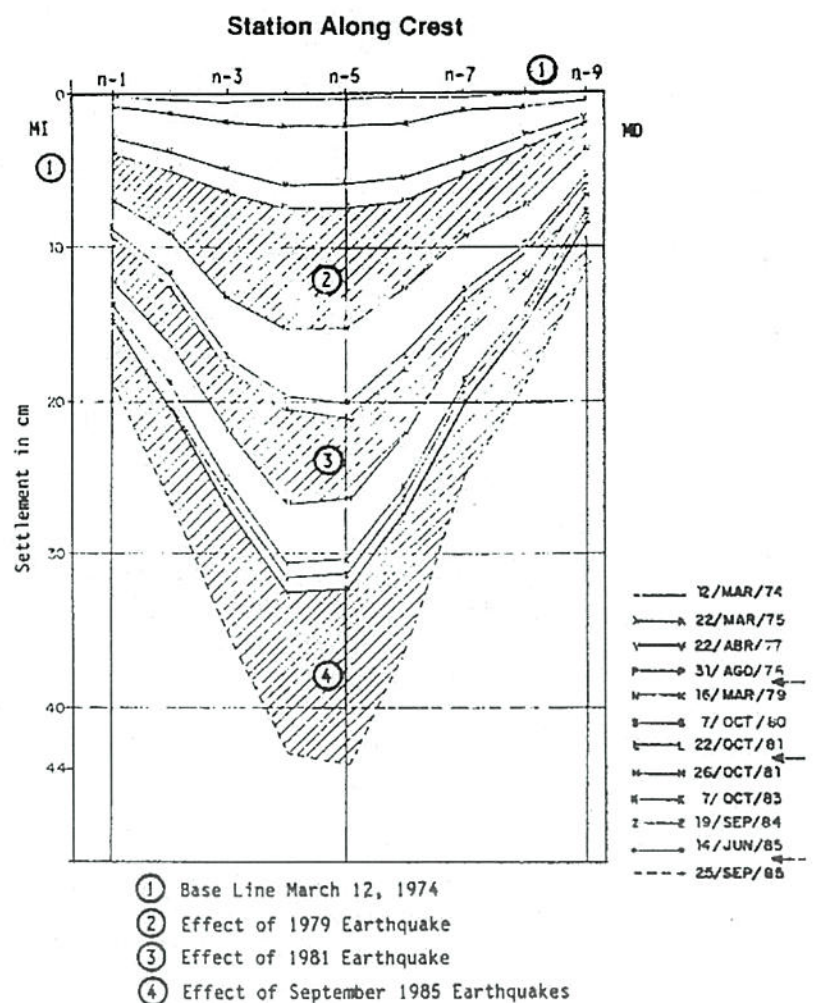


Figure 11

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Guadalupe Dam

(Loma Prieta Quake, 17 October 1989)

Dam Details

Location	California, U.S.A
Type	Zoned Earthfill
Height (m)	43 (142 ft)
Crest Length (m)	211 (695 ft)
Year Built	1935
Owner	Santa Clara Valley Water District
Foundation Geology	
Design Freeboard (m)	3.0 (total) 3.4 (operating)
Freeboard at Time of Earthquake (m)	24 (although was full only 3 months prior)

Special Features of Dam

The dam incorporates concrete panels on the upstream side for erosion protection. Modifications in 1972 involved placing a sloping earthfill buttress against the upstream toe.

To avoid potential confusion, it must be pointed out that this case study refers to the Guadalupe dam located in California, U.S.A., located 37 11.9 N and 121 52.7 W and is to be distinguished from another 'Guadalupe' dam, of similar dimensions located in the State of Mexico, as described by Marsal, R.J. (1982), or any of the dams located in the Guadalupe county, Texas.

Earthquake Details

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	18	18.5	25.8
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	-----	0.4-0.45g*	
Dam Damage Descriptor	Major, class 3		

Special Features of Earthquake

The duration of shaking was generally between 7 and 10 seconds, which is about half of that expected for an event of this magnitude (Harder et al 1996).

* estimation presented in Harder et al

Embankment Details

The Guadalupe dam of California was completed in 1935 as a rolled earth dam. Despite a design for an upstream impervious section, the processes of selection, compaction and placement offered little success in creating distinctly different materials. Consequently, Guadalupe dam is essentially an homogeneous embankment.

The dam incorporates concrete panels on the upstream side for erosion protection.

In 1972 an upstream buttress was added to improve drawdown stability. This involved removing some concrete panels and placing a lightly compacted clayey sloping buttress to 18m below crest. Figure 1 (below) shows the basic embankment structure at the time of the Loma Prieta earthquake (from Harder et al 1996).

Figure 2 shows the dam in plan.

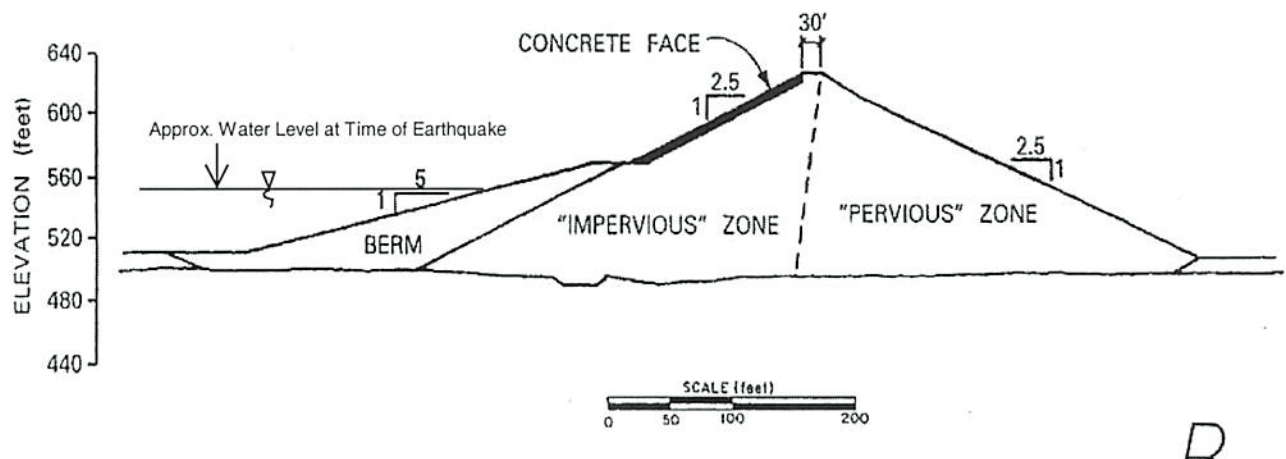


Figure 1 Guadalupe Dam

At the time of the earthquake, the reservoir level was some 24 metres (78 ft) below the crest. However, the reservoir had been full up to about 3 months before the quake and therefore, according to many authors, the embankment could be considered to be saturated at the time of the quake (Harder et al 1996).

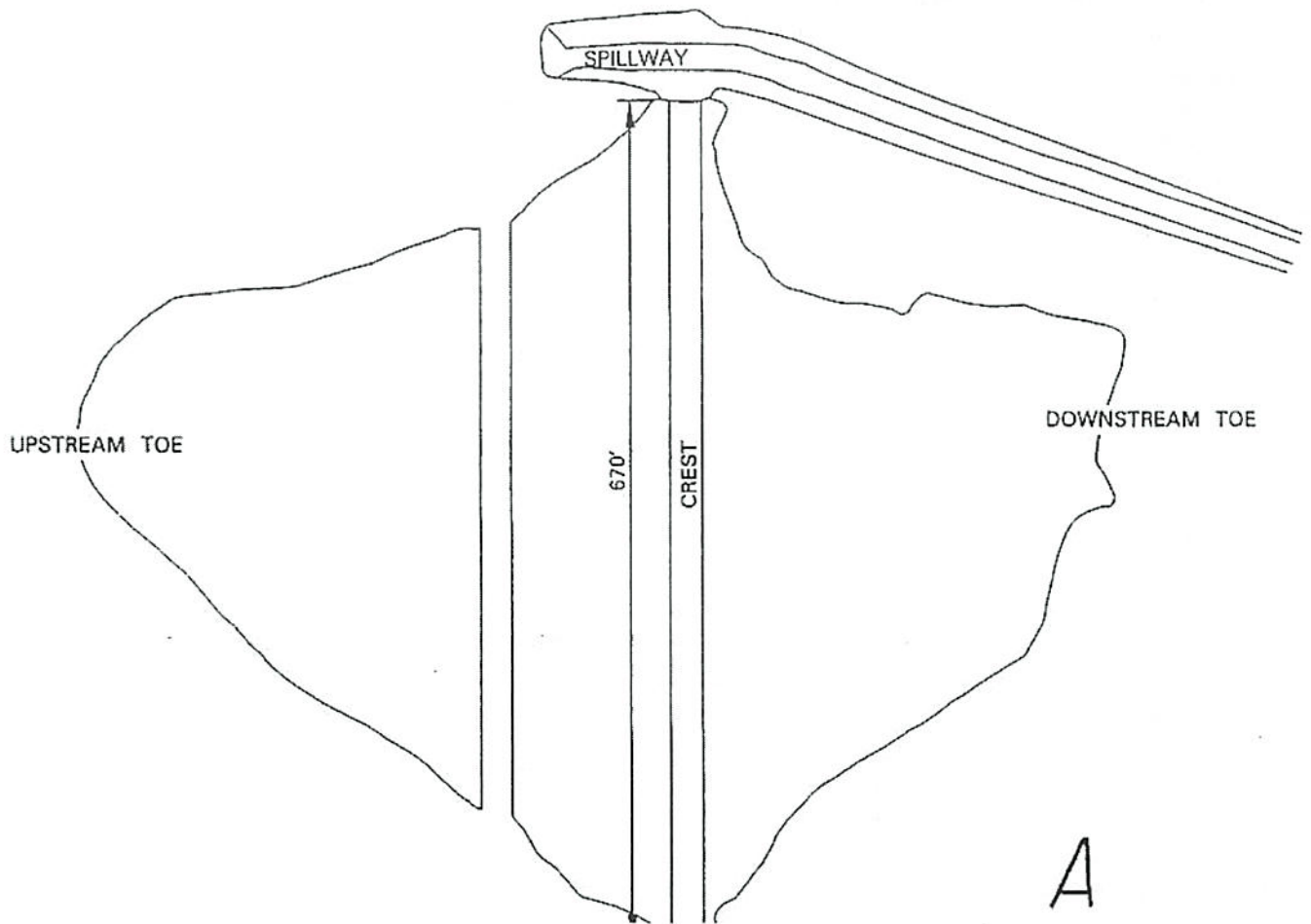


Figure 2. Guadalupe Dam in Plan

Foundation Geology / Materials

None of the references give details on the foundation conditions of Guadalupe dam, except that Swaisgood (1995) records Guadalupe as having no alluvium.

Effects from Earthquake

Damage to the Guadalupe dam from the Loma Prieta earthquake included settlements, deformations and minor longitudinal and transverse cracking.

Harder et al (1996) report **settlement** of up to 200mm on the embankment, which corresponds to a total settlement of 0.45% of the embankment height. The embankment also suffered **deformations** including a 45mm lateral displacement of the crest in the upstream direction.

Minor **longitudinal cracking** occurred on the crest and on the upstream slope, on the upper portion of the buttress. The cracks on the crest were mostly on the downstream slope and within 600mm from the crest (Foster and MacDonald 1997). The cracks on the buttress extended over the length of the dam were recorded after the quake as having a maximum width of 25mm. However, by 5 weeks later these cracks had widened to 100mm and were recorded to extend to a depth of 1.4 metres (Harder et al 1996). Two explanations for the location of these cracks have been given. Firstly, it is possible that the change in geometry resulted in stress concentrations that induced the cracking. Secondly, the placement and dead weight of the berm may have caused settlements on the slope and thus cracking. The past settlements may have formed internal cracks which only surfaced under the earthquake loads (ibid).

Harder et al (1996) also report minor **transverse cracking** at the crest at the abutment contacts.

Minor cracking, spalling and offset occurred in the concrete panels due to the panels pounding against each other under the earthquake movements (ibid).

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La Villita Dam

(Various Earthquakes)

Dam Details

Location	Mexico
Type	Central Core Earth and Rockfill
Height (m)	60
Crest Length (m)	427
Year Built	1964-1968
Owner	
Foundation Geology	Alluvial deposits of up to 70m boulders, gravels, sands and silts
Design Freeboard (m)	3

Special Features of Dam

Flood damage during construction resulted in an area near the centre of the embankment needing repair. The fill used for repair was more compressible than the fill that existed before the flood damage. Consequently, then dam has a feature of inhomogeneous compressibility (Elgamal *et al* 1990) Instrumentation including three strong-motion accelerometers and inclinometers were installed during construction (Resendiz *et al* 1982).

Earthquake Details

The La Villita Dam has been subject to eight significant earthquakes since 1966. Of these, only two have produced any significant deformations and cracking. This case study focuses on those since 1975.

Earthquake No. 1

Name	Mexico		
Date	11 October 1975		
Magnitude (M _s)	4.9		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	52 [⊥]	63 [*]	82
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.35g [⊥]	0.07g [⊥]	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	None / Slight, class 0		

^{*} from USGS database

[⊥] as presented in Elgamal et al 1990

Earthquake No. 2

Name	Mexico		
Date	15 November 1975		
Magnitude (M_s)	5.9		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	10 [⊥]	33 [*]	35
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.21g [⊥]	0.04g [⊥]	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	None / Slight, class 0		

Earthquake No. 3

Name	Guerrero		
Date	14 March 1979		
Magnitude (M_s)	7.6		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	121 [⊥]	49 [*]	131
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.40g [⊥]		
Freeboard at Time of Earthquake (m)	10.5		
Dam Damage Descriptor	Minor, class 1		

Earthquake No. 4

Name	Playa Azul		
Date	25 October 1981		
Magnitude (M_s)	7.3		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	31 [⊥]	28 [*]	42
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.43g [⊥]	0.09g [⊥]	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	None / Slight, class 0		

^{*} from USGS database

[⊥] from Elgamal et al 1990

Earthquake No. 5

Name	Michoacan		
Date	19 September 1985		
Magnitude (M_s)	8.1		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	58 [⊥]	29 [*]	65
Peak Ground	<i>Crest</i>		<i>Foundation</i>
Acceleration (g)	0.79g [⊥]	0.45g [*]	0.12g [⊥]
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	Moderate, class 2		

Earthquake No. 6

Name	Michoacan Aftershock		
Date	21 September 1985		
Magnitude (M_s)	7.5		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	61	42 [*]	74
Peak Ground	<i>Crest</i>		<i>Foundation</i>
Acceleration (g)	0.21g [⊥]	0.16g [*]	0.04g [⊥]
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	Minor, class 1		

^{*} from USGS database

^{*} from USCOLD, 1999

[⊥] from Elgamal et al 1990

Embankment Details

La Villita dam is a symmetrical central earth-core rockfill dam. Figure 2 shows an embankment section at maximum section height and gives brief descriptions of all units. Further properties of these material units, where available, are presented in Table 1.

Figure 1 shows a plan view of the embankment. Although located in a deep narrow gorge, extensive alluvial filling results in a dam height of only 60m (see figure 3). To control seepage through the alluvium, a concrete cutoff wall was constructed from under the core through the alluvium down to the bedrock (74m metres in total). In addition to this, the alluvium behind the wall was grouted to a depth of 26 metres. This grouting was the reinforce the wall and provide a 'second line of defense'. It was also designed to reduce settlement under the core (Resendiz et al 1982).

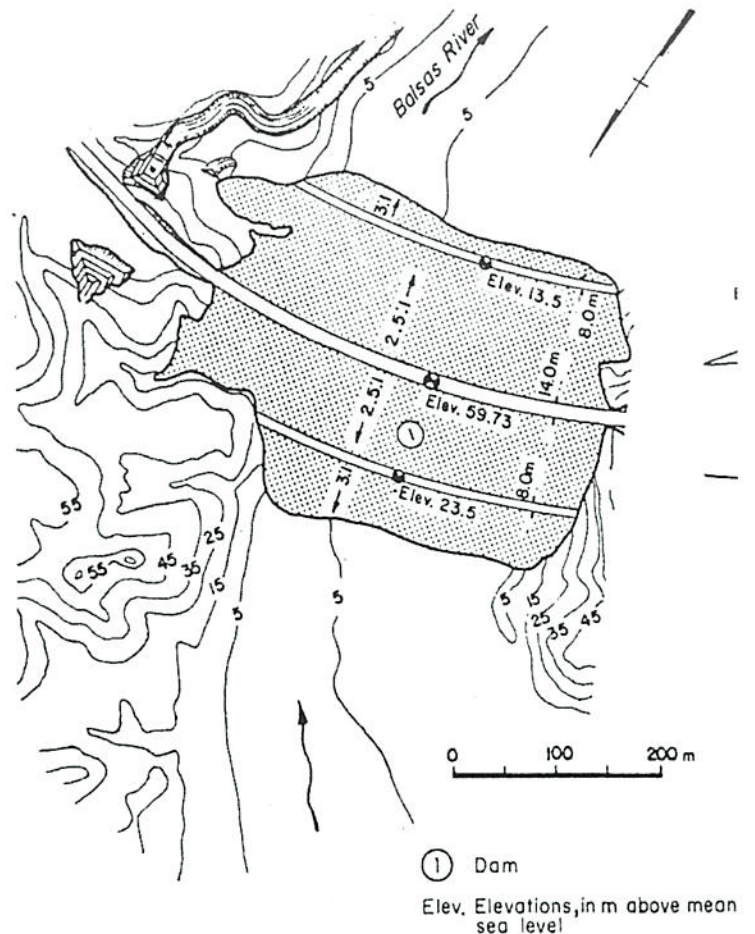
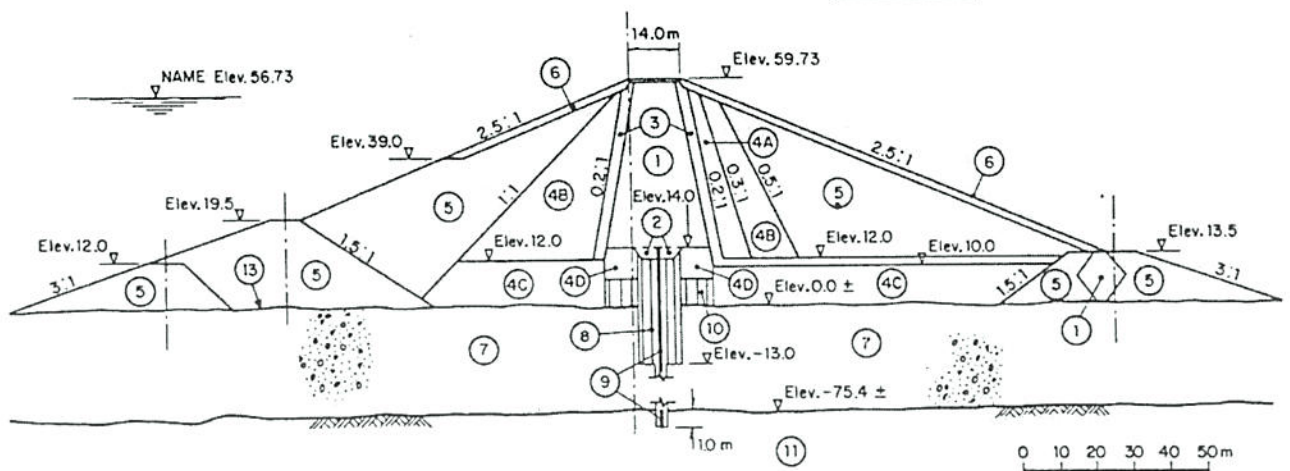


Figure 1



- | | |
|------------------------------------|-----------------------------------|
| 1 Compacted impervious material | 7 Alluvium (gravel and sand) |
| 2 Highly plastic clay | 8 Grout curtain |
| 3 Sand filters | 9 ICOS-type concrete cut-off wall |
| 4A Well graded gravel and sand | 10 Consolidation grouting |
| 4B Gravel and sand | 11 Andesitic breccia |
| 4C Dumped gravel and sand | 12 Dam axis |
| 4D Compacted gravel, sand and muck | 13 Original ground surface |
| 5 Rockfill | Elev. Elevation, in m |
| 6 Selected rockfill | NAME Maximum water level |

Figure 2

The dam was built in two stages, with construction beginning in the dry on the right bank between stations 0+300 and 0+490 (see figure 3). Construction of the left bank followed this, and began with dumping soil underwater between stations 0+150 and 0+300. During this, a large flood event occurred, washing away much riverbed material. Material used for repairs was much more compressible. As a result, the dam has a feature of inhomogeneous compressibility.

Foundation Geology and Design

The dam is founded on a deep alluvial layer of up to 70 metres thick comprising boulders, gravel, sand and silt, which in turn overlies a bedrock of

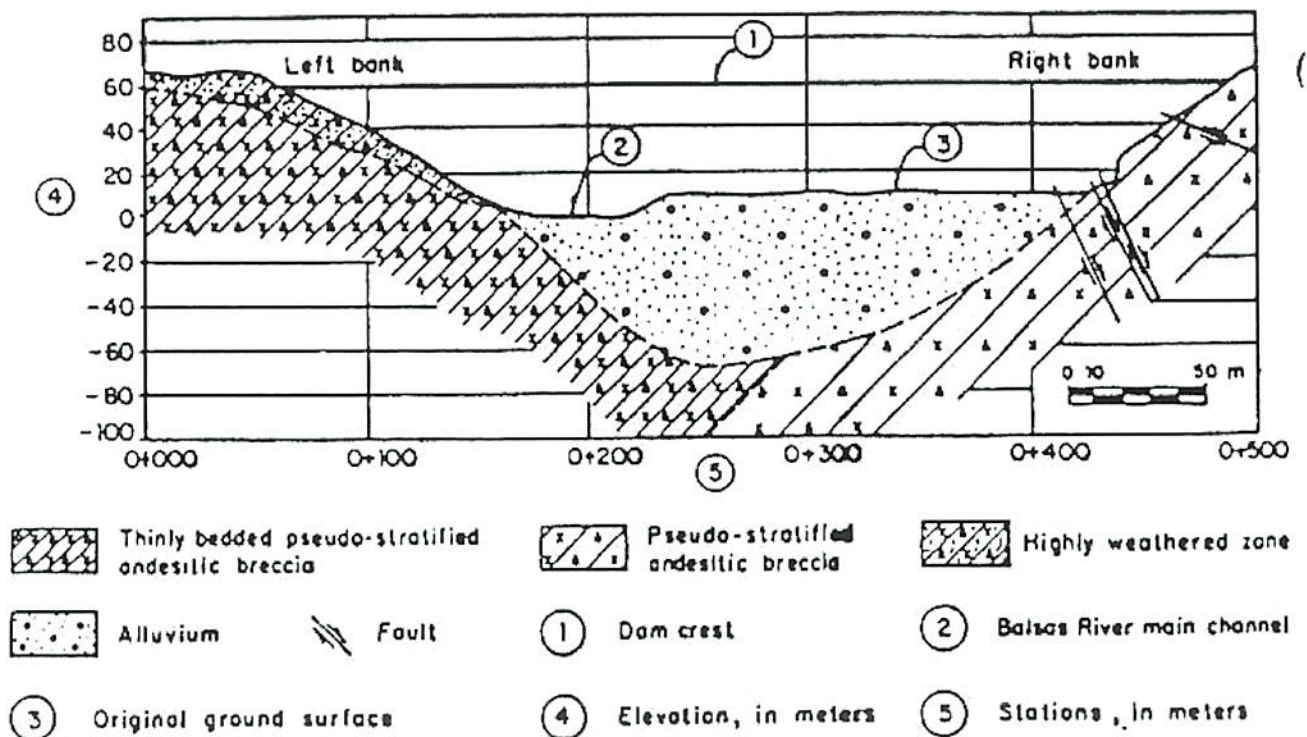


Figure 3

Effects of Earthquakes

The extensive instrumentation installed at La Villita has allowed close monitoring of the embankment since construction and throughout the many earthquakes. Data on the settlement that has occurred from construction until the September 1985 is shown in figure 3 a, b and c. Elgamal et al (1990) report that in all earthquakes, there have been evident 'localised zones of low strength and differential settlements'. They attribute this to: the variable thickness of the alluvium; the inhomogeneous compressibility (see 'Embankment Details' above); and, the zoned cross section.

