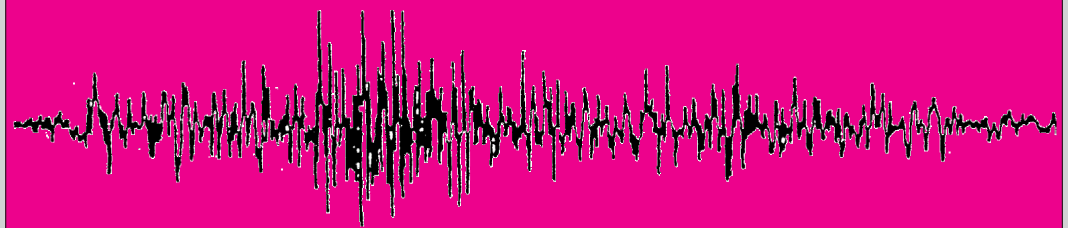
# DESIGN FEATURES OF DAMS TO RESIST SEISMIC GROUND MOTION

*Guidelines and case studies*

# ASPECTS DE LA CONCEPTION PARASISMIQUE DES BARRAGES

*Recommandations et exemples*

**Bulletin 120**



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**2001**

DESIGN FEATURES OF DAMS TO RESIST SEISMIC GROUND MOTION
ASPECTS DE LA CONCEPTION PARASISMIQUE DES BARRAGES

**120**

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Cover photograph (from Dr. M. Wieland)
Sefid Rud Dam (Iran) just after the June, 1990 Manjil Earthquake (M 7.5)

*Photo de couverture (d'après Dr. M. Wieland)*
*Le barrage Sefid Rud (Iran) juste après le Séisme Manjil de juin 1990 (M 7,5)*

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*Guidelines and case histories*

# ASPECTS DE LA CONCEPTION PARASISMIQUE DES BARRAGES

*Recommandations et exemples*

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(*) En anglais seulement

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(*) In English only

# AVANT-PROPOS

Le présent Bulletin a été élaboré après plusieurs années de discussion au sein du Comité des Aspects Sismiques des Projets de Barrages et est destiné à servir de guide utile de conception. Il répond essentiellement à deux questions : « Quel a été le comportement des barrages soumis à des charges sismiques ? » et « Que devons-nous changer dans la conception des barrages situés dans des régions sismiques ? ». Il s'appuie beaucoup sur de précédentes publications de l'USCOLD, telles que « Comportement observé sur des barrages au cours de séismes » et indique les nombreux aspects structuraux à prendre en compte pour divers types de barrage. Il présente des recommandations et constituera un guide précieux pour les prises de décisions en matière de conception parasismique des barrages, depuis le choix du site jusqu'à celui des appareils de mesures.

En se basant sur les travaux du Comité USCOLD des Séismes, le Bulletin a été préparé par J. L. Ehasz (Président du Comité USCOLD), qui fut le coordinateur pour cette partie des activités du Comité CIGB des Aspects Sismiques des Projets de Barrages, une discussion avec présentation d'observations ayant eu lieu au sein de ce Comité.

Il faut signaler l'importance particulière de l'Annexe A du Bulletin constitué de tableaux indiquant « le comportement de barrages au cours de séismes enregistrés dans le passé », tableaux établis par l'USCOLD en tant que revue générale et par le Comité Japonais des Grands Barrages (sous la conduite du Prof. Ch. Tamura) pour les barrages japonais. Ils donnent une image d'ensemble du comportement des barrages à travers le monde.

Une autre partie importante du Bulletin est l'Annexe B présentant une série d'exemples intéressants, extraits de la publication USCOLD « Comportement observé sur des barrages au cours de séismes ».

Le présent Bulletin constitue une source d'informations précieuses sur le comportement des barrages au cours de séismes et présente d'utiles recommandations relatives à la conception des barrages situés dans des zones sismiquement actives.

A. BOZOVIC

Président du Comité des Aspects Sismiques
des Projets de Barrages

# FOREWORD

This report has been considered within the ICOLD Committee on Seismic Aspects of Dam Design for several years in order to serve as a useful design guide document. It essentially responds to the questions : "How have dams performed under seismic loading?" and, "What should we do differently with the design of dams in earthquake country?". It draws heavily on earlier USCOLD publications such as "Observed Performance of Dams during Earthquakes", and then outlines numerous structural features to be considered for various types of dams. It is a guideline document and should be used in guiding decisions on seismic design ranging from site selection to instrumentation.

Using the earlier work of USCOLD Committee on Earthquakes, this Bulletin was prepared by Mr. J. L. Ehasz (Chairman of USCOLD Committee), who was coordinator for this part of Committee activity, and then discussed and commented within the Committee on Seismic Aspects.

Of special value for this Bulletin are the tables of the "Historic Performance of Dams during Earthquakes" given in Appendix A, which were contributed by USCOLD as a general review and by JANCOLD (care of Prof. Ch. Tamura) for the Japanese Dams. Together they result in quite a comprehensive picture of worldwide dam performance.

Another important supporting instrument is the set of Case Histories given in Appendix B, obtained from the USCOLD publication on "Observed Performance of Dams during Earthquakes".

This Bulletin is intended to serve as a source of information on dam behaviour and performance during earthquakes and as Guidelines on design features of dams in seismically active environment.

A. BOZOVIC

Chairman, Committee on Seismic Aspects
of Dam Design

# 1. INTRODUCTION

Quelques barrages ont subi d'importants dégâts sous l'effet de séismes. À travers le monde, une rupture totale, d'origine sismique, n'a affecté qu'une douzaine de barrages ; ces ouvrages étaient principalement des barrages de stériles ou des barrages par remblayage hydraulique, ou des digues en terre, relativement petites, de conception ou de construction anciennes et peut-être inadéquates. Seuls six barrages-poids en béton ou barrages en remblai, de dimensions importantes, ont été sévèrement endommagés. Plusieurs d'entre eux subirent une rupture presque totale et furent remplacés. Aucun barrage-voûte n'a connu d'importants dégâts, même si certains ouvrages de ce type furent soumis à de fortes secousses sismiques. Aux États-Unis, plus de 6 800 barrages ont une hauteur dépassant 15 m, plus de 1 640 une hauteur dépassant 30 m, et plus de 440 une hauteur dépassant 60 m. Si l'on considère le nombre total de grands barrages existant dans le monde, et le nombre limité de ruptures survenues, le comportement observé est donc très satisfaisant.

Cependant, cette excellente constatation peut être due, en grande partie, au fait que peu de barrages ont subi des secousses sismiques d'intensité et de durée suffisantes pour compromettre l'intégrité structurale des barrages. Sauf pour quelques exemples bien connus, des niveaux de mouvements sismiques du sol, équivalant au Séisme de Dimensionnement, n'ont pas mis à l'épreuve la plupart des barrages existants. Réciproquement, quelques barrages ont subi des dégâts importants sous des secousses sismiques inférieures à celles prises en compte dans leurs projets. Au cours du séisme Chi-Chi de septembre 1999 (Taiwan), la présence d'une faille de direction à peu près normale au barrage de dérivation Shih-Kang, en béton, provoqua un déplacement différentiel vertical de 6 à 8 m entre les pertuis vannés près de l'appui rive droite. Il y eut écoulement d'eau à travers les pertuis endommagés, mais sans lâchure catastrophique d'eau de la retenue.

Au cours des trois dernières décennies, d'importants progrès ont été réalisés dans la connaissance de l'action des séismes sur les barrages en béton et les barrages en remblai. La possibilité actuelle d'enregistrer des secousses sismiques, grâce au développement des appareils de mesure, ainsi que les progrès dans les méthodes analytiques utilisant des programmes informatiques, ont permis une meilleure évaluation de la réponse sismique des barrages. Beaucoup d'efforts ont été consacrés à la formulation de méthodes analytiques et numériques en vue de la prévision du comportement sous l'effet d'un séisme donné, généralement représenté par un graphique approprié d'accélération dans le temps. D'importants progrès ont été également réalisés dans la définition des paramètres sismiques caractérisant l'exposition d'un barrage donné aux séismes. Par contre, les tâches concernant la conception parasismique des barrages ont connu un peu moins d'intérêt, probablement du fait que les concepteurs comprennent insuffisamment ce que signifient les données recueillies sur le comportement réel des barrages sous l'effet de charges sismiques.

Des méthodes analytiques peuvent être appliquées pour évaluer la réponse d'un barrage à un séisme. L'analyse nécessite une définition de la géométrie du barrage, des matériaux de construction et de leurs propriétés dynamiques, de la constitution de la

# 1. INTRODUCTION

Historically, earthquakes have damaged few dams significantly. On a worldwide basis, only about twelve dams are known to have failed completely as the result of an earthquake; those dams were primarily tailings or hydraulic fill dams, or relatively small earthfill embankments of older and, perhaps, inadequate design or construction. Only six embankment or concrete gravity dams of significant size have been severely damaged. Several of those experienced near total failure and were replaced. No arch dam has ever suffered significant damage, even though many such structures were subjected to substantial earthquake ground motions. In the United States alone, over 6800 dams are higher than 15 meters; over 1640 exceed 30 meters in height and over 440 exceed 60 meters. Hence, if one considers the total number of existing large dams on a world-wide basis, the current performance record appears outstanding, based on the limited number of failures that have occurred.

This excellent record, however, may be largely related to the fact that few dams have been shaken by earthquakes of sufficient duration and intensity to jeopardize the dams' structural integrity. Except for several well-known examples, levels of ground motion equivalent to the Design Basis Earthquake have not tested most existing dams. Conversely, a few dams have experienced significant damage under shaking less demanding than what had, or should have, been considered in their design. During the September 1999, Chi-Chi, Taiwan Earthquake, a secondary fault passing almost normal to the major Shih-Kang concrete diversion dam caused 6 to 8 m vertical differential displacement between gate bays near the right abutment. Water flowed through the broken bays, but without catastrophic release of the reservoir.

During the last three decades major progress has been made in the understanding of earthquake action on dams, both of concrete and embankment construction. The ability of recording actual earthquake motions, because of instrumentation development, as well as progress with analytical procedures based on computer programs, result in better appraisals of the seismic response of dams. A lot of effort has been devoted to the formulation of analytical and numerical methods to predict behavior caused by any given earthquake, generally represented by an appropriate acceleration time history. Also, considerable progress has been made in the definition of seismic input to characterize the seismic exposure of a given structure. On the other hand, the task of performing earthquake-resistant design of dams has met with somewhat less interest, probably due to the scarcity of the design professionals' understanding of the meaning of observed data on actual dam behavior under earthquake loading.

Analytical methods may be applied to estimate the response of a dam to an earthquake. The analysis requires definition of the dam geometry, construction materials and their dynamic properties, foundation composition and dynamic fondation et de ses caractéristiques dynamiques, des caractéristiques topographiques et sismiques du site du barrage. L'obtention de la corrélation optimale entre le barrage et le site requiert un processus itératif, consistant à ajuster l'utilisation des matériaux de construction disponibles dans la zone du barrage, l'emplacement du barrage dans le site, et la définition de la forme du site. L'ingénieur de construction et le concepteur-analyste, avec parfois des objectifs discordants chez le même ingénieur, procèdent à une itération entre les aspects techniques d'adaptation du site, les matériaux disponibles et la méthode de construction, et les aspects techniques concernant l'estimation du comportement de l'ouvrage sous l'effet d'un séisme. Même les meilleures méthodes analytiques ne fournissent pas de réponses numériques précises, mais quelques données sur les problèmes de conception en cours d'examen.

properties, dam site topographic features, and the earthquake characteristics at the damsite. Achieving the optimal correlation of dam and site requires an iterative process of adjusting the use of available construction materials in the dam section, locating the dam within the site, and the shaping of the site. The construction engineer and the analyst, and sometimes conflicting objectives within the same engineer, iterate between the technical aspects of adapting the site, available materials and the construction process, and the technical aspects of estimating the project's behavior when subjected to an earthquake. Even the best analytical methods do not provide precise numerical answers, but they do provide some insight into the design problems under consideration.

# 2. OBJET DU BULLETIN

L'objet des présentes Recommandations est de décrire brièvement le comportement observé sur des barrages soumis à des séismes, puis d'indiquer et de décrire les aspects structuraux à prendre en considération pour que l'ouvrage résiste efficacement aux mouvements sismiques du sol. De telles considérations porteront essentiellement sur les reconnaissances spéciales, les techniques de conception et de construction qui sont nécessaires pour les barrages situés dans des zones sismiquement actives.

Des modes caractéristiques de rupture concernant les divers types de barrages seront utilisés pour vérifier tout projet donné. L'adoption comme postulat ou hypothèse d'une série de phénomènes susceptibles de conduire à une rupture sous l'effet d'un séisme peut révéler une caractéristique intrinsèquement indésirable dans le projet de barrage en cours d'examen, ou mettre en évidence une modification souhaitable.

Le projeteur doit tenir compte des mouvements résultant de tout séisme prenant naissance à quelque distance du site du barrage, et du mouvement possible de la fondation si un système de failles actives traverse la zone du barrage. La présence d'une faille active sur le site du barrage est parfois inévitable et sera considérée comme un des plus sévères défis nécessitant une particulière attention. Les résultats d'analyses théoriques et leur comparaison avec des comportements réellement observés sur des barrages existants identiques, sous l'effet de séismes, fourniront une série de conclusions préliminaires concernant la forme optimale de divers types de barrages et le traitement des fondations sur un site donné.

# 2. PURPOSE OF THE BULLETIN

The purpose of these Guidelines is to briefly describe the observed performance of dams subjected to earthquakes, and then to outline and describe the structural considerations required to effectively resist seismic ground motions. Such considerations should essentially reflect the special investigations, design and construction techniques that are necessary for dams in a seismically active area.

Typical failure modes for the different kinds of dams should be used to test any given design. Postulating or hypothesizing a sequence of events that may result in a failure due to earthquake action could reveal an inherently undesirable feature of the dam design under consideration, or put in evidence some desirable change.

The designer must take into account the motions resulting from any earthquake at any distance from the damsite, and possible movement of the foundation if an existing active fault system crosses the dam alignment. Having an active fault cross the alignment is sometimes unavoidable, and should be considered as one of the most severe design challenges requiring special attention. The results of theoretical analyses, and their comparison with recorded cases available of actual seismic behavior of similar existing dams, provide a series of preliminary conclusions concerning the optimal shaping of alternative types of dams, and their foundation treatments at a given site

# 3. OBSERVATIONS ET COMPORTEMENT

Si un grand nombre d'informations ont été publiées sur le comportement des barrages, la documentation applicable est souvent très technique et difficilement accessible aux maître d'ouvrage ou au grand public. Un inventaire des principaux barrages ayant subi de fortes secousses sismiques figure dans l'Annexe A «Comportement de barrages au cours de séismes enregistrés dans le passé». Les informations comprennent les éléments suivants lorsqu'ils sont disponibles : paramètres sismiques principaux, type et dimensions du barrage, distance de l'épicentre, et au moins quelques indications grossières sur l'importance des dégâts subis, le cas échéant. Voir également «Comportement observé sur des barrages au cours de séismes», USCOLD, 1992, et Volume II, USCOLD, 1999 (Référence 1) ; et «Bibliographie sur le comportement de barrages au cours de séismes» (Référence 2).

Un ensemble d'exemples représentatifs est donné dans l'Annexe B, avec des informations détaillées sur le comportement des barrages sélectionnés ayant subi des secousses sismiques. L'examen des informations contenues dans les Annexes A et B montre que les barrages peuvent supporter, de façon satisfaisante, des charges sismiques, mais que les secousses sismiques constituent l'une des conditions de charges les plus sévères pour les barrages et doivent être examinées avec beaucoup d'attention.

En vue d'obtenir les données fondamentales permettant d'orienter les études de conception parasismique des ouvrages et de réduire les conséquences néfastes, il est nécessaire d'étudier le comportement observé sur des barrages au cours de séismes, et les conséquences sur l'intégrité structurale des différents types de barrages. Le type et la grandeur des déformations résiduelles, la fissuration observée et tout autre dégât subi par l'ouvrage principal et sa fondation sous l'effet d'un séisme présentent un grand intérêt.

## 3.1. COMPORTEMENT SISMIQUE DES BARRAGES EN REMBLAI

Le 17 janvier 1995, au Japon, le séisme Kobe (Magnitude Mw = 6,9), appelé également séisme HyogoKen Nanbu, survint à 20 km au sud-ouest de Kobe, cité très peuplée de 1,5 million d'habitants environ. Le mouvement de type bilatéral subi par la faille Nojima au cours de ce séisme était très semblable au mécanisme de rupture de faille lors du séisme Loma Prieta de 1989, en Californie. Il comportait une longueur de rupture estimée entre 30 et 50 km.

Trois petits barrages en terre, faisant partie du système de réservoirs de Koyoen, furent sévèrement endommagés. Ils étaient situés dans la zone de l'épicentre, à quelques kilomètres de l'endroit où de vieilles maisons subirent d'importants dégâts. Un autre petit barrage en terre, le barrage Niketo, proche également de la zone de fortes intensités sismiques, fut complètement détruit. Lorsque le séisme se produisit, les réservoirs de Koyoen avaient leur niveau d'eau très bas, par suite d'une longue période

# 3. OBSERVATIONS AND PERFORMANCE

While considerable information has been published on the performance of dams, the applicable literature is often very technical and not easily accessible to dam owners or the general public. An inventory of principal dams that have experienced significant earthquake shaking is presented in Appendix A, entitled "Historic Performance of Dams During Earthquakes". This information includes, where available, principal earthquake parameters, dam dimensions and types, epicentral distances, and at least some crude indications of the severity of the damage incurred, if any had been reported. See also "Observed Performance of Dams During Earthquakes," USCOLD, 1992 and Volume II, 1999 (Reference No 1), and "Bibliography on Performance of Dams During Earthquakes" (Reference No 2).

A representative set of Case Histories is given in Appendix B, with more comprehensive information on the behavior of selected dams subjected to seismic shaking. The combination of information in Appendices A and B constitute the basis that dams can withstand seismic loads satisfactorily, however, seismic shaking still constitutes one of the most severe loading conditions for dams and must be addressed.

The basic input to guide structural design philosophy to produce designs that efficiently resist seismic forces and mitigate adverse consequences, requires the study of observed behavior of dams during earthquakes, and the consequences on the structural integrity of different types of dams. Of particular interest are the kind, type and magnitude of residual deformations, observed cracking, and any other damage to the main dam structure and its foundation resulting from seismic action.

## 3.1. EARTHQUAKE PERFORMANCE OF EMBANKMENT DAMS

The January 17, 1995 Kobe, Japan Earthquake (Mw = 6.9), also named the HyogoKen Nanbu Earthquake, occurred 20 km southwest of Kobe, a densely populated city with a population of approximately 1.5 million people. The bilateral mode of movement of the Nojima Fault experienced during that event was very similar to the fault rupture mechanism of the 1989 Loma Prieta, CA Earthquake. It involved a rupture length estimated at between 30 and 50 km.

Three small earth dams, belonging to the Koyoen Reservoir system were severely damaged. They were located within the epicentral area, a few kilometers away from where extensive damage occurred to older homes. Another small earth dam, Niketo dam, also near the zone of large seismic intensities, collapsed completely. The Koyoen reservoir pools were quite low when the earthquake occurred, due to a prolonged dry period. A post-earthquake reconnaissance report prepared by the U.S.

de sécheresse. Les reconnaissances effectuées après le séisme firent l'objet d'un rapport préparé par l'U.S. Army Corps of Engineers Waterways Experiment Station (WES) (Référence 3), indiquant que les remblais de Koyoen avaient chacun une longueur de 70 m environ, une hauteur de 7,5 à 10 m et des talus de fruit 2 : 1 (h/v). Ils étaient constitués de matériaux de bonne granulométrie, légèrement cohérents, avec des dimensions granulométriques correspondant aux catégories gravier, sable et silt, avec un peu d'argile. Les talus avaient un revêtement en béton.

Les dégâts causés à d'autres barrages en remblai par le séisme Kobe furent peu importants. Le barrage Tokiwa, digue en terre zonée, d'une hauteur de 33 m, situé à 10 km environ de l'épicentre, subit une fissuration modérée dans le revêtement de crête, près des appuis. Une de ces fissures s'étendait jusqu'au noyau, mais restait dans la zone de la revanche. Le barrage Kitamaya, de 24 m de hauteur, constitué d'un remblai en granite décomposé avec un drain cheminée vertical, était situé à 31 km de l'épicentre. Il subit un glissement superficiel, peu profond, sur le talus amont. Aucun autre dégât ne fut observé sur des barrages en terre de hauteur supérieure à 12 m. Toutefois, des barrages en remblai plus petits subirent divers types de dégâts, tels que des fissures longitudinales, des fissures transversales, des tassements, des déformations du corps du barrage, et jusqu'à une rupture totale. Les dégâts limités sur les barrages en remblai peuvent s'expliquer, en partie, par l'évaluation des niveaux d'accélération de pointe sur les sites de barrages, qui furent estimés à environ 0,22 g sur les sites rocheux où les barrages étaient construits.

Le séisme Northridge, Californie, survenu le 17 janvier 1994, était centré à 32 km environ à l'ouest-nord-ouest de la vallée San Fernando sur une faille de chevauchement sans affleurement, avec pendage sud–sud-ouest sous la vallée. Le séisme Northridge, qui causa d'importants dégâts à des bâtiments, à des ponts sur autoroutes, présenta un grand intérêt pour les ingénieurs de barrages. Tout d'abord, il soulignait de nouveau le risque sismique associé aux failles masquées, en Californie, région où les phénomènes tectoniques étaient considérés comme raisonnablement bien connus. Ensuite, ce séisme était le deuxième événement important, en moins de 25 ans, affectant la vallée de San Fernando.

En 1971, le séisme San Fernando, Californie, a sévèrement endommagé plusieurs barrages en remblai (digues par remblayage hydraulique) et causé la rupture presque totale du barrage inférieur (barrage Lower Van Norman) du complexe de retenues Van Norman. Ce barrage est parfois désigné dans la littérature sous le nom de Lower San Fernando. Le séisme Northridge de 1994 provoqua des mouvements du sol, parfois très intenses, sur les sites de 105 barrages se trouvant dans un rayon de 75 km autour de son épicentre. Ces barrages comprenaient la plupart de ceux ayant subi des secousses en 1971.

Le séisme Northridge causa quelques fissures et des mouvements de talus sur onze barrages en terre et en enrochement. Cependant, aucun de ceux-ci ne présenta un danger immédiat pour la vie et les biens. Ce comportement satisfaisant peut résulter, en grande partie, du fait qu'en Californie la plupart des barrages importants ont été réévalués pour le plus grand séisme possible, lors des études entreprises après le séisme San Fernando. Les remblais douteux ou présentant un danger ont été renforcés ou mis hors service, ou bien les maîtres d'ouvrage ont été dans l'obligation d'exploiter les retenues avec une plus grande hauteur de revanche.

Army Corps of Engineers Waterways Experiment Station (WES) (Reference No 3) indicated that the Koyoen embankments were each about 70 m long 7.5 to 10 m high, with slopes of about 2:1 (horizontal to vertical). They were built of a well-graded, slightly cohesive mixture of materials ranging in size from gravel, sand and silt, with some clay. The slopes were faced with concrete.

Damage to other embankment dams from the Kobe Earthquake was limited. Tokiwa Dam, a zoned earthfill dam with a height of 33 m, about 10 km from the epicenter, experienced moderate cracking in the crest pavement, near both of the abutments. One of these cracks extended to the core, but remained confined within the freeboard zone. Kitamaya Dam, a 24 m high embankment, built of decomposed granite with a vertical chimney drain, was about 31 km away from the epicenter. It experienced shallow surficial sliding of its upstream slope. No other damage was observed in earthfill dams taller than 12 m. Smaller embankment dams, however, suffered various forms of damage such as longitudinal cracking, transverse cracking, settlement, deformation of the dam body, and up to complete failure. The limited damage to embankment dams could be partially explained by the overall assessment of peak acceleration levels at dam sites, which was estimated to be of the order of 0.22 g at rock sites where dams were built.

The Northridge Earthquake, CA occurred on January 17, 1994 and was centered about 32 km west-northwest of the San Fernando Valley on a blind thrust fault dipping south-southwest below the valley. In addition to considerable damage being inflicted on buildings, lifelines and highway bridges, the Northridge Earthquake was significant to the dam engineering profession for two reasons. First, it reemphasized the seismic hazard associated with concealed faults in California, a region where the distribution of tectonic features was thought to be reasonably well understood. Secondly, it was the second significant event in less than 25 years to affect the San Fernando Valley.

In 1971, the San Fernando Earthquake severely damaged several embankment (hydraulic fill) dams, and caused near-total failure of the lower dam (Lower Van Norman Dam) of the Van Norman Reservoir complex. This dam has sometimes been referred in the literature as the Lower San Fernando Dam. The 1994 Northridge Earthquake induced ground motions, sometimes quite severe, at 105 dams located with a 75 km radius of its epicenter. These dams included most of those shaken in 1971.

Eleven earthfill and rockfill dams experienced some cracking and slope movements a result of the Northridge Earthquake. Yet, none of these presented an immediate threat to life and property. This satisfactory performance may result, to a significant extent, from the fact that, in California, most significant dams have been reevaluated for the Maximum Credible Earthquake (MCE), during investigations initiated after the San Fernando Earthquake. Questionable or unsafe embankments have been upgraded or decommissioned, or the owners have been required to operate the reservoirs with an increased freeboard.

Un des rares barrages en remblai ayant subi des dégâts notables lors du séisme Northridge fut, de nouveau, le barrage Lower Van Norman, de 38 m de hauteur, construit par remblayage hydraulique. Depuis 1971, le barrage ne sert plus pour le stockage d'eau, la retenue vide étant utilisée pour la maîtrise des crues. Des fissures de 5 à 9 cm d'ouverture, de plusieurs centaines de mètres de longueur, furent constatées. Certaines avaient au moins 1,5 m de profondeur. Des zones de sable en agitation et un trou d'affaissement furent observés également le long du parement amont. Le tassement maximal en crête atteignait 20 cm, et le déplacement horizontal maximal en crête était de 10 cm environ vers l'amont. Les fissures et les mouvements endommagèrent gravement le talus amont et une partie d'un conduit d'évacuation s'effondra (Référence 4). Si le vieux barrage avait stocké autant d'eau qu'en 1971, les dégâts auraient été beaucoup plus importants, ou au moins identiques à ceux causés par le séisme de 1971.

Le barrage Upper Van Norman, de 25 m de hauteur, avec une retenue maintenue également vide depuis qu'il fut sérieusement endommagé en 1971, subit des fissures transversales près de l'appui rive droite, sur le talus aval, et près de l'appui rive gauche, atteignant 18 m de longueur et 5 à 7,5 cm d'ouverture. Les déplacements irréversibles maximaux en crête étaient d'environ 70 cm pour les tassements et de plus de 15 cm pour les mouvements horizontaux vers l'amont.

Le barrage Los Angeles, de 40 m de hauteur, qui remplace maintenant les deux barrages San Fernando, est situé entre les deux barrages en remblai précités, dont les retenues vides servent à la maîtrise des crues. Les secousses du sol furent très fortes, avec des amplitudes parmi les plus élevées jamais enregistrées. Cependant, seules de faibles déformations et une fissuration superficielle furent observées sur le barrage. Malgré les fortes secousses sismiques, la crête du barrage se déplaça seulement de 5,6 cm horizontalement et tassa seulement de 8,9 cm au droit de la section transversale de hauteur maximale. Cette amélioration de comportement résulte des importants progrès dans la science des séismes, d'une meilleure compréhension du comportement des barrages sous l'action des séismes, et des progrès dans les techniques de construction. Le barrage Los Angeles, conçu pour résister à de fortes secousses sismiques, resta intact lors du séisme Northridge. Par contre, les barrages Lower et Upper Van Norman, conçus à partir de normes sismiques moins rigoureuses et exécutés suivant des méthodes de construction de 1912-1915, subirent d'importants dégâts au cours des deux séismes de 1971 et 1994.

Enfin, le séisme Northridge causa de faibles dégâts, sous la forme de fissures transversales et de tassements, aux barrages Lower Franklin (hauteur : 31 m), Santa Felicia (hauteur : 65 m), Sycamore Canyon (hauteur : 12 m), Schoolhouse Debris Basin (hauteur : 11,5 m), Cogswell (hauteur : 80 m), Porter Estate (hauteur : 12,5 m) et Rubio Basin (hauteur : 19,5 m) (Référence 5).

Le 17 octobre 1989, le séisme Loma Prieta (magnitude M 7,1), Californie, affecta une large zone de la Baie de San Francisco, une douzaine de barrages en remblai subissant de fortes secousses. Plus de 100 barrages de diverses dimensions, la plupart étant des barrages en remblai, étaient situés dans un rayon de 100 km autour de l'épicentre. Cet événement démontra une fois de plus que des barrages en remblai bien conçus et bien construits pouvaient faire face, en toute sécurité, à de fortes secousses sismiques. Il indiqua aussi que des barrages situés dans des zones de forte séismicité étaient rarement mis à l'épreuve de la pleine magnitude du mouvement du sol, qui doit être prise en compte dans leur projet.

One of the few embankment dams that suffered noticeable damage from the Northridge Earthquake was, again, the 38 m high Lower Van Norman Dam, a hydraulic fill dam. The dam has been abandoned as a water storage facility since 1971, but is still used with empty reservoir for flood control. It experienced 5 to 9 cm-wide cracks, several hundred meters long. Some of these cracks were at least 1.5 m deep. Sand boils and a sinkhole was also observed along the upstream face. Maximum crest settlement was 20 cm, and maximum horizontal crest movement was about 10 cm toward upstream. Cracking and movement seriously damaged the upstream slope and a portion of an outlet pipe collapsed (Ref. No. 4). Had the old dam been holding as much water as it had in 1971, the damage could have been far greater or at least equal to that of the 1971 earthquake.

The 25 m high Upper Van Norman Dam (also left with an empty reservoir since it was also severely damaged in 1971) experienced transverse cracks near its right abutment, on the downstream slope, and near its left abutment, up to 18 m long and 5 to 7.5 cm wide. Maximum non-recoverable crest displacements were about 70 cm of settlement, and over 15 cm of horizontal upstream movement.

The 40 m high Los Angeles Dam, which now replaces the two San Fernando dams, is located in-between these two flood-control, dry embankments. Ground shaking was very strong, with amplitudes among the highest ever recorded. Yet, the dam showed only minor deformation and only superficial cracking. Despite the intense shaking the crest of the dam moved only 5.6 cm horizontally and settled only 8.9 cm at its maximum section. This improvement in performance comes from major advances in earthquake science, improved understanding of how embankment dams perform during earthquakes and advanced construction practices. The Los Angeles dam, designed to withstand severe shaking, remained intact during the Northridge earthquake. In contrast, the Lower and Upper Van Norman Dams, which were designed to less stringent seismic standards and with 1912-1915 construction methods, suffered major damage during both the 1971 and 1994 earthquakes.