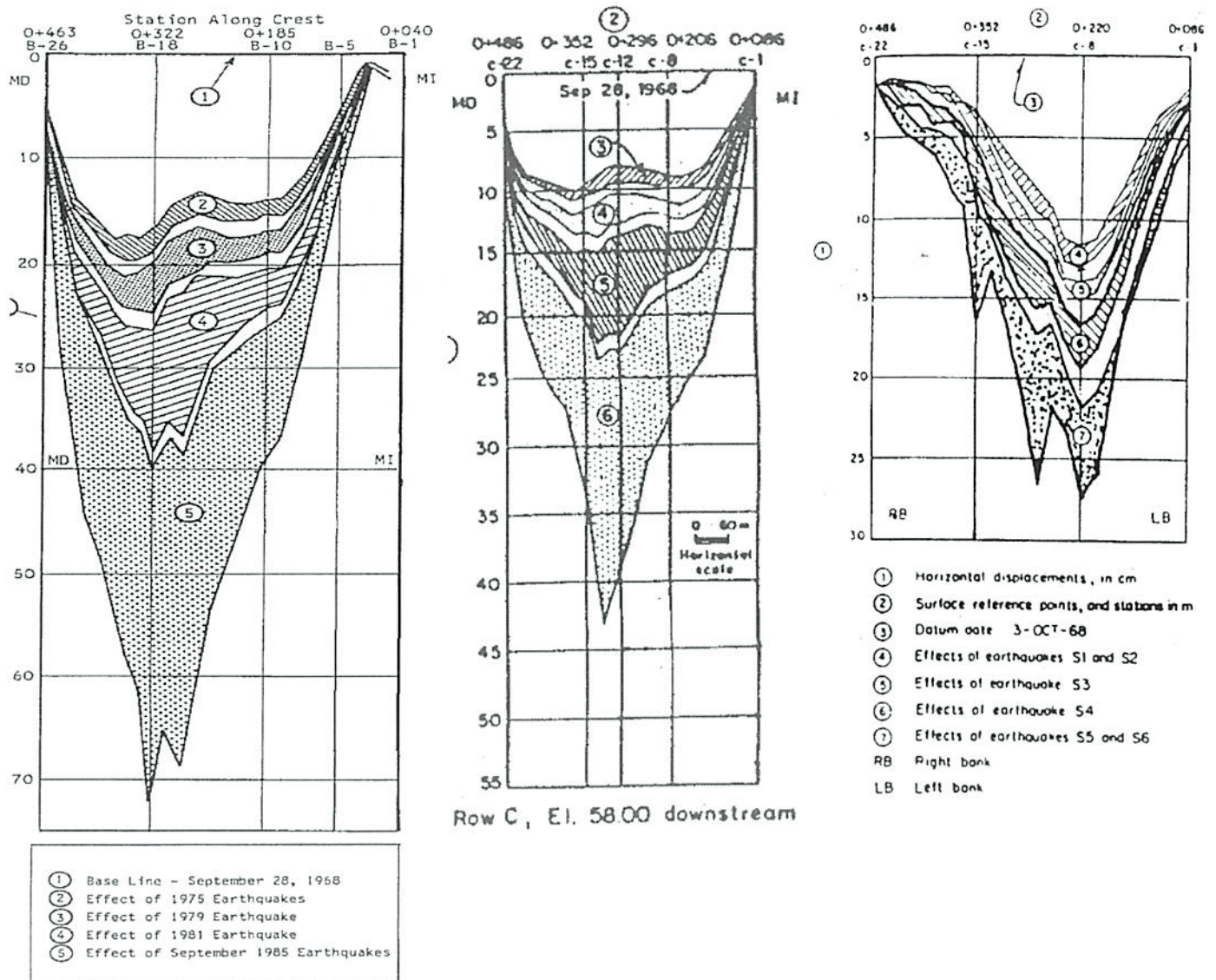


Figure 3 a b and c Showing Displacements in centimeters. A: Vertical Displacement Upstream; B: Vertical Displacements Downstream; C: Horizontal Displacements

Effects of the 1975 Mexico Earthquakes

No significant effects. Slight temporary increase in rate of settlement. Crest horizontal displacement of 16mm reported for both quakes (Succaarieh *et al*, 1993).

Effects of the 1979 Guerrero Earthquake

La Villita dam suffered minor damage as a result of the Guerrero earthquake. **Settlements** of up to 50 mm were recorded on the crest. **Longitudinal cracking** along the crest including one 3 mm in width and 84m long and another 5 mm in width and 39m long. Some **transverse cracking** up to 6 m long width was found adjacent to right abutment. A horizontal displacement of 11 mm was measured on the crest.

Resendiz *et al* (1982) state that the main types of deformation and damage were associated with compaction and shear distortion of the embankment i.e. see Figure 4. They roughly estimate that 17% of the settlement of La Villita dam was due to lateral spreading.

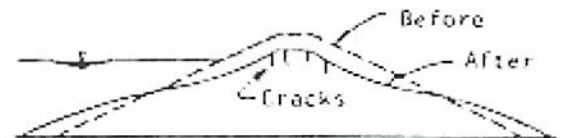


Figure 4

Figure 5 shows locations of cracking on the dam after the Guerrero earthquake.

Resendiz *et al* posit that the distribution of longitudinal cracks align well with the distribution of settlements (see figure jdjjdj).

No transverse cracking is reported by Resendiz *et al* (1982). They suggest that the absence of transverse cracking is due to the relatively small settlements incurred, and the gentle valley profile of the site.

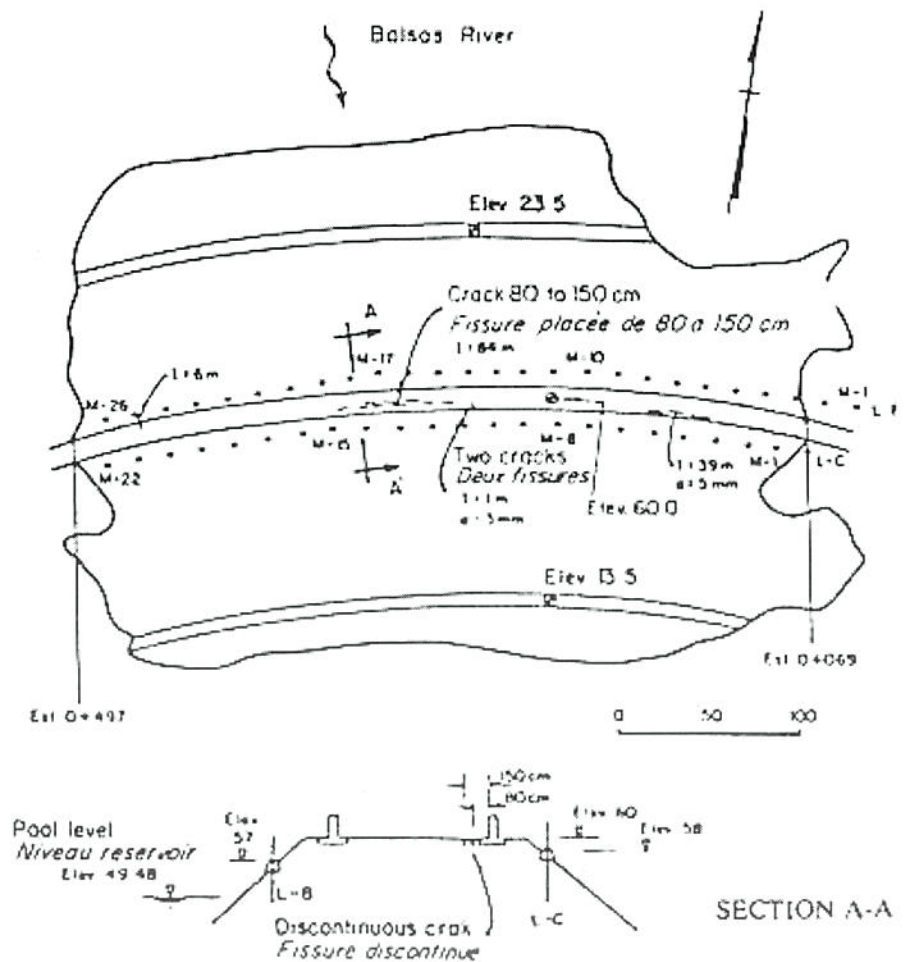


Figure 5

SECTION A-A' (not to scale)

(L-B, L-C) Rows of surface reference points

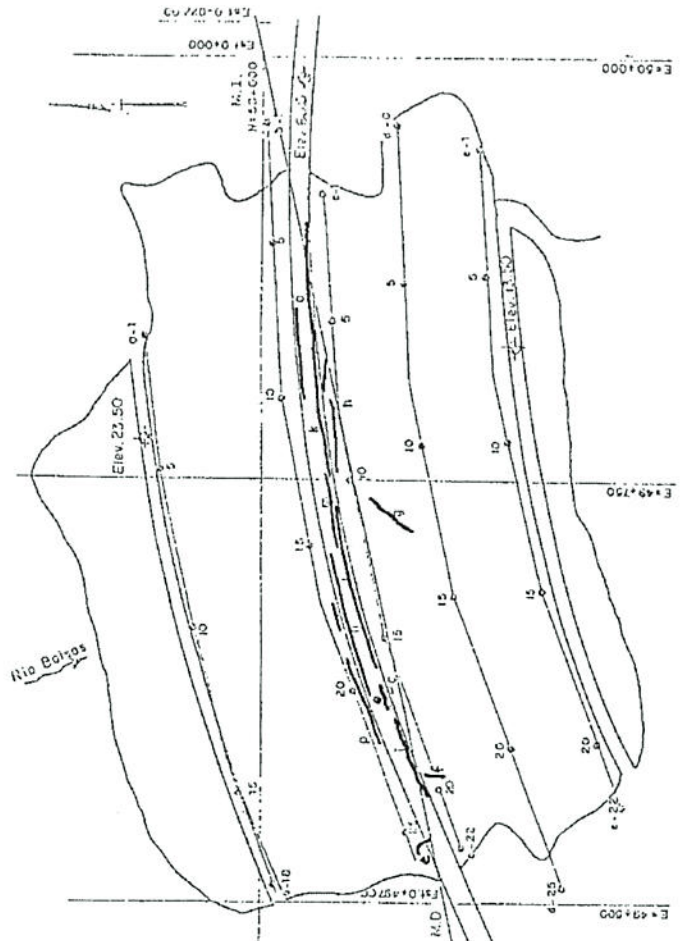
- (l) Crack length, in m.
- (M-i) Bench marks.
- (a) Crack opening, in mm.
- (Elev) Elevation in m.

Effects of the 1981 Playa Azul Earthquake

No significant effects reported, except a crest horizontal displacement of 29 mm. (Succaarieh *et al*, 1993).

Effects of the 1985 Michoacan Earthquake

The dam suffered minor to moderate damage from this earthquake. Damage included cracking, settlement and spreading (ICOLD, 1999). Maximum **settlement** of 320 mm was recorded on the center of upstream side, and 220 mm on the center of the downstream. There were only small settlements adjacent to abutments. **Spreading** of the base resulted in horizontal movements of 100 mm and 165 mm for the downstream and upstream toes respectively. Two main systems of **longitudinal cracks** developed within 5 m of the crest on either side, including one up to 50 mm wide, 1.5m deep and 80m long on the upstream side and another 15 mm wide and 300 m long on the downstream side. The crack on the upstream side also featured a scarp of 15-50mm. Additional minor longitudinal cracks of 50 mm width were also found. Some transverse cracking was found adjacent to the right abutment. The cracking details for this event are depicted in figure 6. ICOLD (1999) report a crest **horizontal displacement** of 55 mm.



Of the cracking pattern, Bureau (1986) comments on close similarities with Leroy Anderson dam in California after the Morgan Hill earthquake.

Because aftershocks are often classed as being the same earthquake as the primary shock, few authors differentiate between damage from each of the shocks. The short time period between events also offers little time to do surveys and damage mapping after individual events. Swaisgood (1995) presents a settlement of 120mm for the aftershock event, but many authors write that there was no further damage from the aftershock. Differentiation between the two event is unclear and thus damage described for this event is considered to be from a combination of both quakes.

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Leroy Anderson Dam

(Morgan Hill Quake, 24 April 1984; Loma Prieta Quake, 17 October 1989)

Dam Details

Location	California, U.S.A.
Type	Central Core Earth and Rockfill
Height (m)	72.5 (pre 1984) 73.5 (post 1987) (71.63 from USGS)
Crest Length (m)	435.9
Year Built	1950
Owner	Santa Clara Valley Water District
Foundation Geology	shale and sandstone
Design Freeboard (m)	4.6 (total)

Special Features of Dam

The dam was built primarily for water conservation purposes. The Anderson reservoir, which it impounds, also serves as a backup for Coyote Dam which lies upstream. Coyote Dam lies on the Calaveras fault and thus has a high seismic risk. Anderson reservoir must always maintain sufficient storage for the Coyote water, were the Coyote Dam to fail. This requirement came into action in 1963 (USCOLD 1988).

The Leroy Anderson Dam also serves as flood control. According to Harder et al (1996), the embankment was raised by about 1.5 metres in 1987 to accommodate larger floods.

Earthquake Details

Earthquake No. 1

Name	Morgan Hill		
Date	24 April 1984		
Magnitude (M_s)	6.2		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	16	8	17.9
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.63g*	0.41g*	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	No or slight / Minor, class 0/1		

Special Features of Earthquake

The peak ground acceleration at the dam is greater for the Morgan Hill quake than for the Loma Prieta quake. However, the duration of strong shaking in the Morgan Hill quake was less than in the Loma Prieta quake (Harder et al 1996).

* Dam instrumented with seismographs. Values as presented in Harder et al

Earthquake No. 2

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	26.9	18.5	32.6
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.43g*	0.26g*	
Freeboard at Time of Earthquake (m)			
Dam Damage Descriptor	Minor, class 1		

Special Features of Earthquake

The duration of shaking was generally between 7 and 10 seconds (12 seconds according to Foster) which is about half of that expected for an event of this magnitude (Harder *et al* 1996). Despite this, the duration of shaking was still longer than for the Morgan Hill quake.

Embankment Details

According to USCOLD (1988) the embankment was designed and constructed to account for seismicity (although it appears to have no conventional filters).

Leroy Anderson dam is composed of central core zones of compacted sandy clay and clayey sand that are flanked by shell zones of rockfill. The earthfill construction is 'rolled' (USCOLD 1988). The rockfill was placed by end dumping from trucks into 10 – 25 foot lifts that were compacted by sluicing with water (Harder *et al* 1996). A basic section of the dam is shown in Figure 1 below.

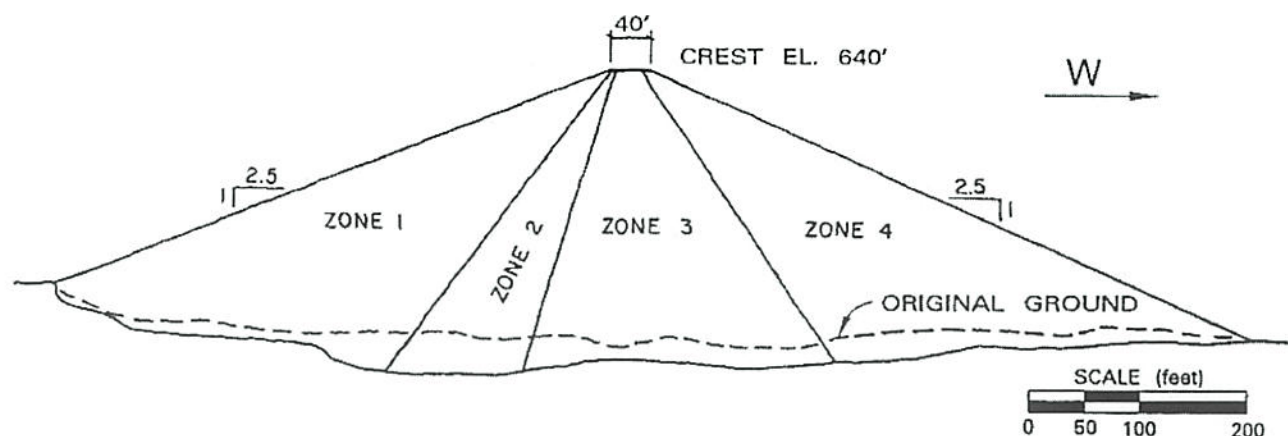


Figure 1 Leroy Anderson Embankment Section (Upstream Left Hand Side)

Zone 1 and **Zone 4** are the rockfill shells, using rock taken from two different quarries near the dam site. **Zone 2** is an 'impervious clay core', which used materials taken from the excavations of the foundation and spillways (USCOLD 1988). **Zone 3** is described by Forster and MacDonald (1998) as a 'compacted semi-pervious sand and gravel zone'. This material was also taken locally and included 15% gravels, 32% sands and 53% fines by weight (USCOLD 1988).

* Dam instrumented with seismographs. Values as presented in Harder *et al*

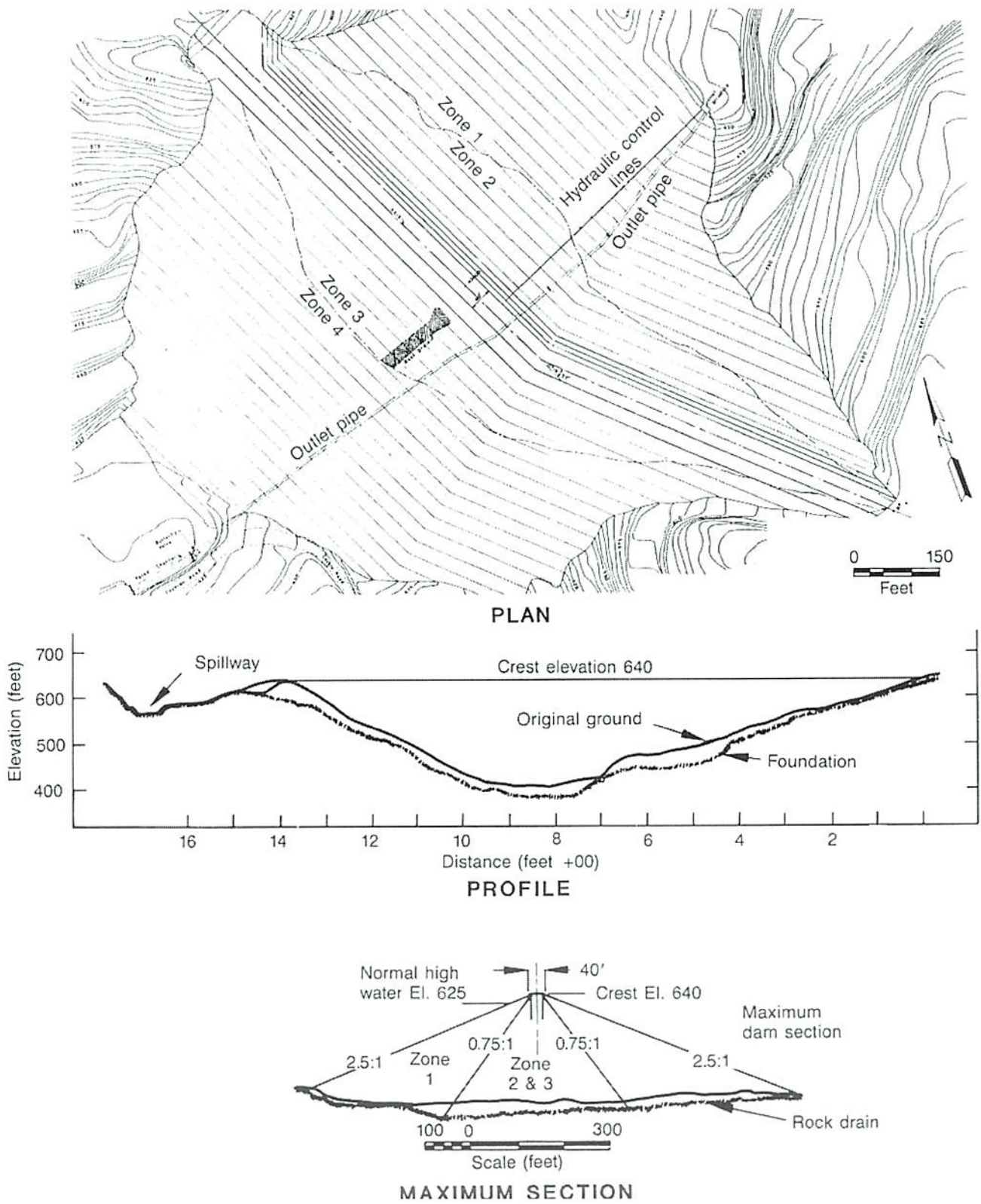


Figure 2 a, b and c Leroy Anderson Plan, Elevation and Maximum Section

Foundation Geology / Materials

The dam is set in a long deep natural gorge on Coyote creek (see photo 1 below) which runs through the Santa Clara Valley District (USCOLD 1988). The Santa Clara Valley is composed mostly of igneous Franciscan rocks. This formation has largely been altered to serpentine, and it extends from the upstream toe of the dam and to the west. East of the dam (upstream), the foothills comprise mostly 'poorly consolidated gravels, sands and clays of Pliocene lakebed origin' (ibid). The reservoir trough is partly filled with poorly consolidated sedimentary deposits and scant remainders of a thin lava flow (ibid).