Lastly, the Northridge Earthquake caused minor damage in the form of transverse cracks and settlement to Lower Franklin Dam (31 m high); Santa Felicia Dam (65 m high); Sycamore Canyon Dam (12 m high); Schoolhouse Debris Basin Dam (11.5 m high); Cogswell Dam (80 m high); Porter Estate Dam (12.5 m high); and Rubio Basin Dam (19.5 m high). (Reference No.5)

The October 17, 1989 Loma Prieta, California, U.S.A. Earthquake (M 7.1) affected a wide region of the San Francisco Bay Area and induced strong shaking in about twelve embankment dams. Over 100 dams of various sizes, most of them embankment dams, were located within 100 km from the epicenter. This event once more documented the ability of well-designed and constructed embankment dams to safely withstand severe ground motions. It also emphasized how rarely dams situated in areas of high seismic exposure are tested to the full magnitude of the ground motion that must be considered in their design.

On peut raisonnablement dire que les barrages affectés par le séisme Loma Prieta peuvent supporter des séismes de plus forte intensité et de plus longue durée que celles enregistrées lors du séisme du 17 octobre 1989. Cela est dû au fait que les secousses maximales (accélérations supérieures à 0,05 g) durèrent moins de huit secondes sur des sites rocheux et de sol compact dans la zone de l'épicentre, durée relativement courte pour une magnitude supérieure à 7. En outre, au moment du séisme, la plupart des retenues étaient remplies entre 10 et 50 % de leur capacité maximale, par suite de plusieurs années consécutives de faible pluviosité. De ce fait, cette sécheresse a pu avoir un effet bénéfique quant à la résistance sismique des barrages en terre affectés, étant donné que les lignes de saturation à l'intérieur des remblais étaient probablement au-dessous de leur situation normale. Les charges hydrodynamiques, affectant les barrages en béton plus que les barrages en remblai, étaient également réduites de façon appréciable du fait des niveaux bas des retenues. Tous les barrages concernés sauf un ont eu un comportement satisfaisant, comme cela avait été généralement prédit dans les précédentes études d'évaluation.

Le barrage Austrian, digue en terre, de 60 m de hauteur, située à 12 km environ de l'épicentre du séisme Loma Prieta, dont le niveau de la retenue était à mi-hauteur au moment du séisme, subit une fissuration importante à l'appui et un tassement maximal en crête de près de un mètre. Les déformations irréversibles induites par le séisme sur le barrage Austrian restaient bien inférieures à la valeur de 3 m à laquelle avait conduit l'application du séisme de dimensionnement (de projet), de magnitude M 8,3, dont l'épicentre était situé sur la faille San Andreas à la distance la plus proche du barrage Austrian. Mais le séisme de 1989 était moins fort que le séisme de dimensionnement local en durée et en énergie sismique. Les tassements observés du remblai en sable graveleux-argileux pouvaient ne pas avoir été prédéterminés sous des conditions de charge identiques à celles appliquées, en se basant sur des méthodes numériques fréquemment utilisées dans l'évaluation de la sécurité des barrages. Le barrage Austrian présentant toujours une sécurité satisfaisante, cette expérience soulignait la nécessité constante de tirer des enseignements du comportement réel des barrages, afin que les méthodes d'évaluation de la sécurité sismique des ouvrages puissent être améliorées.

Outre les séismes susmentionnés, des rapports sur le comportement ou les dégâts subis par des barrages en remblai au cours d'une douzaine de séismes ont été obtenus. Les séismes les plus importants sont les suivants : San Francisco, Californie (1906) ; Kanto, Japon (1923) ; Santa Barbara, Californie (1925) ; Kern County, Californie (1952) ; Hebgen Lake, Montana (1959) ; Tokachi-Oki, Japon (1968) ; San Fernando, Californie (1971) ; Chili (1971, 1985) ; Mexico (1971, 1981, 1985) ; Edgecumbe, Nouvelle-Zélande (1987).

Le séisme San Francisco de 1906 (magnitude M 8,3, estimée) affecta 30 barrages en terre de dimensions moyennes, situés dans un rayon de 50 km autour de la trace de rupture de faille ; 15 de ces barrages étaient à moins de 5 km de cette trace. La plupart de ces barrages ont supporté les secousses avec des dégâts mineurs. Un tel comportement satisfaisant sous des conditions extrêmes de charge a été attribué plus à la nature argileuse de ces remblais qu'à leurs degrés de compactage.

Le séisme Kanto de 1923 (Japon) représente peut-être le premier cas où des dégâts importants survenus à un barrage en remblai ont été signalés. Le barrage Ono, digue en terre de 40 m de hauteur, subit de nombreuses fractures, dont une fissure plongeant de 21 m le long du noyau d'argile corroyée. Le tassement du barrage

The dams affected by the Loma Prieta Earthquake can reasonably be said to be capable of withstanding earthquakes of higher intensities and longer duration than were experienced during the October 17, 1989 event. This because the strong phase of shaking (accelerations greater than 0.05 g) during that earthquake lasted less than eight seconds at rock and firm soil sites in the epicentral area, a relatively short duration for a magnitude greater than 7. Also, at the time of the earthquake, most of the reservoirs were at between 10 to 50 percent of their maximum capacity, due to several consecutive years of low rainfall. Hence, the drought may have been a beneficial factor for the seismic resistance of the affected earthfill dams, since phreatic surfaces within the embankments were probably below normal. Hydrodynamic loads, which affect concrete dams more than embankment dams, were also significantly reduced as a result of low reservoir levels. All but one of the dams concerned performed well, as had been generally predicted in prior evaluation studies.

Austrian Dam, a 60 m high earthfill dam located about 12 km from the Loma Prieta epicenter, with a reservoir water level only at mid-height at the time of the earthquake, experienced substantial abutment cracking and a maximum crest settlement of nearly one-meter. The non-recoverable earthquake-induced deformations of Austrian Dam remained well below the 3 m which the dam had been predicted to experience under the applicable Design Basis Earthquake (DBE), a Magnitude 8.3 event and epicentered on the San Andreas Fault at its closest distance from Austrian Dam. But the 1989 earthquake was considerably less demanding than the local DBE in overall duration and seismic energy content. The observed settlements of this gravely clayey sand embankment might not have been predicted under loading conditions similar to those experienced, based on some frequently used numerical methods of dam safety evaluation. While Austrian Dam is still safe, this experience emphasized the constant need to learn from actual performance of dams, so that seismic safety evaluation procedures can be improved.

In addition to the above earthquakes, performance or damage reports for embankment dams had been obtained from approximately twelve earthquakes. The most significant of these included the San Francisco, CA (1906); Kanto, Japan (1923); Santa Barbara, CA (1925); Kern County, CA (1952); Hebgen Lake, MT (1959); Tokachi-Oki, Japan (1968); San Fernando, CA (1971); Chile (1971, 1985); Mexico (1979, 1981, 1985) and Edgecumbe, New Zealand (1987) earthquakes.

The 1906 San Francisco earthquake (M 8.3, estimated) affected about 30 medium-sized earthfill dams located within 50 km of the fault rupture trace; 15 of these dams were at a distance of less than 5 km from the rupture trace. The majority of these survived the shaking with only minor damage. Such satisfactory performance under extreme loading conditions has been attributed more to the clayey nature of these embankments than to their degrees of compaction.

The 1923 Kanto, Japan earthquake represents perhaps the first documented case of occurrence of significant damage to an embankment dam. Ono Dam, a 40 m high earthfill dam, was fractured in many places including a fissure that extended down 21 m along the puddled clay core. Ono Dam settled nearly 30 cm with longitudinal atteignit près de 30 cm, avec des fractures longitudinales jusqu'à 60 m de longueur et 25 cm d'ouverture ; des glissements locaux d'environ 18 m de longueur depuis l'escarpement jusqu'au pied se produisirent sur le talus aval.

Le séisme Santa Barbara de 1925 (magnitude M 6,3) causa le glissement catastrophique et la rupture du barrage Sheffield à Santa Barbara, Californie. Ce fut probablement un des premiers cas mettant en évidence les effets des secousses du sol sur des matériaux de faible densité, avec ruptures du remblai.

Au cours du séisme Kern County de 1952 (M 7,7), des barrages situés dans le sud de la Californie subirent des dégâts modérés. De sérieux dégâts furent causés au barrage Eklutna, de 6 m de hauteur, par le séisme Alaska de 1964 (M 8,4).

Ce ne fut qu'en 1971 lors du séisme San Fernando (M 6,5), Californie, que les préoccupations des ingénieurs concernant la vulnérabilité de certains types de barrages en terre furent confirmées. Les médias et le grand public portèrent une très grande attention à l'événement de 1971. Une grave catastrophe fut évitée de peu dans une zone très urbanisée. Le barrage Lower Van Norman, barrage par remblayage hydraulique, de 42 m de hauteur, subit une liquéfaction étendue et d'importantes ruptures de talus. Le déversement sur la crête et l'inondation d'une zone aval, où plus de 70 000 habitants étaient menacés, furent évités de justesse, et uniquement du fait que le niveau de retenue était relativement bas pour la saison lorsque le séisme survint. Le barrage Upper Van Norman, de 24 m de hauteur, fut également sérieusement endommagé. Le cas du barrage Lower Van Norman proche de la rupture constitua un jalon dans l'évaluation du comportement des barrages en terre. Il attira l'attention des ingénieurs et des organismes publics concernés par la sécurité des barrages sur la vulnérabilité des remblais constitués de sables et de silts saturés, mal compactés. Il conduisit également à de nombreuses réévaluations des conditions de sécurité de barrages et entraîna d'importants progrès dans le développement des méthodes numériques d'analyse dynamique des barrages.

Un autre événement récent, présentant de l'intérêt, est le séisme Mexico de 1985 (M 8,1) qui affecta deux grands barrages en terre-enrochement et en enrochement, La Villita (60 m de hauteur) et El Infiernillo (147 m de hauteur). Alors qu'aucun de ces barrages ne subit d'importants dégâts lors du séisme de 1985, ils furent soumis à partir de 1975 à une série de séismes faiblement espacés dans le temps, dont cinq avaient une magnitude supérieure à 7,1. Les tassements cumulés induits par les séismes sur le barrage La Villita, digue en terre-enrochement avec un noyau étanche central épais en argile, approchent maintenant de un pour cent de sa hauteur initiale. Les tassements du barrage La Villita ont montré une tendance à augmenter lors d'événements plus récents, ce qui peut résulter d'un affaiblissement progressif des caractéristiques d'une partie des matériaux de remblai. Un tel comportement n'a pas été observé sur le barrage El Infiernillo, digue en enrochement avec noyau en terre, les déformations étant restées faibles et uniformes d'un événement à l'autre. El Infiernillo a une fondation rocheuse, tandis que La Villita a une fondation alluviale.

Des séismes de magnitude modérée, tels que le séisme Edgecumbe (M 6,2) de 1987, en Nouvelle-Zélande, qui endommagea le barrage Matahina de 78 m de hauteur, et le séisme Whittier Narrows (M 6,1) de 1987 en Californie (États-Unis), qui affecta plusieurs barrages en remblai dans la grande zone de Los Angeles, présentent beaucoup d'intérêt sur le plan de la technique des barrages en raison de la grande cracks up to 60 m long and 25 cm wide; local slides about 18 m long from scarp to toe developed on its downstream face.

The 1925 Santa Barbara Earthquake (M 6.3) caused the catastrophic slide and failure of Sheffield Dam in Santa Barbara, California. This was probably one of the early cases that recognized the effects of ground shaking on low-density materials causing embankment failures.

Moderate damage was experienced by dams in southern California during the 1952 Kern County earthquake (M 7.7). The 6 m high Eklutna Dam suffered serious damage during the 1964 Alaska earthquake (M 8.4).

It was not until the 1971 San Fernando, California earthquake (M 6.5) that engineers' concerns regarding the vulnerability of certain types of earth dams were confirmed. The 1971 event received considerable attention from both the media and the general public. A major catastrophe was narrowly avoided in a highly developed urban area. The Lower Van Norman Dam, a 42 m high hydraulic fill dam, experienced widespread liquefaction and major slope failures. Overtopping of the crest and flooding of an area involving over 70 000 downstream residents was barely avoided, and only because the reservoir water level was relatively low for the season when the earthquake occurred. The 24 m high Upper Van Norman Dam was also severely damaged. The near failure of the Lower Van Norman Dam became a true milestone in earthfill dam performance evaluation. It brought the potential vulnerability of embankments constructed of poorly compacted saturated fine sands and silts to the attention of engineers and public agencies involved in dam safety. It also triggered numerous state-mandated re-assessments of dam safety, and led to significant advances in the development of numerical methods of dynamic analysis of dams.

Another recent event of interest is the 1985 Mexico earthquake (M 8.1), that involved two large earth-rock and rockfill dams, La Villita (60 m high) and El Infiernillo (147 m high). While neither of these dams experienced significant damage during the 1985 earthquake, they have been shaken since 1975 by a unique sequence of closely spaced events, five of which were larger than Magnitude 7.1. Cumulative earthquake-induced settlements of La Villita Dam, an earth-rockfill embankment with a wide central impervious clay core, now approach one percent of its original height. La Villita Dam's settlements have shown a tendency to increase in magnitude with more recent events, perhaps due to some progressive weakening of part of the embankment materials. Similar performance has not been observed at El Infiernillo Dam, an earth core rockfill dam (ECRD), the deformations of which have remained small and consistent from one event to the next. El Infiernillo has rock foundation, while La Villita foundation is alluvium.

Events of moderate magnitude, such as the 1987 Edgecumbe earthquake (M 6.2) in New Zealand, which damaged the 78 m high Matahina Dam, or the 1987 Whittier Narrows, CA earthquake (M 6.1), which affected several embankment dams in the greater Los Angeles Area, are considered to be significant from a dam engineering qualité technique des données de comportement et des enregistrements des fortes secousses recueillis lors de ces séismes.

Un examen détaillé des observations et enregistrements effectués lors d'événements passés a permis de constater que les barrages en remblai soumis à de fortes secousses sismiques se sont comportés, certains de façon satisfaisante, d'autres de façon médiocre, leur comportement étant étroitement associé à la nature des matériaux de remblai. Si l'on admet que la plupart des barrages en terre bien construits sont capables de résister à de fortes secousses sismiques sans conséquences néfastes, ceux constitués de matériaux argileux compactés, fondés sur du rocher ou des argiles compactes, ont montré qu'ils résistaient à de très fortes secousses du sol, même lorsque des méthodes de compactage obsolètes ou inefficaces avaient été utilisées. Par contre, des remblais anciens constitués de sables ou silts insuffisamment compactés et des barrages de stériles, de conception ancienne, constituent presque tous les cas connus de ruptures, dont la cause principale est la liquéfaction de ces matériaux. Les barrages par remblayage hydraulique, procédé de construction qui a été pratiquement abandonné, et les barrages de stériles représentent les types de barrage en remblai susceptibles d'être endommagés par des séismes. Par contre, les barrages en enrochement à masque amont en béton de ciment ou en béton bitumineux sont généralement considérés comme des ouvrages intrinsèquement stables sous des charges sismiques extrêmes. Ils représentent les types de barrage qu'il est souhaitable d'adopter dans les zones de forte séismicité.

## 3.2. COMPORTEMENT SISMIQUE DES BARRAGES EN BÉTON

On ne connaît pas de barrages en béton s'étant rompus sous l'action d'un séisme. Dans le cas du séisme Chi-Chi, le barrage de dérivation Shih-Kang était situé à 50 km de l'épicentre et fut soumis à une accélération horizontale de 0,5 g. De sérieux dégâts résultèrent des déplacements d'une faille. Le barrage de dérivation Chi-Chi, situé à moins de 10 km de l'épicentre, subit une accélération horizontale de 0,5 g sans dégâts. Les deux ouvrages sont du type poids en béton, équipés de vannes segment. Une centaine, ou plus, de barrages en béton ont peut-être été soumis à des secousses sismiques, mais une douzaine seulement ont subi des accélérations de pointe enregistrées, ou estimées, de 0,20 g ou au-dessus. Ces barrages en béton comprennent tous les types principaux : voûte, voûtes multiples, poids et contreforts.

Aucun barrage-voûte n'a subi d'importants dégâts, bien que de tels ouvrages aient été soumis à de fortes secousses du sol. Lors du séisme San Fernando de 1971 (M 6,5), Californie, le barrage Pacoima, de 113 m de hauteur, fut exposé à une accélération maximale estimée à 0,70 g à sa base; une accélération de pointe sans précédent de 1,25 g fut enregistrée sur le rocher de l'appui rive gauche, légèrement au-dessus de la crête du barrage. Cependant, on a pensé que cette forte accélération résultait de la forme en cañon de la vallée à cet endroit, et de la fissuration du bedrock à l'emplacement du sismographe pour forte secousse. Le séisme de 1971 ne causa pas de fissures sur le barrage, ni de mouvements relatifs entre plots adjacents de l'ouvrage. L'appui rive gauche fut consolidé au moyen de tirants précontraints afin de stabiliser deux importants coins rocheux qui s'étaient déplacés de plusieurs pouces au cours du séisme. L'accélération de pointe près du sommet de l'appui rive gauche, au cours du séisme Northridge du 17 janvier 1994, atteignit 1,76 g (direction horizontale) et 1,6 g (direction verticale). Le joint entre le massif de butée rive gauche et l'extrémité de la point of view because of the high technical quality of performance data and strong-motion records collected as a result of these events.

From a detailed review of past experience records, it has become apparent that embankment dams have fared both satisfactorily and poorly when subjected to strong earthquake motion, and that their performance has been closely related to the nature of the fill material. While most well-built earthfill dams are believed to be capable of withstanding substantial earthquake shaking with no detrimental effects, those built of compacted clayey materials on stiff clay or bedrock foundations have historically withstood extremely strong levels of ground motion, even when obsolete or inefficient compaction procedures had been used. In contrast, older embankments built of inadequately compacted sands or silts, and older design tailings dams, represent nearly all the known cases of failures, primarily as a result of the liquefaction of those materials. Hydraulic fill dams, a type of construction that has been virtually abandoned, and tailings dams represent the types of embankment dams most vulnerable to earthquake damage. Conversely, rockfill dams, concrete faced rockfill dams (CFRD), and asphaltic concrete faced rockfill dams, are generally considered to be inherently stable under extreme earthquake loading. They represent desirable types of dams in highly seismic areas.

## 3.2. EARTHQUAKE PERFORMANCE OF CONCRETE DAMS

No concrete dam is known to have failed as the result of an earthquake. In the Chi-Chi Earthquake, the Shih-Kang diversion dam was 50 km from epicenter and experienced about 0.5 g horizontal. Severe damage was due to fault displacement. The Chi-Chi diversion dam is less than 10 km from epicenter, experienced about 0.5 g horizontal and had no damage. Both are concrete gravity, radial gated structures. Perhaps one hundred or more concrete dams have been shaken by earthquakes, but only about twelve such dams have experienced peak accelerations recorded, or estimated, at 0.20 g or greater. Those dams included all principal types of concrete structures: arch, multiple arch, gravity and buttress.

No significant damage has ever been suffered by an arch dam, although such structures experienced substantial ground motions. During the 1971 San Fernando, CA, earthquake (M 6.5), the 113 m high Pacoima Dam was subjected to estimated maximum base accelerations of about 0.70 g; an unprecedented peak acceleration of 1.25 g was recorded on rock at the left abutment, slightly above the dam crest. However, that large acceleration is presumed to have been related to the local narrow ridge topography, and to the shattered condition of the bedrock at the location of the strong motion instrument. Pacoima Dam did not develop structural cracks or experience relative movements between adjacent blocks of the dam as a result of 1971 earthquake. The left abutment was strengthened through installation of post-tensioned anchors to stabilize two large rock wedges that had moved several inches during the earthquake. The peak acceleration near the top of the left abutment during the January 17, 1994 Northridge earthquake were 1,76 g horizontal and 1,6 g vertical. The joint between the left abutment and the left end of the arch, opened a max of 5 cm due to voûte s'ouvrit de 5 cm au maximum par suite du mouvement de coins rocheux sur l'appui supérieur. Une légère fissuration horizontale du béton sur l'extrémité rive gauche du barrage et plusieurs petits décrochements horizontaux et verticaux aux joints entre plots se produisirent. Le séisme mit en évidence, pour la première fois, l'ouverture et la fermeture de joints de contraction verticaux au cours d'un séisme. Le barrage Pacoima est un ouvrage de protection contre les crues et le niveau de retenue était à peu près à mi-hauteur du barrage lors des deux séismes.

Le barrage Ambiesta (Italie), voûte de 59 m de hauteur, fut soumis à des secousses lors du séisme Friuli de 1976 (M 6,5). Une accélération du sol de 0,33 g fut enregistrée sur l'appui rive droite. Le barrage ne subit aucun dégât, confirmant les résultats des précédentes études sur modèle physique qui avaient indiqué que des accélérations substantiellement plus fortes (0,75 g ou plus) seraient nécessaires pour causer des dégâts à l'ouvrage.

Parmi d'autres barrages-voûtes ayant subi des séismes, on peut citer le barrage Honenike (Japon), barrage à voûtes multiples de 30 m de hauteur, où une fissure se produisit dans une voûte près d'un contrefort, au cours du séisme Nankai de 1946 (M 7,2). La fissure fut réparée par injection. Plusieurs autres grands barrages-voûtes, dont Santa Anita et Big Tujunga (Californie, États-Unis) ; Barcis et Maina di Sauris (Italie), Kariba (Zambie), Monteynard et Grandval (France), Kurobe (Japon), situés à 50 km ou moins des épicentres de séismes de magnitudes M 4,9 à 6,6 ne furent pas endommagés. Cependant, les intensités locales des secousses sur ces sites étaient probablement modérées.

Plusieurs barrages-poids, dont Aono, Gohonmatsu et Sangari (Japon), situés respectivement à 1,5 , 19 et 30 km environ de l'épicentre du séisme Kobe de 1995, ne furent pas endommagés. Les secousses locales sur ces sites rocheux furent probablement modérées, comme le laissaient à penser les toits de tuiles intacts des maisons voisines. Les barrages Aono et Sangari sont en béton, tandis que le barrage Gohonmatsu est le premier barrage japonais (construit en 1900) réalisé en maçonnerie grossière. Des fissures très fines furent observées dans la chape en béton du parapet de crête, mais aucune fissure n'existait dans le corps du barrage. Deux autres barrages-poids, Nunobiki (32 m de hauteur) et Karasubara (31 m de hauteur) subirent le séisme sans dégâts apparents. Ainsi, des barrages-poids, de taille moyenne, se comportèrent de façon très satisfaisante au cours du séisme Kobe. Comme précédemment indiqué, cela peut résulter des mouvements très modérés du sol qui se sont produits sur les sites rocheux. Cependant, lors d'autres séismes, quelques barrages-poids et à contreforts ont été plus gravement affectés que les barrages-poids et barrages-voûtes japonais précités. Des exemples sont brièvement décrits ci-après.

Le barrage Blackbrook (Grande-Bretagne), barrage-poids en béton, de 30 m de hauteur, avec un parement amont en briques et un parement aval en pierres, est le seul barrage de Grande-Bretagne sur lequel on a signalé des dégâts dus à un séisme (1957). Le séisme avait une magnitude de 5,3 environ à une profondeur de foyer de 7 km. L'épicentre était situé à 6,4 km environ du site du barrage. Le séisme provoqua une fissuration du mortier du parement aval en pierres. Toutes les grandes pierres, qui couronnaient les murs parapets sur les deux côtés de la crête du barrage Blackbrook, furent soulevées sur leur lit de mortier et retournées en arrière, écrasant le mortier.

Le barrage Koyna (Inde), barrage-poids rectiligne, de 102 m de hauteur, et le barrage Hsinfengkiang (Chine), barrage à contreforts, de 104 m de hauteur, furent movement of the rock wedges on the upper abutment. Minor horizontal cracking of concrete at the left end of the dam, and several minor horizontal and vertical block offsets at the joints occurred. The earthquake provided, for the first time evidence of opening and closing of vertical contraction joints during an earthquake. Pacoima is a flood control dam and the reservoir was at about mid-dam height during both earthquakes.

Ambiesta Dam, a 59 m high arch dam in Italy, was shaken during the 1976 Friuli earthquake (M 6.5). Ground acceleration was recorded as 0.33 g at the right abutment. The dam suffered no damage, confirming results of previous physical model studies, which had indicated that substantially larger accelerations (0.75 g or greater) would be required to cause damage to the structure.

Other arch dams shaken by earthquakes include Honenike Dam, in Japan, a 30 m high multiple arch, which developed a crack in an arch near a buttress during the 1946 Nankai earthquake (M 7.2). The crack was repaired by grouting. Several other major concrete arch dams, such as Santa Anita, and Big Tujunga, CA; Barcis, and Maina di Sauris, in Italy; Kariba, in Zambia; Monteynard and Grandval, in France; and Kurobe, in Japan, were located 50 km or less from epicenters of various events with Magnitudes between 4.9 and 6.6, but were undamaged. However, the local intensities of shaking at those sites were probably moderate.

Several gravity dams, including Aono, Gohonmatsu, and Sangari Dams, located about 1.5, 19 and 30 km from the 1995 Kobe epicenter, respectively, were undamaged. Local shaking at these rock sites was probably moderate, as suggested by undisturbed tile roofs observed at nearby houses. Aono and Sangari dams are concrete dams, while Gohonmatsu (32 m high) is the first Japanese dam (built in 1900) constructed built of concrete rubble masonry. Hairline cracks were observed in the capping concrete on the crest railing, but no cracks were observed in the dam body. Two other gravity dams, Nunobiki (32 m high) and Karasubara (31 m high) survived the earthquake with no apparent damage. Hence, medium-size concrete gravity dams performed very well during the Kobe earthquake. As previously stated, this could be due to the modest ground motion experienced at rock sites. However, in other earthquakes, a few concrete gravity and buttress dams have been affected more severely by earthquakes than the above Japanese gravity dams and arch dams, in general. This experience is briefly described below.

Blackbrook Dam, in Great Britain, a 30 m high concrete gravity dam with an upstream brick facing and a downstream stone facing, is the only dam in Great Britain reported to have been damaged by an earthquake (1957). The event, had a local magnitude of about 5.3 at a focal depth of 7 km. The epicenter was about 6.4 km from the dam site. It resulted in cracking of the mortar of the downstream stone facing of the dam. All of the large coping stones, which topped the parapet walls on both sides of the crest of Blackbrook Dam, were lifted from their mortar bed and dropped back, crushing the mortar in the process.

Koyna Dam, in India, a 102 m high straight gravity dam, and Hsinfengkiang Dam, in China, a 104 m high buttress dam, were shaken as the result of nearby soumis à des secousses lors de séismes tout proches, de magnitudes 6,5 (1967) et 6,1 (1962), respectivement. Il fut suspecté que ces deux séismes avaient été induits par les retenues. Ces deux barrages furent affectés par une importante fissuration longitudinale près de la crête. Les dégâts furent attribués à des détails de conception ou de construction, qui ne seraient pas adoptés dans les ouvrages modernes. Les deux barrages furent réparés et sont toujours en service.

Le barrage Sefid Rud (Iran), barrage à contreforts, de 105 m de hauteur, subit des dégâts lors du séisme Manjil du 21 juin 1990 (M 7,5). Le barrage était situé à moins de 32 km de l'épicentre, et l'accélération de pointe du sol fut estimée à 0,7 g. Le barrage fut affecté par diverses formes de dégât, dont une importante fissuration dans la partie supérieure des contreforts. Les contreforts furent conçus en adoptant des charges horizontales pseudo-statiques allant de 0,1 g à 0,25 g. Ainsi, l'expérience de Sefid Rud suscite un grand intérêt car elle représente un exemple d'un barrage en béton soumis à de fortes secousses, beaucoup plus sévères que les charges de projet. Le barrage subit d'importants dégâts, mais eut un comportement global satisfaisant, si l'on considère que le séisme Manjil fut probablement proche de ce qui peut être considéré comme un événement maximal sur ce site.

Le barrage Lower Crystal Springs (États-Unis), barrage-poids incurvé de 38,5 m de hauteur, constitué de blocs de béton avec emboîtement, subit le séisme San Francisco de 1906 (M estimée à 8,3) sans une seule fissure. La rupture de faille principale fut localisée à moins de 180 m du barrage ; un glissement du côté droit, de 3 m environ, fut mesuré à proximité. Le barrage Lower Crystal Springs fut de nouveau soumis à des secousses lors du séisme Loma Prieta de 1989, mais une fois de plus ne fut pas affecté.

Le Lac Mead (États-Unis), qui est crée par le barrage Hoover, du type poids incurvé, de 220 m de hauteur, fut suspecté d'être à l'origine d'une séismicité induite modérée (M 5,0 ou moins), qui n'endommagea pas le barrage. Enfin, le barrage Poiana Usului (Roumanie), du type à contreforts, situé à 60 km environ de l'épicentre du séisme Roumain de 1977 (M 7,2), se comporta de façon satisfaisante.

Globalement, le comportement des barrages en béton sous l'action de séismes a été satisfaisant, ce qui laisse à penser que les barrages de ce type sont plus résistants vis-à-vis des séismes que les barrages en remblai. Cela peut résulter du fait que les barrages en béton ont été réalisés suivant des normes plus rigoureuses que celles adoptées pour certains des anciens barrages en remblai. En outre, les barrages en béton sont probablement moins sujets à un vieillissement, à une détérioration des matériaux, à des percolations et à un défaut d'entretien que les barrages anciens en remblai. Cependant, il reste encore à connaître le comportement réel d'un grand barrage-voûte mince, situé dans une zone fortement sismique, sous l'action de son séisme de dimensionnement.

earthquakes of Magnitudes 6.5 (1967) and 6.1 (1962), respectively. Both of these earthquakes were suspected of being caused by reservoir triggered seismicity. Both of these dams developed substantial longitudinal cracking near the top. Damage was attributed to design or construction details that would be avoided in modern structures. The two dams were repaired and are still in service.

The Sefid Rud Dam, a 105 m high concrete gravity buttress dam in Iran, suffered damage during the Manjil Earthquake of June 21, 1990 (M 7.5). The dam was located less than 32 km from the epicenter, and peak ground acceleration was estimated at 0.7 g. The dam suffered various forms of damage, including severe cracking in the upper part of the buttresses. The buttresses were designed using pseudo-static horizontal loadings ranging from 0.1 g to 0.25 g. Thus, the Sefid Rud experience is important because it represents an example of a concrete dam that was subjected to strong shaking substantially more severe than its design loads. The dam suffered significant damage, but had overall satisfactory performance considering that the Manjil Earthquake was probably not far from what can be expected as a maximum event at that site.

Lower Crystal Springs Dam, a 38.5 m high curved concrete gravity dam built of interlocking concrete blocks, withstood the 1906 San Francisco earthquake (M 8.3, estimated) without a single crack. The primary fault rupture was located less than 180 m from the dam; a right-lateral slip of about 3 m was measured nearby. Lower Crystal Springs Dam again experienced moderate shaking during the 1989 Loma Prieta earthquake, but was once more unaffected.