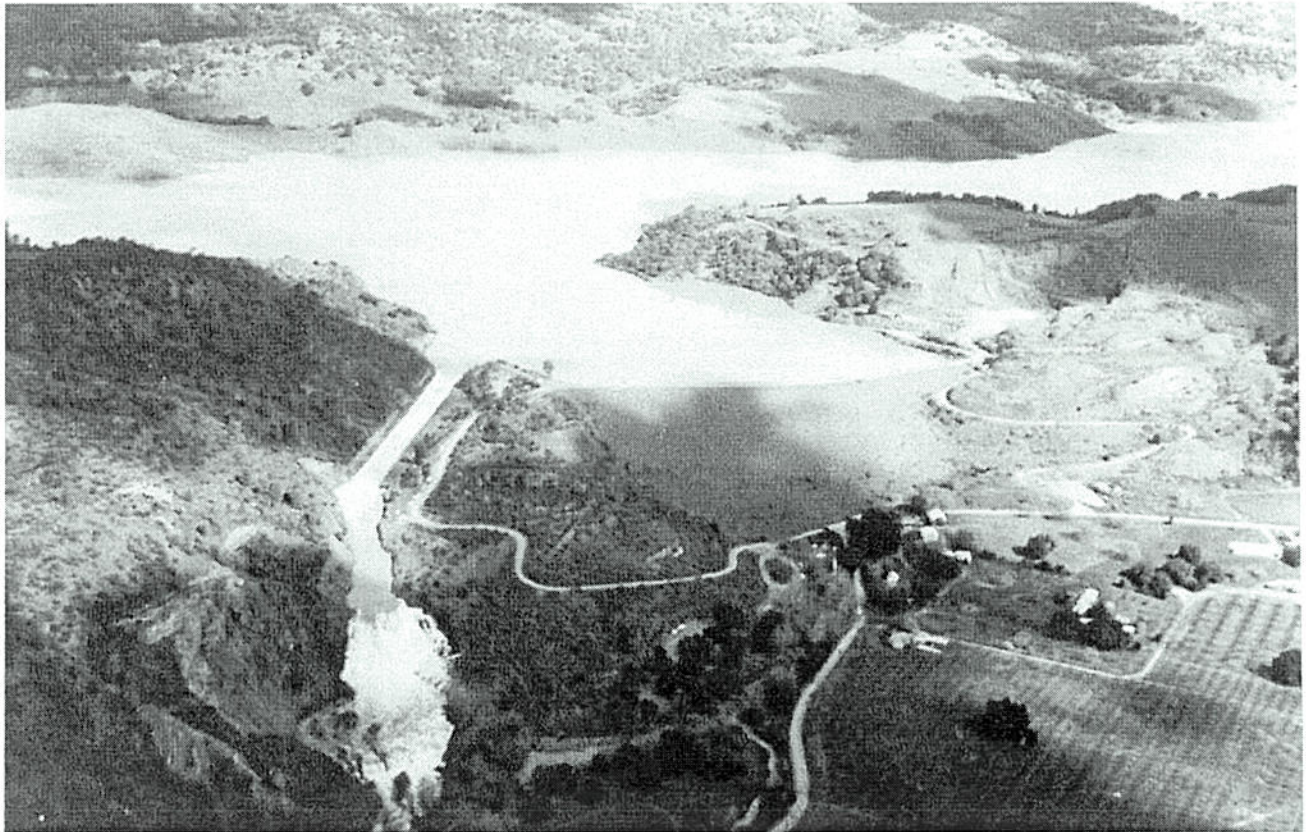


Photo 1 Leroy Anderson Dam and Surrounds

Geology on the left abutment included shale and sandstone outcrops, and on the right abutment included components of serpentine, shale and sandstone. To prepare the foundation extensive excavations were performed to remove the fractured sandstone and fault zones from the bedrock, and expose more 'massive' sandstone. Any remaining cracks were grouted using cement slurry at high pressure through bores in the sandstone. (USCOLD, 1988). According to Swaisgood (1995) there are no alluvial layers left present under the embankment.

Cracking From Earthquake

Morgan Hill Quake

Swaigood (1995) documents a crest **settlement** of 15mm (corresponding to a 'normalised' settlement of 0.02%) after the Morgan Hill earthquake. This settlement is assumed to have occurred within the embankment, as he records that no alluvium underlies the embankment.

Two relatively shallow parallel systems of **longitudinal cracks** about 25 mm wide formed along the crest. The location of the cracks was approximately above the interface between rockfill shells and earth core. Thus, the crack formation is attributed to differential settlement between the well compacted clay core and loosely placed rockfill shell zones (Harder et al 1996). Seven backhoe trenches were used to investigate the crack systems. Basic logs of these trenches are shown in Figure 3 below (from Forster and MacDonald 1998).

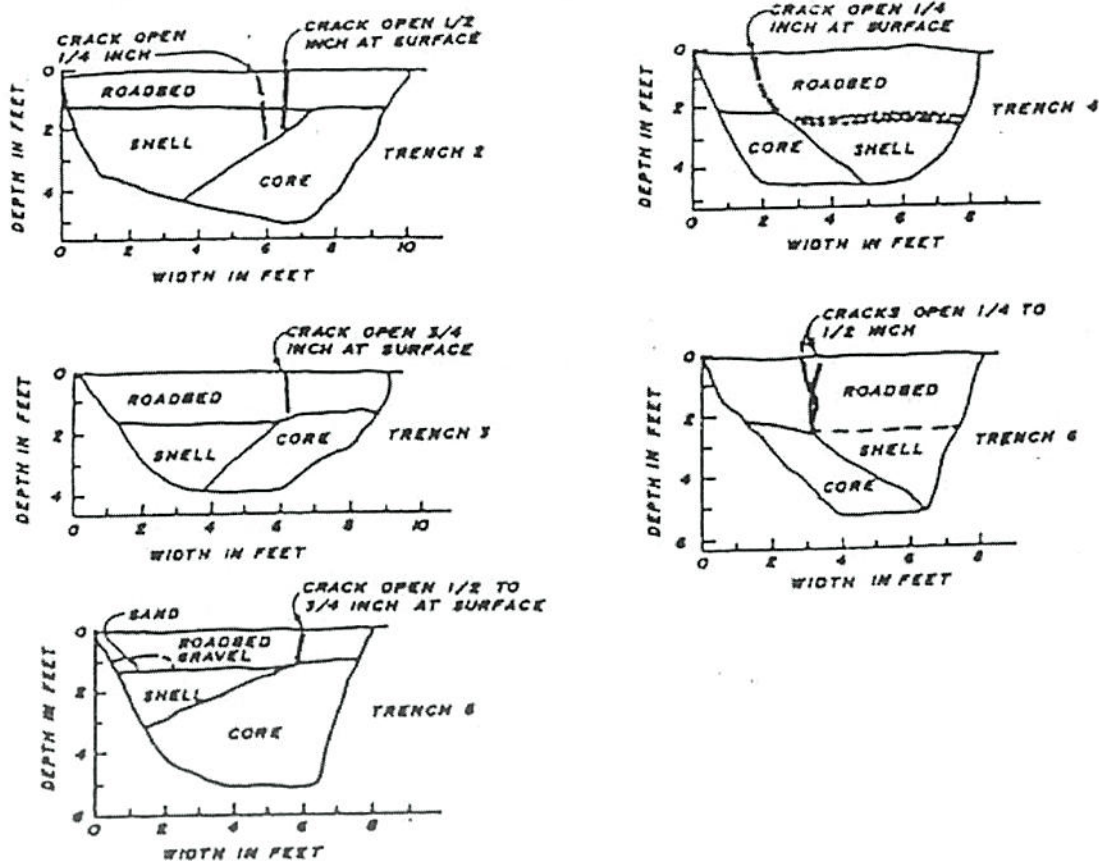


Figure Cracking Found in Test Pits After the Morgan Hill Earthquake

Loma Prieta Quake

Although the peak acceleration at the crest was larger for the Morgan Hill earthquake, the duration of strong shaking was less than that of the Loma Prieta earthquake. According to Harder et al (1996), damage was very similar as for the Morgan Hill earthquake, with only minor damage. **Settlements** of up to 40 mm were recorded (0.06% of the dam height). Minor **longitudinal cracking** occurred to a depth of 0.6m, with maximum width less of than 20mm again above the contact between earthfill and rockfill shells. The crack formation is attributed to differential settlement between the well compacted clay core and loosely placed rockfill shell zones (Harder et al 1996).

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Lexington Dam

(Loma Prieta Quake, 17 October 1989)

Dam Details

Location	California, U.S.A.
Type	Zoned Earthfill
Height (m)	62 (204ft)
Crest Length (m)	246 (807ft)
Year Built	1953
Owner	Santa Clara Valley Water District
Foundation Geology	No alluvium
Freeboard at Time of Earthquake (m)	30
Design Freeboard (m)	-

Special Features of Dam

The upstream zone was originally designed to be pervious, but a specification change, passed during construction, allowed a random zone containing predominantly impervious materials. An attached spillway runs through the left abutment. The outlet works run under the embankment.

Earthquake Details

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1 M_s (USGS Database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	21*	18.5	28
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.45g*	0.45g*	
Dam Damage Descriptor	Moderate / Minor, class 2/1		

Special Features of Earthquake

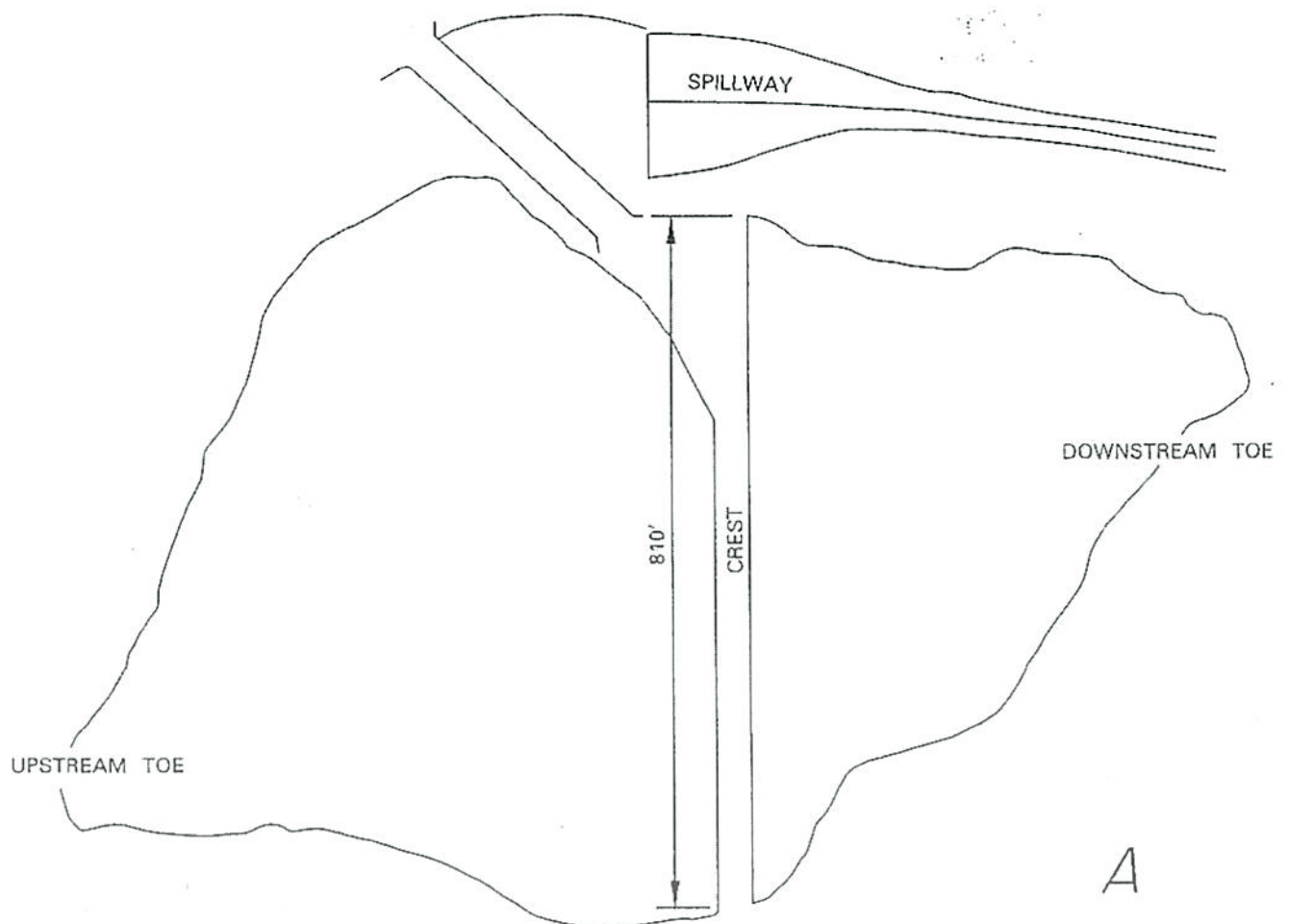
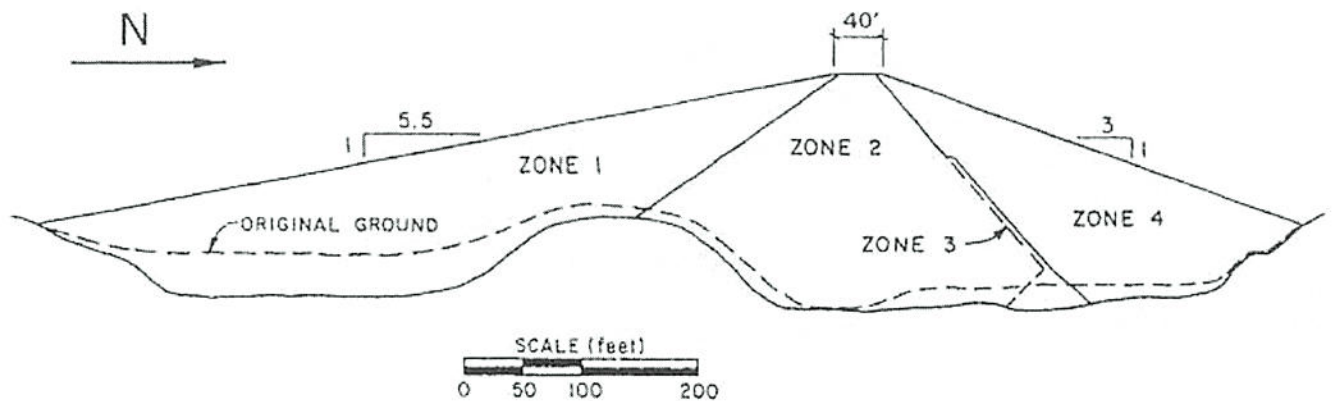
The duration of shaking was generally between 7 and 10 seconds, which is about half of that expected for an event of this magnitude (Harder *et al* 1996).

* USCOLD, 1992. Harder *et al* note that the dam was only 3km from the fault line.

Embankment Details

The dam contains an upstream impervious zone (zone 1), downstream semi-pervious zone (zone 4) and a thick central impervious core (zone 2). A narrow filter zone is placed downstream of the core (zone 3).

Zone 2 is described by Harder *et al* (1996) as a gravelly and sandy clay core, and zones 1 and 4 are described as random shell zones of clayey sands and gravels. Foster and MacDonald (1998) explain that zone 1 was originally intended to be a pervious zone, but specification change resulted in it becoming a random zone containing impervious materials.



Foundation Geology / Materials

No specific details on the foundation geology were found. However, Foster and MacDonald (1998) mention in passing that natural ground at the right abutment comprises “colluvium and highly weathered rock”. According to Swaisgood (1998), there is no layer alluvium left under the embankment.

Cracking From Earthquake

Harder *et al* (1996) report that the earthquake produced longitudinal cracking on both the upstream and downstream faces and transverse cracking at both abutments. They state that cracks were reasonably isolated and extended to depth of 0.6 to 2 metres.

Transverse cracking was found occurred at the abutments in both the embankment and the natural ground. On the left embankment, cracks were confined to the downstream side, on the right embankment, cracking only occurred on the natural ground. On the left abutment, transverse cracking, discontinuous over a length of 58 metres, was reported to be 38mm wide and 1.7 m deep (Fong and Bennett, 1995; Forster and MacDonald, 1998). Bureau *et al* (1989) write that these cracks at the left abutment were confined to the downstream groin between 3 and 30 metres below the crest, and Forster and MacDonald believe the cracking appears to be controlled by steep abutment-embankment contact.

At the right abutment transverse cracking was about 10mm wide and 1.1 metres deep (Fong and Bennett, 1995; Forster and MacDonald, 1998) and occurred in both upstream and downstream directions. Forster and MacDonald (1998) state that this cracking on the right abutment was confined to natural ground to the right of the embankment.

Cracking oriented obliquely to the crest was also found on the crest at approximately 45 metres in from the left abutment (Harder *et al* 1996).

Foster and MacDonald (1998) describe the **longitudinal cracking** as ‘minor cracks of approximately 1 m deep and 10 mm wide’ and state that they appeared on both upstream and downstream faces.

A **settlement** of 260 mm (0.41% dam height) was recorded on the crest at the dams maximum section, and the embankment experienced a downstream movement of 75 mm (Bureau *et al* 1989).

With settlement of 0.41% of embankment height, and longitudinal crack width of about 10mm, this places the dam damage into class 1 / 2 “minor to moderate”.

Approximately 6 weeks after the earthquake event, a large damp area (about 50m long and 10m wide) developed on the downstream face of the dam. The cause of the **seepage** is unverified, but has been attributed by some authors to the presence of old exploration holes, which acted as relief well under the higher earthquake induced pore pressures (Harder *et al* 1996; Forster and MacDonald, 1998).

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Los Angeles Dam

(Northridge Quake, 17 January 1994)

Dam Details

Location	California, U.S.A.
Type	Asphalt Face Embankment
Height (m)	47 (155ft)
Crest Length (m)	1018
Year Built	1979
Owner	Los Angeles Department of Water
Foundation Geology	sandstone.
Freeboard at Time of Earthquake (m)	8
Design Freeboard (m)	-

Special Features of Dam

The dam is located in the Van Norman complex (see figure **BBB**), to replace the (now decommissioned) Upper and Lower San Fernando dams. The Los Angeles dam, together with the North Dyke, form the Los Angeles Reservoir, which is the main water supply for the city of Los Angeles.

Earthquake Details

Name	Northridge		
Date	17 January 1994		
Magnitude (M_s)	6.7 [*] , 7.8 [*] M_s		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	10 ^Γ	18 ^Δ	21
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.56g [∇]	0.32g [∇]	
Dam Damage Descriptor	Minor / Moderate, class 1/2		

Special Features of Earthquake

Maximum peak accelerations of 1.00 g were recorded just 2 km from the Los Angeles dam. Tectonic movements associated with the earthquake event resulted in fairly uniform horizontal and vertical movements across the greater region about the embankment (Davis and Bardet, 1996).

^{*}Bardet and Davis (1996), Swaisgood (1995) and Fong (1997) each record a magnitude of 6.7. The USGS database records 7.8 M_s
^Γ calculated from coordinates of dam and epicenter

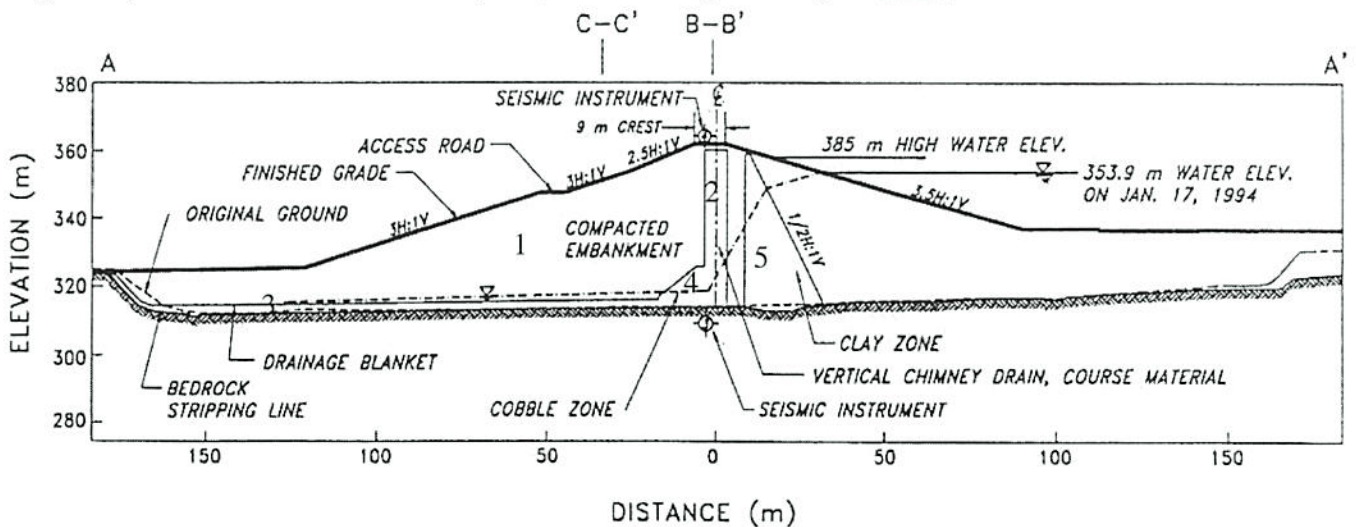
^Δ from USGS database

[∇] from Davis and Bardet, 1995/6

Embankment Details

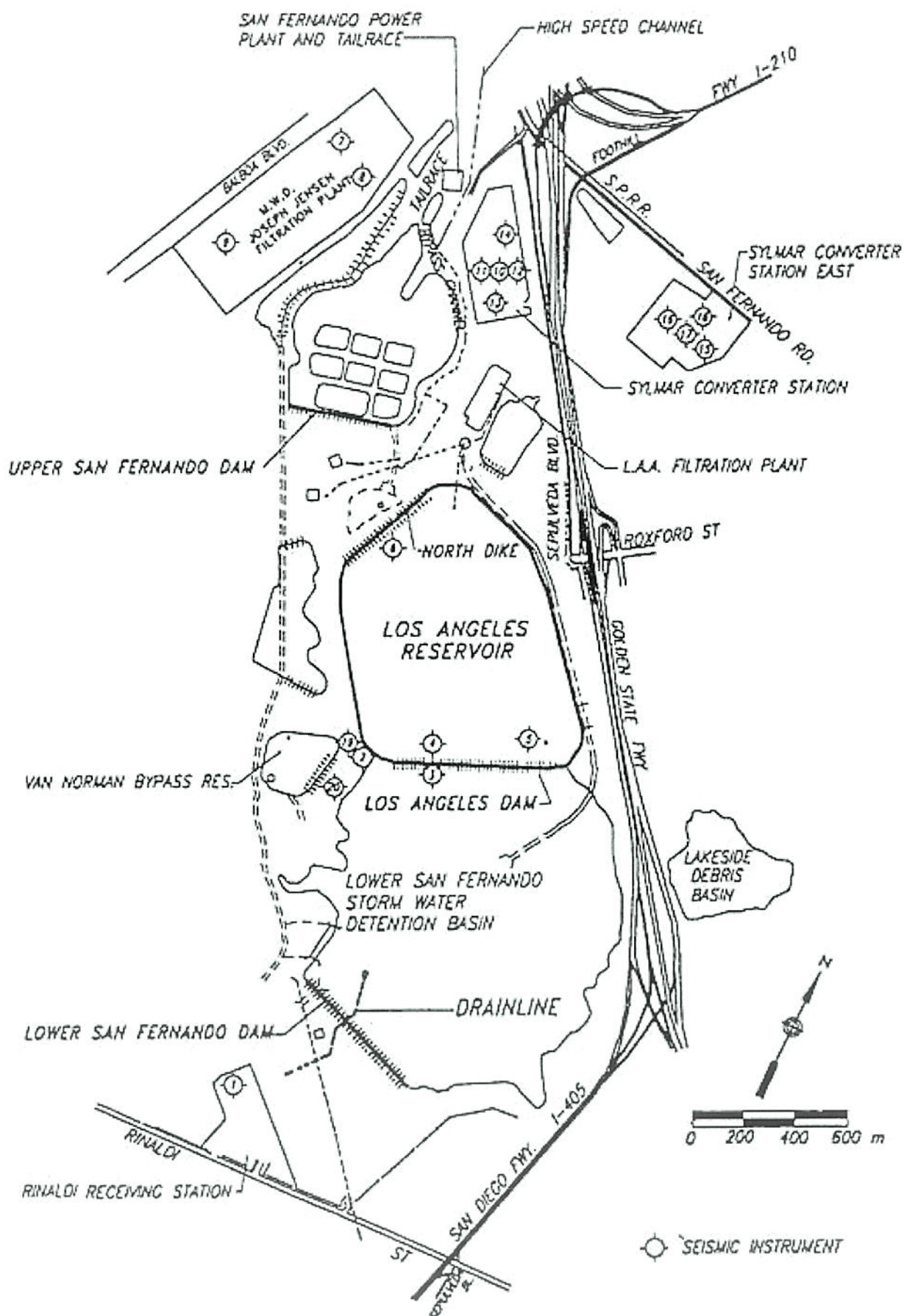
The Los Angeles Dam comprises compacted silty sand shell material on the upstream and downstream slopes (1), a vertical chimney drain (2), a downstream near horizontal drainage blanket (3), a small zone of blended cobbles, gravels and sands (4) located immediately downstream from the lower part of the chimney drain, and a silty clay core (5) upstream from the chimney drain. These units are shown in Figure 1, and basic properties are shown in Table 1. The interior slopes of both embankments are lined with asphalt, but reservoir floor is not lined. (Davis & Bardet, 1996).

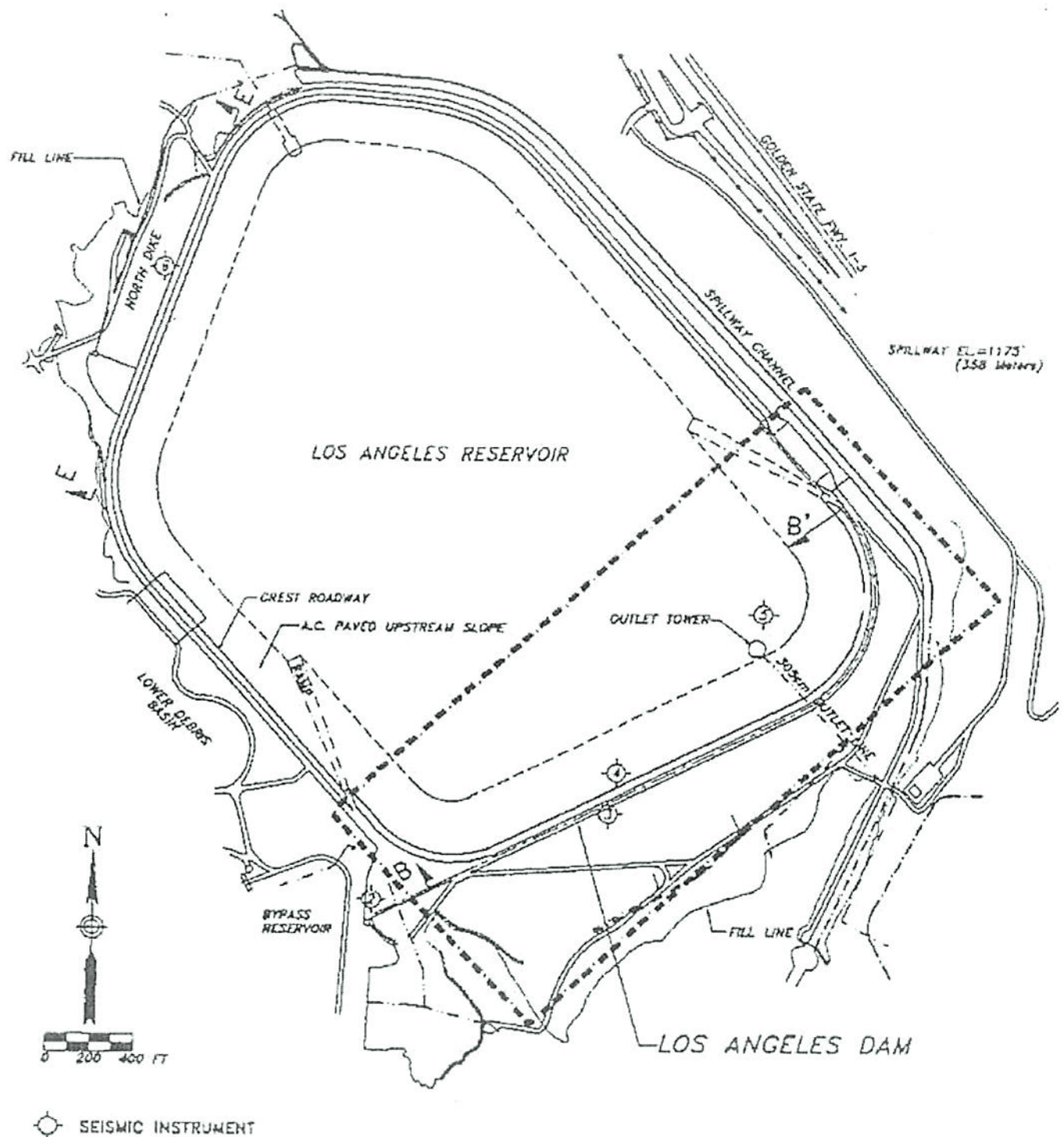
Figure 1 (the location of this section (A-A') is shown in plan in figure (kjhk))



Zone	Name	Description	Properties			Compaction and Placement		Wc %
			γ_d (kN/m ³)	c' (kPa)	ϕ'	Thickness	Compaction	
1	Shells	compacted silty sand	19.2	34 (moist) 16.3 (sat)	37 (moist) 39 (sat)	-	95 to 98 % relative of 33750 ft lbs/ft ³	12
2	Chimney Drain	-	19.5	0	35	-		10
3	Drainage Blanket	-	19.5	0	35	-		10
4	Cobble Zone	small zone of blended cobbles, gravels and sands	21.5	0	38	-	93% modified ASTM D1557 3 layers with compactive effort 1615 kN.m.m ³	10
5	Core clay zone	silty clay	17.5	17.2	26.	-	95% relative of 33750 ft lbs/ft ³	20

Note, - standard Proctor compaction is 12375 ft.lbs/ft³ and modified Proctor compaction is 56250 ft.lbs/ft³ (Holtz and Kovacs 1981)
- compaction procedures were carefully controlled (Bureau et al 1996)





Foundation Geology / Materials

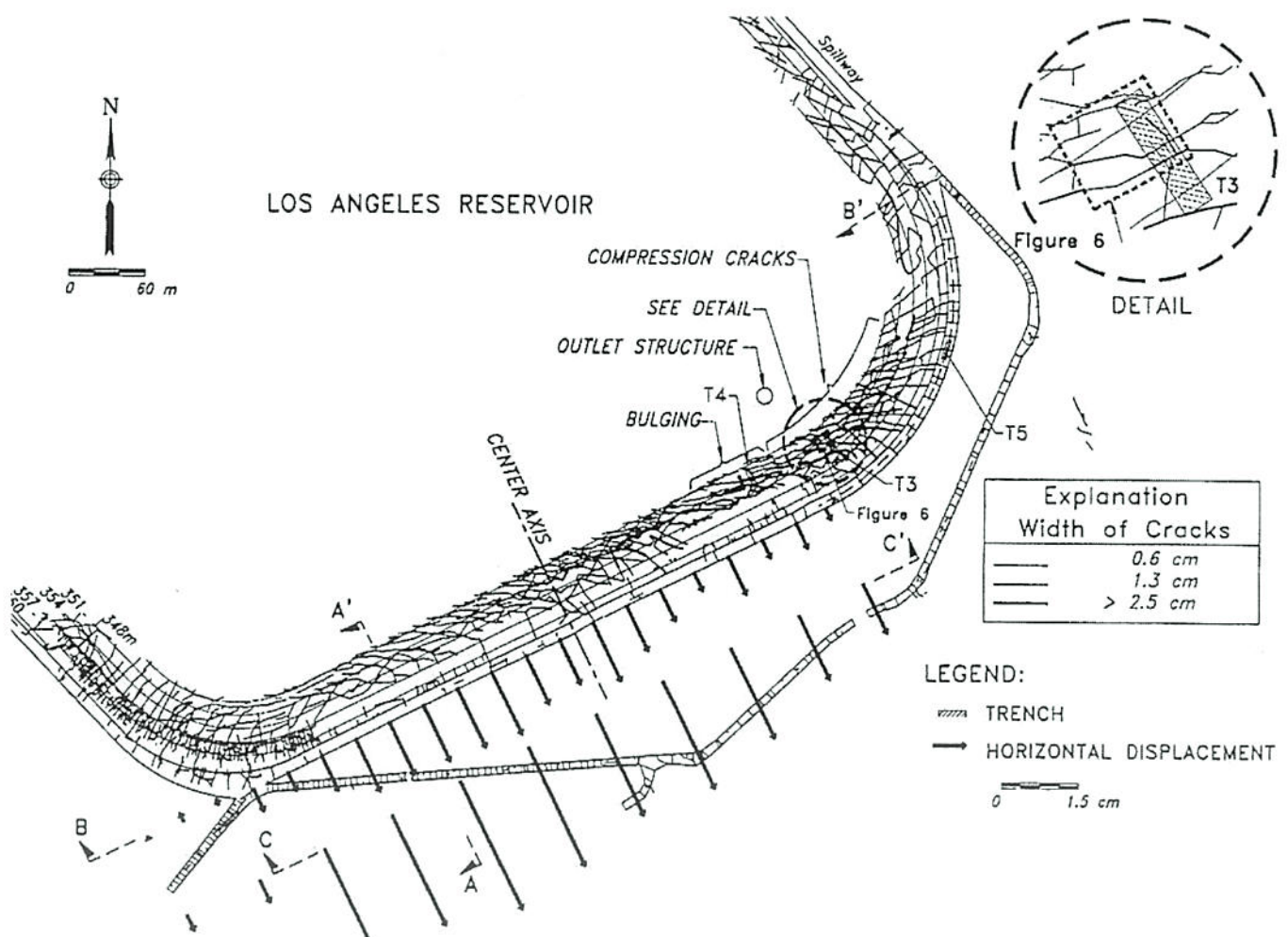
Both embankments are founded on "Saugus formation bedrock", with no alluvial deposits left under the dam (Swaigood, 1998; Davis and Bardet, 1996). ICOLD (1999) describe this formation as comprising claystone, siltstone, sandstone and interbedded pebble-cobble conglomerates. (ICOLD 1999).

Effects of the Earthquake

Tectonic uplift caused horizontal and vertical movements over the whole Van Norman complex. The vertical movement was in the order of 300mm, and had a slight tilt. The horizontal tectonic movement was

uniform between ends of abutments. D&B write that tectonic uplift was small enough such that it 'did not significantly influence the settlements and vertical strains'.

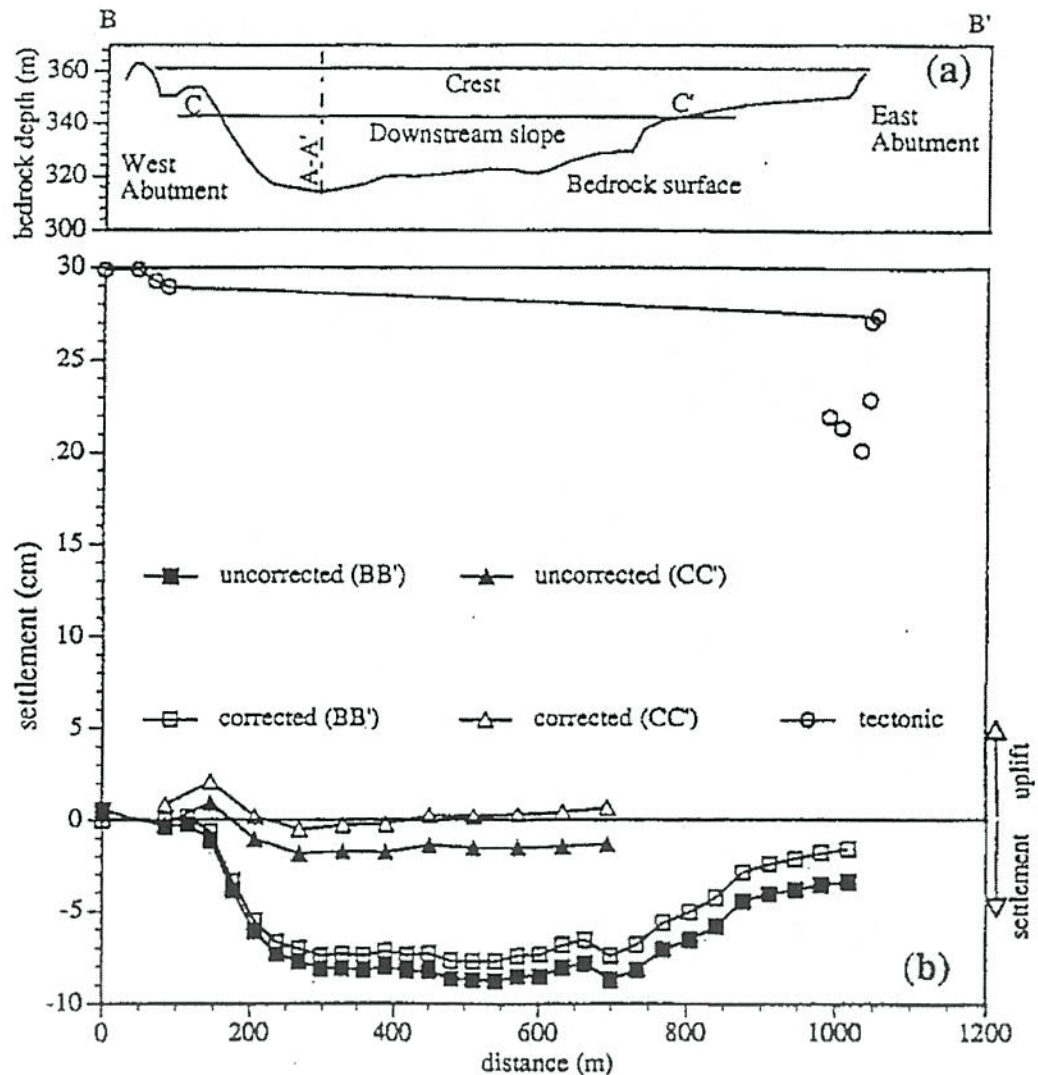
Settlements measured at the site had to be corrected to account for these large scale movements. Figure (KXXX) shows the profile of crest settlements at Los Angeles Dam. The 'corrected' figures are those which take into account large scale tectonic movement that occurred over the whole site - i.e. these figures give an indication of crest settlement relative to the foundation bedrock. Authors agree that the maximum crest settlement for this embankment was about 90mm (0.19% of embankment height). (Fong and Bennet, 1995; Swaisgood, 1998; Bureau *et al* 1996). Swaisgood describes damage as 'moderate'. Davis and Bardet (1996) write that most movement took place during the earthquake, and there was very little additional movements in the subsequent months.



After the earthquake, there was extensive cracking over the embankment. Davis and Bardet (1996) write "the 70mm asphalt lining that covered all of the upstream face featured many cracks in addition to those that were present before the earthquake". Most of the cracking was of the asphalt surface, but there were also transverse cracks through the crest which extended into diagonal ones along the upstream face. The largest crack was at the left abutment and was described by Davis and Bardet (1996) as being of a 'compressional' nature. The rest they described as being "of a shear type". A schematic of these cracks is shown in figure (KXX).

Authors agree that the cracking over the embankment was 'superficial', and trenching of the largest cracks indicated that even these did not extent into the dam body (ICOLD 1999). Fong and Bennett's (1995) record of the incident characterises the crack width as 1.3 mm.

Davis and Bardet (1996) write that the dense configuration of cracks on the asphalt face are not clearly related to permanent displacements and settlement. They suggest that these cracks may have been generated by transient stresses and deformations during the earthquake.



Accorind to ICOLD (1999), seepage levels, piezometers and observation wells indicated increases in pore pressure in and around the embankment after the earthquake event. These returned to normal conditions 'within a short time after the earthquake'. (ICOLD 1999).

Davis and Bardet (1996) summarise damage to the embankment by saying it "slightly moved and settled and sustained small superficial cracks". ICOLD (1999) concludes the performance of this embankment confirms the notion that dams built of well compacted cohesive clays and dense sands can perform satisfactorily during very severe shaking.

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Matahina Dam

(Bay of Plenty / Edgecumbe Earthquake, 2 March 1987)

Dam Details

Location	New Zealand
Type	Earthcore Rockfill
Height (m)	86 (Structural)
Crest Length (m)	400
Year Built	1967
Owner	Electricity Corp ^a of New Zealand
Foundation Geology	Sandy gravel alluvium under base and ignimbrite cliffs at abutments
Freeboard at Time of Earthquake (m)	n/a
Design Freeboard (m)	3

Special Features of Dam

The dam suffered an incident of leakage and piping during first filling which was serious enough to necessitate lowering of the lake and investigations / remedial works.

Instrumentation on the dam includes five strong motion accelerometers.

Earthquake Details

Name	Bay of Plenty / Edgecumbe		
Date	2 March 1987		
Magnitude (M _s)	6.2		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	23	12	25.9
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.42g [⊥]	0.33g [⊥]	
Dam Damage Descriptor	Minor, class 1 (with subsequent major issues)		

Special Features of Earthquake

The main earthquake was preceded and succeeded by numerous foreshocks and aftershocks. Four of the aftershocks and one of the foreshocks had magnitudes over 5.0 and epicenters within 'a few kilometres' of the dam (Gillon 1988).

[⊥] measured by instrumentation on the dam. As presented in Gillon. Note that a larger peak acceleration of 0.48g was recorded at mid height on the embankment.

Embankment Details

Figure 1 shows a section of the embankment at maximum height.

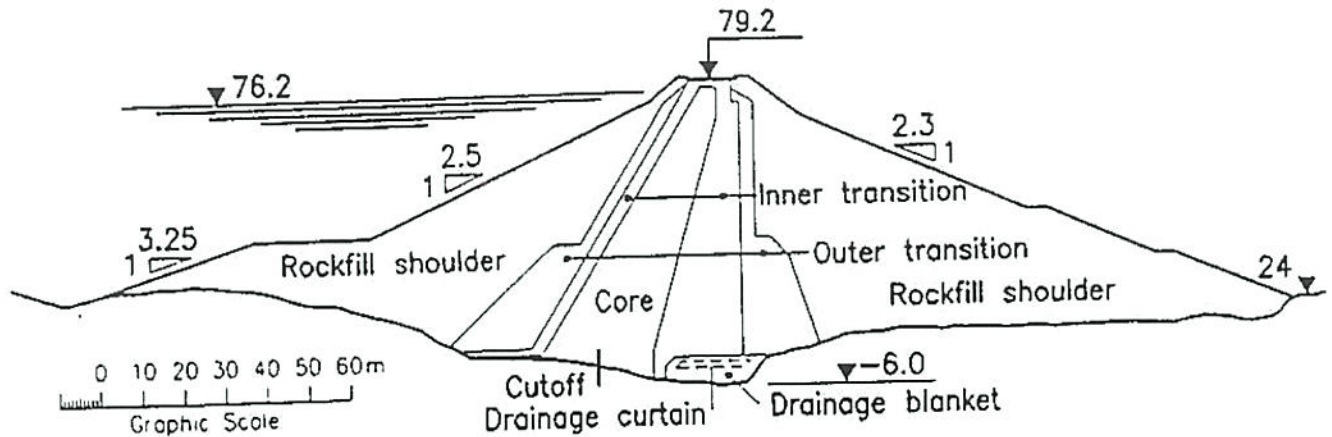


Figure 1

The earthfill core comprises low plasticity gravelly clay of weathered greywacke. It was compacted dry of optimum, forming what is described by Gillon (1997) as a 'dense strong material'. At the base, the core was taken down to the tertiary beds (Galloway, 1967).

The transition zones were constructed from weathered fine ignimbrite / tuff and quarry spoil. The inner transition adjacent to the core was a dense sandy silt with assessed UCS of 100kPa, whereas the outer transition was of sandy gravel / gravelly silt (Gillon, 1997). According to Galloway (1967) the transition materials have no measurable plasticity.

Dam shoulders are constructed from quarried ignimbrite rockfill, comprising angular blocks of sizes 150 to 900 mm. The rockfill was compacted dry in 1 metre layers by track rolling of a heavy tractor (Gillon, 1997).

The core and transitions zones were placed as rolled fills, and full control of compaction and water content was maintained. The outer transition zones and rockfill shells were placed in thick layers with 'arbitrarily defined compaction' (Galloway, 1967).