Lake Mead, which is impounded by Hoover Dam, a 220 m high curved gravity dam, has been suspected of being the cause of moderate reservoir-triggered seismicity (M 5.0 or less), which did not damage the dam. Lastly, Poiana Usului Dam, in Romania, a buttress dam, was located about 60 km away from the epicenter of the 1977 Romanian earthquake (M 7.2), and performed satisfactorily.

Overall, the performance of concrete dams has been satisfactory, and such dams could be implied to be more earthquake-resistant than embankment dams. This is perhaps because concrete dams may have been built to design standards higher than used for some of the earlier embankment dams. Furthermore, concrete dams are probably less susceptible to aging, materials deterioration, and seepage and poor maintenance than are older embankment dams. Yet, the true test of a major thin arch concrete dam in a highly seismic area and subjected to its Design Basis Earthquake has yet to come.

# 4. CONSIDÉRATIONS GÉNÉRALES RELATIVES À LA CONCEPTION

Des considérations sismiques orientent de nombreuses décisions techniques au cours de la conception des barrages, allant du choix du site aux types d'appareils d'auscultation à mettre en place sur le site et sur l'ouvrage. Des reconnaissances et études sismiques feront partie du processus général de conception, avec les études concernant la topographie, la géologie et les aspects géotechniques, nécessaires à la détermination de leur influence sur le projet, la construction et le comportement du barrage. Des facteurs topographiques, tels que la forme de la vallée, peuvent influer sur les caractéristiques sismiques. La géologie des deux appuis, si elle est différente, est susceptible d'avoir un effet sur la réponse sismique du barrage ; les caractéristiques géotechniques des zones de cisaillement ou des fondations alluviales peuvent avoir une influence sur le traitement des fondations et le choix du profil en travers du barrage.

Le problème de la réponse sismique et du comportement des barrages et de leur fondations, formulé en termes généraux – incluant la réponse interactive tridimensionnelle 1) du corps du barrage 2) de l'eau de la retenue et 3) de la fondation du barrage – est extrêmement complexe. Dans le cas de très fortes secousses sismiques, ou lorsque des conditions spéciales du corps du barrage et de la fondation règnent, le problème ne peut être analysé et expliqué que par l'étude de la réponse non-linéaire du système barrage-fondation, ce qui est toujours une importante tâche numérique. L'observation d'un comportement structural de barrage dans le domaine élastique linéaire, tel que cela se présente lors de secousses générées par des vibrateurs, fournit des résultats généralement en accord avec les valeurs calculées dans le cas de secousses de faible intensité. Cependant, il est très difficile d'extrapoler de ces résultats ceux correspondant à la réponse du barrage sous de fortes sollicitations sismiques.

L'évaluation des paramètres sismiques de calcul a été traitée dans le Bulletin CIGB n° 72 (Référence 6). Les problèmes d'observation et de calcul du comportement des barrages sous des charges sismiques a fait l'objet du Bulletin CIGB n° 52 (Référence 7). Ces deux Bulletins, ainsi que les connaissances disponibles sur les comportements et les réponses sismiques de barrages, doivent être pris en considération avant d'établir des recommandations sur la conception structurale d'un barrage et le traitement de sa fondation.

Il faut reconnaître que l'ensemble du domaine des paramètres sismiques et des réponses connexes des barrages connaît un développement continu. Aussi, les présentes recommandations ne doivent-elles être considérées que comme une représentation de l'état actuel de nos connaissances sur le sujet. Néanmoins, les connaissances et expériences actuelles, déduites d'exemples réels, permettent de tirer un certain nombre de conclusions étendues et valables sur les caractéristiques souhaitables de la structure des barrages.

# 4. DESIGN CONSIDERATIONS

Seismic considerations guide many engineering decisions in dam design, ranging from site selection to the type of instrumentation at the dam site. A seismic investigation must be part of the general design process, along with studies of topography, geology and geotechnical aspects necessary to determine their influence on the design, construction and performance of the dam. Topographic effects, such as valley shape, could affect the seismic characteristics. The geology of the two abutments, if different, could affect the seismic response; and the geotechnical properties of shear zones or of alluvial foundations can influence the foundation treatment and dam cross-section.

The problem of seismic response and behavior of dams and their foundations when formulated in general terms – including three-dimensional interactive response of 1) the dam body, 2) the reservoir water and 3) the dam foundations – is extremely complex. For very strong seismic inputs, or for cases when special conditions of the dam body and foundation composition prevail, the problem can be analyzed and understood only by investigating the non-linear response of the dam/foundation system, which is always a major numerical undertaking. The observation of a dam's structural behavior in the linearly elastic range, such as evoked when shaking with vibrators, is yielding results generally in agreement with computed values for low level shaking. However, it is very difficult to extrapolate from those results to obtain the ones anticipated in response to strong seismic inputs.

An evaluation of the seismic input parameters has been the subject of ICOLD Bulletin No.72 (Reference No. 6). The problems inherent in the observation and analysis of dam behavior under seismic loading were the subject of ICOLD Bulletin No. 52, (Reference No 7). Those two Bulletins, together with the available body of knowledge about dam behavior and response under earthquake loading, should be considered before recommending the desirable structural features of a dam and its foundation treatment.

It should be understood that the entire field of seismic input assessment, and the related structural response of dams, is in continuing development. Therefore, these Guidelines on dam design and foundation treatment should be understood only as a "state of the art" reflection on our knowledge of the subject. Nevertheless, current knowledge and experience, as drawn from actual case histories, provide the basis for a number of broad, sound conclusions regarding the desirable structural features of dams.

Les chapitres suivants présentant des considérations générales de conception concernent quatre sujets principaux. Ils portent sur la forme et la géologie du site du barrage, le choix du type de barrage, les aspects structuraux des barrages en remblai et des barrages en béton, et enfin les glissements de terrain et les appareils d'auscultation.

## 4.1. PRISE EN COMPTE DE LA FORME ET DE LA GÉOLOGIE DU SITE DU BARRAGE

Les aspects physiques et topographiques d'un site de barrage sont d'une importance capitale lors du choix du site et lors de la restauration d'un barrage existant. Les aspects topographiques, géologiques, géotechniques et autres, de tout site de barrage sont des paramètres primordiaux. Du point de vue topographique, il importe de se rendre compte que la forme particulière d'un cañon peut amplifier les vibrations sismiques ou les rendre asymétriques au cours d'un séisme. La réponse du profil en travers de la vallée peut être estimée qualitativement et parfois l'analyse apportera une aide dans le choix de la disposition générale de l'ouvrage. Le cas du barrage Pacoima, où de très fortes accélérations furent enregistrées sur l'appui rive gauche et où des mouvements de coins rocheux se produisirent dans cette zone, illustre la possibilité de dégâts sur le corps d'un barrage ou ses appuis par suite d'une configuration topographique particulière (« Amplification d'arête » d'une secousse sismique).

Un site particulier peut avoir une direction prédominante de secousses sismiques, résultant de la configuration de la vallée, de la géologie de la fondation et de la profondeur relative des dépôts alluviaux, pouvant modifier la direction et l'intensité des secousses sur le site. Le concepteur devra en tenir compte pour prendre des mesures appropriées destinées à faire face aux diverses intensités simultanées de secousses dans des zones contiguës du barrage, causant des concentrations de déformations et de contraintes, ou même un choc possible d'une section contre une autre.

Toutes les caractéristiques géologiques d'un site de barrage revêtent une grande importance. Une attention particulière doit être portée aux changements brusques de conditions géologiques, tels que les formations différentes sur les versants d'une gorge. Toute caractéristique particulière existante fera l'objet d'études poussées et son influence sur la réponse sismique du barrage sera évaluée. Au cours des secousses sismiques, l'interaction barrage-fondation peut avoir des effets nuisible sur le réseau de diaclases d'une fondation rocheuse. Lors des études de projet, des mesures concernant la fondation peuvent comprendre des traitements spéciaux : fouilles plus profondes, injection de consolidation plus importante ou drainage complémentaire. La présence de zones de cisaillement dans la fondation, souvent remplies de matériaux fortement cisaillés, est susceptible de nécessiter la construction de clavettes de cisaillement conçues pour les charges sismiques. Les conditions de fondation, en particulier dans le cas de formations rocheuses sédimentaires de faible résistance, peuvent nécessiter la réalisation d'une galerie de visite et de drainage.

La présence de dépôts sableux sans consistance sous les recharges d'un barrage en remblai constitue un danger potentiel de liquéfaction au cours d'un séisme, avec perte de résistance au cisaillement et rupture possible du barrage. Afin que la conception assure une sécurité satisfaisante, on peut être conduit à enlever les couches

The following sections on design considerations are divided into four main topics. They cover considerations on the damsite shape and geology, the selection of dam type, structural features of embankment dams and concrete dams and, and finally landslides and instrumentation.

## 4.1. CONSIDERATION OF DAMSITE SHAPE AND GEOLOGY

The physical and topographic aspects of a damsite are of fundamental importance during the site selection stage, and when considering the design for remediation of an existing dam. Topographic, geological, geotechnical and other unique aspects of any damsite are primary variables. From the topographic point of view, it is important to realize that the particular shape of a canyon could amplify seismic vibrations or make them unsymmetrical during an earthquake. The valley section's response may be estimated qualitatively, and sometimes the analysis will help in the layout of the structure. The case of Pacoima Dam, where very high accelerations were recorded on the left abutment and where movement of the rock wedges occured at the same location, illustrates the possibility of damaging effects on a dam structure or its abutments caused by a peculiar topographic configuration. ("Ridge amplification" of earthquake shaking).

A particular site may have a predominant direction of shaking that would relate to the river valley's configuration, the foundation geology, and the relative depth of alluvial deposits that could modify the direction and intensity of shaking across the site. The designer should take this into account to provide adequate measures to resist different, simultaneous intensities of shaking in contiguous dam sections, causing stress and strain concentrations, or even the possibility of pounding of one section against another.

Of significant importance are all geologic aspects of a damsite. Special attention should be paid to any sharp changes in geologic conditions, such as different formations on the sides of a gorge. Intensive studies should be made of any existing special feature and its significance to the seismic response of the dam being evaluated. During seismic shaking, the dam foundation interaction may adversely affect the joint system of the foundation rock. Design measures for the foundation may require special treatment by deeper excavation, more intensive consolidation grouting or additional drainage. The presence of foundation shear zones, often filled with highly sheared materials or gouge, may require the construction of shear keys that must be designed for earthquake loading. Foundation conditions, particularly for weak sedimentary rock formations, could require the addition of an inspection and drainage gallery.

The presence of loose sandy deposits under the shells of an embankment dam could present the potential danger of liquefaction during an earthquake, with the resulting loss of shear strength and potential failure of the dam. A safe design may suspectes, ou à les compacter par des méthodes dynamiques, ou à installer des dispositifs de drainage.

Le projet d'un barrage de grande longueur doit tenir compte des effets particuliers tridimensionnels résultant de vibrations et mouvements non uniformes et déphasés le long de l'alignement du barrage, problème spécialement rencontré dans les projets de pipelines, galeries, ponts et autres ouvrages de grande longueur.

La présence de gorges épigéniques étroites dans la fondation d'une vallée relativement large est susceptible de conduire à des réponses dynamiques indésirables des contreforts ou plots de barrages en béton. Une telle situation peut nécessiter le choix d'un autre type de barrage, par exemple un barrage en enrochement, avec un noyau étanche ou un masque amont en béton de ciment ou en béton bitumineux, ou la construction d'un mur parafouille épais, en béton, dans la gorge épigénique.

## 4.2. CHOIX DU TYPE DE BARRAGE

Le choix d'un site de barrage peut nécessiter une comparaison entre divers types de barrage sur différents sites possibles de barrage, en tenant compte des charges sismiques très probables et de leur influence sur le type et les dispositions générales du barrage. Une estimation de la réponse sismique des différents formations, accidents et couches géologiques, ainsi que des dépôts alluviaux sur le site du barrage, est essentielle pour un choix rationnel du type de barrage, de ses caractéristiques de conception et pour son adaptation au site. Le choix du meilleur type de barrage pour un site donné nécessite la connaissance de l'interaction de chaque type avec les caractéristiques du site. La réponse sismique de chaque type de barrage sera donc étudiée.

require either the removal of the suspected layers, their compaction by dynamic methods, or the installation of drainage features.

The design of a long dam should take into account the special three dimensional effects that may result from non-uniform and out-of-phase vibrations and movements along the dam alignment, a problem typically encountered in the design of pipelines, tunnels, bridges and other long structures.

The presence of buried narrow gorges in a relatively wide valley foundation could lead to undesirable dynamic responses of concrete dam buttresses or monoliths. Such a condition could require either the selection of a different type of dam, for example a rockfill embankment, either with an impervious core, or a concrete, or asphaltic concrete face, or the construction of a massive concrete plug within the buried gorge.

## 4.2. SELECTION OF A DAM TYPE

The selection of a damsite may require a comparison of various dam types at alternative damsites with consideration of the seismic loadings most likely to occur, and their influence on the dam type and configuration. An appreciation of the seismic response of the different geological formations, features and strata, as well as the alluvial deposits at the damsite, is essential for a rational selection of the dam type, design features and its adaptation to the site. The selection of the best type of dam for a particular site requires understanding the interaction of each dam type with the characteristics of the site. Therefore, the seismic response of each dam type should be considered.

# 5. ASPECTS PARTICULIERS DE LA CONCEPTION SISMIQUE

Les secousses dynamiques causées par les séismes induisant des contraintes et des déformations en plus de celles résultant des conditions statiques normales, il importe que les charges additionnelles soient incorporées dans le projet initial ou dans le projet de restauration d'un barrage existant. Les sous-chapitres suivants essaient de répondre à la question : **«Quelles dispositions particulières doit-on adopter dans la conception des barrages situés dans des zones sismiques ?».** Les barrages en terre, ou en terre-enrochement, ayant des réponses différentes aux secousses dynamiques, comparativement aux barrages en béton, ces deux catégories de barrages seront examinées séparément.

## 5.1. ASPECTS STRUCTURAUX CONCERNANT LES BARRAGES EN REMBLAI

Comme décrit dans le chapitre 3, le comportement sismique de la plupart des barrages en remblai a été satisfaisant. Les seuls barrages dont on connaît une rupture totale sous l'action de secousses sismiques sont des barrages de stériles ou des barrages par remblayage hydraulique, ou des barrages en terre relativement petits, de conception et construction anciennes et, peut-être, déficientes. Une des conséquences les plus dangereuses des charges dynamiques agissant sur un barrage en remblai est la liquéfaction de zones de la fondation ou du remblai, contenant des matériaux à grains fins, sans cohésion, saturés, et/ou des matériaux non compactés. Les recommandations relatives à la conception et à la construction des barrages en remblai sujets à de sévères secousses sismiques sont les suivantes :

- Les fouilles en fondation doivent descendre jusqu'aux matériaux très denses ou au rocher. Une autre solution consiste à densifier les matériaux de fondation sans consistance, ou à les enlever et les remplacer par des matériaux fortement compactés, afin de se prémunir contre une liquéfaction ou perte de résistance.

- Les matériaux de remblai susceptibles de connaître des pressions interstitielles élevées au cours de fortes secousses sismiques ne doivent pas être utilisés à l'amont de l'organe d'étanchéité, ou au-dessous de la ligne de saturation, afin de parer à une liquéfaction ou perte de résistance.

- Toutes les zones du remblai doivent être suffisamment compactées pour prévenir une perte de résistance et des tassements excessifs sous l'action des séismes.

- La pente transversale de la fondation au droit de la zone du noyau sera horizontale ou légèrement en pente vers l'amont, le long des 30 m supérieurs des appuis des barrages de grande hauteur, afin d'assurer des contacts étanches après les secousses sismiques et/ou prévenir des tassements excessifs des matériaux du remblai.

# 5. SPECIAL FEATURES FOR SEISMIC DESIGN

Since dynamic shaking caused by earthquakes induces stresses and deformations over and above the normal static conditions, it is important that additional loadings be incorporated into the original design, or into the design for remediation of an existing dam. The following sub-section attempts to respond to the question: **"What should we do differently in the design of a dam in earthquake country" ?** Since earth, or earth-rockfill, dams respond differently to dynamic shaking than concrete dams, they are discussed separately.

## 5.1. STRUCTURAL FEATURES FOR EMBANKMENT DAMS

As described in Section 3, the seismic performance of most embankment dams has been outstanding. The only dams that have been known to fail completely as a result of seismic shaking were tailings or hydraulic fill dams, or relatively small earthfill embankments of older and, perhaps, inadequate design and construction. One of the most dangerous consequences of the dynamic loading of an embankment dam is the liquefaction of foundations or embankment zones, that contain saturated fine grained cohesionless and/or uncompacted materials. Recommendations for design and construction of embankment dams subject to severe earthquake shaking are as follows:

- Foundations must be excavated to very dense materials or rock; alternatively the loose foundation materials must be densified, or removed and replaced with highly compacted materials, to guard against liquefaction or strength loss.

- Fill materials which tend to build significant pore water pressures during strong shaking must not be used upstream from the water barrier, or below the water table, to guard against liquefaction or strength loss.

- All zones of the embankment must be thoroughly compacted to prevent strength loss and earthquake induced excessive settlement.

- The transverse foundation slope across the core zone should be either horizontal, or slope gently upstream, along the upper 30 m of the abutments of high dams, to assure watertight contacts after shaking and/or preclude excessive settlement of the embankment materials.

• Le contact noyau-fondation aura une forme douce, sans arêtes vives rentrantes. La pente transversale de la fondation au droit de la zone du noyau sera inférieure à 1 : 4 (verticale : horizontale) de l'amont vers l'aval, et de préférence inférieure à 1 : 2 (verticale : horizontale) le long de l'axe longitudinal du barrage pour prévenir toute tendance à des fissures transversales.

• Tous les barrages en remblai, et spécialement les barrages homogènes, doivent avoir des zones drainantes internes de forte capacité pour intercepter les percolations provenant de fissures transversales causées par des séismes, et pour garantir que les zones de remblai conçues pour être non saturées le restent après tout phénomène ayant pu conduire à une fissuration.

• Des filtres doivent être prévus sur les fondations rocheuses fracturées afin de prévenir la pénétration de matériaux dans la fondation résultant d'une érosion interne, si des fissures sont ouvertes sous l'action d'un séisme.

• Des zones filtrantes et drainantes plus épaisses que la normale seront adoptées pour assurer une continuité en cas de décrochements de zones, et pour colmater toutes fissures transversales causées par des séismes.

• Les zones de transition amont et/ou aval seront auto-colmatantes et auront une granulométrie permettant de colmater également des fissures à l'intérieur du noyau.

• Le contact du noyau le long des zones supérieures des appuis sera évasé afin d'allonger les chemins de percolation à travers les fissures causées dans les appuis par les séismes.

• Des matériaux de remblai de nature « fragile » ne doivent pas être utilisés pour constituer des organes d'étanchéité et seront remplacés par des matériaux plus plastiques dans les zones où des tractions risquent de se manifester lors de secousses sismiques.

• Des tassements et des fissures transversales sous l'effet de fortes secousses étant possibles, le barrage doit avoir une hauteur de revanche plus importante que la normale. Une revanche suffisante sera prévue en vue de faire face aux tassements probables d'origine sismique et aux seiches possibles.

• Une fissuration étant possible en crête, celle-ci sera plus large que la normale, afin d'allonger les chemins de percolation le long des fissures transversales susceptibles d'être causées par des séismes.

## 5.2. ASPECTS STRUCTURAUX CONCERNANT LES BARRAGES EN BÉTON

Comme indiqué dans le chapitre 3, le comportement sismique des barrages en béton a été généralement excellent. Les principaux types de barrages en béton sont les suivants :

- Barrage-voûte mince, à double courbure
- Barrage-voûte cylindrique

- The shaping of the foundation/core contact should be gentle and free of sharp and re-entrant edges. The transverse foundation slope across the core zone should be less steep than 4:1 (horizontal:vertical) from upstream to downstream, and preferably less steep than 2:1 (horizontal:vertical) along the longitudinal or centerline of the dam to preclude the tendency for transverse cracking.

- All embankment dams, and especially homogeneous embankment, must have high capacity internal drainage zones to intercept seepage from any transverse cracking caused by earthquakes, and to assure that embankment zones designed to be unsaturated remain so after any event that may have led to cracking.

- Filters must be provided on fractured foundation rock to preclude piping of embankment into the foundation, should the fractures be opened by the earthquake.

- Wider than normal filter and drain zones must be used to provide continuity should zone offset occur and to heal any transverse cracks caused by earthquakes.

- The upstream and/or downstream transition zones should be self-healing, and of such gradation as to also heal cracking within the core zone.

- The core contact along the upper portions of the abutments should be flared to assure long seepage paths through cracks in the abutments caused by earthquakes.

- “Brittle” soils should be avoided for use as water barriers, and/or should be replaced with more plastic materials in areas where tension is more likely to develop during earthquake shaking.

- Since settlement and transverse cracking under strong shaking are possible, the dam should have a larger freeboard than normal to increase lateral dimensions at the maximum water surface. Sufficient freeboard should be provided in order to cover the settlement likely to occur during the earthquake and possible seiches.

- Since cracking of the crest is possible, the crest width should be wider than normal to produce longer seepage paths through any transverse cracks that may develop during earthquakes.

## 5.2. STRUCTURAL FEATURES FOR CONCRETE DAMS

As described in Section 3, the seismic performance of concrete dams generally has been excellent. The basic types of concrete dams are:

- Thin arch dams with double curvature
- Cylindrical arch dams

- Barrage à voûtes multiples
- Barrage-poids
- Barrage à contreforts
- Barrage-poids évidé
- Combinaison des types précités
- Combinaison d'un barrage en béton avec un barrage en terre ou en enrochement adjacent à l'ouvrage en béton.

En général, les barrages-voûtes ont eu un meilleur comportement que les autres barrages en béton. Aucun dégât important n'a été subi par un barrage-voûte, même sous l'action de fortes secousses sismiques. Même sous des accélérations sismiques extrêmement élevées, comme celles subies par le barrage Pacoima plusieurs fois, les dégâts ont été remarquablement faibles.

Plusieurs détails de conception contribuent au comportement satisfaisant constaté sur des barrages-voûtes :

- Mise au point d'un projet présentant une géométrie régulière et sans saillie (symétrie souhaitable mais non essentielle).
- Maintien d'une charge de compression continue le long de la fondation, en profilant celle-ci et en concevant également une plinthe comme socle du barrage et pour le transfert des charges à la fondation, si nécessaire.
- Limitation du rapport longueur en crête/hauteur pour éviter une déformation excessive du barrage dans les modes élevés de vibration au cours des secousses sismiques.
- Aménagement de joints de contraction avec emboîtement approprié.
- Amélioration de la résistance dynamique et consolidation de la fondation rocheuse au moyen de fouilles et du remplacement de la roche de faible qualité, des zones de cisaillement, des cavités, etc, par du béton, et au moyen d'injections.
- Préparation soignée des surfaces de reprise afin de rendre maximales les résistances à l'adhérence et à la traction.
- Augmentation de l'épaisseur en crête afin de réduire l'incidence de fissures locales.
- Réduction de masse dans la partie supérieure du barrage.

Le comportement de barrages-poids et de barrages à contreforts a été affecté par des séismes, en particulier par des secousses parallèles à l'axe du barrage. Diverses dispositions constructives permettent d'améliorer leur comportement sismique :

- Adoption de basses températures de bétonnage pour réduire les contraintes initiales de traction d'origine thermique et la fissuration due au retrait.
- Mise en place et maintien d'un bon système de drainage.
- Préparation soignée des surfaces de reprise afin de rendre maximales les résistances à l'adhérence et à la traction.

- Multiple arch dams
- Full gravity dams
- Buttress dams
- Hollow gravity dams
- Combinations of the above types
- Combinations of any concrete type with an earth or rockfill embankment adjacent to the concrete structure.

Generally speaking, arch dams have performed better than other concrete dams. No significant damage has ever been experienced by an arch dam, even though large earthquakes have shaken such dams. Even under extremely large earthquake accelerations, as experienced by Pacoima Dam on several occasions, the damage was remarkably low.

There are several design details that are regarded as contributing to the outstanding records of arch dams, as follows:

- Development of a design with regular and smooth geometry (symmetry is desirable, but not essential).
- Maintenance of continuous compressive loading along the foundation, by shaping of the foundation and also designing a plinth structure to support the dam and transfer load to the foundation, if found necessary.
- Limiting the crest length to height ratio, to assure that the dam doesn't distort excessively in the higher modes of vibration during earthquake shaking.
- Providing contraction joints with adequate interlocking.
- Improving the dynamic resistance and consolidation of the foundation rock by extensive foundation benefaction, consisting of excavation and replacement with concrete of lesser quality rock, shears,cavities, etc, and grouting.
- Provision of well-prepared lift surfaces to maximize bond and tensile strength.
- Increasing the crest width, to reduce the incidence of local through– cracking.
- Minimizing mass in the upper portion of the dam.

The performance of gravity and buttress dams has been affected by earthquakes, especially by shocks parallel to the dam axis. Various structural features which are considered to improve seismic performance are:

- Maintenance of low concrete placing temperatures to minimize initial, heat-induced tensile stresses and shrinkage cracking.
- Development and maintenance of a good drainage system.
- Providing well-prepared lift surfaces to maximize bond and tensile strength.

• Utilisation de clavettes de cisaillement dans les joints de construction.

• Réduction des discontinuités dans le corps du barrage, telles que murs parapets, massifs de butée, piles dans les ouvrages annexes, et des concentrations de contraintes associées.

• Augmentation de l'épaisseur en crête afin de réduire les conséquences de fissures locales.

• Absence de changement de pente sur le parement aval d'un barrage-poids afin d'éliminer les concentrations locales de contraintes.

• Prise en compte du fait que les barrages à contreforts sont particulièrement sensibles aux secousses sismiques parallèles à l'axe du barrage.

Le comportement d'ouvrages mixtes, tels que diverses combinaisons d'ouvrages en béton et d'ouvrages en terre/enrochement, donne matière à préoccupation dans des conditions de séismicité. Une attention particulière doit être portée aux ouvrages en terre, ou en terre/enrochement, adjacents à un barrage-poids ou à un évacuateur de crue en béton. L'interaction dynamique et les déplacements différentiels des ouvrages adjacents en terre et en béton seront étudiés avec soin. Chaque ouvrage aura une réponse différente et des déplacements différents, qui seront analysés pour vérifier la compatibilité dynamique des deux différentes structures au cours des secousses sismiques. L'adoption d'une pente sur l'interface de béton en vue de maintenir une charge normale de remblai contre le béton, et l'utilisation de matériaux plastiques et de larges zones de filtre dans les remblais sont des dispositions spéciales de projet pour la zone d'interface.

Un autre type d'ouvrage nécessitant une attention particulière est le masque en béton de ciment ou en béton bitumineux sur un barrage en enrochement. Les détails des raccordements de la dalle avec la dalle de pied et l'ancrage de celle-ci à la fondation pour résister aux charges sismiques seront soigneusement étudiés. Les waterstops seront étudiés en détail et une redondance de waterstops sera, le cas échéant, adoptée. Les matériaux drainants sous la dalle de parement doivent être choisis avec beaucoup de soin, et le système filtre-enrochement sous la dalle doit être parfaitement compacté. Il y aura lieu de prévoir une galerie de visite le long de la dalle de pied, pour le cas exceptionnel où des dégâts seraient causés par de fortes secousses sismiques. La mise en place d'une zone de remblai à grains fins à l'amont, et au raccordement du masque avec la dalle de pied, peut contribuer au colmatage de toute fissure ou séparation du béton. Les dispositions constructives constituent un problème important dans le cas d'ouvrages mixtes. L'appui et la plinthe en béton ou la dalle de pied doivent avoir une forme permettant un bon compactage de la terre ou de l'enrochement afin d'éviter des vides qui pourraient résulter des tassements d'origine sismique.

- Utilization of shear keys in the construction joints.

- Minimizing of discontinuities in the dam body, such as parapet walls, thrust blocks, piers with appurtenant structures and related, sympathetic, local stress concentrations.

- Increasing the crest width to reduce the consequences of local through-cracking.

- Avoiding a break in slope on the downstream faces of gravity dams to eliminate local stress concentrations.

- Recognition of the fact that buttress dams are particularly sensitive to earthquake shocks parallel to the dam axis.

The performance of combination structures, such as various combinations of concrete and earth/rockfill structures, is of concern under seismic conditions. Of particular interest are dams where the earth, or earth-rock, structures are wrapped around the end of a gravity dam or concrete spillway structure. Of concern are the dynamic interaction, and differential displacement of the adjacent earth and concrete structures. Each structure will have a different response and displacement, which should be analyzed to determine the dynamic compatibility of the two different structures during earthquake shaking. Sloping and battering of the concrete interface to maintain a normal embankment loading against the concrete, and the use of plastic soils and wider filter zones in the embankments, are special design considerations for the interface area.

Another combination type structure that needs particular attention is the concrete facing, or asphaltic concrete facing on a rockfill dam. The details of the concrete connections of the slab with the toeslab, and the anchorage of the toeslab to the foundation to sustain the seismically induced loads, should be carefully considered. The water stop details should be studied, and even redundant waterstops should be utilized. The drainage materials utilized beneath the face slab should be designed very carefully, and the rockfill and filter system beneath the face slab must be well compacted. Consideration should be given to providing an inspection gallery along the toeslab, for the unusual case of potential damage due to a strong seismic shaking. Placement of a fine-grained earthfill zone upstream, and on the facing connection with the toeslab, can aid in the plugging of any crack or concrete separation. Constructability is a major concern for combination structures. The abutment, and the concrete plinth or toe slab, must be shaped to allow good compaction of the earthfill or rockfill to prevent voids that could be caused by earthquake induced settlements.

# 6. GLISSEMENTS DE TERRAIN DANS LA RETENUE

La possibilité de glissements de terrain provoqués par des séismes doit être considérée lors de l'évaluation des risques généraux de glissements dans les retenues. La hauteur de la revanche sera déterminée en tenant compte des seiches, ainsi que des grandes vagues causées par des glissements de terrain dans la retenue. Ces glissements peuvent également bloquer les écoulements dans les ouvrages annexes.

# 6. LANDSLIDES INTO THE RESERVOIR

The possibility of earthquake triggered landslides must be considered when assessing the general hazards of landslides into reservoirs. The freeboard should include considerations associated with seiche, as well as large waves caused by landslides into the reservoir. Landslides also could block flow into appurtenant structures.

# 7. APPAREILS D'AUSCULTATION

Des appareils classiques d'auscultation seront installés, avec, en plus, des appareils pour fortes secousses (voir « Appareils d'auscultation pour barrages sujets à de fortes secousses – Recommandations sur leurs choix, installation, exploitation et entretien » – Référence 10). Si la retenue est suffisamment grande pour être considérée comme source potentielle de séisme induit, un réseau de sismographes sensibles sera mis en place pour enregistrer l'activité microsismique avant la mise en eau de la retenue. Voir « Séismicité induite par une retenue » (Référence 9).