Table 1 presents some material parameters.

Zone	Name	Description	Properties			Compaction and Placement			Wc %
			γ_d (kN/m ³)	c' (kPa)	ϕ'	Thickness	Compaction	Density Ratio	
1	Earth core	Weathered greywacke, gravelly clay, low plasticity, dense, strong.	15.5*	5 [±]	32	-	'rolled'	-	Dry of optimum
2	Inner Transition	Dense sandy silt	12.6*	7.5 [±]	35	-	'rolled'	-	-
3	Outer Transition	Sandy gravel / gravelly silt	12.9*	-	36	-	'track rolling of heavy tractor'	-	-
4	Rockfill	Quarried ignimbrite, using blocks 150 – 900mm	-	-	-	1 m	'track rolling of heavy tractor'	-	'Compacted dry'

* Assuming specific gravity of soil particles of 2.7

[±] these values were presented in Galloway (1970) without units. They were assumed to be in kPa.

Grading envelopes of all embankment materials are shown in Figure 2 below.

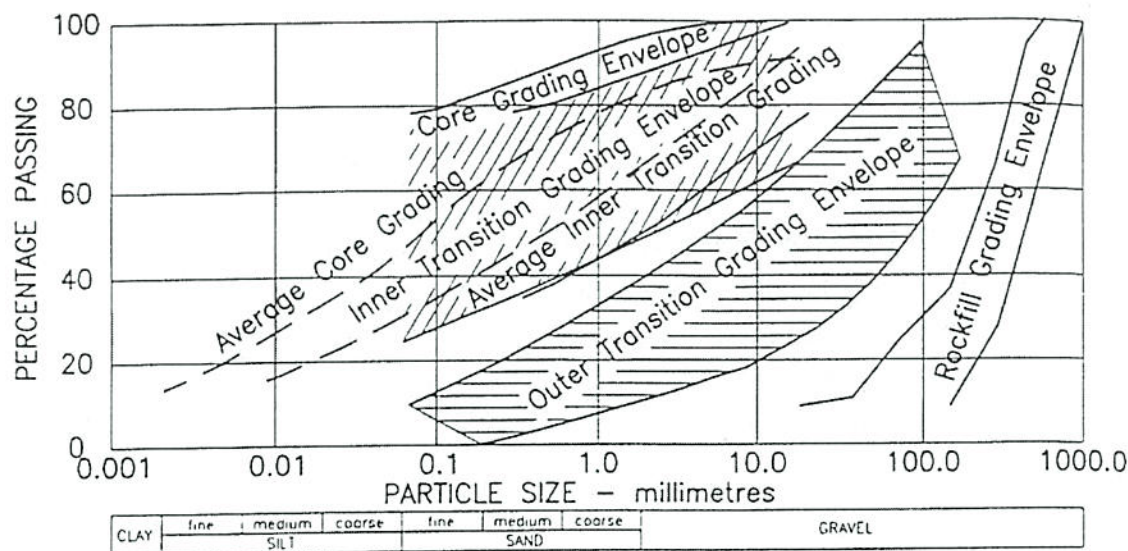


Figure 2 - Particle Size Distribution of Embankment Materials

Seepage under the dam is controlled by a shallow cutoff wall below the core and a 30m deep curtain of drainage holes. The flow from these holes is discharged to a drainage blanket and flows from the blanket are monitored on a weir downstream of the dam (Gillon, 1988).

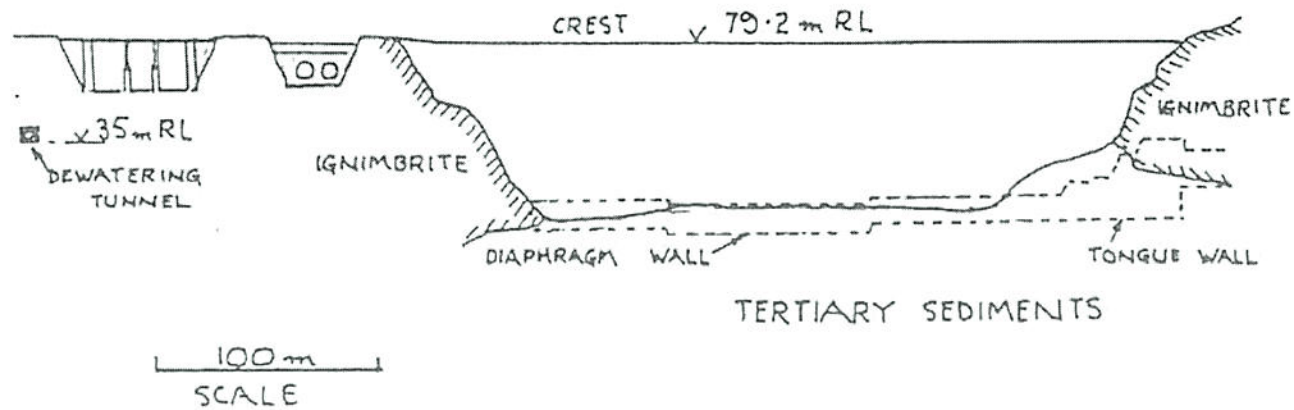


Figure 3 - Elevation of Embankment and Diaphragm Wall

Foundation Geology and Design

The Matahina Dam is located on the margin of the “Central Volcanic Region” of New Zealand. Figure 4 below shows the geology of the region and the dam and epicenter locations. Of interest is the Waiohau fault which passes within 500m of the dams’ left abutment.

This regions’ geology is characterized by young igneous (rhyolitic and andesitic) rocks which feature as extensive ignimbrite sheets. These sheets extend to the location of the dam, where they are manifest as ‘hard columnar jointed welded tuff’. (Gillon, 1988). Underlying the ignimbrite sheets are beds of ‘compact silty clays, clay gravels, sands and silty sands’ (Galloway, 1967). The silty clays are tertiary sediments with low plasticity. The sands have pumiceous origin and thus feature low unit weight and ‘appreciable’ compressibility. Permeability in the tertiary beds is about 10^{-3} ms^{-1} and up to 10^{-6} ms^{-1} in the pumiceous beds (Galloway, 1967).

The dam itself is located on a bend in the river in a steep valley formed by erosion through such an ignimbrite flow and into the underlying materials. The abutments are thus formed as hard ignimbrite and the base from the tertiary deposits. (Gillon 1988).

Figure 5 shows a longitudinal section of the dam site, including basic geology and fault locations.

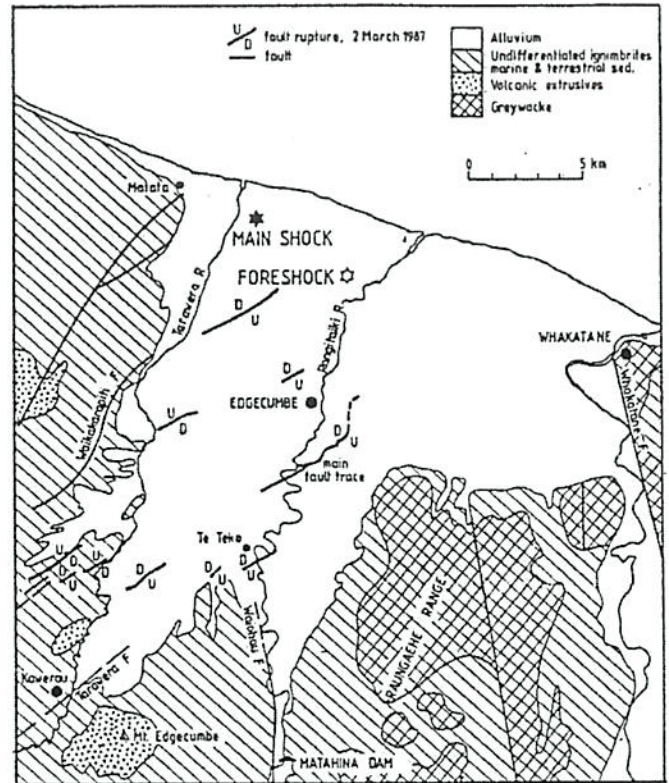


Figure 4 Dam, Epicentre and Geology

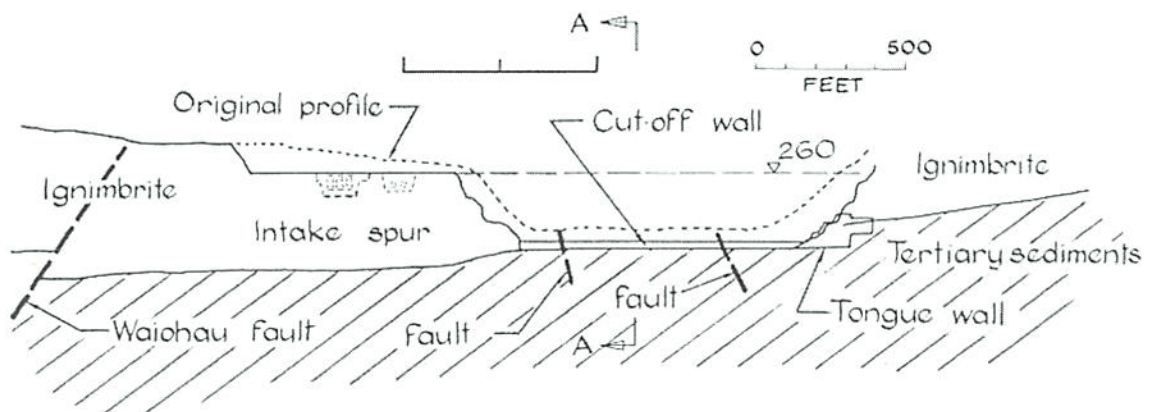


Figure 5 - Longitudinal Elevation of Site

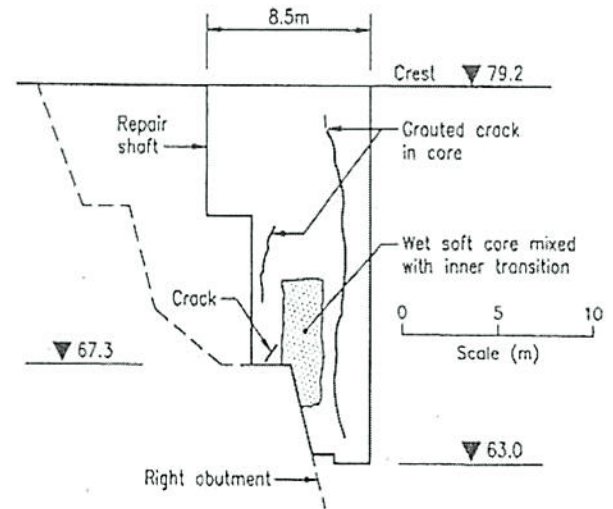
At the base, the embankment is founded on the alluvial sandy gravel deposits of the river bed, except for the core which is extended to the compact sediments that underlie the ignimbrite. As mentioned, a shallow cutoff wall extends from under the core into the sediments. The zone of contact between core and abutment uses concrete facing slabs (Gillon, 1997).

The abutments were trimmed to a stable slope and grouted where necessary. Consolidation grouting of the rock was performed in the core contact zone. Benches in the abutment rock were widened at places to act as temporary roads. Although efforts were made to trim or infill these with concrete, miscommunication resulted in at least one of these being left in place – near the top of the right abutment (Galloway, 1970).

Effect of the Earthquake

So as not to form an incorrect view of the earthquake damage at Matahina Dam, it is necessary to first describe damage that the dam had suffered prior to the earthquake, as they are closely related.

During first filling of the reservoir (in January 1967), an increase in seepage was noted. This seepage developed until, approximately two weeks later, an erosion cavity was detected near the right abutment. The lake was lowered and investigations revealed that the core had cracked adjacent to a 1.8m wide bench in the abutment, and that water had passed through, eroding as much as 100 cubic metres of transition materials. Investigations also found that erosion was being stemmed by inner transition materials being drawn into the crack.. The area was repaired by placing a plastic concrete patch behind the damaged core and by grouting the damaged core area with cement-bentonite. Later investigations (after the earthquake) showed that these repairs were not entirely effective (Gillon, 1997). The details of this event are depicted in Figure 6. The cause of the cracking was attributed, at least in part, to differential settlements being set up adjacent to the irregular abutment



1967 Right Abutment Core Damage

Figure 6



Figure 7 - Right Abutment incident during first filling

As shown in Figure 8, measurements from the inspection gallery 'do not suggest significant foundation settlement'. Settlements continued for 'several weeks' after the earthquake (ibid). Settlements of 800mm were measured on the upstream slope. Investigations of the region found no indications of slope failure. The settlements were concluded to be the result of earthquake induced compaction of the rockfill (ibid). After the earthquake (1987), increased **seepage** was found over the weir from the left abutment and so the reservoir was drawn down by 2.5 metres and the dam inspected. The draw down succeeded in stopping seepage flows. However, there is uncertainty with the seepage levels recorded at the weir as investigations revealed some connectivity between groundwater and drainage blanket flows. Thus the levels at the weir were affected by reservoir and tailwater levels.

Inspections showed minor local settlements and surface **cracking** at the abutments. At the right abutment, investigations revealed some cracking through the abutment and embankment materials, and that some piping damage had been underway. However, the evidence for piping was attributed to the 1967 incident, not directly to the earthquake. Cracking in the core material adjacent to benches in the abutments were found at depths up to 29 m, although it is unsure whether these were a direct result of the earthquake (Gillon, 1997).

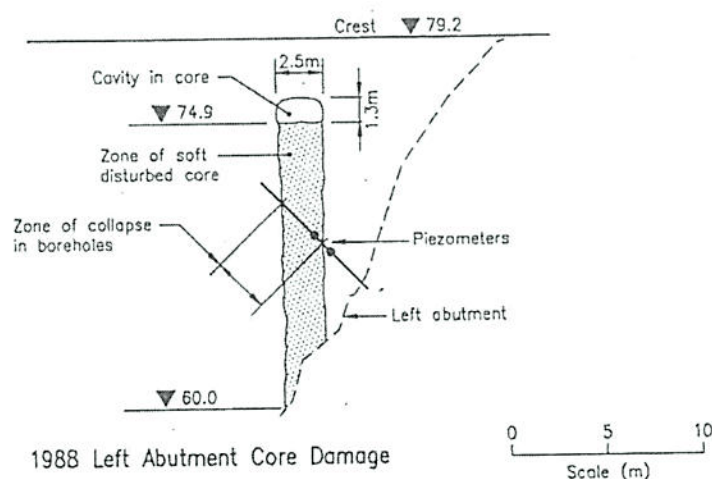


Figure 10

Investigations into the core at the left abutment found soft collapsing zones and piezometric levels close to lake level. However, as investigation proceeded into the left abutment (some 9 months after the earthquake), a cavity formed on the crest, above the downstream transition zone. The reservoir was drawn down and excavations found areas of significant piping induced erosion, featuring a 2.5m wide, 1.3m high cavity in the core zone. There was no evidence that self healing would have occurred and the downstream filter on the



left abutment was described as 'ineffective'. It was concluded minor cracking and settlement associated with the earthquake had reactivated semi-stable erosion damage that had existed since first filling. This had continued undetected for 10 months after the quake while other investigations were being performed. It is noted that the 2.5m draw down may well have prevented dam breaching (Gillon, 1997)

References

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Oroville Dam

(Oroville Earthquake, 1 August 1975)

Dam Details

Location	U.S.A.
Type	Zoned Earthfill
Height (m)	230
Crest Length (m)	2104
Year Built	1968
Owner	CA Dept. of Water Resources
Foundation Geology	Rock, no alluvium
Freeboard at Time of Earthquake (m)	12.5 m
Design Freeboard (m)	6.7 m (22 ft) below crest.

Special Features of Dam

A primary purpose of the Dam was to provide flood control. As the lake fills during heavy rains or large spring snow melts, water is carefully released to prevent flooding downstream, saving both lives and costly property damage (UC Website).

Stroppini (1976) described the dam as 'one the most highly instrumented dams in existence'.

Earthquake Details

Name	Oroville
Date	1 August 1975
Magnitude	6.15 M_w (USGS Database)
Distance (km)	<i>Epicentral</i> <i>Depth</i> <i>Focal</i>
	12 [♦] 8.8 [♦] 15
Peak Ground	<i>Crest</i> <i>Foundation</i>
Acceleration (g)	0.2g [*] 0.1g [*]
Dam Damage Descriptor	No or slight, class 0

Special Features of Earthquake

Some authors believe the event was due to reservoir induced seismicity. Topozada and Morrison (1982) argue that two factors suggest that Lake Oroville contributed to both the location and timing of the 1975 earthquake. The first factor is the proximity of the earthquake to the lake (with respect to local geological features), and the second factor is the occurrence of the earthquake following an unprecedented seasonal fluctuation in lake levels.

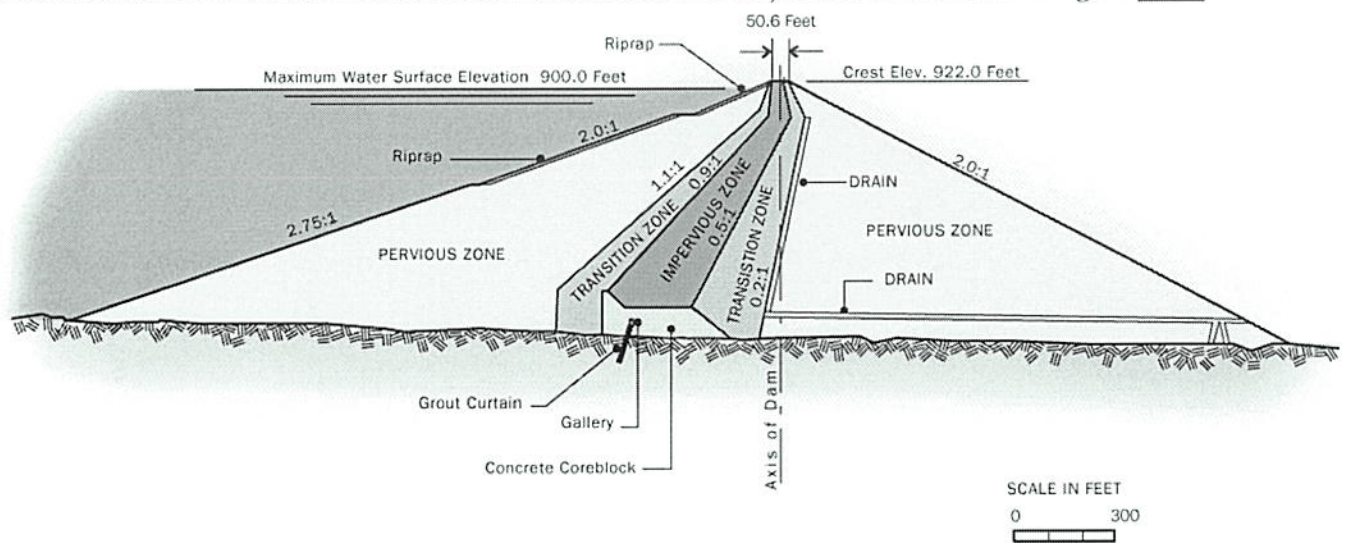
The earthquake sequence ($M > 3$) consisted of five foreshocks, a main shock, and numerous aftershocks. The sequence included seven earthquakes of magnitude greater than 4.6.