# 7. INSTRUMENTATION

Conventional dam instrumentation should be provided, plus strong motion instruments (see “Strong Motion Instruments at Dams – Guideline’s for their Selection, Installation, Operation and Maintenance”– Reference No. 10). If the reservoir will be large enough to be considered as a potential source of Reservoir Triggered Earthquakes (RTS), a network of sensitive seismographic instruments should be established to record micro-seismic activity prior to reservoir filling. Refer to “Reservoir Triggered Seismicity” (Reference No. 9).

# 8. RÉSUMÉ ET CONCLUSIONS

Les barrages de tous types peuvent être conçus et construits pour résister de façon satisfaisante aux charges sismiques dans les zones de forte séismicité. L'expérience montre que des barrages modernes ayant subi de fortes secousses sismiques ont eu un bon comportement. En fait, des barrages soumis à des séismes plus forts que leur séisme de dimensionnement ont eu un bon comportement.

Des considérations sismiques orientent de nombreuses décisions techniques au cours de la conception des barrages, allant du choix du site aux types d'appareils d'auscultation à installer sur le site et le barrage. Il importe d'avoir une bonne connaissance de la configuration, de la géologie et des conditions géotechniques du site du barrage. Le type de barrage choisi et les caractéristiques de projet doivent être compatibles avec le site du barrage et son environnement sismique. Les présentes recommandations décrivent plusieurs aspects structuraux, concernant les barrages en remblai et les barrages en béton, à prendre en compte et à incorporer dans la conception, le calcul et la construction de barrages dans des régions sismiques. Tous ces aspects doivent être considérés par le projeteur et évalués. Lorsque l'occasion se présente, le comportement de tout barrage soumis à de fortes secousses sismiques sera analysé. Un programme continu de comparaisons des comportements avec les évaluations analytiques de ceux-ci sera maintenu en vue de réaliser des progrès dans la connaissance du comportement des barrages sujets à des charges sismiques.

# 8. SUMMARY AND CONCLUSIONS

Dams of all types can be designed and constructed to satisfactorily resist earthquake loadings in areas of high seismicity. Experience shows that good performance has been experienced by modern dams that have been shaken by strong earthquakes. In fact, dams that have been shaken by earthquakes stronger than their design basis earthquake (DBE), have performed well.

Seismic considerations guide many decisions in dam design, ranging from site selection to the type of instrumentation to be installed at the dam. There is a need to take particular cognizance of damsite configuration, geology and geotechnical conditions. The dam type selected, and the design features, should be compatible with the damsite and its seismic environment. These guidelines describe several structural features for embankment and concrete dams that should be considered and incorporated in the design, analysis and construction of dams in seismic regions. All of these features should be considered by the designer and evaluated. When the opportunity exists, the performance of any dam subjected to strong shaking should be analyzed. A continuing program of behavior comparisons with analytical estimates of behavior should be maintained in order to advance the knowledge of the behavior of dams subjected to earthquakes.

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# ANNEXES / APPENDICES (*)

| | | | |
|---|---|---|---|
| Annexe A - | Tableau : Comportement de barrages au cours de séismes enregistrés dans le passé | Appendix A - | Table : Historic performance of dams during earthquakes |
| Annexe B - | Exemples : Comportement observé sur des barrages au cours de séismes | Appendix B - | Case histories : Observed performance of dams during earthquakes |

(*) En anglais seulement / In English only

# APPENDIX A

**TABLE**

**HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES**

* See explanation for dam type at end of table

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **AUGUSTA** | GA, USA | E | --- | Charleston | 31-Aug-1886 | 7,0 | 180,0 | Collapse |
| **STEPHENSON CREEK** | CA, USA | M | --- | --- | 13-Jul-1894 | --- | 64,0 | Minor |
| **O. SAN ANDREAS** | CA, USA | E | 8,5 | San Francisco | 19-Apr-1906 | 8,3 | 0,0 | Minor |
| **N. SAN ANDREAS** | CA, USA | E | 29,4 | San Francisco | 19-Apr-1906 | 8,3 | 0,0 | Minor |
| **LAKE RANCH** | CA, USA | E | 11,5 | San Francisco | 19-Apr-1906 | 8,3 | 0,1 | Moderate |
| **BEAR GULCH** | CA, USA | E | 13,6 | San Francisco | 19-Apr-1906 | 8,3 | 3,2 | None |
| **PILARCITOS** | CA, USA | E | 31,2 | San Francisco | 19-Apr-1906 | 8,3 | 3,2 | None |
| **SARATOGA** | CA, USA | E | --- | San Francisco | 19-Apr-1906 | 8,3 | 0,1 | Moderate |
| **U. HOWELL** | CA, USA | E | 11,5 | San Francisco | 19-Apr-1906 | 8,3 | 0,2 | Moderate |
| **L. HOWELL** | CA, USA | E | 10,9 | San Francisco | 19-Apr-1906 | 8,3 | 0,2 | Moderate |
| **CROCKER** | CA, USA | E | 13,6 | San Francisco | 19-Apr-1906 | 8,3 | 2,0 | None |
| **BURLINGAME** | CA, USA | E | 7,3 | San Francisco | 19-Apr-1906 | 8,3 | 1,6 | None |
| **EMERALD LAKE No.1** | CA, USA | E | 17,3 | San Francisco | 19-Apr-1906 | 8,3 | 2,2 | None |
| **NOTRE DAME** | CA, USA | E | 15,2 | San Francisco | 19-Apr-1906 | 8,3 | 3,2 | None |
| **U. CRYSTAL SPRINGS** | CA, USA | E | 22,7 | San Francisco | 19-Apr-1906 | 8,3 | 0,0 | Moderate |
| **L. CRYSTAL SPRINGS** | CA, USA | GA | 38,5 | San Francisco | 19-Apr-1906 | 8,3 | 0,4 | None |
| **SEARSVILLE** | CA, USA | GA | 19,4 | San Francisco | 19-Apr-1906 | 8,3 | 60,0 | None |
| **LAGUNITA** | CA, USA | E | 4,5 | San Francisco | 19-Apr-1906 | 8,3 | 6,4 | None |
| **BELVEDERE** | CA, USA | E | 14,5 | San Francisco | 19-Apr-1906 | 8,3 | 8,0 | None |
| **U.MOUNT N. BASIN** | CA, USA | E | 5,2 | San Francisco | 19-Apr-1906 | 8,3 | 8,0 | None |
| **LAGUNITAS** | CA, USA | E | 14,5 | San Francisco | 19-Apr-1906 | 8,3 | 8,0 | None |
| **COWELL** | CA, USA | E | 15,2 | San Francisco | 19-Apr-1906 | 8,3 | 19,2 | None |
| **ESTATES** | CA, USA | E | 28,2 | San Francisco | 19-Apr-1906 | 8,3 | 28,8 | None |
| **BERRYMAN** | CA, USA | E | 12,1 | San Francisco | 19-Apr-1906 | 8,3 | 28,8 | None |
| **SUMMIT** | CA, USA | E | 6,4 | San Francisco | 19-Apr-1906 | 8,3 | 30,4 | None |
| **LAKE CHABOT** | CA, USA | E | 40,9 | San Francisco | 19-Apr-1906 | 8,3 | 46,4 | None |
| **PACIFIC GROVE** | CA, USA | E | 6,1 | San Francisco | 19-Apr-1906 | 8,3 | 41,6 | None |
| **LAKE RALPHINE** | CA, USA | E | 10,6 | San Francisco | 19-Apr-1906 | 8,3 | 35,2 | None |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **TEMESCAL** | CA, USA | E | 31,8 | San Francisco | 19-Apr-1906 | 8,3 | 29,0 | Minor |
| **U. SAN LEANDRO** | CA, USA | E | 37,9 | San Francisco | 19-Apr-1906 | 8,3 | 37,0 | None |
| **PIEDMONT No. 1** | CA, USA | E | 15,8 | San Francisco | 19-Apr-1906 | 8,3 | 30,0 | Minor |
| **PORT COSTA** | CA, USA | E | 13,6 | San Francisco | 19-Apr-1906 | 8,3 | 44,8 | None |
| **FORREST LAKE** | CA, USA | E | 18,2 | San Francisco | 19-Apr-1906 | 8,3 | 44,8 | None |
| **LAKE HERMAN** | CA, USA | E | 15,2 | San Francisco | 19-Apr-1906 | 8,3 | 51,2 | None |
| **L. ST HELENA** | CA, USA | E | 15,2 | San Francisco | 19-Apr-1906 | 8,3 | 51,2 | None |
| **U. ST HELENA** | CA, USA | E | 15,2 | San Francisco | 19-Apr-1906 | 8,3 | 51,2 | None |
| **LAKE CAMILLE** | CA, USA | E | 9,1 | San Francisco | 19-Apr-1906 | 8,3 | 52,8 | None |
| **LAKE FREY** | CA, USA | E | 25,2 | San Francisco | 19-Apr-1906 | 8,3 | 59,2 | None |
| **VOLCANO LAKE** | Mexico | E | 3,6 | Imperial V. | 22-Jun-1915 | 5,3 | 0,0 | Collapse |
| **FAIRMONT** | CA, USA | E | --- | Imperial V. | 22-Oct-1916 | 6,0 | 22,0 | None |
| **SHEFFIELD** | CA, USA | E | 7,6 | Santa Barbara | 29-Jun-1925 | 6,3 | 11,2 | Collapse |
| **BARAHONA** | Chile | T | 60,6 | Talca | 01-Oct-1928 | 8,4 | 160,0 | Collapse |
| **CHATSWORTH No. 2** | CA, USA | HF | 13,3 | Santa Monica | 30-Aug-1930 | 5,3 | 1,0 | Moderate |
| **TUAI** | New Zlnd | CG | 4,2 | New Zealand | 02-Feb-1931 | X | --- | None |
| **TUAI DIVERSION** | New Zlnd | E | 5,2 | New Zealand | 02-Feb-1931 | X | --- | Minor |
| **HOOVER** | NV, USA | GA | 220,0 | Induced eqk | 1936 | 5,0 | 8,0 | None |
| **MALPASO** | Peru | CCRD | 77,3 | Peru | 10-Oct-1938 | | ---- | Minor |
| **VOLCANO LAKE** | Mexico | E | 3,6 | El Centro | 18-May-1940 | 7,1 | 0,0 | Collapse |
| **LAGUNA** | CA, USA | E | 15,2 | El Centro | 18-May-1940 | 7,1 | 67,0 | Minor |
| **COGOTI** | Chile | CFRD | 83,3 | Illapel | 04-Apr-1943 | 7,9 | 89,0 | Minor |
| **NORTH END** | CA, USA | E | --- | Calipatria | 20-Jul-1950 | 5,4 | 3,0 | Minor |
| **POGGIO CANCELLI** | Italy | E | 17,0 | Gran Sasso | 05-Sep-1950 | 5,5 | 6,4 | None ? |
| **BOUQUET CANYON** | CA, USA | E | 57,6 | Kern County | 21-Jul-1952 | 7,7 | 73,6 | None |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **ISABELLA** | CA, USA | E | 56,1 | Kern County | 21-Jul-1952 | 7,7 | 86,0 | None |
| **DRY CANYON** | CA, USA | HF | 20,0 | Kern County | 21-Jul-1952 | 7,7 | 70,0 | Moderate |
| **BUENA VISTA** | CA, USA | E ? | 6,1 | Kern County | 21-Jul-1952 | 7,7 | 28,0 | Moderate |
| **SOUTH HAYWEE** | CA, USA | HF | 24,5 | Kern County | 21-Jul-1952 | 7,7 | 152,0 | Minor |
| **FAIRMONT** | CA, USA | HF | 36,7 | Kern County | 21-Jul-1952 | 7,7 | 57,6 | None |
| **DRINKWATER** | CA, USA | E | 31,8 | Kern County | 21-Jul-1952 | 7,7 | 67,2 | None |
| **TEJON STORAGE** | CA, USA | E | 9,7 | Kern County | 21-Jul-1952 | 7,7 | 6,4 | Minor |
| **LAHONTAN** | NV, USA | E | 37,9 | Fallon | 23-Aug-1954 | 6,7 | 48,0 | None |
| **COLEMAN** | NV, USA | Comp | --- | Fallon | 23-Aug-1954 | 6,7 | 24,0 | Collapse |
| **SAGUSPE** | NV, USA | E | --- | Fallon | 23-Aug-1954 | 6,7 | 24,0 | Collapse |
| **ROGERS** | NV, USA | M | --- | Fallon | 23-Aug-1954 | 6,7 | 80,0 | Collapse |
| **PONTEBA** | Algeria | CG | 17,9 | Orleansville | 09-Sep-1954 | 6,8 | 3,5 | Major |
| **STEEG** | Algeria | CGB | 89,4 | Orleansville | 09-Sep-1954 | 6,8 | 70,0 | None |
| **OUEDD FODDA** | Algeria | CG | 100,3 | Orleansville | 09-Sep-1954 | 6,8 | 5,0 | None |
| **ARCATA** | CA, USA | E | 16,7 | Eureka | 21-Dec-1954 | 6,6 | 8,0 | Moderate |
| **ST MARY'S** | CA, USA | M | 15,2 | Daly City | 23-Oct-1955 | 5,4 | 3,0 | Minor |
| **BLACKBROOK** | Gr. Britain | G | 30,3 | ? | 11-Feb-1957 | 5,6 | 6,4 | Moderate |
| **PINZANES** | Mexico | CFRD | 66,7 | Mexico | 28-Jul-1957 | 7,5 | ---- | None |
| **HEBGEN** | MN, USA | E | 27,3 | Hebgen Lake | 17-Aug-1959 | 7,1 | 16,0 | Serious |
| **XIGEER** | China | E | 6,4 | Bachu | 13-Apr-1961 | 6,8 | 35,0 | Major |
| **HSINFENGKIANG** | China | CGB | 104,2 | Hsinfengkiang | 19-Mar-1962 | 6,1 | 1,1 | Serious |
| **MONTEYNARD** | France | CA | 154,1 | Induced eqk | - -1962 | 4,9 | 4,0 | None |
| **KARIBA** | Rhodesia | CA | 127,3 | Induced eqk | 23-Sep-1963 | 6,1 | ---- | None |
| **EKLUTNA** | AK, USA | Comp | 6,1 | Good Friday | 27-Mar-1964 | 8,4 | 100,0 | Serious |
| **LA CALERA** | Mexico | ECRD | 27,9 | Mexico | - -1964 | VIII | ---- | Serious |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **EL SOLDADO** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | ---- | Collapse |
| **LLIU LLIU** | Chile | E | 20,0 | Chile | 28-May-1965 | 7,1 | --- | Moderate |
| **EL CERRADO** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 37,0 | Moderate |
| **CATAPILCO** | Chile | T | 13,6 | Chile | 28-May-1965 | 7,1 | --- | Serious |
| **LA PATAGUA** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 22,0 | Collapse |
| **SAUCE** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 66,0 | Moderate |
| **RAMAYANA** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 85,0 | Serious |
| **CERRO BLANCO** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 96,0 | Minor |
| **BELLAVISTA** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 55,0 | Serious |
| **EL COBRE** | Chile | T | --- | Chile | 28-May-1965 | 7,1 | 35,0 | Collapse |
| **KREMASTA** | Greece | CA | 163,9 | Induced eqk ? | 05-Feb-1966 | 6,2 | 11,0 | None |
| **FUYANG RIVER** | China | E | | Xingtai | 22-Mar-1966 | 7,2 | 21,0 | Major |
| **KOYNA** | India | CG | 102,4 | Koyna | 11-Dec-1967 | 6,5 | 3,0 | Serious |
| **VIR** | India | E | 23,9 | Koyna | 11-Dec-1967 | 6,5 | 8,0 | Moderate |
| **SHOREY** | Peru | T | --- | Peru | 1969 | | ---- | Collapse |
| **WANGWU** | China | E | 25,7 | Bohai Gulf | 18-Jul- 1969 | 7,2 | ? | Major |
| **YEYUAN** | China | E | 24,9 | Bohai Gulf | 18-Jul- 1969 | 7,2 | ? | Serious |
| **HUACHOPOLCA** | Peru | T | --- | Peru | 1970 | | ---- | Collapse |
| **LOPEZ** | CA, USA | E | 50,3 | San Fernando | 09-Feb-1971 | 6,5 | 8,0 | None |
| **YARNELL DEBRIS** | CA, USA | E | 14,8 | San Fernando | 09-Feb-1971 | 6,5 | 8,8 | None |
| **DRY CANYON** | CA, USA | HF | 20,0 | San Fernando | 09-Feb-1971 | 6,5 | 10,4 | None |
| **SAN FERNANDO DK. B** | CA, USA | E | 10,9 | San Fernando | 09-Feb-1971 | 6,5 | 10,4 | Moderate |
| **CHANNEL DIV. DIKE** | CA, USA | E | 13,0 | San Fernando | 09-Feb-1971 | 6,5 | 10,4 | Minor |
| **HANSEN** | CA, USA | E | 29,4 | San Fernando | 09-Feb-1971 | 6,5 | 14,4 | None |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **GREEN VERDUGO** | CA, USA | E | 35,5 | San Fernando | 09-Feb-1971 | 6,5 | 14,4 | None |
| **DRINKWATER** | CA, USA | E | 31,8 | San Fernando | 09-Feb-1971 | 6,5 | 14,4 | None |
| **PORTER ESTATE** | CA, USA | E | 12,4 | San Fernando | 09-Feb-1971 | 6,5 | 16,0 | None |
| **BOUQUET CANYON** | CA, USA | E | 57,6 | San Fernando | 09-Feb-1971 | 6,5 | 17,6 | None |
| **RESERVOIR No. 1** | CA, USA | E | 10,6 | San Fernando | 09-Feb-1971 | 6,5 | 22,4 | None |
| **CHATSWORTH** | CA, USA | HF | 13,3 | San Fernando | 09-Feb-1971 | 6,5 | 23,0 | None |
| **SEPULVEDA** | CA, USA | E | 17,3 | San Fernando | 09-Feb-1971 | 6,5 | 24,8 | None |
| **HAROLD** | CA, USA | E | 9,1 | San Fernando | 09-Feb-1971 | 6,5 | 26,4 | None |
| **DIEDERICH** | CA, USA | E | 18,2 | San Fernando | 09-Feb-1971 | 6,5 | 27,2 | None |
| **ENCINO** | CA, USA | E | 50,9 | San Fernando | 09-Feb-1971 | 6,5 | 26,2 | None |
| **SANTA FELICIA** | CA, USA | E | 60,6 | San Fernando | 09-Feb-1971 | 6,5 | 26,2 | None |
| **RUNKLE** | CA, USA | E | 12,4 | San Fernando | 09-Feb-1971 | 6,5 | 28,8 | None |
| **U. HOLLYWOOD** | CA, USA | E | 26,4 | San Fernando | 09-Feb-1971 | 6,5 | 28,8 | None |
| **U. FRANKLIN** | CA, USA | E | 16,7 | San Fernando | 09-Feb-1971 | 6,5 | 28,8 | None |
| **U. STONE CANYON** | CA, USA | E | 33,3 | San Fernando | 09-Feb-1971 | 6,5 | 28,8 | None |
| **GLENOAKS** | CA, USA | E | 18,8 | San Fernando | 09-Feb-1971 | 6,5 | 30,4 | None |
| **STONE CANYON** | CA, USA | E | 56,1 | San Fernando | 09-Feb-1971 | 6,5 | 30,4 | None |
| **FAIRMONT** | CA, USA | HF | 36,7 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **L. FRANKLIN** | CA, USA | HF | 31,2 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **EAGLE ROCK** | CA, USA | E | 34,2 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **ROWENA** | CA, USA | E | 8,8 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **RUBIO DEBRIS BASIN** | CA, USA | E | 16,7 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **SAWTELL** | CA, USA | E | 10,3 | San Fernando | 09-Feb-1971 | 6,5 | 33,6 | None |
| **J.W.WISDA** | CA, USA | E | 15,2 | San Fernando | 09-Feb-1971 | 6,5 | 33,6 | None |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **SILVER LAKE** | CA, USA | HF | 16,1 | San Fernando | 09-Feb-1971 | 6,5 | 33,6 | None |
| **EATON WASH** | CA, USA | E | 19,1 | San Fernando | 09-Feb-1971 | 6,5 | 33,6 | None |
| **ELYSIAN** | CA, USA | E | 21,8 | San Fernando | 09-Feb-1971 | 6,5 | 35,2 | None |
| **WOOD RANCH** | CA, USA | E | 44,2 | San Fernando | 09-Feb-1971 | 6,5 | 35,2 | None |
| **ASCOT** | CA, USA | E | 22,1 | San Fernando | 09-Feb-1971 | 6,5 | 35,2 | None |
| **CHEVY CHASE** | CA, USA | E | 10,6 | San Fernando | 09-Feb-1971 | 6,5 | 28,8 | None |
| **L. SAN FERNANDO** | CA, USA | HF | 42,4 | San Fernando | 09-Feb-1971 | 6,5 | 11,2 | Major |
| **U. SAN FERNANDO** | CA, USA | HF | 24,2 | San Fernando | 09-Feb-1971 | 6,5 | 11,2 | Serious |
| **PACOIMA** | CA, USA | CA | 112,7 | San Fernando | 09-Feb-1971 | 6,5 | 5,0 | None |
| **BIG TUJUNGA** | CA, USA | CA | 76,1 | San Fernando | 09-Feb-1971 | 6,5 | 32,0 | None |
| **SANTA ANITA** | CA, USA | CA | 76,1 | San Fernando | 09-Feb-1971 | 6,5 | 27,0 | None |
| **EL COBRE** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | 80,0 | Serious |
| **CATAPILCO** | Chile | E | 13,9 | Chile | 08-Jul- 1971 | 7,5 | 110,0 | Moderate |
| **COLLAGUA** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | ---- | None |
| **EL MELON** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | ---- | None |
| **SALAMANCA** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | 110,0 | Collapse |
| **ILLAPEL** | Chile | T | 7,9 | Chile | 08-Jul- 1971 | 7,5 | 100,0 | Collapse |
| **LIMAHUIDA** | Chile | T,E | 10,0 | Chile | 08-Jul- 1971 | 7,5 | 100,0 | Moderate |
| **LLIU LLIU** | Chile | E | 20,0 | Chile | 08-Jul- 1971 | 7,5 | ---- | Serious |
| **CERRO NEGRO** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | ---- | Collapse |
| **LOS MAQUIS** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | ---- | Moderate |
| **LAS PATAGUAS** | Chile | T | --- | Chile | 08-Jul- 1971 | 7,5 | ---- | Moderate |
| **SHIMEN** | China | E | 44,6 | Haicheng | 04-Feb-1975 | 7,3 | 33,0 | Serious |
| **OROVILLE** | CA, USA | ECRD | 233,3 | Oroville | 01-Aug-1975 | 5,7 | 6,9 | None |
| **LA VILLITA** | Mexico | ECRD | 59,7 | Mexico | 11-Oct-1975 | 5,9 | 40,0 | None |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Mexico | 11-Oct-1975 | 5,9 | 79,0 | None |
| **LA VILLITA** | Mexico | ECRD | 59,7 | Mexico | 15-Nov-1975 | 7,2 | 27,0 | None |
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Mexico | 15-Nov-1975 | 7,2 | 23,0 | None |
| **PAIHO (BAIHE)** | China | E | 64,5 | Tangshan | 28-Jul- 1976 | 7,8 | 150,0 | Moderate |
| **SIATHINZE** | China | CGA | 19,9 | Tangshan | 28-Jul- 1976 | 7,8 | ? | Moderate |
| **TOUHO (DOUHE)** | China | E | 21,8 | Tangshan | 28-Jul- 1976 | 7,8 | ---- | Serious |
| **AMBIESTA** | Italy | CA | 58,8 | Friuli | 06-May-1976 | 6,5 | 22,0 | None |
| **LUMIEI** | Italy | CA | 135,2 | Friuli | 06-May-1976 | 6,5 | 30,0 | None |
| **MAINA DISAURIS** | Italy | CA | 135,2 | Friuli | 06-May-1976 | 6,5 | 43,0 | None |
| **BARCIS** | Italy | CA | 49,7 | Friuli | 06-May-1976 | 6,5 | 48,0 | None |
| **IZVORUL MONTELVI** | Romania | CG | 126,2 | Vrancea | 04-Mar-1977 | 7,2 | 100,0 | None |
| **POIANA USULUI** | Romania | CB | 79,4 | Vrancea | 04-Mar-1977 | 7,2 | 60,0 | None |
| **PEREZ CALDERA** | Chile | T | --- | Argentina | 24-Nov-1977 | 7,4 | 350,0 | Minor |
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Guerrero | 14-Mar-1979 | 7,6 | 110,0 | Minor |
| **LA VILLITA** | Mexico | ECRD | 59,7 | Guerrero | 14-Mar-1979 | 7,6 | 110,0 | Minor |
| **VERMILION** | CA, USA | E | 45,5 | Mammoth Lakes | 27-May-1980 | 6,2 | 21,0 | None |
| **LONG VALLEY** | CA, USA | E | 38,2 | Mammoth Lakes | 27-May-1980 | 6,2 | 5,0 | None |
| **LA VILLITA** | Mexico | ECRD | 59,7 | Mexico | 25-Oct-1985 | 7,3 | 31,0 | None |
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Mexico | 25-Oct-1985 | 7,3 | 55,0 | None |
| **LEROY ANDERSON** | CA, USA | ECRD | 71,2 | Morgan Hill | 24-Apr-1984 | 6,2 | 16,0 | Minor |
| **COYOTE** | CA, USA | E | 42,4 | Morgan Hill | 24-Apr-1984 | 6,2 | 24,0 | None |
| **CERRO NEGRO** | Chile | T | --- | Chile | 03-Mar-1985 | 7,7 | ---- | Collapse |
| **LOS LEONES** | Chile | ECRD | 107,3 | Chile | 03-Mar-1985 | 7,7 | ---- | None |
| **VETA DE AGUA** | Chile | T | --- | Chile | 03-Mar-1985 | 7,7 | ---- | Collapse |
| **RAPEL** | Chile | CA | 109,3 | Chile | 03-Mar-1985 | 7,7 | ---- | Moderate |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **LA VILLITA** | Mexico | ECRD | 59,7 | Michoacan | 19-Sep-1985 | 8,1 | 44,0 | Minor |
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Michoacan | 19-Sep-1985 | 8,1 | 75,0 | Minor |
| **LA VILLITA** | Mexico | ECRD | 59,7 | Michoacan | 21-Sep-1985 | 7,5 | 61,0 | None |
| **EL INFIERNILLO** | Mexico | ECRD | 147,0 | Michoacan | 21-Sep-1985 | 7,5 | 80,0 | None |
| **TUAI DIV.** | New Zlnd | CG | 5,2 | Bay of Plenty | 02-Mar-1987 | 6,2 | 11,0 | None |
| **MATAHINA** | New Zlnd | ECRD | 78,5 | Bay of Plenty | 02-Mar-1987 | 6,2 | 23,0 | Moderate |
| **GARVEY RESERVOIR** | CA, USA | E | 48,5 | Whittier | 01-Oct-1987 | 6,1 | 3,0 | None |
| **ORANGE COUNTY RES.** | CA, USA | E | 34,5 | Whittier | 01-Oct-1987 | 6,1 | 23,0 | None |
| **PUDDINGSTONE** | CA, USA | E | 44,8 | Whittier | 01-Oct-1987 | 6,1 | 57,6 | None |
| **WHITTIER-NARROWS** | CA, USA | E | 28,5 | Whittier | 01-Oct-1987 | 6,1 | 4,0 | None |
| **AUSTRIAN** | CA, USA | E | 56,1 | Loma Prieta | 17-Oct-1989 | 7,1 | 11,5 | Serious |
| **NEWELL** | CA, USA | E | 55,2 | Loma Prieta | 17-Oct-1989 | 7,1 | 18,4 | Moderate |
| **LEXINGTON** | CA, USA | E | 62,1 | Loma Prieta | 17-Oct-1989 | 7,1 | 20,6 | Minor |
| **VASONA PERCOLATION** | CA, USA | E | 10,3 | Loma Prieta | 17-Oct-1989 | 7,1 | 24,5 | Minor |
| **LEROY ANDERSON** | CA, USA | E | 71,2 | Loma Prieta | 17-Oct-1989 | 7,1 | 26,9 | Minor |
| **ELMER J. CHESBRO** | CA, USA | E | 28,8 | Loma Prieta | 17-Oct-1989 | 7,1 | 19,0 | Moderate |
| **GUADALUPE** | CA, USA | E | 43,0 | Loma Prieta | 17-Oct-1989 | 7,1 | 18,1 | Minor |
| **RINCONADA** | CA, USA | E | 12,1 | Loma Prieta | 17-Oct-1989 | 7,1 | 26,2 | Minor |
| **ALMADEN** | CA, USA | E | 33,3 | Loma Prieta | 17-Oct-1989 | 7,1 | 15,5 | Minor |
| **COASTWAYS** | CA, USA | E | 13,9 | Loma Prieta | 17-Oct-1989 | 7,1 | 37,9 | Minor |
| **SODA LAKE** | CA, USA | E | 10,6 | Loma Prieta | 17-Oct-1989 | 7,1 | 28,2 | Moderate |
| **MILL CREEK** | CA, USA | E | 23,0 | Loma Prieta | 17-Oct-1989 | 7,1 | 30,2 | None |
| **L. CRYSTAL SPRINGS** | CA, USA | CG | 38,5 | Loma Prieta | 17-Oct-1989 | 7,1 | 69,0 | None |
| **SAN ANTONIO** | CA, USA | E | 48,5 | Pomona Valley | 28-Feb-1990 | 5,5 | 3,0 | Minor |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **SEFID-RUD** | Iran | CGB | 105,5 | Manjil | 21-Jun-1990 | 7,3 | 32,0 | Moderate |
| **AMBUKLAO** | Luzon | E | 129,1 | Philippines | 16-Jul-1990 | 7,7 | 80,0 | Moderate |
| **AYA** | Luzon | E | 10,6 | Philippines | 16-Jul-1990 | 7,7 | 46,4 | Minor |
| **BINGA** | Luzon | E | 101,5 | Philippines | 16-Jul-1990 | 7,7 | 48,0 | Moderate |
| **CANILI** | Luzon | E | 69,7 | Philippines | 16-Jul-1990 | 7,7 | 16,0 | Minor |
| **DIAYO** | Luzon | E | 59,7 | Philippines | 16-Jul-1990 | 7,7 | 32,0 | Moderate |
| **MASIWAY** | Luzon | E | 24,8 | Philippines | 16-Jul-1990 | 7,7 | 19,2 | Serious |
| **PANTABANGAN** | Luzon | E | 106,4 | Philippines | 16-Jul-1990 | 7,7 | 48,0 | Minor |
| **COGSWELL** | CA, USA | R | 80,6 | Sierra Madre | 28-Jun-1991 | 5,8 | 3,8 | None |
| **BEAR VALLEY** | CA, USA | MA | --- | Bear Mountain | 28-Jun-1992 | 6,6 | 14,4 | None |
| **WIDE CANYON** | CA, USA | E | 25,5 | Landers | 28-Jun-1992 | 7,4 | --- | Minor |
| **BEAR VALLEY** | CA, USA | MA | 24,2 | Landers | 28-Jun-1992 | 7,4 | 44,8 | None |
| **FENA** | Guam | E | 40,9 | Guam | 08-Aug-1993 | 8,1 | 60,0 | Moderate |
| **L. SAN FERNANDO** | CA, USA | HF | 37,9 | Northridge | 17-Jan-1994 | 6,7 | 9,4 | Serious |
| **U. SAN FERNANDO** | CA, USA | HF | 24,8 | Northridge | 17-Jan-1994 | 6,7 | 10,2 | Moderate |
| **LOS ANGELES** | CA, USA | E | 39,4 | Northridge | 17-Jan-1994 | 6,7 | 9,8 | Minor |
| **LAS LLAJAS** | CA, USA | E | 29,1 | Northridge | 17-Jan-1994 | 6,7 | --- | Minor |
| **LOWER FRANKLIN** | CA, USA | E | 31,2 | Northridge | 17-Jan-1994 | 6,7 | 18,1 | Minor |
| **SANTA FELICIA** | CA, USA | E | 64,5 | Northridge | 17-Jan-1994 | 6,7 | 33,1 | Minor |
| **PACOIMA** | CA, USA | CA | 110,6 | Northridge | 17-Jan-1994 | 6,7 | 18,2 | Minor |
| **SYCAMORE CANYON** | CA, USA | E | 12,1 | Northridge | 17-Jan-1994 | 6,7 | 24,0 | Minor |
| **SCHOOLHOUSE D. B.** | CA, USA | E | 11,5 | Northridge | 17-Jan-1994 | 6,7 | 13,8 | Minor |

| DAM NAME | Country | Type | H [ m ] | Earthquake Name | Date | M or MMI | Dist. [km] | Damage Rating |
|---|---|---|---|---|---|---|---|---|
| **MORRIS S. JONES** | CA, USA | E | 14,8 | Northridge | 17-Jan-1994 | 6,7 | 42,7 | Minor |
| **RESERVOIR 5, LA CTY** | CA, USA | E | 10,9 | Northridge | 17-Jan-1994 | 6,7 | 19,0 | Minor |
| **COGSWELL** | CA, USA | E | 80,6 | Northridge | 17-Jan-1994 | 6,7 | 52,3 | Minor |
| **BRAND Diversion Basin** | CA, USA | E | 13,6 | Northridge | 17-Jan-1994 | 6,7 | --- | Minor |
| **PORTER ESTATE** | CA, USA | E | 12,4 | Northridge | 17-Jan-1994 | 6,7 | 5,3 | Moderate |
| **RUBIO DEBRIS BASIN** | CA, USA | E | 19,4 | Northridge | 17-Jan-1994 | 6,7 | 38,7 | Minor |
| **ZHONG HAI** | China | CG | --- | Lijiang | 03-Feb-1996 | 7,0 | 4,0 | Serious |

LEGEND

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

# HISTORIC PERFORMANCE OF DAMS DURING EARTHQUAKES
## Earthquakes classified by the Richter scale of 6.5 or over, with some exceptions in which damage has occurred

Dams constructed before the World War II(up to 1945)

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| BUSHUKO | Fukui JAPAN | TE | 20,3 | Fukui | 28-juin-48 | 7,1 | 24 | None |
| HONENIKE | Kagawa JAPAN | MV | 32,1 | Nankai | 21-déc-46 | 8,0 | 50 | Minor |
| IMAWATARI | Gifu JAPAN | PG | 34,2 | | 09-sept-69 | 6,6 | 41 | None |
| KANEYAMA | Gifu JAPAN | PG | 36,1 | | 09-sept-69 | 6,6 | 38 | None |
| KASAGI | Gifu JAPAN | PG | 40,6 | | 09-sept-69 | 6,6 | 43 | None |
| KASAGI | Gifu JAPAN | PG | 40,6 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 48 | None |
| KAWABE | Gifu JAPAN | PG | 27,0 | | 09-sept-69 | 6,6 | 36 | None |
| KOSE | Nara JAPAN | PG | 36,4 | To-Nankai | 07-déc-44 | 7,9 | 80 | None |
| KOSE | Nara JAPAN | PG | 36,4 | Yoshino | 18-juil-52 | 6,8 | 33 | None |
| KUSAKI | Hyogo JAPAN | PG | 24,8 | Kitatajima | 23-mai-25 | 6,8 | 48 | None |
| MAKABE | Gunma JAPAN | PG | 25,8 | Nishisaitama | 21-sept-31 | 6,9 | 40 | None |
| MITAKI | Tottori JAPAN | CB | 23,6 | Tottori | 10-sept-43 | 7,2 | 33 | None |
| MIURA | Nagano JAPAN | PG | 82,7 | | 09-sept-69 | 6,6 | 27 | None |
| MIURA | Nagano JAPAN | PG | 82,7 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 19 | None |
| MURAYAMACHOSEICHI-KAMIENTEI | Tokyo JAPAN | TE | 23,9 | Kanto | 01-sept-23 | 7,9 | - | Moderate |
| MURAYAMACHOSEICHI-SHIMOENTEI | Tokyo JAPAN | TE | | Kanto | 01-sept-23 | 7,9 | - | Moderate |
| NISHIMURA | Gifu JAPAN | PG | 19,4 | Kitamino | 19-août-61 | 7,0 | 41 | None |
| NISHIMURA | Gifu JAPAN | PG | 19,4 | | 09-sept-69 | 6,6 | 8 | None |

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| NISHIMURA | Gifu JAPAN | PG | 19,4 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 37 | None |
| NUNOBIKIGOHONMATSU | Hyogo JAPAN | PG | 33,0 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 19 | None |
| OCHIAI | Gifu JAPAN | PG | 33,0 | | 09-sept-69 | 6,6 | 48 | None |
| OCHIAI | Gifu JAPAN | PG | 33,0 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 34 | None |
| OGUCHI No.1 | Ishikawa JAPAN | PG | 28,2 | Kitamino | 19-août-61 | 7,0 | 27 | None |
| OHARA | Toyama JAPAN | PG | 51,8 | Kitamino | 19-août-61 | 7,0 | 45 | None |
| OI | Gifu JAPAN | PG | 53,0 | | 09-sept-69 | 6,6 | 45 | None |
| OI | Gifu JAPAN | PG | 53,0 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 40 | None |
| OMATAZAWA | Kanagawa JAPAN | PG | 18,5 | Kanto | 01-sept-23 | 7,9 | 36 | None |
| OMATAZAWA | Kanagawa JAPAN | PG | 18,5 | Kitaizu | 26-nov-30 | 7,3 | 39 | None |
| ONBARA | Okayama JAPAN | CB | 23,9 | Tottori | 10-sept-43 | 7,2 | 25 | None |
| ONO | Yamanashi JAPAN | TE | 37,0 | Kanto | 01-sept-23 | 7,9 | 51 | Serious |
| OSODANI | Wakayama JAPAN | TE | 26,4 | | 12-janv-38 | 6,8 | 45 | None |
| OSODANI | Wakayama JAPAN | TE | 26,4 | | 15-juin-48 | 6,7 | 19 | None |
| SEBATANI | Nagano JAPAN | PG | 22,7 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 39 | None |
| SHIMOHARA | Gifu JAPAN | PG | 23,6 | | 09-sept-69 | 6,6 | 13 | None |
| SHIMOHARA | Gifu JAPAN | PG | 23,6 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 39 | None |
| TOKIWA | Nagano JAPAN | PG | 23,9 | | 09-sept-69 | 6,6 | 47 | None |
| TOKIWA | Nagano JAPAN | PG | 23,9 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 4 | None |
| TSUZURAO | Nara JAPAN | PG | 26,4 | To-Nankai | 07-déc-44 | 7,9 | 83 | None |
| TSUZURAO | Nara JAPAN | PG | 26,4 | Yoshino | 18-juil-52 | 6,8 | 33 | None |
| YAMASHITAIKE | Oita JAPAN | TE | 18,2 | | 21-avr-75 | 6,4 | 8 | Severe |
| YOSHINODANI | Ishikawa JAPAN | PG | 24,2 | Kitamino | 19-août-61 | 7,0 | 30 | None |
| YURAGAWA | Kyoto JAPAN | PG | 15,2 | Kitatango | 07-mars-27 | 7,3 | 28 | None |

Dams constructed after the World War II(1946 -     )

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| AIGAERI | Osaka JAPAN | PG | 33,03 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 37 | None |
| AINONO | Akita JAPAN | TE | 40,61 | | 16-oct-70 | 6,2 | 16 | Minor |
| AKI | Oita JAPAN | PG | 34,85 | | 26-août-83 | 6,8 | 9 | None |
| AKIGAMI | Gifu JAPAN | PG | 73,64 | | 09-sept-69 | 6,6 | 40 | None |
| AKIGAMI | Gifu JAPAN | PG | 73,64 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 35 | None |
| ANSEI | Hyogo JAPAN | TE | 28,79 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 33 | None |
| AONO | Hyogo JAPAN | PG | 28,79 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 43 | None |
| ARAKINE | Chiba JAPAN | TE | 33,33 | | 17-déc-87 | 6,7 | 24 | None |
| ARASAWA | Yamagata JAPAN | PG | 60,61 | Niigata | 16-juin-64 | 7,5 | 51 | None |
| ASAHI | Gifu JAPAN | PG | 86,36 | | 09-sept-69 | 6,6 | 42 | None |
| ASAHI | Gifu JAPAN | PG | 86,36 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 35 | None |
| AYUYAGAWA | Hyogo JAPAN | PG | 46,06 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 39 | None |
| DAINICHIGAWA | Hyogo JAPAN | PG | 42,42 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 47 | None |
| DONDO | Hyogo JAPAN | PG | 71,21 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 19 | None |
| EIRAKU | Osaka JAPAN | PG | 39,70 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 39 | None |
| FUNAKIIKE | Hyogo JAPAN | TE | 30,91 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 30 | None |
| FUTAKAWA | Hokkaido JAPAN | PG | 30,30 | Urakawa-oki | 21-mars-82 | 7,1 | 39 | None |
| FUTAKAWA | Hokkaido JAPAN | PG | 30,30 | | 14-janv-87 | 7,0 | 34 | None |
| HAGINARI | Akita JAPAN | PG | 60,61 | | 29-mars-85 | 6,5 | 43 | None |
| HANEKAWA | Akita JAPAN | TE | 17,58 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 95 | None |
| HATOGAYA | Gifu JAPAN | PG | 62,73 | Kitamino | 19-août-61 | 7,0 | 29 | None |
| HATSUOGAWA | Hyogo JAPAN | PG | 30,91 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 40 | None |
| HAYAKUCHI | Akita JAPAN | PG | 60,61 | | 29-mars-85 | 6,5 | 17 | None |
| HIGASHIUEDA | Gifu JAPAN | PG | 17,88 | Kitamino | 19-août-61 | 7,0 | 42 | None |
| HIGASHIUEDA | Gifu JAPAN | PG | 17,88 | | 09-sept-69 | 6,6 | 21 | None |
| HIGASHIUEDA | Gifu JAPAN | PG | 17,88 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 35 | None |

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| HIRATA | Yamagata JAPAN | TE | 15,76 | Niigata | 16-juin-64 | 7,5 | 95 | None |
| HIROTO | Miyazaki JAPAN | PG | 65,76 | | 03-déc-96 | 6,6 | 38 | None |
| HITOKURA | Hyogo JAPAN | PG | 74,55 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 49 | None |
| HOKKAWA | Osaka JAPAN | PG | 45,15 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 39 | None |
| HONJOGAWA | Hyogo JAPAN | PG | 47,27 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 50 | None |
| INAGAWA | Nagano JAPAN | PG | 42,73 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 17 | None |
| INEKOKI | Nagano JAPAN | VA | 59,70 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 43 | None |
| INEKURAIKE | Osaka JAPAN | ER | 32,12 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 43 | None |
| ISHIBUCHI | Iwate JAPAN | ER | 52,73 | Miyagi-ken-Hokubu | 30-avr-62 | 6,5 | 46 | None |
| IWASHIMIZU | Hokkaido JAPAN | PG | 29,70 | Urakawa-oki | 21-mars-82 | 7,1 | 52 | None |
| IWASHIMIZU | Hokkaido JAPAN | PG | 29,70 | | 14-janv-87 | 7,0 | 33 | None |
| IWAYA | Gifu JAPAN | ER | 126,67 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 38 | None |
| KAERUGAWA | Niigata JAPAN | TE | 18,79 | Niigata | 16-juin-64 | 7,5 | 92 | None |
| KAMEYAMA | Chiba JAPAN | PG | 34,24 | | 17-déc-87 | 6,7 | 40 | None |
| KANAYAMA | Chiba JAPAN | TE | 28,18 | | 17-déc-87 | 6,7 | 47 | None |
| KANNO | Yamagata JAPAN | PG | 44,24 | Niigata | 16-juin-64 | 7,5 | 74 | None |
| KANOGAWA | Ehime JAPAN | PG | 60,61 | | 06-août-68 | 6,6 | 32 | None |
| KASABORI | Niigata JAPAN | PG | 73,94 | Niigata | 16-juin-64 | 7,5 | 97 | None |
| KATSUURA | Chiba JAPAN | TE | 28,79 | | 17-déc-87 | 6,7 | 37 | None |
| KIJIYAMA | Yamagata JAPAN | HG | 45,76 | Niigata | 16-juin-64 | 7,5 | 67 | None |
| KISO | Nagano JAPAN | PG | 34,85 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 6 | None |
| KITAYAMA | Hyogo JAPAN | TE | 24,24 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 31 | Minor |
| KOSHIBU | Nagano JAPAN | VA | 104,24 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 44 | None |
| KOUCHI | Hyogo JAPAN | ER | 23,94 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 12 | None |
| KOUJIYA | Hyogo JAPAN | ER | 43,94 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 49 | None |
| KUGUNO | Gifu JAPAN | PG | 26,67 | | 09-sept-69 | 6,6 | 40 | None |
| KUGUNO | Gifu JAPAN | PG | 26,67 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 35 | None |
| KURAMOCHI | Chiba JAPAN | TE | 17,27 | | 17-déc-87 | 6,7 | 26 | None |
| KUROISHI | Hyogo JAPAN | TE | 29,39 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 50 | None |

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| KUSHIKINO | Kagoshima JAPAN | ER | 31,52 | | 13-mai-97 | 6,1 | 19 | Minor |
| KUZURYU | Fukui JAPAN | ER | 127,27 | | 09-sept-69 | 6,6 | 36 | None |
| MAKIO | Nagano JAPAN | ER | 103,94 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 4 | Minor |
| MAKOMANAI | Hokkaido JAPAN | ER | 34,24 | Hokkaido-Nansei-oki | 12-juil-93 | 7,8 | 69 | Minor |
| MARUYAMA | Gifu JAPAN | PG | 97,58 | | 09-sept-69 | 6,6 | 41 | None |
| MARUYAMA | Gifu JAPAN | PG | 97,58 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 43 | None |
| MASEGAWA No.2 | Gifu JAPAN | PG | 44,24 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 43 | None |
| MATSUKAWA | Nagano JAPAN | PG | 83,94 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 38 | None |
| MAWARISEKIOIKE | Aomori JAPAN | TE | 17,88 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 115 | Major |
| MEYA | Aomori JAPAN | PG | 57,58 | | 29-mars-85 | 6,5 | 37 | None |
| MIBORO | Gifu JAPAN | ER | 130,30 | Kitamino | 19-août-61 | 7,0 | 18 | None |
| MIBORO | Gifu JAPAN | ER | 130,30 | | 09-sept-69 | 6,6 | 42 | None |
| MIDONO | Nagano JAPAN | VA | 94,85 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 40 | None |
| MINASE | Akita JAPAN | ER | 66,06 | Niigata | 16-juin-64 | 7,5 | 145 | Minor |
| MINASE | Akita JAPAN | ER | 66,06 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 201 | Minor |
| MINOOGAWA | Osaka JAPAN | ER | 46,67 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 48 | None |
| MIOMOTE | Niigata JAPAN | PG | 82,12 | Niigata | 16-juin-64 | 7,5 | 47 | Minor |
| MIWA | Nagano JAPAN | PG | 68,79 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 45 | None |
| MORIYOSHI | Akita JAPAN | PG | 61,52 | | 29-mars-85 | 6,5 | 30 | None |
| NAGARA | Chiba JAPAN | TE | 51,82 | | 17-déc-87 | 6,7 | 29 | None |
| NAGAWADO | Nagano JAPAN | VA | 154,24 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 36 | None |
| NAGUMA | Chiba JAPAN | TE | 18,48 | | 17-déc-87 | 6,7 | 22 | None |
| NARAI | Nagano JAPAN | ER | 59,70 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 26 | None |
| NARIAIIKE | Hyogo JAPAN | PG | 32,73 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 42 | None |
| NARUDE | Gifu JAPAN | PG | 53,03 | Kitamino | 19-août-61 | 7,0 | 39 | None |
| NARUGO | Miyagi JAPAN | VA | 93,94 | Miyagi-ken-Hokubu | 30-avr-62 | 6,5 | 38 | None |
| NICHINAN | Miyazaki JAPAN | PG | 46,67 | | 03-déc-96 | 6,6 | 40 | None |
| NIIHO | Niigata JAPAN | TE | 38,48 | Niigata | 16-juin-64 | 7,5 | 79 | None |

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| NIIKAPPU | Hokkaido JAPAN | ER | 102,12 | | 14-janv-87 | 7,0 | 31 | None |
| OGURAGAWA | Niigata JAPAN | TE | 33,03 | Niigata | 16-juin-64 | 7,5 | 84 | None |
| OKAWASE | Hyogo JAPAN | PG | 50,61 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 39 | None |
| OKUNIIKAPPU | Hokkaido JAPAN | VA | 60,91 | | 14-janv-87 | 7,0 | 25 | None |
| OKUNO | Shizuoka JAPAN | ER | 59,70 | | 20-févr-90 | 6,5 | 26 | None |
| ONO | Kyoto JAPAN | PG | 60,91 | | 14-août-67 | 6,6 | 23 | None |
| OTAKIGAWA | Nagano JAPAN | PG | 18,18 | | 09-sept-69 | 6,6 | 32 | None |
| OTAKIGAWA | Nagano JAPAN | PG | 18,18 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 13 | None |
| OTANI | Hyogo JAPAN | TE | 16,36 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 6 | None |
| OYA | Ishikawa JAPAN | ER | 56,06 | | 07-févr-93 | 6,6 | 32 | None |
| PIRIKA | Hokkaido JAPAN | PG/ER | 39,70 | Hokkaido-Nansei-oki | 12-juil-93 | 7,8 | 89 | None |
| SAKAGAMI | Gifu JAPAN | PG | 23,33 | Kitamino | 19-août-61 | 7,0 | 46 | None |
| SAMANI | Hokkaido JAPAN | PG | 43,64 | Urakawa-oki | 21-mars-82 | 7,1 | 31 | None |
| SAMANI | Hokkaido JAPAN | PG | 43,64 | | 14-janv-87 | 7,0 | 41 | None |
| SASOGAWA | Fukui JAPAN | PG | 75,45 | Kitamino | 19-août-61 | 7,0 | 28 | None |
| SASOGAWA | Fukui JAPAN | PG | 75,45 | | 09-sept-69 | 6,6 | 47 | None |
| SAWA-IKE | Osaka JAPAN | PG | 20,91 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 38 | None |
| SEKISHIBATAMEIKE | Fukushima JAPAN | TE | 29,70 | Niigata | 16-juin-64 | 7,5 | 103 | Minor |
| SERIKAWA | Oita JAPAN | PG | 51,82 | | 03-déc-96 | 6,6 | 46 | None |
| SHIGERI | Hyogo JAPAN | ER | 16,97 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 50 | None |
| SHIMO-NIIKAPPU | Hokkaido JAPAN | PG | 45,76 | | 14-janv-87 | 7,0 | 33 | None |
| SHIZUNAI | Hokkaido JAPAN | PG | 65,76 | Urakawa-oki | 21-mars-82 | 7,1 | 39 | None |
| SHIZUNAI | Hokkaido JAPAN | PG | 65,76 | | 14-janv-87 | 7,0 | 33 | None |
| SHUNBETSU | Hokkaido JAPAN | PG | 26,97 | Urakawa-oki | 21-mars-82 | 7,1 | 54 | None |
| SHUNBETSU | Hokkaido JAPAN | PG | 26,97 | | 14-janv-87 | 7,0 | 27 | None |
| SUBARI | Akita JAPAN | PG | 71,52 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 96 | None |
| SUBARI | Akita JAPAN | PG | 71,52 | | 29-mars-85 | 6,5 | 31 | None |
| TAKANE No.1 | Gifu JAPAN | VA | 132,12 | | 09-sept-69 | 6,6 | 44 | None |
| TAKANE No.1 | Gifu JAPAN | VA | 132,12 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 27 | None |

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| TAKANE No.2 | Gifu JAPAN | HG | 68,48 | | 09-sept-69 | 6,6 | 42 | None |
| TAKANE No.2 | Gifu JAPAN | HG | 68,48 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 28 | None |
| TAKI | Iwate JAPAN | PG | 69,70 | | 09-janv-87 | 6,6 | 34 | None |
| TAKIHATA | Osaka JAPAN | PG | 61,52 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 50 | None |
| TANIYAMA | Hyogo JAPAN | TE | 28,18 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 7 | None |
| TATSUO | Aomori JAPAN | TE | 14,85 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | - | Severe |
| TENNO | Hyogo JAPAN | PG | 33,64 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 16 | None |
| TOBE | Aomori JAPAN | PG | 42,73 | | 29-mars-85 | 6,5 | 14 | None |
| TOKIWA | Hyogo JAPAN | TE | 33,33 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 7 | Minor |
| TOKUHATA | Hyogo JAPAN | ER | 15,45 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 50 | None |
| TOTTORIIKE | Osaka JAPAN | PG | 29,39 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 39 | None |
| TSUBAKIHARA | Gifu JAPAN | PG | 67,88 | Kitamino | 19-août-61 | 7,0 | 36 | None |
| TSUNOKAWA | Gifu JAPAN | PG | 21,52 | Kitamino | 19-août-61 | 7,0 | 43 | None |
| USHINO | Miyagi JAPAN | ER | 21,82 | Miyagi-ken-oki | 12-juin-78 | 7,4 | 125 | Minor |
| UTSUBO | Gifu JAPAN | PG | 25,45 | Kitamino | 19-août-61 | 7,0 | 50 | None |
| YAMAGUCHI | Nagano JAPAN | PG | 38,48 | | 09-sept-69 | 6,6 | 49 | None |
| YAMAGUCHI | Nagano JAPAN | PG | 38,48 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 25 | None |
| YOMIKAKI | Nagano JAPAN | PG | 31,82 | | 09-sept-69 | 6,6 | 49 | None |
| YOMIKAKI | Nagano JAPAN | PG | 31,82 | Nagano-ken-Seibu | 14-sept-84 | 6,8 | 16 | None |
| YUNOSAWA | Akita JAPAN | TE | 17,58 | | 16-oct-70 | 6,2 | 12 | Moderate |
| YUNOSAWA | Akita JAPAN | TE | 17,58 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 94 | Severe |
| YUZURUHA | Hyogo JAPAN | PG | 41,82 | Hyogo-ken-Nanbu | 17-janv-95 | 7,2 | 43 | None |

Local irrigation Dams constructed under empirical technique (before the year of 1956 which the Standard of Dam Design, 'Earthdam for Local irrigation Dams,' was set)

| DAM NAME | LOCATION | ID | H (m) | EQ.NAME | DATE | MAG. | DIST. (km) | DAMAGE |
|---|---|---|---|---|---|---|---|---|
| BIRUZAWA * | Yamagata JAPAN | TE | 23,33 | Niigata | 16-juin-64 | 7,5 | 100 | Moderate |
| BIRUZAWA * | Yamagata JAPAN | TE | 23,33 | Miyagi-ken-oki | 12-juin-78 | 7,4 | 173 | Major |
| FUJITA * | Yamagata JAPAN | TE | 17,58 | Niigata | 16-juin-64 | 7,5 | 88 | Serious |
| HACHIMENZAWA * | Akita JAPAN | TE | 20,30 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 130 | Moderate |
| HASEIKE No.2 * | Yamagata JAPAN | TE | 15,45 | Niigata | 16-juin-64 | 7,5 | - | Minor |
| HIGASHIDAIZAWA * | Akita JAPAN | TE | 16,97 | Nihonkai-Chubu | 26-mai-83 | 7,7 | - | Major |
| HONGO * | Aomori JAPAN | TE | 21,82 | Nihonkai-Chubu | 26-mai-83 | 7,7 | 134 | Major |
| IWAKURA * | Akita JAPAN | TE | 16,67 | Oga | 01-mai-39 | 6,8 | 39 | Serious |
| KAMONOTANI * | - JAPAN | TE | 14,85 | Niigata | 16-juin-64 | 7,5 | - | Severe |
| KOGANEZAWA * | Aomori JAPAN | TE | 20,91 | Tokachi-oki | 16-mai-68 | 7,9 | 236 | Moderate |
| KONAKA * | Chiba JAPAN | TE | 20,61 | | 17-déc-87 | 6,7 | 23 | Moderate |
| NISHINO * | - JAPAN | TE | 17,88 | Niigata | 16-juin-64 | 7,5 | - | Minor |
| NIWAIKIMINE * | Hokkaido JAPAN | TE | 14,85 | Hokkaido-Nansei-oki | 12-juil-93 | 7,8 | 74 | Serious |
| OIKE * | Nagano JAPAN | TE | 16,06 | Matsushiro Swarm | 1965 to 1970 | (5.4) | - | Minor |
| OKURA * | Yamagata JAPAN | TE | 15,76 | Niigata | 16-juin-64 | 7,5 | 102 | Minor |
| OOTSUTSUMI * | Akita JAPAN | TE | 15,45 | Nihon-kai-Chubu | 26-mai-83 | 7,7 | 76 | Minor |
| OTANI-IKE * | Ehime JAPAN | TE | 26,97 | Nankai | 21-déc-46 | 8,0 | 80 | Moderate |
| SHIONOIRI * | Nagano JAPAN | TE | 18,48 | Matsushiro Swarm | 1965 to 1970 | (5.4) | - | Minor |
| TAKINOSAWA * | Yamagata JAPAN | TE | 14,85 | Niigata | 16-juin-64 | 7,5 | 85 | Moderate |
| TANOSAWA * | Aomori JAPAN | TE | 22,73 | Tokachi-oki | 16-mai-68 | 7,9 | 221 | Moderate |
| UMAGAMI * | Yamagata JAPAN | TE | 24,55 | Niigata | 16-juin-64 | 7,5 | 84 | Minor |
| UMAKURA * | Akita JAPAN | TE | 23,33 | Niigata | 16-juin-64 | 7,5 | 152 | Moderate |

Local irrigation Dams denoted by an asterisk have not been constructed under the Standard of Dam Design.

IDs in the list are subject to the ones of the Description of types of dams set by ICOLD except HG, Hollow Gravity.

# APPENDIX B

## OBSERVED PERFORMANCE OF DAMS DURING EARTHQUAKES

### CASE HISTORIES

1. Ambiesta, Italy; Friuli Earthquake (1976)
2. Ambuklao, Philippines; Philippines Earthquake (1990)
3. Bear Valley, CA; Landers Earthquake (1992)
4. Binga, Philippines; Philippines Earthquake (1990)
5. Cerro Negro, Chile; Central Chile Earthquake (1985)
6. Chabot, CA; San Francisco Earthquake (1906)
7. Cogoti, Chile; Illapel Earthquake (1943)
8. La Villita, Mexico; Michoacan Earthquake (1985)
9. Los Angeles, CA; Northridge Earthquake (1994)
10. Los Leones, Chile; Central Chile Earthquake (1985)
11. Masiway, Philippines; Philippines Earthquake (1990)
12. Mochikoshi, Japan; Izu-Ohshima-Kinkai Eqk. (1990)
13. Pantabangan, Philippines; Philippines Earthquake (1990)
14. Sefid-Rud, Iran; Manjil Earthquake (1990)
15. Sheffield, CA; Santa Barbara Earthquake (1925)
16. Vermilion, CA; East Sierra Nevada Eqk. Sequence (1980)

# 1. AMBIESTA DAM, ITALY

Ambiesta Dam is a 59 m high concrete arch dam located in Northern Italy. On May 6, 1976, the dam was subjected to the Gemona-Friuli (Friuli) Earthquake, an earthquake of magnitude 6.5 that resulted in hundreds of deaths and extensive property damage. A peak ground acceleration of 0.33 g was recorded at the site. The dam did not suffer any damage from the main shock, nor from any of its foreshocks and aftershocks.

## AMBIESTA DAM

Ambiesta Dam is located near Tolmezzo, in the Eastern Alps, Italy, approximately northwest of the City of Udine (Fig. 1.1). The 59 m high dam is built across the Ambiesta River, a tributary of the Tagliamento River. It has a crest length of 145 m, a crest thickness of 2.0 m, a bottom thickness of 7.8 m, and a reservoir storage of 3.6 $hm^3$. The dam was designed between 1949 and 1954. Construction of the dam began in 1955 and was completed in 1956. The dam was constructed to impound a supply reservoir for the Medio Tagliamento-Somplago hydroelectric plant.

Located in an area of recognized high seismicity, Ambiesta Dam was designed to be earthquake-resistant. The dam was constructed as a symmetrical, double curvature arch with a marked downstream overhang, referred to as a "cupola" arch (Fig. 1. 2). The designers felt that this type of construction would offer the best capacity to withstand severe overloads. The double curvature arch abuts on a "pulvino", which is essentially a thickened perimeter concrete joint, poured along the dam footprint.

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

In anticipation of potential earthquake effects on the structure, seismic analyses were performed during the design phase, using horizontal earthquake load coefficients. Experimental tests were also conducted on four 1:50 and 1:75 scale models of the structure (Semenza et al., 1958). Tests were first conducted by regularly increasing horizontal loads simulating hydrostatic pressure on the 1:50 scale model, until its complete failure. Failure occurred for loads about twelve times the magnitude of normal hydrostatic load. Two of the 1:75 scale models were tested for horizontal seismic forces, using a specially constructed frame and cyclic loading of the chord of the arch. Failure of the upper part of the model, at full reservoir condition, corresponded to an equivalent applied acceleration of 0.75 g. Tests were also performed to simulate vertical earthquake loading on another 1:75 scale model. Collapse of the upper part of the arch occurred under repetitive vertical loads equivalent to 0.76 g acceleration. It was felt at the time by the designers that the applied horizontal and vertical oscillatory "earthquake" forces would largely exceed those expectable at the Ambiesta site, a recognized highly seismic area. Based on the results of these model studies, the sill of the overflow spillway structure (Fig. 1.3) was stiffened to increase the load-carrying capacity of the crest of the arch.