[♦] from Stroppini, 1976

^{*} from Seed, 1981

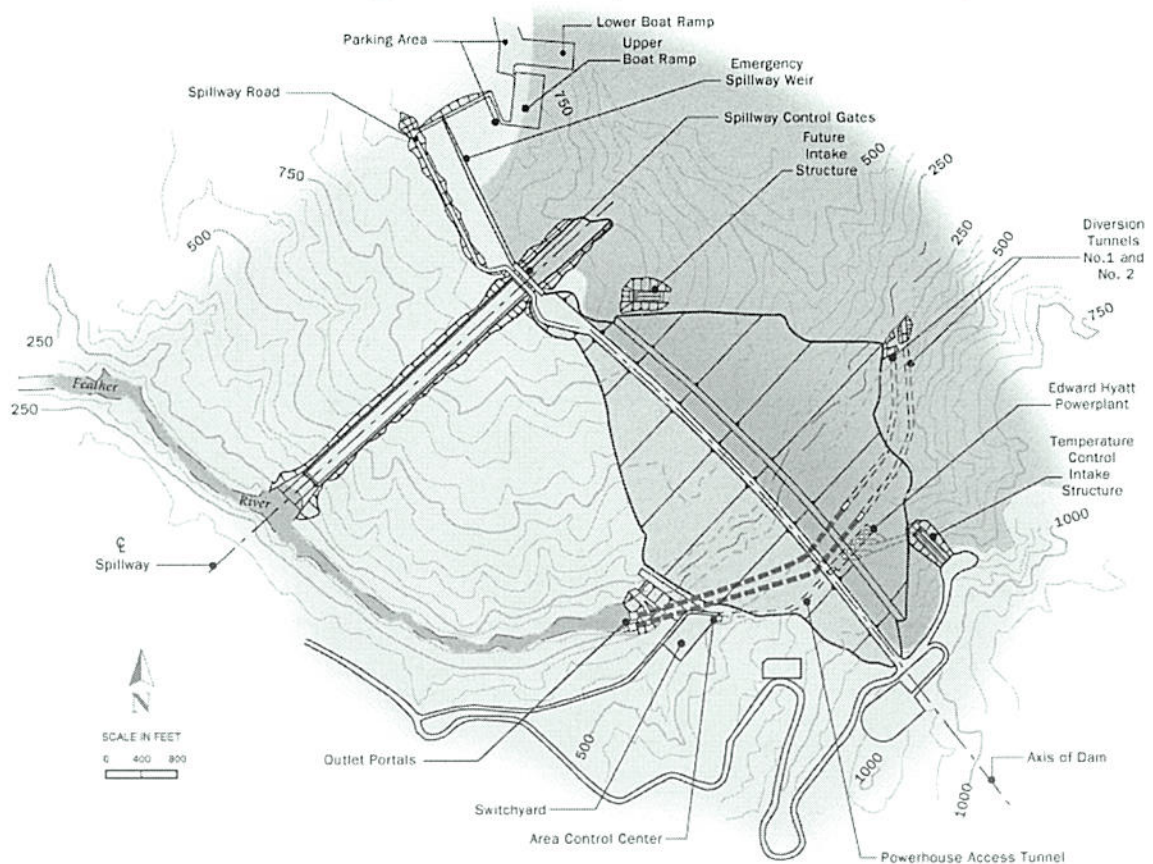
Embankment Details

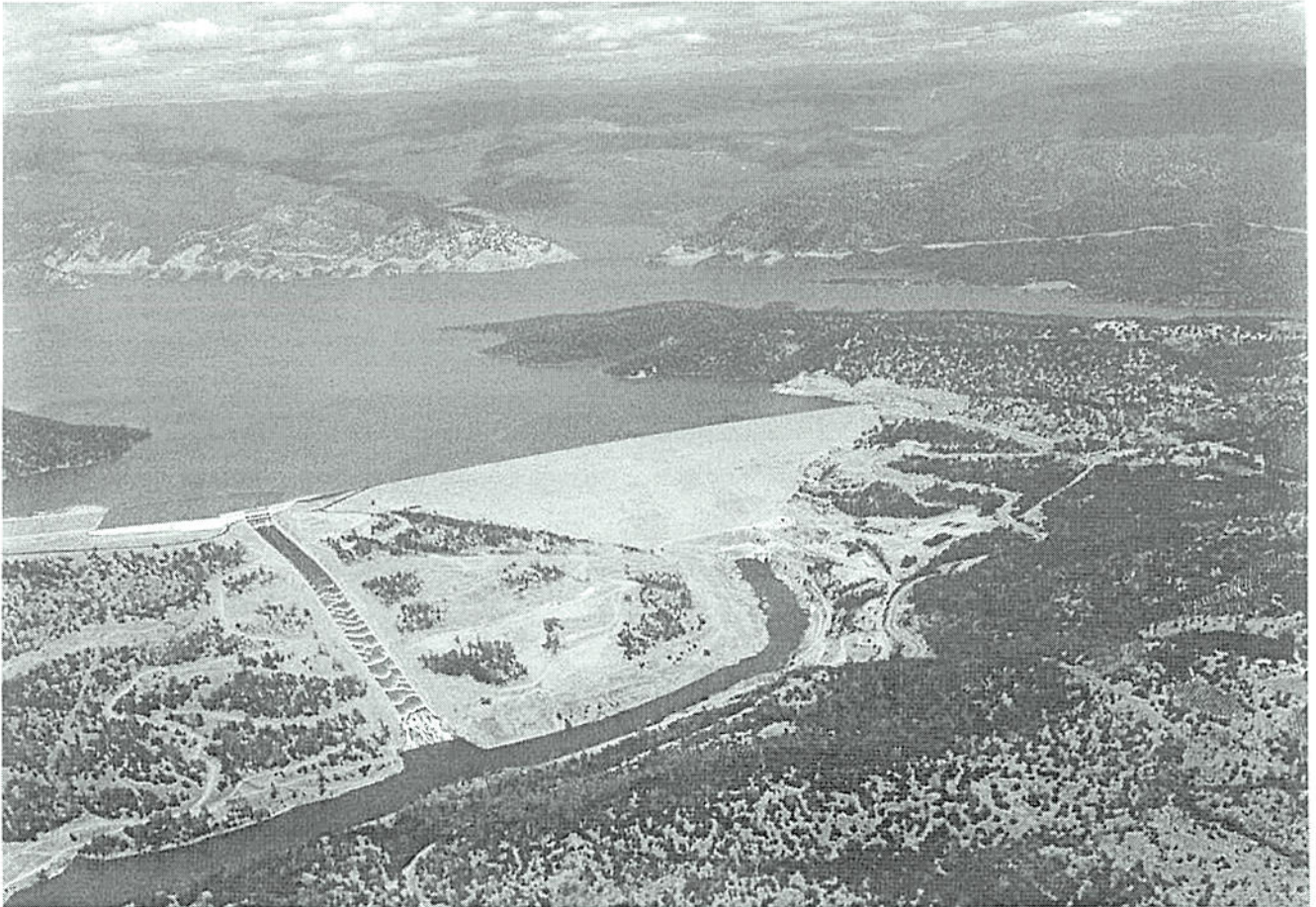
The Oroville Dam is a zoned earthfill embankment and its major section is shown in figure XXXX below.



Only basic data on the embankment materials was found. Stroppini (1976) described the embankment as “an impervious clay core resting on a concrete core block, appropriate transitions, and sand gravel-cobble shells”.

Most authors describe the embankment as zoned earthfill, but USCOLD (1992) and Swaisgood (1998) describe it as an earthcore rockfill embankment. This may just be an error, but may also indicate that the pervious outer shells, with their ‘sand gravel-cobble’ composition are in fact bordering on rockfill.





Foundation Geology / Materials

Little information was found on the embankments' foundations. However, Swaisgood (1998) reports that no alluvium remains under the foundations, and figure XXX shows it is founded directly on rock. Stroppini (1976) describes regional geology as being characterised as being part of the 'western metamorphic belt of the Sierra Nevada'.

Effects of The Earthquake

Shaking at the foundation of 0.13 g (transverse) and 0.09g (upstream-downstream) was recorded. The upstream / downstream direction accelerations were magnified in the embankment to maximum recorded values of 0.13g in the crest (although larger values could have occurred, as instrumentation in the crest did not record for the first two or three seconds). (Stroppini, 1976).

Under these loading the maximum movement of the embankment was 30mm, with maximum settlement of 10mm (maximum horizontal upstream movement of 15mm (ibid). This settlement amounts to 0.004% of dam height, which places the event in the 'class 0, no or slight' classification.

No cracking was reported in any of the publications.

Stroppini (1976) reported increased pore pressure in the embankment by up to a maximum of 16 meters, although this values was still less than values recorded during previous periods of high reservoir stages.

There were minor changes in seepage reported, which dissipated slowly after the event.

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San Justo Dam

(Loma Prieta Quake, 17 October 1989)

Dam Details

Location	California, U.S.A.
Type	Zoned Earth and Rockfill
Height (m)	41
Crest Length (m)	330
Year Built	1986
Owner	San Benito County Water District
Foundation Geology	14 m clay/sand alluvium
Freeboard at Time of Earthquake (m)	“reservoir 48% full”
Design Freeboard (m)	

Special Features of Dam

The dam is heavily instrumented with strong motion sensors (Harder *et al* 1996). It is located only 2.4 km from the San Andreas fault and was designed to withstand larger seismic loading than experienced from the Loma Prieta Earthquake.

Earthquake Details

Name	Loma Prieta		
Date	17 October 1989		
Magnitude (M_s)	7.1 M_s (USGS Database)		
Distance (km)	<i>Epicentral</i>	<i>Depth</i>	<i>Focal</i>
	27*	18.5	33
Peak Ground	<i>Crest</i>	<i>Foundation</i>	
Acceleration (g)	0.4g*	0.26g*	
Dam Damage Descriptor	Minor, class 1		

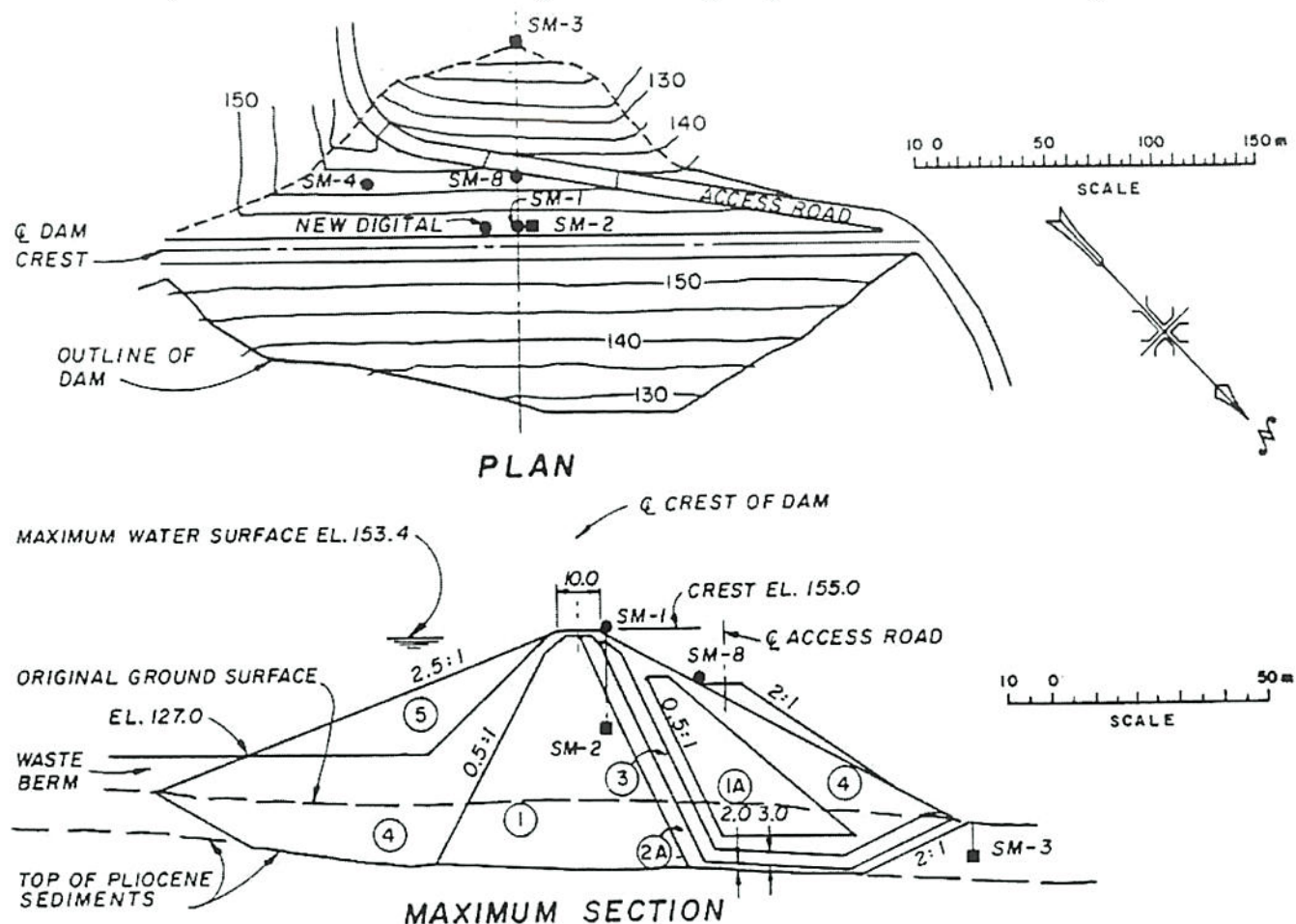
Special Features of Earthquake

The duration of shaking was generally between 7 and 10 seconds, which is about half of that expected for an event of this magnitude (Harder *et al* 1996).

* presented in Harder *et al*, 1992

Embankment Details

USCOLD (1992) describes the embankment as a “zoned earthfill with clayey silty core, double chimney and blanket drains, one of sand and the other of gravel”. See Figure (both from USCOLD 1992)



Zone	Description	Compaction and Placement	
		Thickness	Compaction
1	Selected CL, SC, SM and ML materials	150mm compacted	Tamping rollers
1a	Selected CL, SC, SM and ML materials	300mm when loose	Pneumatic Tyred Rollers
2a	Processed SP materials	300mm when loose	Vibratory rollers
3	Processed GP materials	600mm when loose	Vibratory rollers
4	Decomposed Granite materials	300mm when loose	Pneumatic Tyred Rollers
5	Limestone materials	1000mm when loose	Vibratory rollers

All info from USCOLD 1992

Table A

Foundation Geology / Materials

USCOLD (1992) briefly describe foundation material as “dense, variably cemented bedded clays and sands, both marine and continental origins”. Swaisgood (1998) records that 14 meters of alluvium remains under the embankment.

Effects From Earthquake

There was no significant damage to San Justo dam after the Loma Prieta earthquake. Published damage descriptions include 'none', 'no significant damage' and 'almost negligible' (Harder *et al* 1996; Swaisgood, 1998; USCOLD 1992). However, maximum crest settlements of 38mm ($\sim 0.08\%$ of the dam height) were recorded (*op cit*), which places it in 'class 1, minor' damage, using the system of this report.

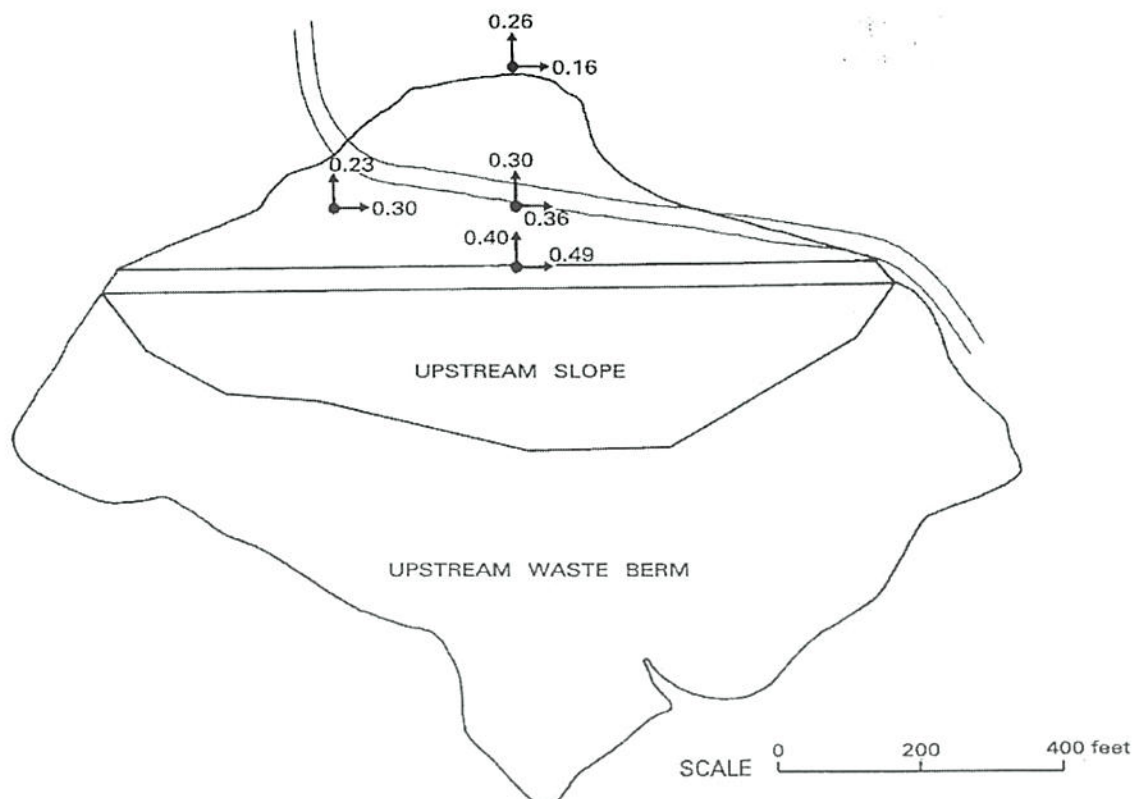
USCOLD (1992) report that stations on the embankment were closely monitored since 1986 and a slight increase in the rate of crest settlement during the time interval spanning the earthquake was noted at locations both upstream and downstream of the centre of the dam.

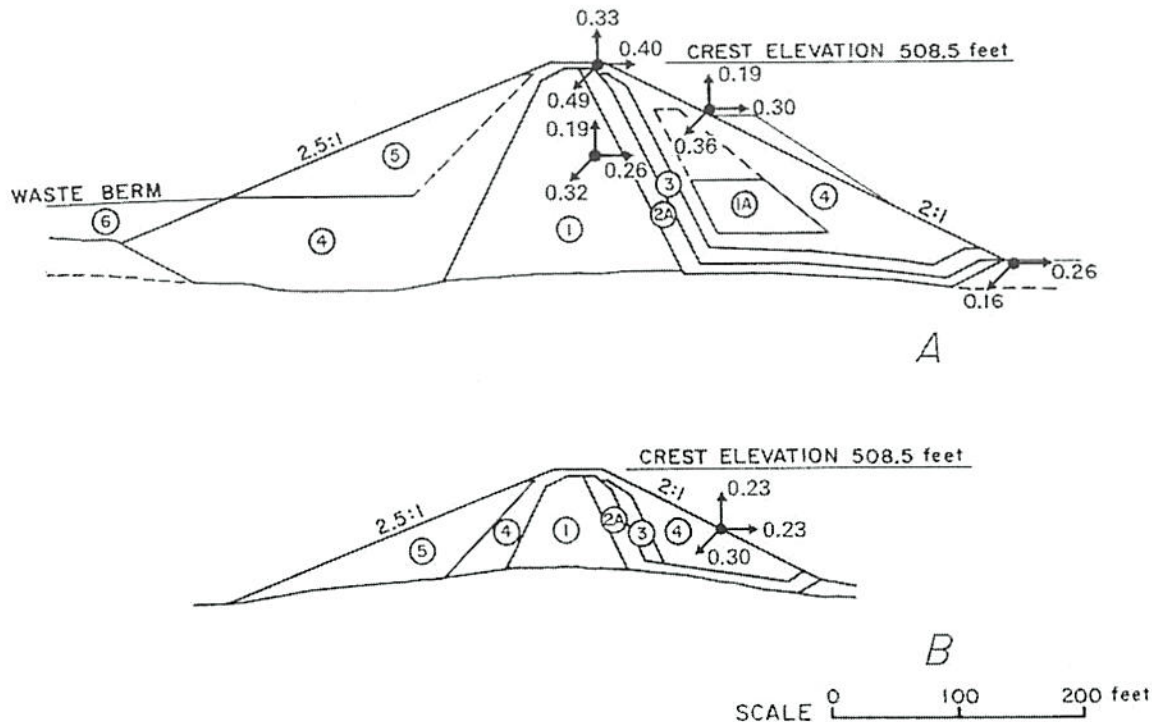
Despite the minimal damage, due to the degree of instrumentation on the embankment this case study offers important insights into the dams' dynamic response. The dam is heavily instrumented with strong motion sensors, as shown in figures *)*)(*. Instrumentation includes a sensor located inside the embankment, 19 metres below the crest (Harder *et al* 1996).

Harder *et al* (1996) note that an amplification factor of two is recorded from foundation (0.2 g) to crest (0.4 g) in the transverse direction, and that the sensor in the centre of the embankment recorded motions of 0.26 g in this direction, showing that most of the amplification takes place in the top half of the dam. This is an interesting insight, considering the fact that most seismically induced longitudinal cracking of embankments occurs within this region. This is concordant with Lane's (1980) explanation of longitudinal cracking (see section 2.2 of the main report), that amplified shaking in the transverse direction (in this upper region of the embankment) causes in horizontal and vertical forces in excess of friction forces, resulting in cracking.

Movements that were in the longitudinal direction (i.e. parallel to the crest) were amplified by a factor of three, but these took place mostly in the bottom half of the dam (Harder *et al* 1996).

Both pictures from Harder et al 1996 Plan of embankment showing peak accelerations recorded at various locations by strong motion sensors.





Less than half of the piezometers installed within the embankment, foundation and in nearby natural ground showed any response to the shaking. However, some differed from anticipated normal levels by over 10 meters. All returned to normal by 10 to 30 days after the event

References

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APPENDIX C – Data Sets Used for the Damage – Seismic Intensity Relationship

Data For Earthfill Dams

Data Set	Dam Name	Mag	PGA			Damage Level			Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs	Chosen			
1 Damage based on Cracking Data and Given Settlement. PGA Given or Recorded	Ono	8.2	0.30	0.31	-	4	3	3/4	puddle clay core	11	Liquefaction
	Matway	7.7	0.65	0.70	-	4	4	4	zoned earthfill	3	Liquefaction
	Lexington	7.1	0.45	0.50	0.55	2	1	2	zoned earthfill	0	
	Elmer J Chetbro	7.1	0.42	0.66	0.46	3	2	2/3	zoned earthfill	-	
	Vascona	7.1	0.37	0.45	0.51	2	1	1/2	earthfill	8	Liquefaction
	La Marquesa	7.8	0.60	0.28	-	5	5	5	earthfill	-	
	Guadalupe	7.1	0.42	0.56	0.64	3	2	2/3	earthfill	0	
	Murayma Lower (Shimo)	8.2	0.20	0.30	-	3	2	3	earthfill	-	Calcs. Suggest PGA should be higher. Used 0.2
	Hebgen	7.6	0.71	1.31	-	4	4	4	earth / rock / concrete core	10	
	Austrian	7.1	0.57	0.68	0.71	4	3	4	homogeneous earthfill	0	
2 Ditto but PGA Calculated	Chetbro	7.1	-	0.50	0.48	3	-	3	earthfill	-	
3 Damage based on Cracking Data and Calculated Settlement. PGA given or recorded	San Luis	7.1	0.10	0.16	-	0	0	0	homogeneous earthfill	-	
	Los Angeles North Dyke	6.7	0.42	0.53	-	1	-	1	embankment	0	Some Liquefaction
4 Damage based on Given Settlement data only. PGA given or recorded	Los Angeles	6.7	0.32	0.53	0.53	1	2	1/2	asphalt face embankment	0	Some Liquefaction
	Yucapa No. 1	6.6	0.15	-	0.32	1	1	1	earthfill	9	
	Almaden	7.1	0.43	0.61	0.66	1	1	1	earthfill	-	
	Wide Canyon	7.5	0.20	-	0.92	1	1	1	earthfill	-	
	Yucapa No. 2	6.6	0.15	-	0.33	0	1	0/1	earthfill	9	
	Robert Matthews	6.9	0.07	-	0.05	0	0	0	earthfill	0	
	Lake Mary Upper	5.5	0.02	-	-	0	0	0	earthfill	1	
	Porter Estates	6.7	0.59	-	0.58	1	4	2	rolled fill	0	
	Santa Felicia	6.7	0.15	-	0.24	2	1	1/2	zoned earthfill	0	
	Vermilion	6.3	0.24	0.30	0.31	1	1	1	zoned earthfill	80	
	Burna Vista	7.7	0.30	-	-	5	3	4	earthfill - some puddle core	-	
	Yuwack	7.8	0.15	1.42	-	1	0	1	earthfill embankment	-	
	Pedmont	7.8	0.35	0.89	0.86	3	1	2/3	homogeneous embankment	-	
	La Palma	7.8	0.46	0.13	-	5	5	5	earthfill	-	Liquefaction
	San Andreas	7.8	0.80	1.95	1.69	2	1	2/1	puddle core, rolled fill	15	1906 Quake
	Cryptal Springs Upper	7.8	0.80	1.95	-	3	2	3/2	earthfill puddle core	-	1906 Quake
7 Ditto but PGA Calculated	Vic	6.5	-	0.24	-	2	2	2	earthfill	-	