## THE MAY 6, 1976 GEMONA-FRIULI EARTHQUAKE

The May 6, 1976 earthquake, with a magnitude of 6.5, caused 965 deaths, injured 2286 people, and inflicted extensive property damage, estimated at $2.8 billion. The dam was located 14 miles from the epicenter. A maximum acceleration of 0.33 g was recorded at the right abutment of the dam. The May 6 earthquake was preceded by a foreshock of magnitude 4.5, about one minute before the main shock. Major aftershocks of magnitude 5.1, 5.5, 5.9 and 6.0, respectively, occurred in the area over a period of approximately four months following the main shock.

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Ambiesta Dam, as well as thirteen other concrete arch dams in the affected region, did not suffer damage from the 1976 Gemona-Friuli earthquake sequence. Two of the other dams within the epicentral area were also thin arch dams, Maina di Sauris Dam (height 136 m), located 43 km from the epicenter, and Barcis Dam (50 m high). According to the references consulted for the preparation of this write-up, no differential movements within Ambiesta Dam body, and especially at the "pulvino", were reported by the Italian engineers who inspected the dam after the earthquake.

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

Following the largest aftershock (September 15, 1976) of the Friuli Earthquake, the Instituto Sperimentale Modelli E Strutture (ISMES) installed an automatic recording system on Ambiesta Dam, including 30 seismometers, to record horizontal motions of the aftershocks. Fig. 1.3 shows the layout of these instruments on the dam. There were five foundation locations; 20 locations along the downstream face, with two sensors mounted transversely and parallel to the valley; and 20 additional locations along the downstream face, with one sensor mounted radially. From October 8 to October 27, 1976, many smaller aftershocks were recorded, the largest with measured peak velocities of 0.254 cm/s at the base of the dam and 1.04 cm/s at the right abutment. Analysis of the aftershocks records indicated a 5.8:1 amplification factor between crest center and base records, in the stream (radial) direction, and a 10.6:1 amplification factor at the left abutment quarter point. Largest spectral amplifications of the recorded motions occurred at frequencies between 8 and 10 Hertz. The recorded responses of the structure to several of the aftershocks of the earthquake were compared with the corresponding theoretical responses obtained from a dynamic finite element analysis of the dam, using the processed acceleration histories of those aftershocks as input excitations. The mathematical model of the dam had been calibrated through the use of forced vibration testing with a 10-ton mechanical actuator, delivering sinusoidal oscillations at frequencies ranging from 2 to 20 Hertz. The dam analyses assumed an infinitely rigid foundation. Fig. 1.4 shows a comparison between recorded and computed crest responses to some aftershocks of the earthquake.

## CONCLUSION

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

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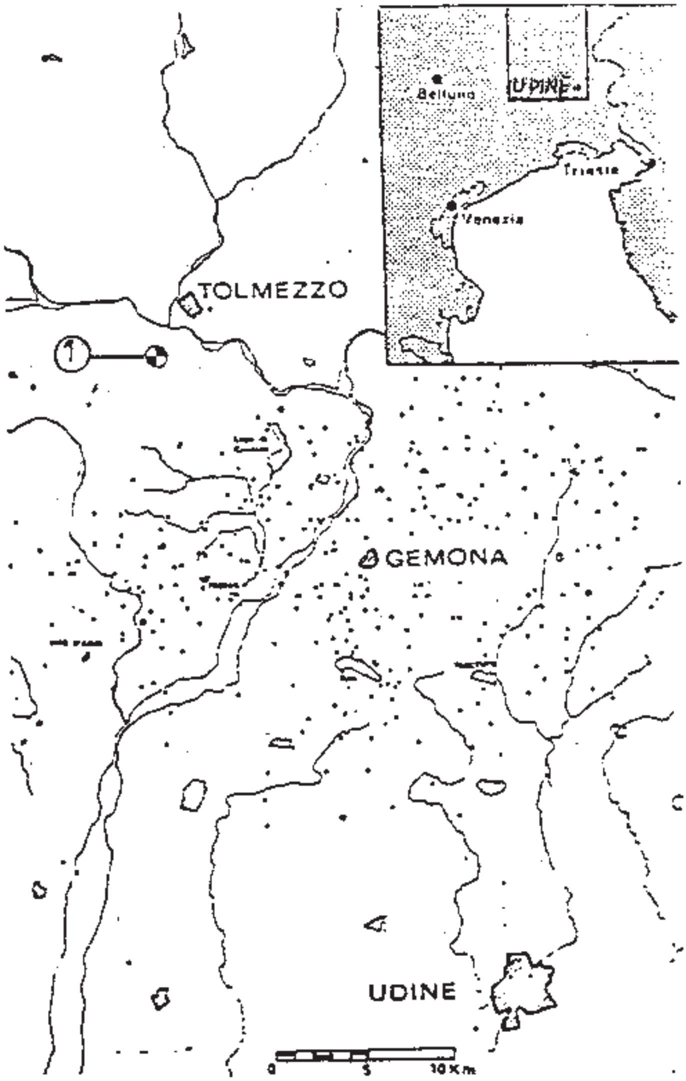


Fig. 1.1
Ambiesta Dam – Epicenters of Shocks (From Castoldi, 1978)
*Barrage Ambiesta – Épicentres des secousses (D'après Castoldi, 1978)*

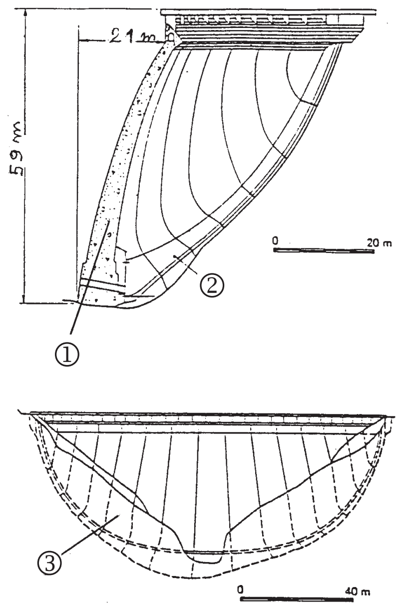


Fig. 1.2
Ambiesta Dam – Section and Elevation
*Barrage Ambiesta – Coupe transversale et élévation*

1) Maximum Section
2) Pulvino
3) Upstream Elevation

(From Hansen, 1979)

*1) Coupe transversale maximale*
*2) Pulvino*
*3) Élévation amont*

*(D'après Hansen, 1979)*

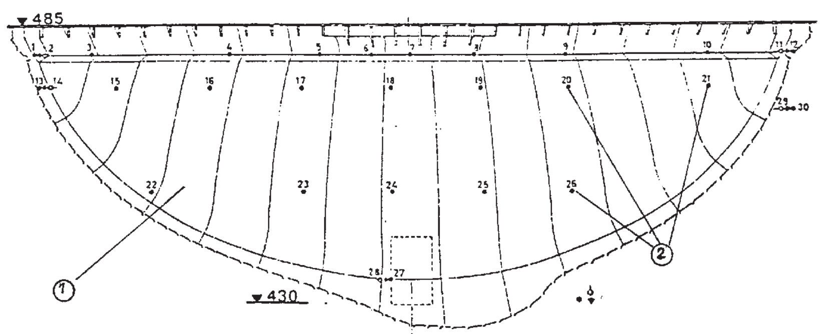


Fig. 1.3
Ambiesta Dam – Layout of Measuring Points
*Barrage Ambiesta – Disposition des points de mesure*

1) Downstream Face
2) Seismometers

(From Castoldi, 1978)

*1) Parement aval du barrage*
*2) Séismomètres*

*(D'après Castoldi, 1978)*

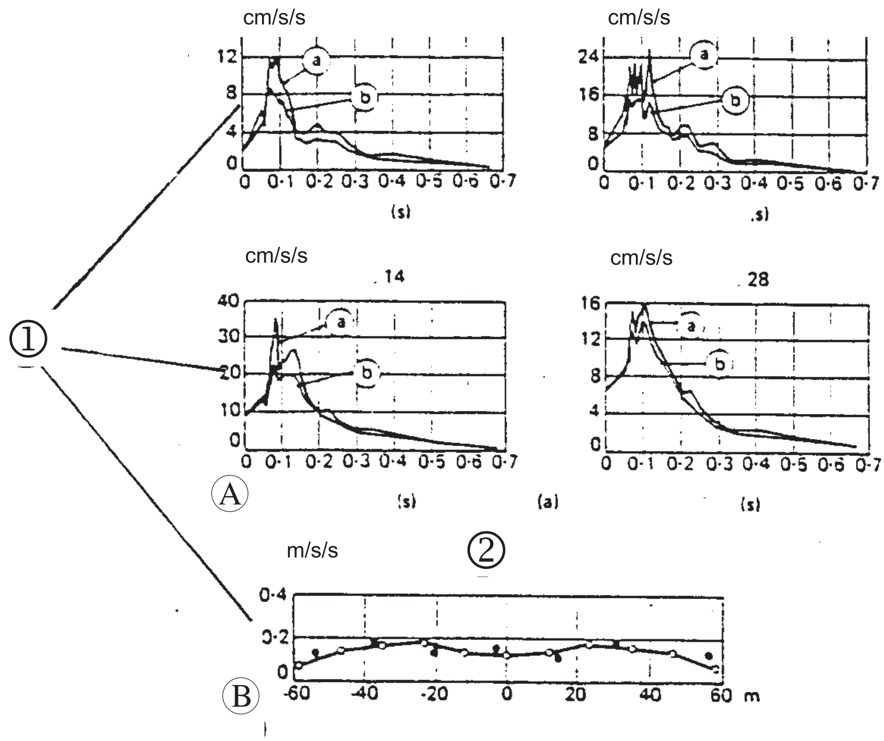


Fig. 1.4
Ambiesta Dam
*Barrage Ambiesta*

A) Response Spectra to Aftershocks
B) Comparison between Computed and Recorded Accelerations

*A) Spectres de réponse aux secousses consécutives*
*B) Comparaison entre les accélérations calculées et observées*

1) Accelerations
2) Crest Arch

*1) Accélérations*
*2) Arc de crête*

a) Computed Values
b) Measured Values

*a) Valeurs calculées*
*b) Valeurs mesurées*

(From Fanelli, 1985)

*(D'après Fanelli, 1985)*

# 2. AMBUKLAO DAM, PHILIPPINES

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

## AMBUKLAO DAM

The Ambuklao Project was placed in service in 1956. Ambuklao Dam, Luzon, Philippines, is a 130 meter high vertical core dumped rockfill dam, see Fig. 2.1 and 2.2. The layout of the dam is shown on Fig. 2.2. Crest width is 12.17 m. The upper part of the upstream and downstream slopes were built at 1.75:1 (horizontal to vertical) and the lower part of both slopes at 2:1 (h to v). The upstream and downstream slopes of the central clayey core slope at 1:4 (h to v). Both sides of the core are protected by thin filter zones. Other project features include a concrete chute spillway, an intake and power-tunnel and an underground powerhouse, see Fig. 2.2. On July 16, 1990, the date of the earthquake, the reservoir level was at El. 752 m. In the following nineteen days after the earthquake, the reservoir was lowered and reached a restricted elevation of 742.5 m.

## THE JULY 16, 1990 EARTHQUAKE

On July 16, 1990, the heavily populated Island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 50 000 square km. At least 1700 people were killed and perhaps 1000 were missing. At least 3500 persons were severely injured. Over 4000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake; even many of the better hotels were damaged.

Seismologically, the July 16 Earthquake is particularly difficult to characterize since it appears to have had two centers of energy release that were apparently triggered within a few seconds of each other, see Fig. 2.3. The first one was located on the Philippine Fault near the city of Cabanatuan; the second center of energy release was on the Digdig Fault, which belongs to the same system as the Philippine Fault and branches off northeast from that feature. The two faults broke along a combined length of about 75 km. The fault displacements were left-lateral strike– slip. The maximum mapped displacement was on the order of 6 meters.

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

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

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

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

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

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

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

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

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

## CONCLUSION

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

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

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

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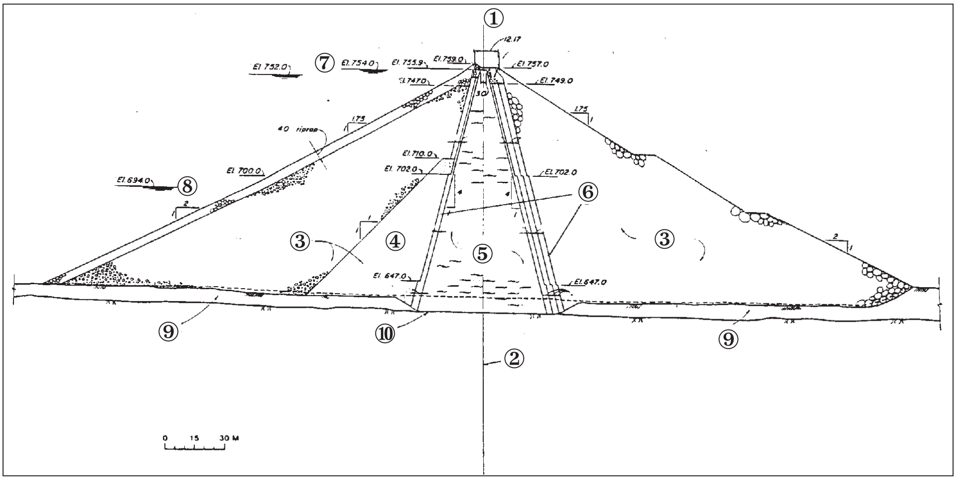


Fig. 2.1

Ambuklao Dam – Maximum Cross Section

*Barrage Ambuklao – Coupe transversale maximale*

1) Axis of Dam
2) Grout Curtain
3) Rolled Pervious Fill
4) Quarried Sand and Gravel
5) Impervious Fill
6) Filters
7) Maximum Storage Level
8) Minimum Storage Level
9) River Gravel
10) Bedrock

*1) Axe du barrage*
*2) Rideau d`injection*
*3) Remblai perméable compacté au rouleau*
*4) Sable et gravier de carrière*
*5) Remblai imperméable*
*6) Filtres*
*7) Niveau maximal de retenue*
*8) Niveau minimal de retenue*
*9) Gravier fluviatile*
*10) Fond rocheux*

Fig. 2.2

Ambuklao Dam – Dam Site Layout

*Barrage Ambuklao – Vue en plan du barrage*

1) Horizontal Axis of Dam — *1) Axe horizontal du barrage*

2) Spillway — *2) Évacuateur de crue*

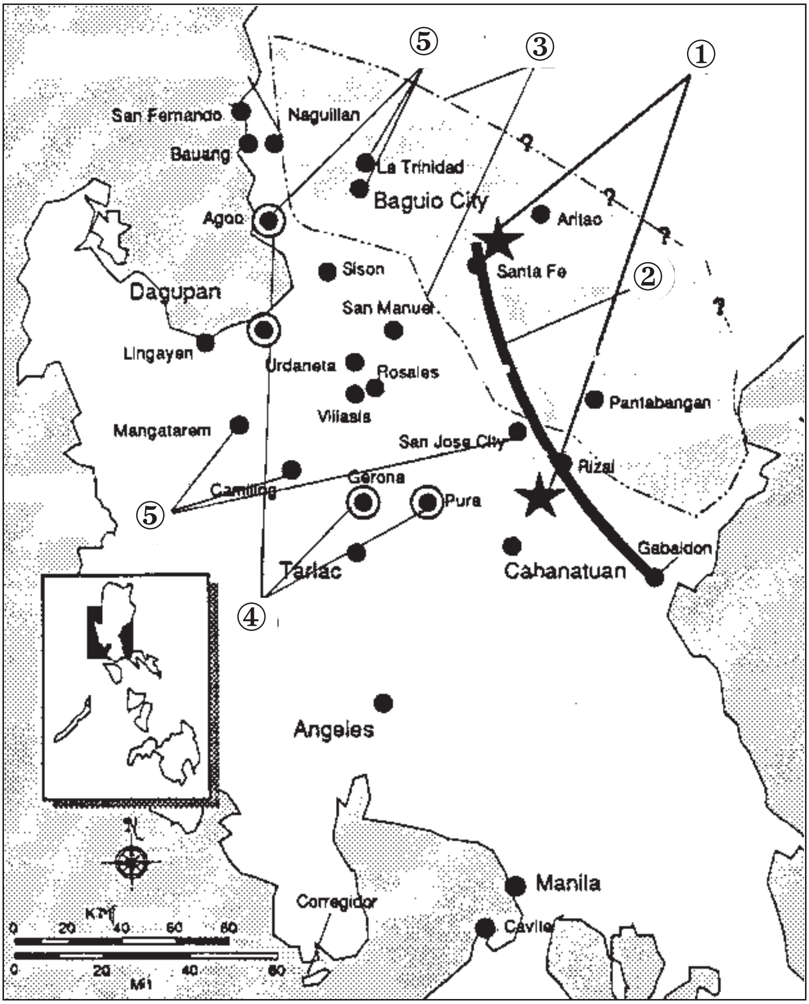


Fig. 2.3

Ambuklao Epicenters Location

*Emplacement des épicentres concernant Ambuklao*

| | |
|---|---|
| 1) Epicenters | *1) Épicentres* |
| 2) Observed Faulting | *2) Mouvements de faille observés* |
| 3) Region of Extensive Landslides | *3) Région de glissements étendus* |
| 4) Extensive Liquefaction Observed | *4) Liquéfaction étendue observée* |
| 5) Structural Damages and Failures | *5) Dégâts causés aux ouvrages* |

# 3. BEAR VALLEY DAM, CALIFORNIA, USA

For the second time in two years, Southern California was jolted awake on June 28, 1992 by the M 7.4 Landers and the Bear Mountain M 6.6 earthquakes, on the anniversary of the 1991 magnitude 5.8 Sierra Madre event. Bear Valley Dam, a rehabilitated 24-m-high concrete dam, was strongly shaken by these two events. The closest distance between the dam and the fault ruptures were 45 km (Landers) and 14.5 km (Big Bear). Thorough inspections after the earthquakes disclosed that the dam was not damaged. The only indication of the shaking was possible slight displacement of girders on the highway bridge located on the dam crest. Estimated peak ground accelerations at the dam site were between 0.40 g and 0.50 g during the second event.

## BEAR VALLEY DAM

Bear Valley Dam is located on Bear Creek in the San Bernardino Mountains, 129 km east of Los Angeles. It impounds 3.2 $hm^3$ Big Bear Lake, a year-round recreation facility in Southern California.

Bear Valley Dam was constructed in 1911-1912 as a 24-m-high, 110-m-long multiple arch structure. There are nine 5.2-m radius (extrados) arches, with a crest elevation of El. 2055 m. The thicknesses of the arches vary from 30 cm at the top, to a maximum of 44 cm. A two-lane concrete girder-type highway bridge is supported by the dam buttresses. Several years prior to the earthquake, concerns over the structural adequacy of the dam during possible severe earthquake shaking or overtopping by large floods had led to reanalysis and rehabilitation of the dam.

The structural upgrade method was conversion of the multiple arch to a gravity dam by infilling the arch bays with conventional mass concrete (Fig. 3.1). The existing arches and buttresses functioned as the upstream and side forms for the mass concrete. The downstream slope was formed at 0.25:1 (horizontal to vertical), except for the top 14 m, which are vertical. Approximately 11 500 cubic metres of concrete were placed. The original dam and mass concrete were made monolithic by providing a gap at their interfaces and contact grouting later. The rehabilitation was accomplished in 1988 and 1989.

The strengthening of the dam included seismic considerations. Two Maximum Credible Earthquakes (MCE) were considered, a M 8.3 earthquake centered along the San Andreas Fault (16 km away), with a peak ground acceleration (PGA) of 0.45 g and 35 seconds of bracketed duration (duration between the first and last peak of 0.05 g or greater). The other was a M 6.0 event, centered on the Helendale Fault, also 16 km away, with a 0.22 g PGA and 10 seconds bracketed duration.

## JUNE 28, 1992 EARTHQUAKES

At 4:58 a.m. on June 28, 1992 the M 7.4 Landers Earthquake occurred on the Johnson Valley-Homestead Valley-Emerson-Camp Rock faults, near the juncture of the Mojave Desert and the San Bernardino Mountains, see Fig. 3.2. The rupture zone stretched north-northwest from Sky Valley for more than 70 km, cutting across several of these known fault traces, rather than following a single previously recognized fault trace. Dramatic fault scarps and up to 6 m of lateral offsets in the Johnson Valley have resulted from this event. Stress changes in the earth's crust resulting from this earthquake caused the M 6.6 Big Bear Earthquake on an unnamed fault to occur at 8:05 a.m. in response to the first rupture sequence. One death, due to falling masonry from a fireplace, and 400 injuries were attributed to the earthquakes. The sparse population on the desert and in the mountains is the reason for these relatively low casualty figures.

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

The closest distance between the dam and the fault ruptures were 45 km (Landers) and 14.5 km (Big Bear). Thorough inspections after the earthquakes disclosed that the Bear Valley Dam had not been damaged. No indication of cracks or distress was visible for both the old and newer parts of the structure. The only indication of the shaking sustained by the dam was possible evidence of slight displacement of girders on the highway bridge located on the dam crest.

## INSTRUMENTATION AND STRONG MOTION RECORDS

Bear Valley Dam was not instrumented to record earthquake motions. Accelerations of as much as 1 g were recorded in Lucerne Valley. Two instruments located in Big Bear Lake City (4 miles away from the dam) and at the Forest Fall Post Office (29 km) away provide indications of the shaking that may have been experienced at the site. At Big Bear Lake City, 0.18 g (horizontal) and 0.08 g (vertical) were recorded during the M 7.4 Landers Earthquake; PGA's of 0.57 g (h) and 0.21 g (v) were recorded during the Bear Valley Earthquake. At Forest Falls P.O., PGA's of 0.12 g (h) and 0.09 g (v) were measured during the first event, and 0.26 g (h) and 0.30 g (v) during the second event.

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

## CONCLUSION

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

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

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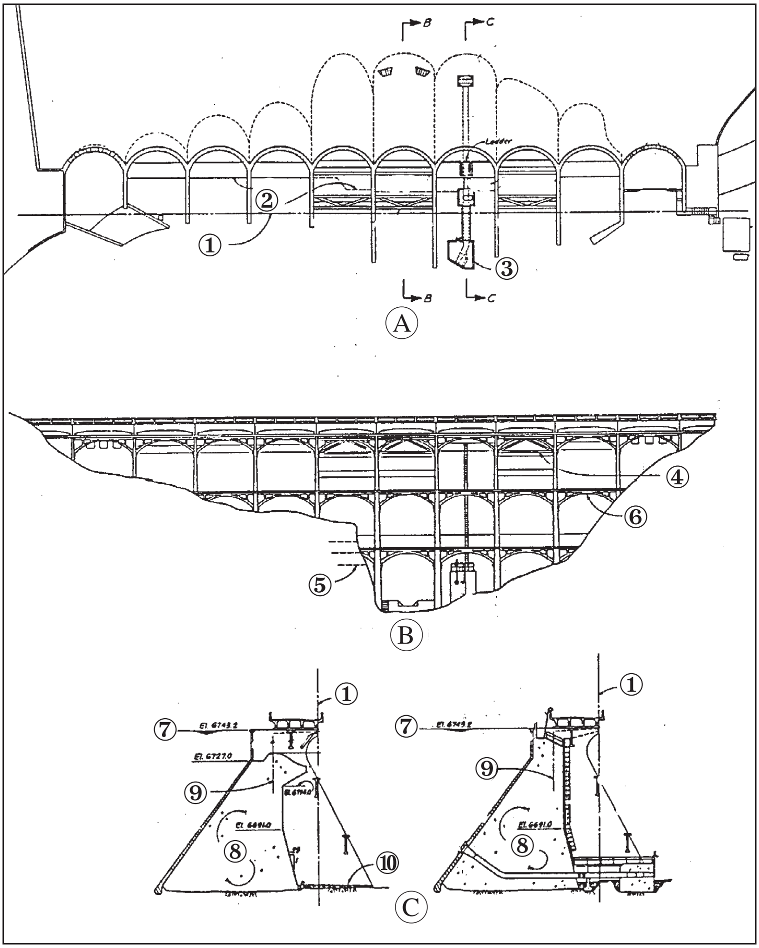


Fig. 3.1

Bear Valley Dam Strengthening Scheme

*Schéma de renforcement du barrage Bear Valley*

| | |
|---|---|
| A) Layout | *A) Vue en plan* |
| B) Downstream Elevation | *B) Élévation aval* |
| C) Sections B-B and C-C | *C) Sections B-B et C-C* |
| 1) Base Line | *1) Ligne de référence* |
| 2) Toe of Gravity Infill | *2) Pied aval du remplissage en béton* |
| 3) Outlet Works | *3) Vidange de fond* |
| 4) Braced Frame | *4) Support avec tirant* |
| 5) Rock Anchors | *5) Tirants d'ancrage du rocher* |
| 6) Existing Buttress Concrete Struts | *6) Entretoises existantes des contreforts* |
| 7) Normal Pool Level | *7) Niveau normal de retenue* |
| 8) Gravity Infill | *8) Remplissage en béton* |
| 9) Extrados Spring Line | *9) Ligne de naissance d'extrados* |
| 10) Crushed Rock Protection | *10) Protection en enrochement* |

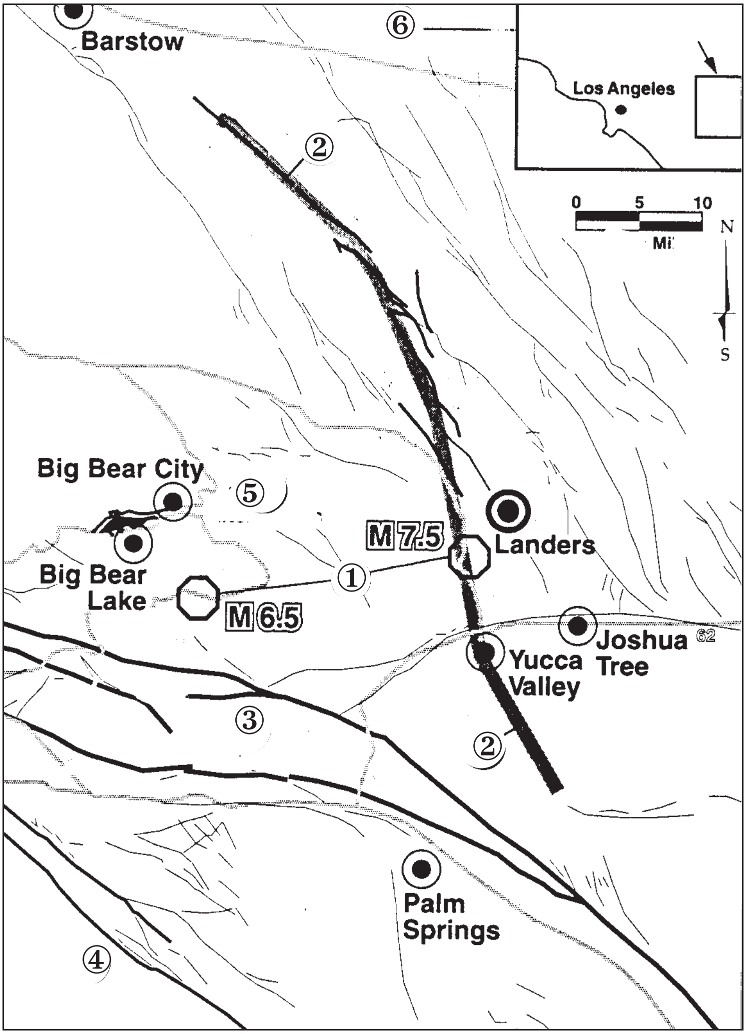


Fig. 3.2

Bear Valley Dam – Epicentral and Fault Locations

*Barrage Bear Valley – Emplacement des failles et des épicentres*

1) Epicenters and Magnitudes
2) Rupture Zone
3) San Andreas Fault Zone
4) San Jacinto Fault Zone
5) Zone of Aftershocks
6) Large Scale Location

*1) Épicentres et magnitudes*
*2) Zone de ruptures*
*3) Zone de la faille San Andreas*
*4) Zone de la faille San Jacinto*
*5) Zone des répliques sismiques*
*6) Situation à grande échelle*

# 4. BINGA DAM, PHILIPPINES

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

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

## BINGA DAM

The Binga Dam Project includes a 102 meter high inclined core rockfill dam. Portions of the rockfill on both sides of the inclined core were rolled (compacted). The outer shells consist of dumped rockfill. The dam layout and cross-section are shown on Fig. 4.1 and 4.2. Other project features include a concrete chute spillway, an intake and power tunnel and an underground powerhouse. The Binga Project was placed in service in 1960. On July 16, 1990, the date of the earthquake, the reservoir was at El. 575 m. By August 4, 1990, the reservoir had been drawn drown to El. 555 m.

## THE JULY 16, 1990 EARTHQUAKE

On July 16, 1990, the heavily populated Island of Luzon, Philippines was shaken by a large earthquake (M 7.7). The earthquake affected an area over 32 000 square km. At least 1700 people were killed and perhaps 1000 were missing. At least 3500 persons were severely injured. Over 4000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake; even many of the better hotels were damaged.

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

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

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

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

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

— Tensile stresses caused by differential settlements induced by changes of geometry in the foundation of the dam’s right abutment, producing diagonal cracks across the crest; and

— Embankment settlement causing tensile stresses at the contact with the spillway structure and producing a transverse crack across the crest.

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

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

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

## CONCLUSION

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

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

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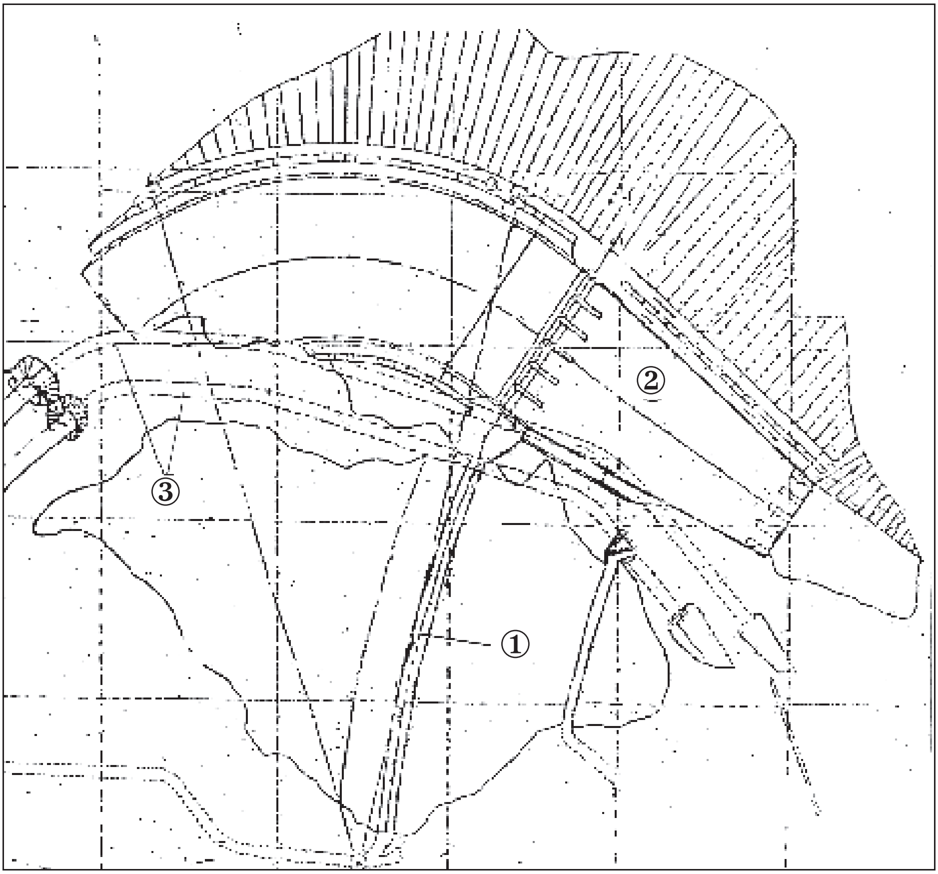


Fig. 4.1
Binga Project Layout
*Vue en plan de l'aménagement Binga*

1) Axis of Dam
2) Spillway
3) Diversion Tunnels

*1) Axe du barrage*
*2) Évacuateur de crue*
*3) Galeries de dérivation*

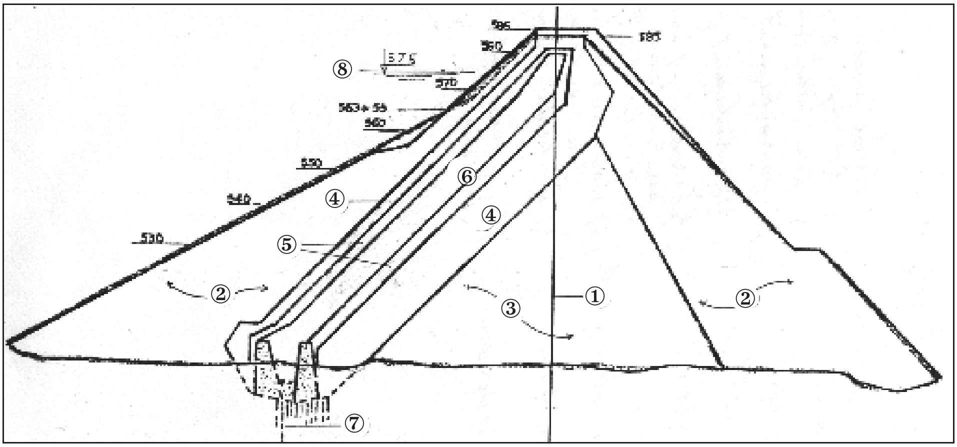


Fig. 4.2

Binga Dam Maximum Section

*Coupe transversale maximale du barrage Binga*

1) Axis of Dam
2) Dumped Rockfill
3) Coarser Rockfill
4) Rolled Rockfill
5) Filter
6) Impervious Rolled Earth Core
7) Grout Curtain
8) Normal High Water Level

*1) Axe du barrage*
*2) Enrochement déversé*
*3) Enrochement grossier*
*4) Enrochement compacté au rouleau*
*5) Filtre*
*6) Noyau étanche en terre, compacté au rouleau*
*7) Rideau d'injection*
*8) Niveau normal de crue*

# 5. CERRO NEGRO TAILINGS DAM, CHILE

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

## CERRO NEGRO DAM NUMBER 4

The Cerro Negro tailings impoundment Number 4 was built outward from a valley side slope. The dam consisted of three sections, a central section roughly parallel to the valley floor, and two transverse sections linking the central section to the valley slope, see Fig. 5.1.

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

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

Three borings were drilled in 1987 at the locations shown on Fig. 5.1. (Castro and Troncoso, 1989). Three zones were encountered: an outer zone consisting of the sand fraction; an intermediate zone of stratified sands and slimes; and an interior zone of slime. The boundaries shown on Fig. 5.1 are based on boring logs data and observations of the walls of a large crevasse formed during the failure. This zoning confirmed the information regarding construction, which indicated a procedure intermediate between upstream and centerline construction methods. All three borings encountered a natural foundation material consisting of a dense gravelly sand.

The zone forming the sand fraction of the tailings, see Fig. 5.1, actually ranged from a silty fine sand, with about 20 percent of silt, to a non-plastic sandy silt. The percentage of fines increased with distance from the outer slope, as shown on Fig. 5.2. Standard penetration testing (SPT) corrected blowcounts within the sand zone increased gradually with depth from about 10 blows/foot near the surface to about 30 blows/foot at a depth of about 20 m.

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

## THE MARCH 3, 1985, CHILE EARTHQUAKE

On March 3, 1985, at 19:47 local time, a strong earthquake shook Central Chile, causing widespread destruction and resulting in 180 deaths, over 2500 persons injured and about 2.6 billion dollars of property damage. The earthquake had a magnitude of 7.8 (Ms). Its focus was located off the coast of Chile, at a depth of 16 km, and within a recognized subduction zone where the Nazca tectonic plate underrides the South American tectonic plate. The epicenter location and peak accelerations instrumentally recorded at various sites within the mesoseismal area are shown on Fig. 5.3. The primary earthquake damage involved both old and modern buildings, industrial facilities, bridges, road embankments, and small earth and tailings dams.

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

A detailed description of the earthquake effects on the Cerro Negro Dam Number 4 can be found in Castro and Troncoso (1989). As a result of the earthquake, a portion of the central section of Dam Number 4 dam failed, and about 130 000 metric tons of slimes and sands were released, forming a large crevasse and breaching the impoundment, see Fig. 5.1. Piles of sand, up to three metres in height, were found within about 100 m of the dam, while some slimes flowed into the Pitipeumo Creek and downstream along the valley for distances of about eight kilometers. A witness indicated that the failure had been preceded by noticeable sloshing of the slimes during the earthquake, and it occurred rather suddenly.

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

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

The 1987 investigation (Castro and Troncoso, 1989) revealed that the outer core of sands was medium dense, with a friction angle of about 36 degrees and undrained steady-state strength (Sus) values ranging from about 127 kPa to 280 kPa, with a median of 196 kPa. The undrained strengths of the slimes were estimated based on laboratory vane shear test results, and were the following:

Undrained Peak Strength, Sup: Sup/vc = 0.27

Undrained Steady-State Strength, Sus: Sus/vc = 0.07

where vc represents the vertical effective overburden pressure.

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

No strong motion records were obtained at or near the dam, but peak ground accelerations were recorded in the general area surrounding the site, see Fig. 5.3. From an examination of Fig. 5.3, it appears that the horizontal peak ground acceleration (PGA) at the dam site probably ranged from 0.30 to 0.40 g. Acceleration time histories with similar PGA's recorded during this event generally show at least ten pulses with accelerations half of the PGA or greater, i.e., between 0.15 to 0.20 g.

## CONCLUSIONS

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

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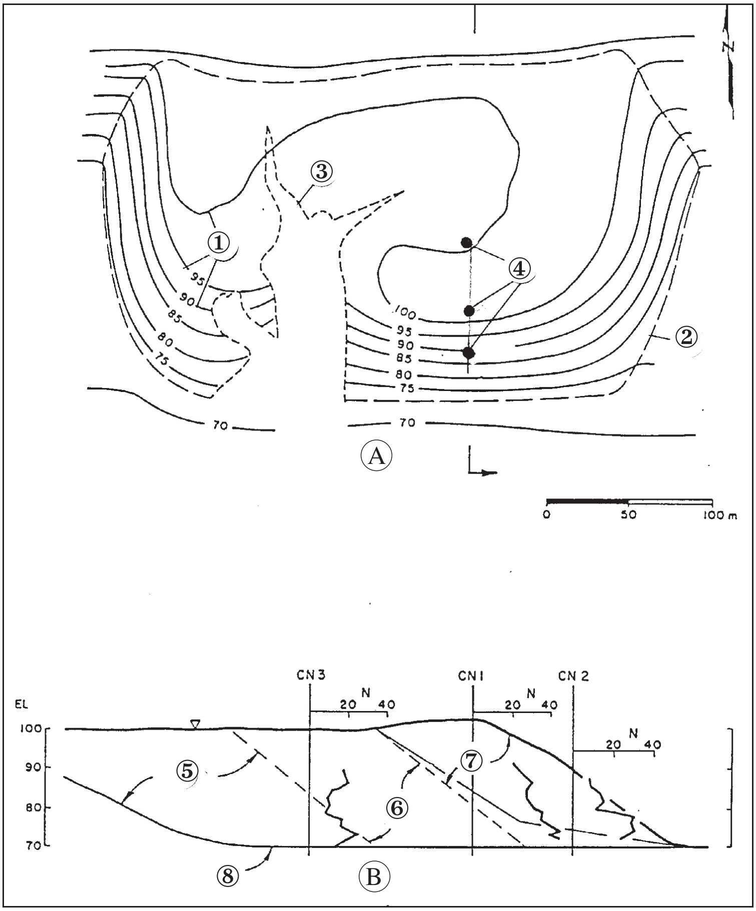


Fig. 5.1

Cerro Negro Tailings Dam – Plan and Section

*Barrage de stériles Cerro Negro – Vue en plan et coupe transversale*

1) Contour Lines
2) Limit of Tailings
3) Limit of Failure (Crevasse)
4) Borings (1987)
5) Slimes
6) Stratified Sand and Slimes
7) Sand
8) Natural Ground Surface

(After Castro and Troncoso, 1989)

*1) Lignes de niveaux*
*2) Limite de stériles*
*3) Limite de la zone de rupture (crevasse)*
*4) Forages (1987)*
*5) Boues*
*6) Boues et sable stratifiés*
*7) Sable*
*8) Surface du terrain naturel*

*(D'après Castro et Troncoso, 1989)*

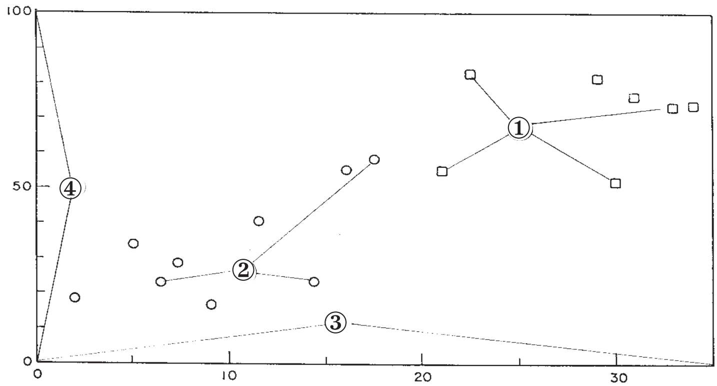


Fig. 5.2

Cerro Negro Tailings Dam – Percent Fines Content

*Barrage de stériles Cerro Negro – Pourcentage de fines*

1) Boring CN1
2) Boring CN2
3) Horizontal Distance from Outer Slope (m)
4) Percent Fines

*1) Forage CN1*
*2) Forage CN2*
*3) Distance horizontale depuis le talus extérieur (m)*
*4) Pourcentage de fines*

(After Castro and Troncoso, 1989)

*(D'après Castro et Troncoso, 1989)*

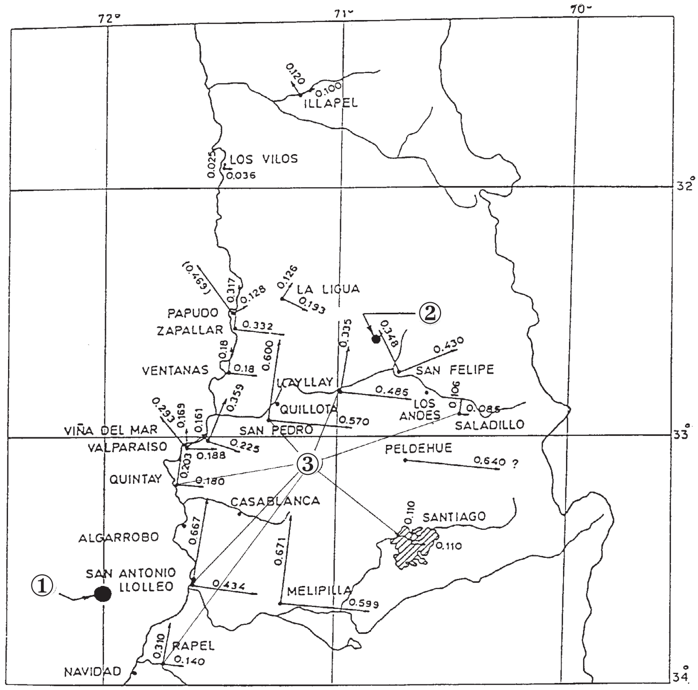


Fig. 5.3

Cerro Negro Tailings Dam – Epicenter and Recorded Accelerations

*Barrage de stériles Cerro Negro – Épicentre et accélérations enregistrées*

1) Epicenter of M=7.8, March 3, 1985 ERQ
2) Location of Cerro Negro Tailings Dam
3) Recorded Peak Horizontal Accelerations as a Fraction of Gravity

*1) Épicentre du séisme M=7.8, 3 mars 1985*
*2) Situation du barrage de stériles Cerro Negro*
*3) Accélérations horizontales maximales, en fraction de la gravité*

# 6. CHABOT DAM, CALIFORNIA, USA

On April 18, 1906 the San Francisco Bay Area was shaken by a magnitude 8+ earthquake that ruptured about 434 km of the San Andreas Fault. This earthquake, known as the "Great San Francisco Earthquake", resulted in substantial damage and destruction. Chabot Dam, a compacted clayey fill embankment about 40 m high, is located about 30 km from the San Andreas Fault and was strongly shaken by the earthquake. Peak ground acceleration at the dam site was estimated to be about 0.40 g. The dam suffered no significant or observable damage.

## CHABOT DAM

Construction of Chabot Dam, formerly known as Lower San Leandro Dam, was begun in 1874 and completed in 1892 (McLean, 1937). Chabot Dam is situated on San Leandro Creek near the eastern boundary of San Leandro and the southern boundary of Oakland, California (Fig. 6.1). The reservoir impounded by the dam has a storage capacity of approximately 14.8 $hm^3$ at the lower spillway crest elevation of 69.3 m. There is a higher spillway crest at Elevation 71 m. The crest of the dam is at Elevation 74 m (with a 0.6 m high concrete parapet wall extending to Elevation 74.7 m) and is about 122 m long. At its maximum section, the dam rises about 50 m above bedrock and 46 m above the original streambed. The embankment contains about 475 000 cubic meters of material. A cross-section through the embankment is shown on Fig. 6.2. This Figure apply to the original construction of Chabot Dam. In 1984, the embankment was raised when a new spillway was built.

Construction began in 1874 with stripping of the proposed dam "footprint" area to a depth of up to 1 m to remove vegetation, roots, and loose topsoil. A core trench ranging from 3 to 10 m in depth was excavated to bedrock. The main body of the dam (referred to as "wagon fill" in Fig. 6.2) was constructed during 1874 and 1875 as a rolled-fill structure, employing teams of horses and horse-drawn equipment for transporting and compacting fill material. Although mention was made that selected material was placed in the "core," subsequent exploration programs showed that the wagon fill can be characterized as a homogeneous mass of predominantly silty sandy clay with clayey sand and gravel.

In 1875, the dam crest was at Elevation 71 m. During the period from 1875 to 1888, the wagon fill was reinforced on the downstream slope by a sluiced fill buttress (referred to as "Hydraulic Fill" in Fig. 6.2), which was initially constructed to Elevation 55 m. Between 1890 to 1891, this hydraulic fill buttress was raised to Elevation 68 m. During the period of 1891 to 1892, the wagon fill was raised to the present day crest elevation of 74 m. Also, at that time, a berm was placed on the upstream face of the dam where an apparent slide had occurred during construction. During 1892 to 1895, local sandstone riprap was added to the upstream face from Elevation 61 m to 73 m. This riprap was grouted in 1912, at which time the concrete parapet wall was constructed along the upstream edge of the dam crest to Elevation 74.7 m.

## THE SAN FRANCISCO 1906 EARTHQUAKE

This probably was the greatest shock felt historically in California. It originated on the San Andreas Fault, north of San Francisco, and had a surface fault rupture of about 434 km. Maximum horizontal surface displacements of 6.5 m were observed near Tomales Bay. Ground fissuring along the San Andreas Fault was observed from Upper Mattole in Humbolt County to San Juan Bautista in San Benito County. Damage in the filled areas of the Cities of San Francisco, Santa Rosa, and San Jose was extensive. The earthquake had an estimated magnitude of about 8.3, caused at least 700 deaths and about 400 million (1906) dollars in damage.

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

During the 1906 San Francisco Earthquake, the estimated intensities in the vicinity of the dam site were of the order of VII to VIII on the Modified Mercalli Intensity (MMI) Scale. The dam was in normal operation at the time of the earthquake, with the reservoir level at Elevation 71 m (see Fig. 6.3). Older files of the Contra Costa Water Company show no records of any reported damage and, likewise, a review of the "Report of the State Investigation Commission" (Lawson, 1908) indicates that neither the dam nor the reservoir experienced any problem as a result of the earthquake (Woodward– Lundgren & Associates, 1974). It should be noted, however, that subsequent studies by Makdisi et al. (1978) suggested that Chabot Dam may have settled between 0.1 and 0.12 m as a result of that event.

Chabot Dam provided the opportunity for one of the first witness descriptions of reservoir seiching. In an inspection report retrieved from the files of the California Division of Safety of Dams (dated May 27, 1930), Mr. G.F. Engle included a testimony from the dam resident caretaker, Mr. Tierney, as follows: *"Mr. Tierney also interestingly relates that a few minutes after the earthquake of April 18, 1906, he arrived at the dam and was surprised to find the water about 1 m lower than it had been the night before. Thinking it had escaped through a rupture in the dam he commenced an investigation. In a few minutes, however, a wave traveled down the reservoir to slap up against the dam and return the water to its normal level of the night before. Apparently during the quake a tidal effect occurred in which the water was piled up in the upper reaches of the reservoir and soon returned in a prominent wave. Mr. Tierney says that no damage to the dam or appurtenant structures was evident as a result of the shock....at the time the reservoir was full."*

## INSTRUMENTATION AND STRONG MOTION RECORDS

Chabot Dam was not instrumented at the time of the 1906 San Francisco Earthquake. The intensity of ground shaking (based on observed damage) in the San Francisco area was estimated at between VII and XI on the Rossi-Forell scale (Lawson, 1908). Chabot Dam is located approximately 19 miles east of the San Andreas Fault, and peak bedrock acceleration at the dam site due to the 1906

San Francisco Earthquake was estimated at about 0.40 g (Woodward– Lundgren & Associates, 1974; Makdisi et al., 1978).

## CONCLUSIONS

Chabot Dam, a 40-m-high compacted embankment, was strongly shaken during the 1906 San Francisco Earthquake. The dam suffered no significant or observable damage. The embankment consisted of predominantly sandy clays with clayey sands and gravel and this experience confirms that, generally, well-compacted clayey dams can withstand severe ground motion shaking without experiencing significant damage. Estimated peak ground acceleration at the dam site were about 0.40 g with a duration of significant shaking of about 50 seconds. Detailed dynamic finite element analyses were performed (Makdisi et al, 1978) to estimate embankment deformation due to ground motions similar to those experienced during the 1906 earthquake. The estimated settlement was found to be in reasonable agreement with the observed performance of the embankment and the lack of observed damage.

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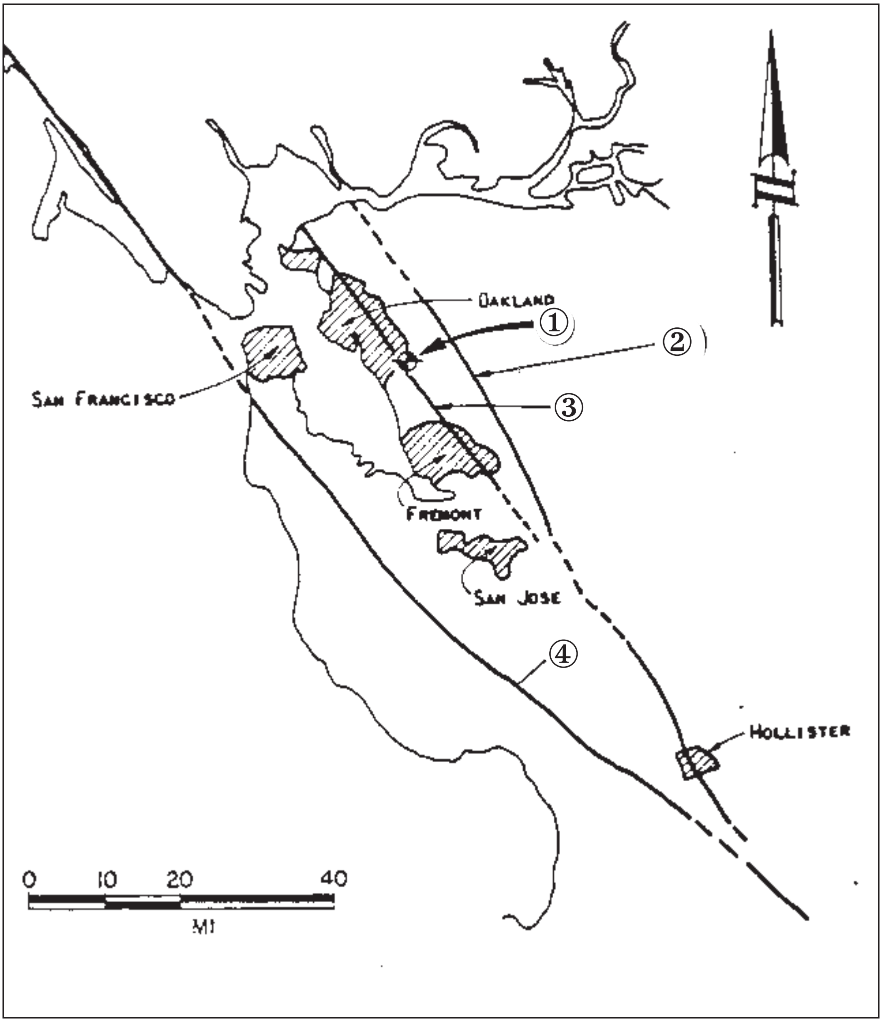


Fig. 6.1

Chabot Dam – Location of Chabot Dam relative to San Andreas Fault

*Barrage Chabot – Situation du barrage Chabot par rapport à la faille San Andreas*

| | |
|---|---|
| 1) Chabot Dam | *1) Barrage Chabot* |
| 2) Calaveras Fault | *2) Faille Calaveras* |
| 3) Hayward Fault | *3) Faille Hayward* |

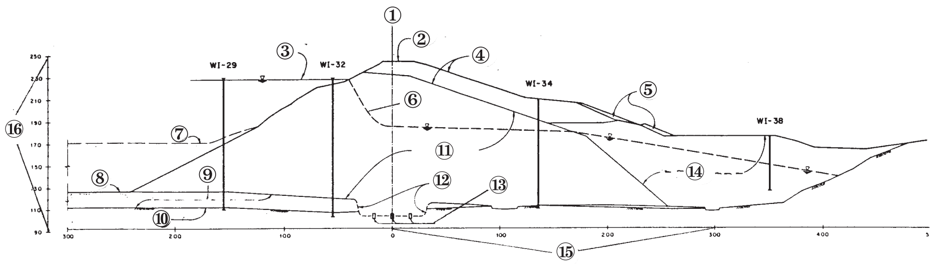


Fig. 6.2

Chabot Dam – Cross Section through Chabot Dam in 1906

*Barrage Chabot – Coupe transversale du barrage Chabot en 1906*

| | |
|---|---|
| 1) Dam Axis | *1) Axe du barrage* |
| 2) Crest Elevation 243 ft (74 m) | *2) Cote de la crête 243 pieds (74 m)* |
| 3) Spillway Crest Elevation 227.2 ft (69.2 m) | *3) Cote du déversoir 227.2 pieds (69.2 m)* |
| 4) Embankment Fill (1890 – 1892) | *4) Remblais (1890 – 1892)* |
| 5) Fillet Fit 1965 | *5) Correction de pente en 1965* |
| 6) Phreatic Line (1964 – 72) | *6) Ligne de saturation (1964 – 72)* |
| 7) Existing Reservoir Bottom | *7) Fond de retenue existant* |
| 8) Original Ground Surface | *8) Surface du terrain initial* |
| 9) Foundation Soils | *9) Sols de fondation* |
| 10) Approximate Bedrock Position | *10) Position approximative du fond rocheux* |
| 11) Wagon Fill (1874 – 1879) | *11) Remblais mis en place au moyen de chariots (1874 – 1879)* |
| 12) Approximate Limits of Cut-off Trench | *12) Limites approximatives du parafouille* |
| 13) Concrete Cut-off Walls | *13) Murs parafouilles en béton* |
| 14) Hydraulic Fill (1875 – 88) | *14) Remblai hydraulique (1875 – 88)* |
| 15) Distance from Centerline (ft) | *15) Distance à partir de l'axe (pieds)* |
| 16) Elevation (ft) | *16) Cote (pieds)* |

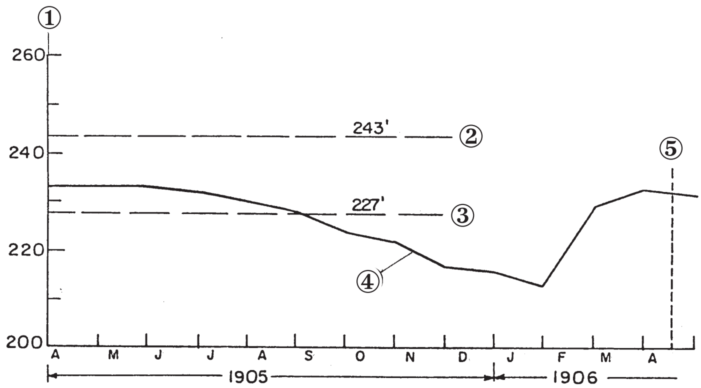


Fig. 6.3

Chabot Dam – Water Levels in the Reservoir of Chabot Dam before and during 1906 Earthquake

*Barrage Chabot – Niveau d'eau dans la retenue du Barrage Chabot, avant et pendant le séisme de 1906*

| | |
|---|---|
| 1) Elevation (ft) | *1) Cote (pieds)* |
| 2) Crest Elevation of Dam | *2) Cote de la crête du barrage* |
| 3) Spillway Sill Crest Elevation | *3) Cote du seuil du déversoir* |
| 4) Earthquake | *4) Le Séisme* |
| 5) Line of Water Level in the Storage | *5) Ligne du niveau d'eau dans la retenue* |

# 7. COGOTI DAM, CHILE

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

## COGOTI DAM

Cogoti Dam, a concrete face rockfill dam, is located in the Province of Coquimbo, Chile, about 75 km from the City of Ovalle. The dam site is situated within the foothills of the Andes Mountains, downstream from the confluence of the Pama and Cogoti rivers, and in a deep gorge naturally carved by the Cogoti River. Cogoti Dam, shown in plan and cross section on Fig. 7.1, has a maximum height of 85 m, a crest length of 160 m and a total rockfill volume of about 700 000 cubic meters. The upstream slope averages 1.4:1 (horizontal to vertical) and the downstream slope is about 1.5:1 (horizontal to vertical). The dam is primarily used for irrigation purposes and impounds a reservoir of 148 $hm^3$ capacity.

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

The flexible, impervious, segmented reinforced concrete face was placed on a 2 meters-thick bedding zone of hand-placed, small-size, rock. It was designed as individually formed slabs, of 10 x 10 m average size, with a thickness tapering from 0.8 m at the upstream toe to 0.2 meters at the crest of the dam. Horizontal and vertical joints with 0.6 m wide copper waterstops and rivets, were provided. The spacing and bar sizes of the steel reinforcement vary as a function of elevation along the dam face, starting with a double curtain of one-inch bars at 0.3 m spacing near the toe and ending with a single curtain of 20 mm bars at 0.2 m spacing at the crest.

The spillway is an ungated channel with a reinforced concrete side– channel having broad crested weir control, and was excavated in the left abutment rock. It has a design capacity of 4990 $m^3/s$.

## THE APRIL 6, 1943 EARTHQUAKE

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

The Illapel earthquake was centered about 95 km from Cogoti Dam. An intensity IX on the Rossi-Forell scale was reported at the dam site. The reservoir is believed to have been at its normal operating level at the time of occurrence of the earthquake. The principal observed effect of the 1943 earthquake on Cogoti Dam was to produce an instantaneous settlement of up to 0.4 m. Settlement occurred throughout the length of the crest and the extreme upper part of the concrete face slab was exposed from the downstream side, as quoted in an internal report by ENDESA S.A., Santiago, Chile (1972). It is of interest to note that the maximum earthquake-induced settlement was about equal to that observed in the 4.5 years since the end of construction. The point where this settlement was measured was near the center of the crest, where the dam height is about 63 m. This is not the highest dam section, which is located close to the right abutment. The settlement at the maximum dam height was less, presumably because of a restraining effect due to the nearby presence of the very steep abutment. Minor rock slides also occurred along the downstream slope of the dam.

Leakage had been observed at Cogoti Dam since the reservoir's first filling in 1939. Intermittent records have been kept over the years, which indicate leakage to be directly related to the elevation of the reservoir and probably coming through the abutment or foundation, rather than the dam itself. No significant increase in dam leakage was observed as a result of the 1943 earthquake. No face cracks were caused by the earthquake. Yearly settlement and leakage data at Cogoti Dam are presented on Fig. 7.2 and 7.3.

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

## CONCLUSION

Cogoti Dam, a 85m-high concrete face rockfill dam, was constructed in 1938 and subjected in 1943 to ground motion of probably significant amplitudes and duration. Although significant settlement occurred, the dam performed extremely well and no seismic damage was observed to the concrete face. Although a rockfill construction method now obsolete had been used (which explains the observed settlement), Cogoti Dam's performance substantiates the generally accepted belief that concrete face rockfill dams have an excellent inherent capacity to withstand substantial earthquake motion without experiencing significant damage. Although Cogoti Dam's leakage has increased over the years, this has been related to aging and spalling of the concrete and joint squeezing, not to the 1943 Illapel nor to any of the subsequent earthquakes to which the dam was exposed.