Data Set	Dam Name	Mag	PGA		Damage Level			Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs	Chosen		
8 Damage based on calculated (Swaigood) settlement only. PGA given. Alluvial thickness assumed = 0 where not given (required for Swaigood)	Mackay	7.3	0.22	0.23	-	1	1	1	-	embankment
	Rinconada	7.1	0.37	0.42	0.48	2	1	2	-	earthfill
	Douke	7.8	0.40	-	-	3	4	3/4	-	homogeneous earthfill
	Howell Lower	7.8	0.80	1.95	0.29	5	4	4/5	-	earthfill
	Santa Felicia	6.5	0.24	0.32	0.18	1	0	1	0	zoned earthfill
	Alonso	6.5	0.15	-	-	1	0	1/0	-	earthfill
	Pablo Marin	7.8	0.05	0.04	-	1	4	-	-	earthfill
	O'Neill	7.1	0.10	0.14	0.06	1	0	-	-	earthfill
	Bouquet Canyon	7.7	0.12	-	-	1	0	-	-	earthfill
	Drinkwater	7.7	0.13	-	-	1	0	-	-	earthfill
	Bouquet Canyon	6.5	0.36	0.48	0.61	1	0	-	-	earthfill
	Kelly Cabin Canyon	7.1	0.20	0.38	0.18	1	0	-	-	earthfill
	Peabody	7.1	0.25	0.54	0.24	1	0	-	-	earthfill
	Reservoir #1	6.5	0.29	0.38	0.38	1	0	-	-	earthfill
	Lake Frey	8.3	0.20	0.36	0.32	2	0	-	-	earthfill
	Sempervirens	7.1	0.30	0.54	0.27	2	0	-	-	earthfill
	Drinkwater	6.5	0.43	0.55	0.48	2	0	-	-	earthfill
	Lake Chabot	7.8	0.25	0.53	0.48	2	0	-	-	earthfill
	Calico	7.1	0.40	0.66	0.60	2	0	-	-	earthfill
	Oak Site	7.1	0.35	0.62	0.32	2	0	-	-	earthfill
	Forest Lake	7.8	0.25	0.56	0.12	2	0	-	-	earthfill
	Lake Herman	7.8	0.25	0.46	0.44	2	0	-	-	earthfill
	St. Helena Lower	7.8	0.25	0.46	-	2	0	-	-	earthfill
	St. Helena Upper	7.8	0.25	0.46	-	2	0	-	-	earthfill
	Port Costa	7.8	0.25	0.56	-	2	0	-	-	earthfill
	Uvas	7.1	0.45	0.73	0.50	2	0	-	-	earthfill
	Solvang No. 2	7.1	0.40	0.69	0.38	2	0	-	-	earthfill
	Lake Camille	7.8	0.25	0.43	0.36	3	0	-	-	earthfill
	Estates	7.8	0.35	0.66	0.84	3	0	-	-	earthfill
	Howell Lower	7.1	0.43	0.80	0.52	3	1	-	-	earthfill
	Lake Ranch	7.1	0.43	0.76	0.44	3	1	-	-	earthfill
	Almaden Valley	7.1	0.43	0.69	0.62	3	0	-	-	earthfill
	Howell Upper	7.1	0.43	0.80	0.52	3	0	-	-	earthfill
	Bay Street	7.1	0.43	0.60	0.53	3	1	-	-	earthfill
	Debell	7.1	0.48	0.75	0.40	3	0	-	-	earthfill
	Cowell	7.1	0.48	0.56	-	3	0	-	-	earthfill
	Pacific Grove	7.8	0.30	0.62	0.12	3	0	-	-	earthfill
	Berryman	7.8	0.35	0.96	0.89	3	0	-	-	earthfill
	Lake Ralphine	7.8	0.35	0.76	0.26	3	0	-	-	earthfill
	Cowell	7.8	0.45	1.33	-	3	0	-	-	earthfill
	Lagunas	7.8	0.60	1.81	-	4	0	-	-	earthfill
	Belvedere	7.8	0.60	1.81	-	4	0	-	-	earthfill

Data Set	Dam Name	Mag	PGA		Damage Level			Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs	Chosen		
	Lagunita	7.8	0.65	1.85	-	5	0	-	-	earthfill
	Emerald Lake #1	7.8	0.75	1.94	1.27	5	0	-	-	earthfill
	Notte Dame	7.8	0.75	1.93	1.13	5	0	-	-	earthfill
	Bear Gulch	7.8	0.75	1.93	0.83	5	0	-	-	earthfill
	Crocker	7.8	0.75	1.94	1.52	5	0	-	-	earthfill
	Porter Estates	6.5	0.40	0.51	0.38	2	0	-	0	rolled fill
	Wood Ranch	6.5	0.17	0.22	0.13	1	0	-	-	rolled fill
	Stone Canyon	6.5	0.22	0.27	0.29	1	0	-	-	rolled fill
	Eagle Rock	6.5	0.20	0.25	0.22	1	0	-	-	rolled fill
	Elyan	6.5	0.18	0.22	0.21	1	0	-	-	rolled fill
	Ascot	6.5	0.18	0.22	-	1	0	-	-	rolled fill
	Encina	6.5	0.24	0.32	-	1	0	-	-	rolled fill
	Easton Wash	6.5	0.19	0.23	0.17	1	0	-	-	rolled fill
	Stone Canyon Upper	6.5	0.23	0.28	0.31	1	0	-	-	rolled fill
	J.W.Wanda	6.5	0.19	0.23	0.21	1	0	-	-	rolled fill
	Hollywood Upper	6.5	0.23	0.29	0.31	1	0	-	-	rolled fill
	Glenoaks	6.5	0.22	0.27	0.25	1	0	-	-	rolled fill
	Savrell	6.5	0.19	0.23	-	1	0	-	-	rolled fill
	Franklin Upper	6.5	0.23	0.29	-	1	0	-	-	rolled fill
	Diedrich	6.5	0.24	0.31	0.30	1	0	-	-	rolled fill
	Rowena	6.5	0.20	0.25	-	1	0	-	-	rolled fill
	Brand Park	6.5	0.28	0.36	0.34	1	0	-	-	rolled fill
	Runkle	6.5	0.23	0.29	0.18	1	0	-	-	rolled fill
	Cherry Chase	6.5	0.23	0.29	0.26	1	0	-	-	rolled fill
	Sepulveda	6.5	0.27	0.34	-	1	0	-	-	rolled fill
	Ivanhoe	6.5	0.20	0.25	-	1	0	-	-	rolled fill
	Harold	6.5	0.25	0.32	0.23	1	0	-	-	rolled fill
	Highland	6.5	0.20	0.27	-	1	0	-	-	rolled fill
	Green Verdugo	6.5	0.43	0.55	0.52	2	0	-	-	rolled fill
	Hansen	6.5	0.43	0.55	-	2	0	-	-	rolled fill
	Lopez	6.5	0.60	0.70	0.01	3	0	-	-	rolled fill
	San Fernando Dyke	6.5	0.55	0.65	-	3	2	-	-	rolled fill
	Yarrell Debris	6.5	0.68	0.68	0.58	4	0	-	-	rolled fill
	Howell Upper	7.8	0.80	1.95	0.30	5	2	-	-	earthfill
	Lake Ranch	7.8	0.80	1.95	0.34	5	0	-	-	earthfill
	Bullingame	7.8	0.80	1.94	-	5	0	-	-	earthfill
	Summit	7.8	0.35	0.90	0.88	3	0	-	-	embankment
	Univ Mound N Basin	7.8	0.60	-	1.77	5	0	-	-	embankment

Data Set	Dam Name	Mag	PGA		Damage Level			Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs	Chosen		
9 Damage based on calculated (Swaisgood) settlement only. This calculation relies on the PGA which was also calculated. Alluvial thickness assumed = 0 where not given. (required for Swaisgood).	Waruta	5.6	-	0.11	-	0	0	0	-	earthfill
	Puama	7.1	-	0.04	-	1	1	1	-	earthfill
	Puama	7.5	-	0.08	-	1	1	1	-	earthfill
	Laguna	7.1	-	0.12	-	1	1	1	-	earthfill
	Necolita	7.9	-	0.11	-	1	1	1	-	earthfill
	San Antonio	5.5	-	0.38	0.00	1	1	1	-	earthfill
	Tanacaza	7.9	-	0.02	-	1	2	1/2	-	compacted homogeneous
	Hosorogi	7.3	-	0.85	-	5	5	5	-	earthfill
	Volcano Lake	7.1	-	0.89	-	5	5	5	-	earthfill
	Cullmo	7.1	-	0.10	-	1	1	1	-	earthfill
	Bai River	7.8	-	0.04	-	1	4	-	-	earthfill
	Biruvava	7.4	-	0.02	-	1	0	-	-	compacted earthshore earthfill
	Biruvava	7.5	-	0.06	-	1	0	-	-	compacted earthshore earthfill
	Colbun	7.8	-	0.03	-	0	-	-	-	compacted earthshore earthfill
	Liu-Liu	7.1	-	0.05	-	1	-	-	-	earthfill
	Huechun	7.1	-	0.05	-	1	-	-	-	earthfill
	Onanike	7.2	-	0.08	-	1	2	-	-	earthfill
	Sakuhitamamike	7.5	-	0.06	-	1	0	-	-	earthfill
	Higashidaiawa	7.7	-	0.02	-	1	0	-	-	earthfill
	Kogazavava	7.9	-	0.02	-	1	0	-	-	earthfill
	Orange County Reservoir	6.1	-	0.26	0.24	1	0	-	-	earthfill
	Ruangue	7.5	-	0.08	-	1	-	-	-	earthfill
	Hongou	7.7	-	0.05	-	1	0	-	-	earthfill
	Liu-Liu	7.5	-	0.09	-	1	4	-	-	earthfill
	Cataploco	7.1	-	0.11	0.10	1	-	-	-	earthfill
	Hasike	7.5	-	0.08	-	1	0	-	-	earthfill
	Takinosava	7.5	-	0.07	-	1	0	-	-	earthfill
	Cullmo	7.5	-	0.16	-	1	-	-	-	earthfill
	Rubio Div. Debris	6.5	-	0.25	0.19	1	0	-	-	earthfill
	Cataploco	7.5	-	0.23	0.38	2	4	-	-	earthfill
	Takamatsukashiroike	7.2	-	0.31	-	2	0	-	-	earthfill
	Garvey Reservoir	6.1	-	0.53	0.52	2	0	-	-	earthfill
	Whitler-Narrows	6.1	-	0.52	-	2	0	-	-	earthfill
	Damonike	7.2	-	0.76	-	4	0	-	-	earthfill
	Obanike	7.2	-	0.79	-	5	2	-	-	earthfill
	Arata	6.6	-	0.86	0.44	5	2	-	-	earthfill
	Tatemichikamike	7.2	-	0.85	-	5	2	-	-	earthfill
	Nabetanike	7.2	-	0.87	-	5	0	-	-	earthfill
	Ivanike	7.2	-	0.90	-	5	2	-	-	earthfill
	Rotanike	7.2	-	0.93	-	5	2	-	-	earthfill
	Tokyo W.S.	8.2	-	0.90	-	5	1	-	-	earthfill
	Shovavake	7.2	-	1.06	-	5	1	-	-	embankment
	Gekze	7.8	-	0.28	-	2	0	-	-	homogeneous earthfill
	Klayama	7.2	-	0.42	-	2	1	-	-	zoned earthfill
	Tanyama	7.2	-	1.00	-	5	1	-	-	zoned earthfill
	Onari	7.2	-	1.00	-	5	1	-	-	homogeneous embankment
	Lo Ovalle	7.5	-	0.07	-	1	-	-	-	-

Data For Rockfill Dams

Data Set	Dam Name	Magnitude	PGA			Damage Level		Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs			
1 Damage based on Cracking Data and Given Settlement. PGA Given or Recorded	La Villita	8.1	0.12	0.29	-	2	1	earthcore rockfill	70	
	El Infiernillo	7.6	0.12	0.05	-	2/1	1	earthcore rockfill	0	
	Leroy Anderson	7.1	0.23	0.41	0.36	1	1	earthcore rockfill	0	
	Coyote	6.2	0.63	0.28	0.27	1/2	1	earthcore rockfill	0	
	El Infiernillo	8.1	0.13	0.23	-	3	2	earthcore rockfill	0	
	Newell Creek	7.1	0.43	0.55	0.49	3	2	earthcore rockfill	-	
2 Ditto but PGA Calculated	Surgu	5.9	-	0.38	-	3	2	rockfill	-	
4 Damage based on Given Settlement data only. PGA given or recorded	Tsengwen	5.3	0.16	-	-	0	-	earthcore rockfill	-	
	Oroville	5.8	0.10	0.34	0.37	0	1	earthcore rockfill	0	
	Makio	6.8	0.57	0.18	-	2	1	earthcore rockfill	29	
	La Villita	7.5	0.04	0.14	-	1	1	earthcore rockfill	70	
	Aya	7.7	0.58	-	-	1	1	earthcore rockfill	0	
	Diayo	7.7	0.38	-	-	1	1	earthcore rockfill	0	
	Liyutan	7.6	0.10	-	-	1	1	earthcore rockfill	-	
	La Villita	7.6	0.02	0.05	-	1	1	earthcore rockfill	70	
	Canli	7.7	0.38	-	-	1	1	earthcore rockfill	-	
	Leroy Anderson	6.2	0.41	0.40	0.40	0	1	earthcore rockfill	0	
	Miboro	7	0.15	-	-	0	1	earthcore rockfill	0	
	Ambuklao	7.7	0.60	0.68	-	3	2	earthcore rockfill	10	
	La Villita	7.3	0.09	0.33	-	1	0	earthcore rockfill	70	
	San Justo	7.1	0.26	0.41	0.01	1	0	earthcore rockfill	14	

Data Set	Dam Name	Magnitude	PGA		Damage Level			Dam Type	Alluvium	Comment
			Given	calc, dist given	calc, dist calc	Ours	Theirs	Chosen		
	El Infiernillo	7.3	0.05	0.17	-	1	0	1/0	0	Damage through piping
	La Villita	5.9	0.04	0.14	-	0	0	0	70	
	El Infiernillo	4.9	0.08	0.01	-	0	0	0	0	
	El Infiernillo	5.9	0.09	0.11	-	0	0	0	0	
	Magat	7.7	0.05	-	-	0	0	0	0	
	Matahina	6.3	0.33	0.29	-	1	4	3	-	
	Nagara	6.7	0.27	-	-	1	0	1/0	-	
	Namioka	7.7	0.10	0.05	-	1	0	1	0	
	Binga	7.7	0.60	0.68	-	4	2	3	-	
	Jen-I-Tan	7.6	0.19	0.22	-	3	1	2	-	
6 Damage based on Cracking Data only. PGA given.										
8 Damage based on calculated (Swaisgood) settlement only. PGA given. Alluvial thickness assumed = 0 where not given (required for Swaisgood)	El Infiernillo	7.5	0.06	0.11	-	0	2	-	0	
	Coyote	7.1	0.19	0.49	0.28	1	0	-	0	
	La Villita	4.9	0.07	0.01	-	0	0	0	70	
	Makio	6.9	0.55	-	-	3	0	-	29	
	Makomanai	7.8	0.18	-	-	1	0	-	-	
9 Damage based on calculated (Swaisgood) settlement only. This calculation relies on the PGA which was also calculated. Alluvial thickness assumed = 0 where not given (required	Tanumizu	7.4	-	0.06	-	1	0	-	-	
	Minoogawa	7.2	-	0.22	-	1	0	-	-	
	Kojiya	7.2	-	0.22	-	1	0	-	-	
	Fukatani	7.2	-	0.38	-	2	1	-	-	
	Ushino	7.4	-	0.04	-	1	0	-	-	

Data For Hydraulic Fill Dams

Data Set	Dam Name	Magnitude	PGA			Damage Level		Dam Type	Alluvium	Comments
			Given	calc, dist given	calc, dist calc	Ours	Theirs			
1 Damage based on Cracking Data and Given Settlement. PGA Given or Recorded	Haiwee South	7.7	0.05	-	-	1	1	hydraulic earthfill	38	
	Chatsworth #2	5.3	0.40	0.23	-	2/3	2	hydraulic earthfill	-	
	San Fernando Lower	6.7	0.47	-	0.52	2/3	4	hydraulic earthfill	11	Liquefaction
	Dry Canyon	7.7	0.12	-	-	2	2	semi-hydraulic fill	20	
	Franklin Lower	6.7	0.10	-	0.08	1	-	hydraulic earthfill	-	One source gave PGA 0.3
4 Damage based on Given Settlement data only. PGA given or recorded	Murayama Upper (Kami)	8.2	0.30	0.30	-	3	2/1	semi-hydraulic fill	3	given PGA 0.12
	San Fernando Lower	6.5	0.48	0.53	0.48	5	5	hydraulic earthfill	11	Liquefaction
	San Fernando Upper	6.7	0.42	-	0.53	3	3	hydraulic earthfill	18	Liquefaction
	San Fernando Upper	6.5	0.55	0.55	0.48	4	4	hydraulic earthfill	18	Liquefaction
	San Leandro	7.8	0.35	0.89	-	2	1	semi-hydraulic fill	5	1906 Quake
6 Damage based on Cracking Data only. PGA given.	Chatsworth	5.3	0.09	0.14	0.15	2	1	hydraulic earthfill	-	
8 Damage based on calculated (Swaigood) settlement only. PGA given. Alluvial thickness assumed = 0 where not given (required for Swaigood)	Fairmont	6.5	0.20	0.25	0.33	1	0	hydraulic earthfill	-	
	Franklin Lower	6.5	0.20	0.25	0.28	1	0	hydraulic earthfill	-	
	Silver Lake	6.5	0.19	0.23	-	2	0	hydraulic earthfill	-	
	Fairmont	7.7	0.18	-	-	2	0	hydraulic earthfill	-	
	Chatsworth	6.5	0.28	0.37	0.26	2	0	hydraulic earthfill	-	
	Hawkins	7.1	0.25	0.31	0.12	2	1	hydraulic earthfill	-	
	Dry Canyon	6.5	0.55	0.65	0.52	4	0	semi-hydraulic fill	20	
	San Leandro (Chabot)	7.1	0.10	0.14	-	1	0	semi-hydraulic fill	5	
	Como	7.3	0.06	0.02	0.03	1	1	semi-hydraulic fill	-	
	Lake Temescal	7.8	0.35	0.95	0.86	4	1	semi-hydraulic fill	-	

Data For Concrete Face Rockfill Dams

Data Set	Dam Name	Magnitude	PGA			Damage Level		Dam Type	Allvm	Comments
			Given	calc, dist given	calc, dist calc	Ours	Theirs			
4 Damage based on Given Settlement data only. PGA given or recorded	Cogoti	7.9	0.19	0.10	-	2	1	concrete face rockfill	0	
	Cogswell	5.8	0.46	-	0.00	1	1	concrete face rockfill	0	
	Cogswell	6.7	0.10	0.13	0.09	0	1	concrete face rockfill	0	
	Cogoti	7.5	0.05	0.03	-	0	1	concrete face rockfill	0	
	Cogoti	7.1	0.04	0.05	-	0	0	concrete face rockfill	0	
	Cogoti	7.7	0.03	0.03	-	0	0	concrete face rockfill	0	
	Minase	7.5	0.08	0.03	-	1	1	concrete faced rockfill	-	
8 Damage based on calculated (Swaisgood) settlement only. PGA given. Alluvial thickness assumed = 0 where not given (required for Swaisgood)	Pinzanes	7.5	0.05	-	-	0	0	concrete faced rockfill	-	

APPENDIX D – Data Sets Used for the Cracking – Seismic Intensity Relationships

Data Set	Dam Name	Dam Type	Earthquake	PGA	MAG	Crack Width	Comments
Cases that recorded both Transverse and Longitudinal Cracking	Oroville	earthcore rockfill	Oroville	0.1	5.8		-
	El Infernillo	earthcore rockfill	Guerrero	0.12	7.6		-
	Dry Canyon	semi-hydraulic fill	Kern County / Arvin-Tehachapi	0.12	7.7		-
	La Villita	earthcore rockfill	Michoacan-Guerrero	0.12	8.1		-
	Buena Vista	earthfill - some puddle core	Kern County / Arvin-Tehachapi	0.3	7.7		-
	Los Angeles	asphalt face embankment	Northridge	0.32	6.7	1	liquefaction
	Matahina	earthcore rockfill	Bay of Plenty	0.33	6.3		-
	Piedmont	homogeneous embankment	San Francisco	0.35	8.3		-
	Vascona	earthfill	Loma Prieta	0.37	7.1	3	-
	Fukatani	earthcore rockfill	Hyougoken Nambu	0.38	7.2		-
	Los Angeles North Dyke	embankment	Northridge	0.42	6.7	6	liquefaction
	San Fernando Upper	hydraulic earthfill	Northridge	0.42	6.7	75	liquefaction
	Guadalupe	earthfill	Loma Prieta	0.42	7.1	12	-
	Elmer J Chesbro	zoned embankment	Loma Prieta	0.42	7.1	25	-
	Almaden	earthfill	Loma Prieta	0.43	7.1	3	-
	Lake Ranch	earthfill	Loma Prieta	0.43	7.1		-
	Lexington	zoned embankment	Loma Prieta	0.45	7.1	38	-
	San Fernando Lower	hydraulic earthfill	Northridge	0.47	6.7		liquefaction
	Austrian	homogeneous earthfill	Loma Prieta	0.57	7.1	250	-
	Birga	earthcore rockfill	Philippine / Luzon	0.6	7.7		-
	Masiway	zoned earthfill	Philippine / Luzon	0.65	7.7		liquefaction
	Hebgen	earth / rock / concrete core	Hebgen Lake / Yellowstone	0.71	7.6	75	-
	San Andreas	puddle core, rolled fill	San Francisco	0.8	8.3		-
	Crystal Springs Upper	earthfill puddle core	San Francisco	0.8	8.3		-
	Howell Upper	earthfill	San Francisco	0.8	8.3		-
	Tokiwa	zoned embankment	Hyougoken Nambu	0.93	7.2		-
Cases that recorded only Transverse Cracking	Birusawa	compacted earthcore earthfill	Miyagikan-Oki	0.02	7.4	50	-
	Franklin Lower	hydraulic earthfill	Northridge	0.08	6.7	6	-
	Liyutan	earthcore rockfill	Chi Chi	0.1	7.6		-
	Sheffield	concrete face earthfill	Santa Barbara	0.15	6.3		liquefaction
	Rubio	embankment ...	Northridge	0.15	6.7	5	-
	Wide Canyon	earthfill	Landers	0.2	7.5	12	-
	Brand DB	embankment ...	Northridge	0.28	6.7	1.25	-
	Sycamore Canyon	embankment ...	Northridge	0.28	6.7	2	-
	Las Lajas	embankment	Northridge	0.4	6.7	1	-
	Taniyama	zoned earthfill	Hyougoken Nambu	1	7.2		-

Data Set	Dam Name	Dam Type	Earthquake	PGA	MAG	Crack Width	Comments
Cases that Recorded Both Longitudinal and Transverse Cracking	Oroville	earthcore rockfill	Oroville	0.1	5.8		-
	El Infiernillo	earthcore rockfill	Guerrero	0.12	7.6	35	-
	Dry Canyon	semi-hydraulic fill	Kern County / Arvin-Tehachapi	0.12	7.7	50	-
	La Villita	earthcore rockfill	Michoacan-Guerrero	0.12	8.1	50	-
	Buena Vista	earthfill - some puddle core	Kern County / Arvin-Tehachapi	0.3	7.7		-
	Los Angeles	asphalt face embankment	Northridge	0.32	6.7		liquefaction
	Matabina	earthcore rockfill	Bay of Plenty	0.33	6.3		-
	Piedmont	homogeneous embankment	San Francisco	0.35	8.3		-
	Vascona	earthfill	Loma Prieta	0.37	7.1	18	-
	Fukatani	earthcore rockfill	Hyougoken Nannbu	0.38	7.2		-
	Los Angeles North Dyke	embankment	Northridge	0.42	6.7		liquefaction
	San Fernando Upper	hydraulic earthfill	Northridge	0.42	6.7		liquefaction
	Elmer J Chesbro	zoned embankment	Loma Prieta	0.42	7.1	100	-
	Guadalupe	earthfill	Loma Prieta	0.42	7.1	100	-
	Almaden	earthfill	Loma Prieta	0.43	7.1		-
	Lake Ranch	earthfill	Loma Prieta	0.43	7.1		-
	Lexington	zoned embankment	Loma Prieta	0.45	7.1	20	-
	San Fernando Lower	hydraulic earthfill	Northridge	0.47	6.7	100	liquefaction
	Austrian	homogeneous earthfill	Loma Prieta	0.57	7.1	460	-
	Binga	earthcore rockfill	Philippine / Luzon	0.6	7.7	300	-
	Masiway	zoned earthfill	Philippine / Luzon	0.65	7.7	100	liquefaction
	Hebgen	earth / rock / concrete core	Hebgen Lake / Yellowstone	0.71	7.6	300	-
	San Andreas	puddle core, rolled fill	San Francisco	0.8	8.3	75	-
	Crystal Springs Upper	earthfill puddle core	San Francisco	0.8	8.3	150	-
	Howell Upper	earthfill	San Francisco	0.8	8.3		-
	Tokiwa	zoned embankment	Hyougoken Nannbu	0.93	7.2	25	-
	La Villita	earthcore rockfill	Guerrero	0.02	7.6		-
	Lliu-Lliu	earthfill	La Ligua	0.05	7.1		-
	Cogoti	concrete face rockfill	Papudo-Zapallar	0.05	7.5		-
	Haivee South	hydraulic earthfill	Kern County / Arvin-Tehachapi	0.05	7.7	25	-
	El Infiernillo	earthcore rockfill	Michoacan aft shk	0.06	7.5		-
	Lo Ovalle	homogeneous embankment	Papudo-Zapallar	0.07	7.5		-

Cases of only longitudinal cracking						
Chatsworth	hydraulic earthfill	Santa Monica	0.09	5.3	75	-
San Luis	homogeneous earthfill	Loma Prieta	0.1	7.1	6	-
Murayama Lower (Shimo)	earthfill	Kwanto	0.12	8.2	25	-
Murayama Upper (Kami)	semi-hydraulic fill	Kwanto	0.12	8.2	125	-
El Infernillo	earthcore rockfill	Michoacan-Guerrero	0.13	8.1	150	-
Ainono	earthfill	Akita-Nantou	0.15	6.5		-
Culimo	earthfill	Papudo-Zapallar	0.16	7.5		-
Jen-I-Tan	earthcore rockfill	Chi Chi	0.19	7.6	160	-
Mackay	embankment	Borah Peak	0.22	7.3		-
Leroy Anderson	earthcore rockfill	Loma Prieta	0.23	7.1	30	-
Vir	earthfill	Koyna	0.24	6.5	38	-
Hawkins	hydraulic earthfill	Loma Prieta	0.25	7.1		-
Coastways	earthfill	Loma Prieta	0.25	7.1		-
Mill Creek	hydraulic earthfill	Loma Prieta	0.27	7.1		-
Ono	puddle clay core	Kwanto	0.3	8.2	250	-
Lake Temescal	semi-hydraulic fill	San Francisco	0.35	8.3		-
Surgu	rockfill	Malatya	0.38	5.9	200	-
Chatsworth #2	hydraulic earthfill	Santa Monica	0.23	5.3	75	-
Doube	homogeneous earthfill	Tangshan	0.4	7.8		liquefaction
Leroy Anderson	earthcore rockfill	Morgan Hill	0.41	6.2		-
Newell Creek	earthcore rockfill	Loma Prieta	0.43	7.1	225	-
Bay Street	earthfill	Loma Prieta	0.43	7.1		-
Almaden Valley	earthfill	Loma Prieta	0.43	7.1		-
Howell Lower	earthfill	Loma Prieta	0.43	7.1		-
Shui Shui	earthfill concrete core	Chi Chi	0.45	7.6	70	-
La Palma	earthfill	Chilean	0.46	7.8	1200	liquefaction
San Fernando Lower	hydraulic earthfill	San Fernando	0.48	6.5		liquefaction
Chesbro	earthfill	Loma Prieta	0.5	7.1	100	
San Fernando Upper	hydraulic earthfill	San Fernando	0.55	6.5		liquefaction
Ambuklao	earthcore rockfill	Philippine / Luzon	0.6	7.7		-
La Marquesa	earthfill	Chilean	0.6	7.8	800	liquefaction
Coyote	earthcore rockfill	Morgan Hill	0.63	6.2	300	-
Otani	zoned earthfill	Hyougoken Nannbu	1	7.2		-
Toulh	-	Chi-Chi	1	7.6		-
Showa-ike	earthfill	Hyougoken Nannbu	1.06	7.2		-

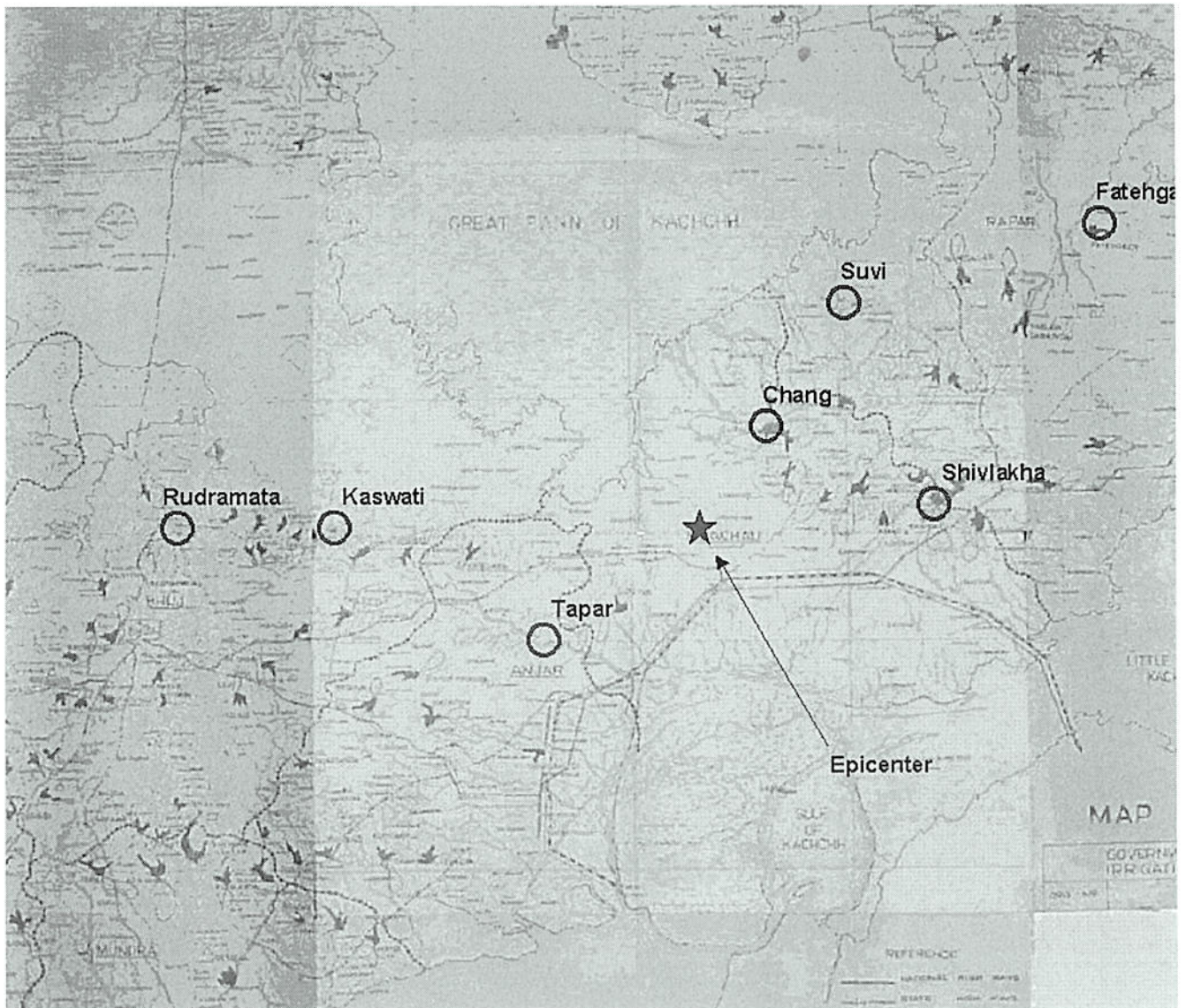
APPENDIX E – Preliminary Report on Effect of Gujarat Earthquake on Embankment Dams

Dams

This preliminary report investigates the following seven dams:

- [Shivlakha Dam](#)
- [Chang Dam](#)
- [Fatehgadh Dam](#)
- [Kaswati Dam](#)
- [Rudramata Dam](#)
- [Suvi Dam](#)
- [Tapar Dam](#)

These dams are located in a region of Gujarat, which is arid with little topography and covered by scrub brush and stretches of desert. The monsoons inundate the region for a few months every year bringing much needed water. The soil tends to be very fertile when irrigation water is available. The number of dams in this region is in the range of 180. The term used to describe the regional water management tactics is "water farming," as the dams here are more for retention than detention. The dams capture the monsoon deluge, taking out the peak to decrease the hazard of flooding and erosion, and temporarily hold some of the water. By this process the groundwater is effectively recharged for the coming growing season and some surface water is retained in the larger reservoirs year round.



Location of the seven dams described in this preliminary report and the location of the epicenter of the main shock of January 26.

Shivlakha Dam



Shivlakha Dam experienced a failure that cut through the core. Cracking and slumping were observed on the up and downstream faces. Extensive failure of the upstream face occurred with cracks on the order of 3m vertical and horizontal. The abutment rock is composed of sandstone and mudstone.



Longitudinal crest cracking was evidence of both upstream and downstream failures.



Large cracks on the upstream face.



A bulge occurred at the upstream toe. A local farmer just upstream of the dam reported that water issued from ground fissures for approximately 0.5 hours on the day of the earthquake.

Chang Dam



This dam experienced catastrophic failure of both the upstream and down stream faces with an overall crest drop of up to 10m. Chang Dam is located approximately 2km from observed tectonic ground deformations.



The large tree on the far right side of this picture is at the original crest elevation of the dam. Cracks can be seen along the entire length of the dam.





Numerous sand boils and fissures were observed along the toe region of the dam. Wide spread liquefaction occurred in and near the dam, and also along a large portion of the edge of the reservoir where lateral spreading on the order of 5m occurred. The ejecta was a silty sand with a clay crust on the surface. Huge quantities of ejecta covered a region of 200m² at the toe of the dam.



The silty sand ejecta remained saturated even after being exposed to direct sun light and high temperatures for up to 20 days after the main shock. Here the ejecta is shown to liquefy quite easily due to an impact load from a rock lobbed into the air.



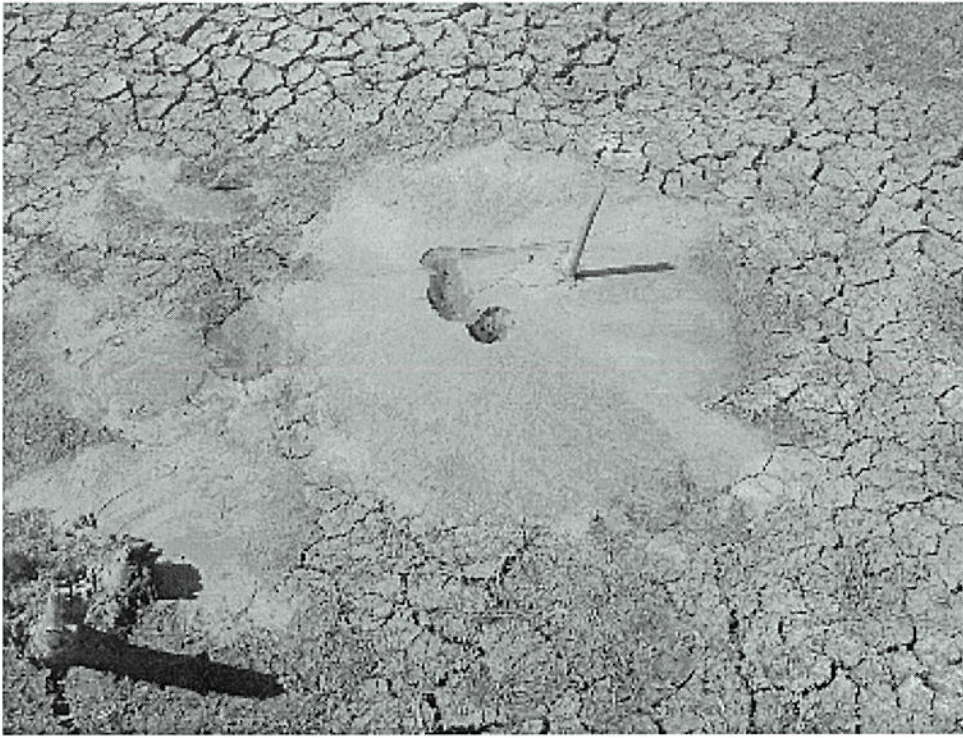
And the addition of a slight vibrational loading to the embedded rock results in further, more extensive, liquefaction.

Fatehgadh Dam

This dam is 4km long and crosses at least two main stream channels.



The observed damage and deformation occurred in conjunction with the location of these two stream channels.



Sand boils and ground cracking due to lateral spreading were found along the upstream toe of the dam at both locations where large deformations of the dam occurred.



Cracks up to 6m deep were found on the upstream face with both translation and rotation evident.



Large toe bulges consisting of silt and clay along the upstream toe.

Rudramata Dam



Cracking and slumping were observed along the upstream face. Scarps were up to 1.5m high with some evident rotation. Longitudinal cracks along the crest of the dam as well as toe bulges were observed.





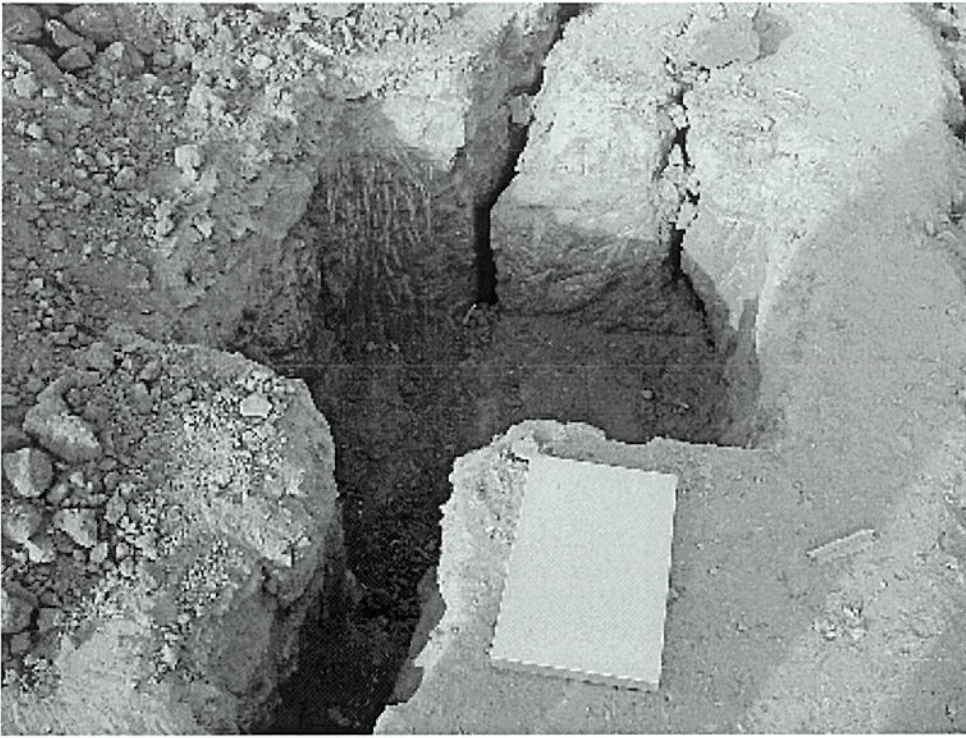
Uncontrolled seepage at the downstream toe began 2 days after the main shock. The seepage location corresponds to the damage on the upstream face.

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Kaswati Dam



Complex failures occurred along the entire length of this dam, extending to the bedrock abutment.



Longitudinal crest cracks up to 75mm wide and 1m deep stretch over a significant length of the dam.



Cracking was observed on the upstream face of the dam up to 2m deep and 0.75m across.



Toe bulging of silt and clay material.

Suvi Dam



This 2.7km long dam had poor construction details. The crest was raised without a corresponding raise in the core, resulting in several meters of pervious material at the crest. A parapet wall was placed at the top but not tied into the core, resulting in a top section of the dam that could allow uncontrolled through-flow. The parapet wall was damaged along 60% of the dam length.



Significant longitudinal crest cracking with a 1m drop in crest height at the deepest section. Collapse of a sluice gate and associated stairway occurred. Cracking and deformation was observed on the upstream face with scarps on the order of 0.75m high.

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Tapar Dam



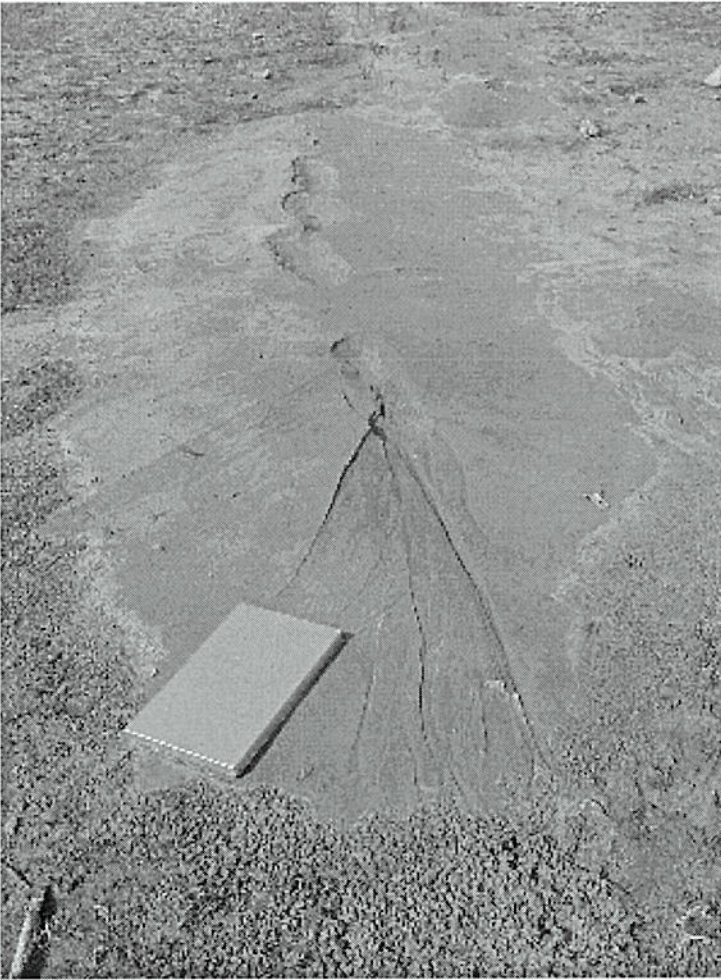
Tapar dam is the most important dam to the region, supplying drinking and agricultural water to the surrounding urban and rural cities and towns. Longitudinal crest cracking up to 1m wide and 300m long was observed.



Failures on the upstream face with scarps on the order of 0.75m high.



Lateral spreading at the toe and along the lower upstream face.



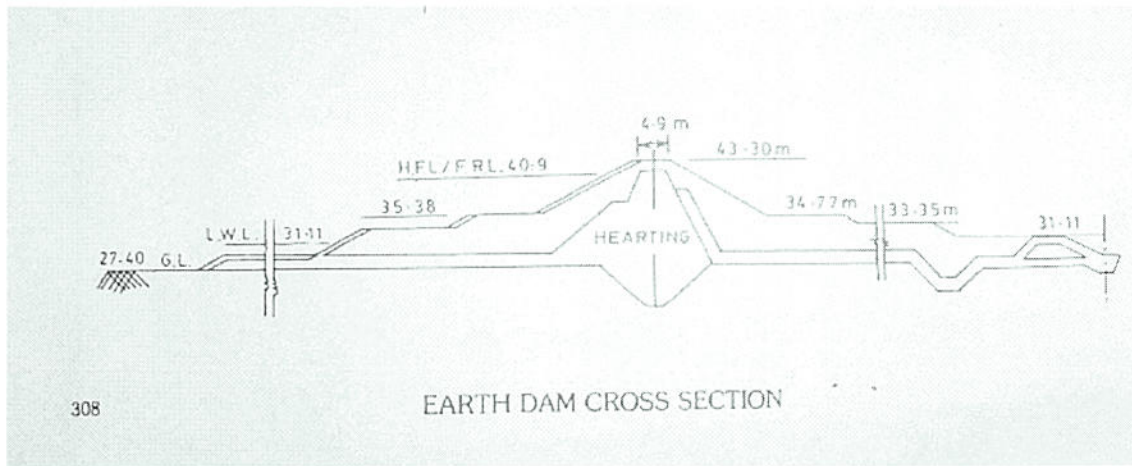
Extensive liquefaction along the upstream toe with ejecta consisting of silty sand. Sand boils also occurred downstream.



Lateral spreading resulting in a down-dropped section.



Cracking here associated with liquefaction along the upstream toe.



Cross section of Tapar Dam.