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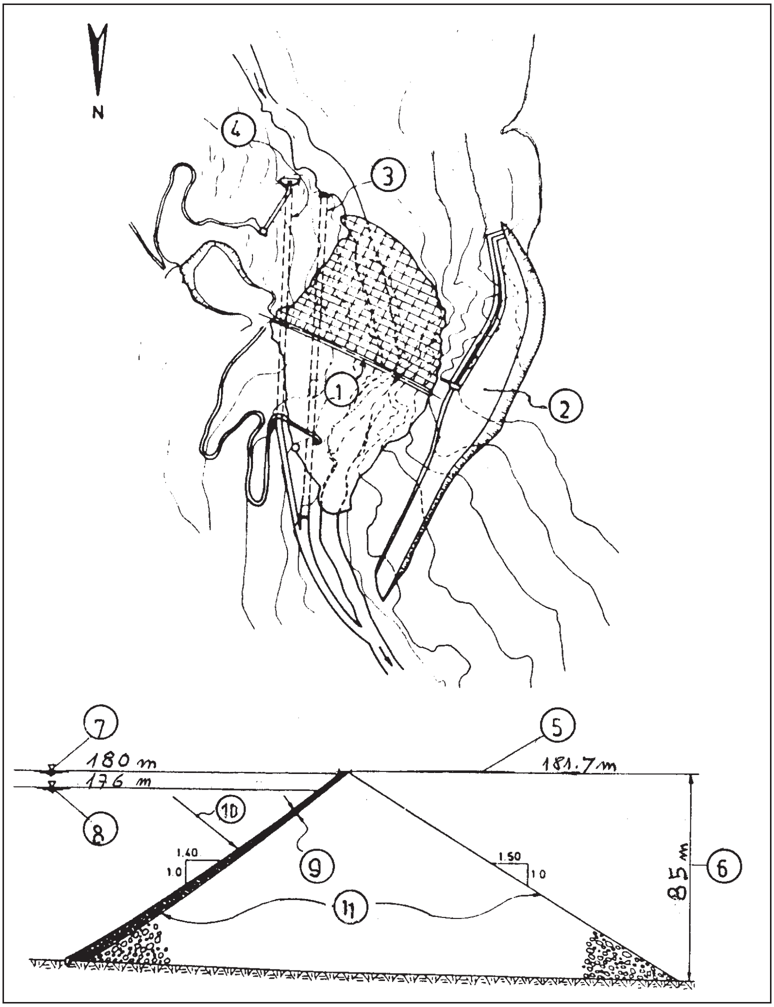


Fig. 7.1

Layout and Cross Section of Cogoti Dam

*Vue en plan et coupe transversale du barrage Cogoti*

| | |
|---|---|
| 1) Dam Axis | *1) Axe du barrage* |
| 2) Spillway | *2) Évacuateur de crue* |
| 3) Diversion Tunnel and Bottom Outlet | *3) Galerie de dérivation et vidange de fond* |
| 4) Low Level Outlet | *4) Vidange à cote basse* |
| 5) Dam Crest Elevation | *5) Cote de la crête du barrage* |
| 6) Maximum Dam Height | *6) Hauteur maximale du barrage* |
| 7) Maximum Water Level | *7) Niveau maximal de retenue* |
| 8) Normal Pool Level | *8) Niveau normal de retenue* |
| 9) Concrete Facing | *9) Masque en béton* |
| 10) Radius of Vertical Curvature of Concrete Facing R=1000 m | *10) Rayon de courbure du masque en béton dans un plan vertical (R = 1000 m)* |

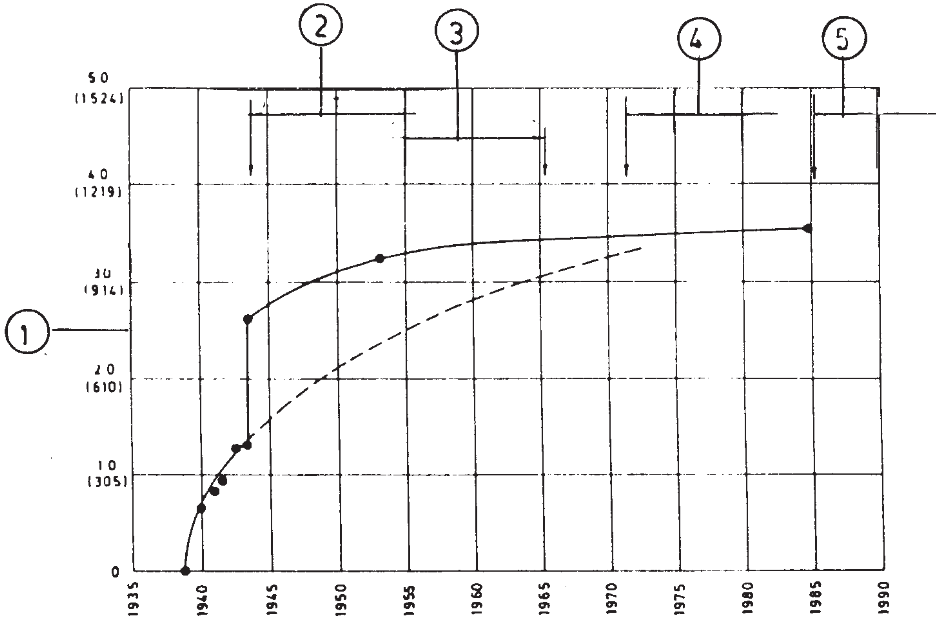


Fig. 7.2
Settlement Records of Cogoti Dam
*Tassements observés sur le barrage Cogoti*

1) Settlements in ft and (mm)
2) 1943 Earthquake
3) 1965 Earthquake
4) 1971 Earthquake
5) 1985 Earthquake

*1) Tassements en pieds et en (mm)*
*2) Séisme de 1943*
*3) Séisme de 1965*
*4) Séisme de 1971*
*5) Séisme de 1985*

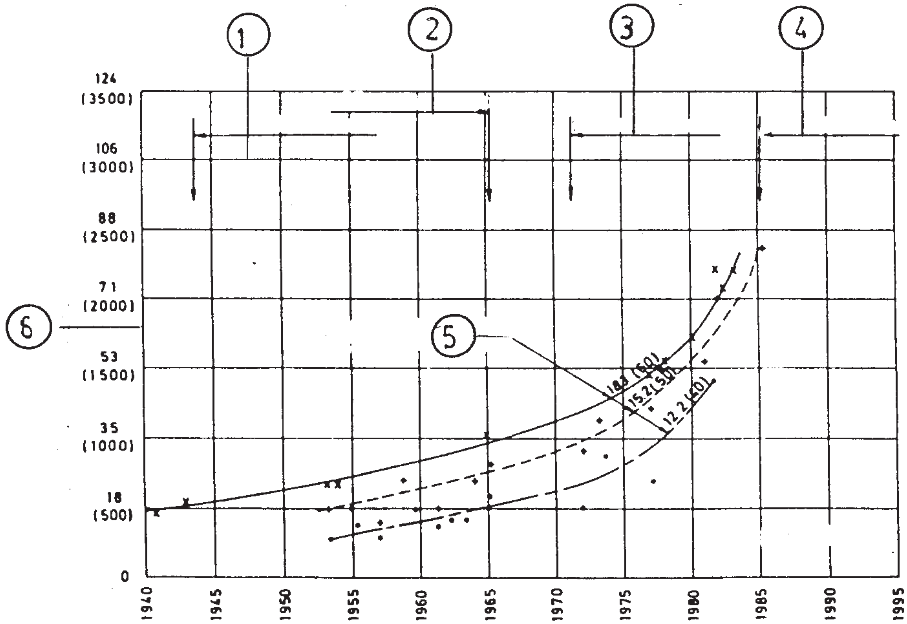


Fig. 7.3
Leakage Records of Cogoti Dam
*Fuites enregistrées au barrage Cogoti*

1) 1943 Earthquake
2) 1965 Earthquake
3) 1971 Earthquake
4) 1985 Earthquake
5) Reservoir Head in ft and m
6) Leakage in $ft^3$/s and lit/s

*1) Séisme de 1943*
*2) Séisme de 1965*
*3) Séisme de 1971*
*4) Séisme de 1985*
*5) Charge d'eau de la retenue en pieds et en m*
*6) Fuites en pieds*$^3$/s et en l/s

# 8. LA VILLITA DAM, MEXICO

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

## LA VILLITA DAM

Jose Maria Morelos (La Villita) Dam, an earth-and-rockfill embankment, was constructed from 1965 to 1967, about 8 miles (13 km) inland from the mouth of the Balsas River. The principal component of a 304 MW multi-purpose hydroelectric, irrigation and flood-control development, the embankment stands 60 m high and has a crest length of about 430 m. It was designed with a symmetrical cross-section with a central impervious clay core, well-graded filter and transition zones and compacted rockfill shells, see Fig. 8.1. The dam layout is shown on Fig. 8.2. Upstream and downstream faces slope at 2.5:1 (horizontal to vertical). The dam crest is slightly concave toward downstream.

La Villita Dam is founded on up to 76 m thick, well-graded alluvial deposits from the Balsas River. The alluvium is composed of boulders, gravels, sands and silts which taper toward the abutments. The abutments consist of stratified layers of andesite and andesitic breccias. A two-foot wide central concrete cutoff wall extends to bedrock across the entire dam foundation. The alluvium below the core was grouted on both sides of the cutoff wall to a depth of 26 m.

## THE SEPTEMBER 19, 1985 EARTHQUAKE

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

September 19 and 21 produced the most extensive strong motion data sets yet obtained in Mexico.

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

Two main systems of longitudinal cracks developed at the crest of La Villita Dam, parallel to its axis, some 16 feet away from the crest edges. These cracks formed along the buried shoulders of the central core and most likely resulted from differential settlement between the core and adjacent filter zones. A 75 m long crack, 0.6 to 5 cm wide at the surface, formed along the upstream side of the dam crest. Vertical offsets of 5 to 10 cm occurred between the lips of the crack, the upstream side settling the most. On the downstream side, another major crack system appeared, about 300 m long, 1 to 1.5 cm wide, with vertical offsets ranging from 1 to 2 cm, the side toward the face of the dam being downthrown. Several other longitudinal cracks, up to two inches wide, but less extensive than the two principal crack systems, were also found. The location of the cracks is shown on Fig. 8.3.

The most significant cracks were investigated by trenching immediately after the earthquake and were confirmed to be only about 5-feet deep. Along the trench walls, several cracks were open between 5 and 10 cm and extended in depth for about 0.6 m through the aggregate base layer of the paved road at the crest of the dam. They faded to hairline when reaching the sands of the filter zone. The cracks were concluded not to reach the impervious core zone and were found to disappear below two-feet depth, except near the right abutment, where one of the cracks was delineated as a closed fissure through a clay lens embedded at about three-foot depth within the filter sands.

La Villita Dam settled and spread laterally during the 1985 earthquake. Post-earthquake surveys showed that, in its central part, the dam settled between 20 and 32 cm on the upstream side and between 9 and 22 cm on the downstream side. Based on inclinometer readings, settlements decreased in magnitude to near zero toward the abutments and seemed to be evenly distributed within the dam cross-section, rather than associated with distinct surfaces. The downstream half of the dam moved horizontally up to 10 cm in the downstream direction and the upstream half up to 16.5 cm in the upstream direction. Downstream horizontal displacements were somewhat irregular, although generally more symmetrical with respect to the center of the dam than the upstream displacements. Settlements were particularly noticeable at several piezometer locations, where the piezometer tubes which extend down to deep within the embankment remained in place, while their protective concrete boxes settled along with the face of the dam.

The powerhouse and other appurtenant facilities were essentially unaffected by the earthquake. Mechanical and electrical equipment remained fully operational and no damage occurred at the spillway, spillway gates, power plant, substation and switchyard. Two 130– ton transformers (13.8 kV/230 kV), adjacent to the power plant building, showed evidence of about 1 cm of horizontal sliding on their pedestals, but were otherwise unaffected.

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

On September 19, 1985, the accelerometer at the center of the crest of the dam recorded a peak horizontal acceleration of 0.45 g and, on the following day, a peak acceleration of 0.16 g was measured during the strongest aftershock. Peak horizontal bedrock acceleration was recorded at 0.13 g for the main event and 0.04 g for the aforementioned aftershock. Bedrock records for the main event are shown on Fig. 8.4. As can be seen on Fig. 8.4 and as confirmed from post-earthquake seismological research studies, the September 19 earthquake resulted from two distinct bursts of energy lasting about 16 seconds each and separated by about 25 seconds. This dual rupture mechanism was more conspicuous on records from other strong motion stations closer to the epicenter than from the La Villita instruments.

Survey monuments, inclinometers and extensometers were essential to provide detailed information on the earthquake-induced deformations of La Villita Dam. Of particular interest was the fact that the dam had previously been shaken by several significant earthquakes in the twelve years that preceded the 1985 event. Fig. 8.5 shows a record of crest settlements from 1968 to 1985. Earthquake-induced settlements have been found to exceed static post-construction settlements and appear to increase in magnitude from one earthquake to the other, perhaps indicating a change in stiffness of the dam materials or a slow, cumulative, deterioration of part of the embankment. Inclinometer records confirmed that permanent deformations decreased in magnitude from crest to bottom of the embankment and did not involve the foundation materials.

## CONCLUSION

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

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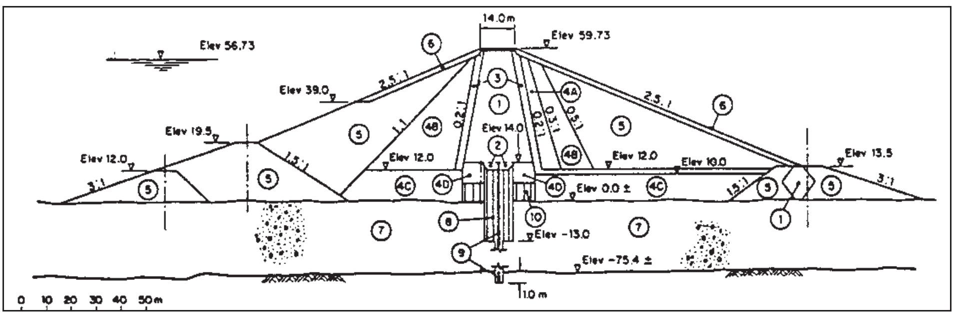


Fig. 8.1

Cross Section of La Villita Dam

*Coupe transversale du barrage La Villita*

| | |
|---|---|
| 1) Compacted Impervious Material | *1) Matériau compacté étanche* |
| 2) Highly Plastic Clay | *2) Argile de haute plasticité* |
| 3) Sand Filters | *3) Filtres de sable* |
| 4A) Well Graded Gravel and Sand | *4A) Sable et gravier de bonne granulométrie* |
| 4B) Gravel and Sand | *4B) Sable et gravier* |
| 4C) Dumped Gravel and Sand | *4C) Sable et gravier déversés* |
| 4D) Compacted Gravel, Sand and Muck | *4D) Sable, gravier et débris compactés* |
| 5) Rockfill | *5) Enrochement* |
| 6) Selected Rockfill | *6) Enrochement sélectionné* |
| 7) Alluvium (Gravel and Sand) | *7) Alluvions (sable et gravier)* |
| 8) Grout curtain | *8) Rideau d`injection* |
| 9) ICOS Type Concrete Cut-off Wall | *9) Parafouille en béton du type ICOS* |
| 10) Consolidation Grouting | *10) Injections de consolidation* |

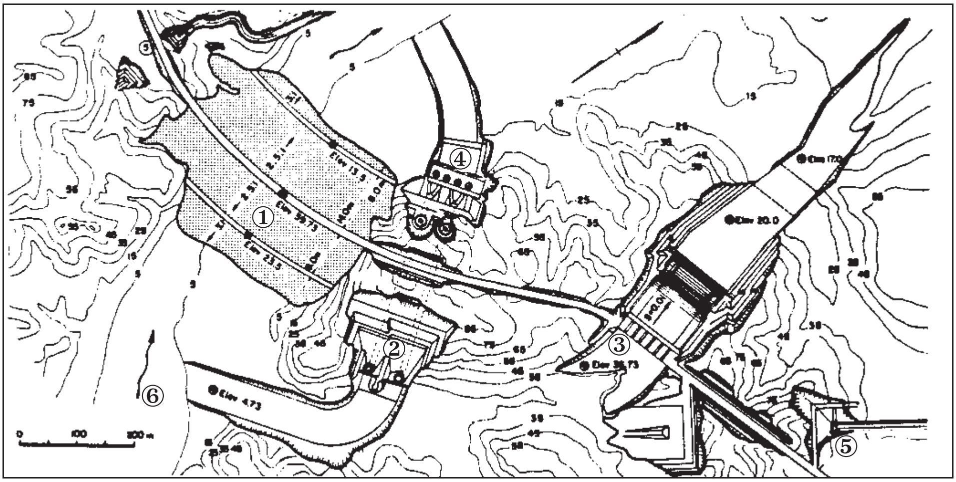


Fig. 8.2

La Villita Dam Layout

*Vue en plan du barrage La Villita*

1) Dam
2) Intake Works
3) Spillway
4) Powerhouse
5) Irrigation Outlet
6) Balsas River

*1) Barrage*
*2) Prises d'eau*
*3) Évacuateur de crue*
*4) Usine*
*5) Ouvrage de restitution des débits d'irrigation*
*6) Rivière Balsas*

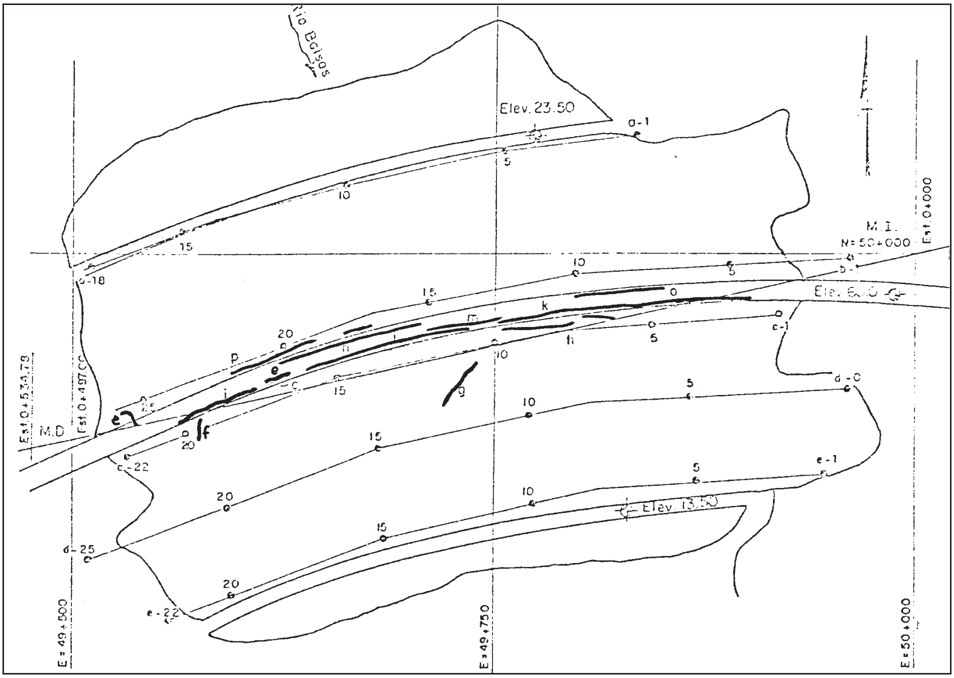

Fig. 8.3
La Villita Dam – Crack Location at La Villita Dam
*Barrage La Villita – Emplacement des fissures sur le barrage La Villita*

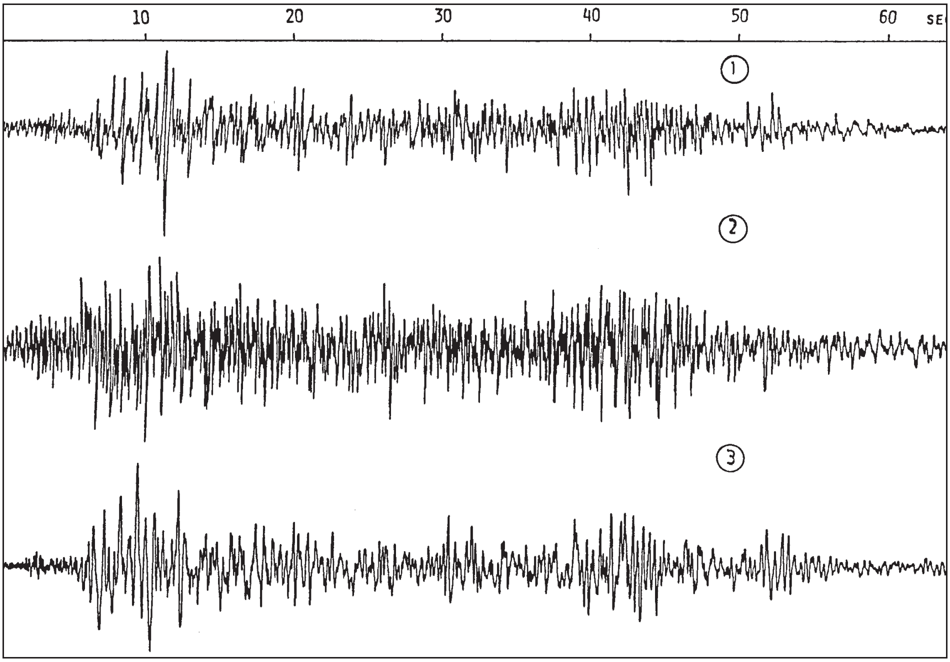


Fig. 8.4

Bedrock Acceleration Records at La Villita Dam

*Accélérogrammes enregistrés sur la fondation rocheuse du barrage La Villita*

1) East-West, 122 gal
2) Vertical, 58 gal
3) North –South, 125 gal

1*) Est–Ouest, 122 gal*
*2) Vertical, 58 gal*
*3) Nord-Sud, 125 gal*

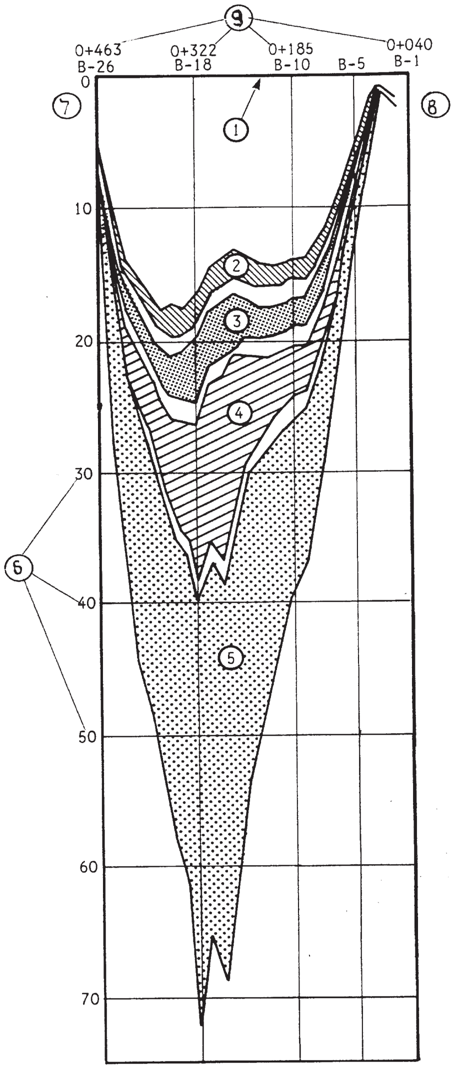


Fig. 8.5

La Villita Dam – Historic Crest Settlements (1968 to 1985)

*Barrage La Villita – Tassements de la crête de 1968 à 1985*

| | |
|---|---|
| 1) Base Line – September 28, 1968 | *1) Ligne de référence – 28 septembre 1968* |
| 2) Effect of 1975 Earthquake | *2) Effet du séisme de 1975* |
| 3) Effect of 1979 Earthquake | *3) Effet du séisme de 1979* |
| 4) Effect of 1981 Earthquake | *4) Effet du séisme de 1981* |
| 5) Effect of Septembre 1985 Earthquake | *5) Effet du séisme de septembre 1985* |
| 6) Settlement in cm | *6) Tassement en cm* |
| 7) Right Bank | *7) Rive droite* |
| 8) Left Bank | *8) Rive gauche* |
| 9) Benchmark Stations along Crest | *9) Repères le long de la crête* |

# 9. LOS ANGELES DAM, CALIFORNIA, USA

The January 17, 1994 Northridge Earthquake (Mw 6.7) affected the dams of the Van Norman Complex (VNC), owned and operated by the Los Angeles Department of Water and Power (LADWP). Epicentral distance was about 10 km. The VNC includes the decommissioned Upper and Lower San Fernando dams and their replacement, Los Angeles Dam (LAD), built in 1979. LAD, a 47 m high modern compacted embankment, experienced up to 9 cm of crest settlement and surficial cracking of the asphalt concrete facing along its upstream slope. Since 1971, the VNC has been extensively instrumented. These instruments have provided an exceptional crop of local strong motion records at and near the dam for the Northridge event, with peak ground accelerations (PGA) approaching the acceleration of gravity (1 g). Especially, strong motions were recorded at the dam crest of Los Angeles Dam, at the west abutment and in the lower part of the foundation. Peak ground accelerations were 0.32 g at foundation level and 0.33 g at the right abutment. Crest acceleration was 0.60 g.

## LOS ANGELES DAM

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

Los Angeles Dam (LAD) was completed in 1979 and impounds Los Angeles Reservoir, of 12 $hm^3$ capacity, see Fig. 9.1. It is a modern, well-instrumented embankment of maximum height 47 m, founded on bedrock of the Sunshine Ranch Member and the upper member of the Saugus Formation. The foundation rock consists of claystone, siltstone, sandstone and interbedded pebble-cobble conglomerates, with average shear wave velocities measured at about 975 m/s in the foundations of the nearby Upper and Lower San Fernando dams.

LAD includes upstream and downstream shells of compacted silty sand, a central vertical chimney drain, a downstream near– horizontal drainage blanket, and a silty clay core upstream from the chimney drain. A small zone of blended cobbles, gravel and sand is also located immediately downstream from the lower part of the chimney drain. The dam crest is 10 m wide and the slopes were constructed at about 3:1 (downstream, h to v) and 3.5:1 (upstream, h to v). The embankment materials were compacted to 93 percent relative compaction (33 750 ft.lbs/$ft^3$). Compaction was carefully controlled during construction. The interior slopes of Los Angeles Reservoir, including the upstream face of the dam, are lined with an asphalt concrete membrane. The reservoir bottom is unlined. Fig. 9.1 and 9.2 show the reservoir layout and the maximum cross section of LAD.

## THE JANUARY 17, 1994 EARTHQUAKE

At 4.31 A.M. PST on January 17, 1994, the Northridge Earthquake (moment magnitude Mw 6.7) affected the greater Los Angeles area. The earthquake was centered along a blind thrust segment of the Oakridge Fault, about 10 km southwest of the VNC, at a focal depth of about 19 km. The earthquake produced some of the largest peak ground accelerations ever recorded, many in the range of 0.50 g to 1.0 g, and Modified Mercalli Intensities of up to IX were assigned at several locations. Some of the recorded response spectra were twice as large as the building code spectrum over a significant part of the period range.

Casualties included 57 deaths and at least 5000 persons injured. Structural damage included numerous cases of partial or total failure, including steel and concrete buildings, apartment buildings, parking structures, highway overpasses and lifelines. This event ended up being the costliest natural disaster in the United States history, with over 20 billions dollars in estimated property damage. As a result of this earthquake, the attention of the public and engineering community focused on extensive damage caused to welded beam-to-column connections in steel moment–resisting frame (SMRF) buildings. Of about 1500 SMRF in Los Angeles, at least 137 sustained connection failures during the Northridge Earthquake.

Thirteen dams in the area were found to have experienced cracking or some movement (Sanchez, 1994). Most of the cracking and movement were concluded to be minor. Most significant observations were large longitudinal open cracks at the decommissioned Lower and Upper San Fernando dams, and the 5 cm opening of a joint between the left abutment and the concrete thrust block of Pacoima Dam. This joint opening was accompanied by about 1 cm of movement of the thrust block downstream relative to the dam crest.

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

Davis and Sakado (1994) and Davis and Bardet (1996) have described in considerable detail the performance of LAD during the Northridge Earthquake. Extensive surficial cracking of the asphalt concrete crest roadway and 7.5 cm thick asphalt lining that covers the upstream slope of the dam was observed, see Fig. 9.3. Most cracks were of the shear type and, near the left abutment, were associated with waving, bulgy surfaces caused by compression of some of the embankment materials. Trenching of the largest cracks indicated that they did not extend deep within the body of the dam. A few cracks existed prior to the seismically-induced cracks, but Davis and Bardet concluded that most cracks were probably caused by transient stresses and deformations during the earthquake.

Immediately after the earthquake, ten survey profiles were taken along the crest axis and downstream slope. The embankment experienced a maximum crest settlement of 9 cm and about 2.5 cm of horizontal nonrecoverable crest displacement, see Fig. 9.4. The downstream slope settled up to 2 cm, and moved laterally slightly in excess of 5 cm downstream.

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

LAD is extensively instrumented with survey monuments, strong motion accelerographs, and piezometers. Strong motion instruments recorded the dam response during the Northridge Earthquake. These strong motion records have been corrected by Professor Trifunac of the University of Southern California (USC). Stations 2 (west abutment) and 3 (foundation) are on bedrock (see Fig. 9.1). Station 4 (crest) is at the maximum cross-section. Peak accelerations of the corrected dam records were 0.27 g (transverse), 0.32 g (longitudinal) and 0.12 g (up) at foundation level; 0.60 g (transverse), 0.42 g (longitudinal) and 0.38 g (up) at the crest; and 0.42 g (transverse), 0.33 g (longitudinal) and 0.32 g (up) at the right abutment. Peak ground accelerations (PGA) recorded at the dam are lower than elsewhere in the VNC. PGA's of 0.85 g and 1.00 g were recorded on alluvium 1340 m south and 2564 m north of LAD, respectively (Bardet and Davis, 1996). The records of the Northridge Earthquake obtained at the foundation and abutment of LAD are shown on Fig. 9.5.

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

## CONCLUSION

The example of LAD confirms that dams built of well-compacted cohesive clays and dense sands can perform satisfactorily during very severe earthquake shaking. Of particular interest is the fact that strong motion records obtained at LAD were generally significantly less severe than those recorded elsewhere in the VNC at distance of less than 3000 m away. This is perhaps the only dam where the base motion below the maximum section of the embankment was recorded. Comparisons of such motions with those obtained at the abutment indicate less severe shaking at depth than at the abutment, and this especially for the vertical component of motion. Lastly, observed performance of LAD during the Northridge Earthquake was found to reasonably match the settlement and acceleration histories subsequently calculated through nonlinear analysis procedures, thereby providing another verification of the validity of modern dam evaluation procedures.

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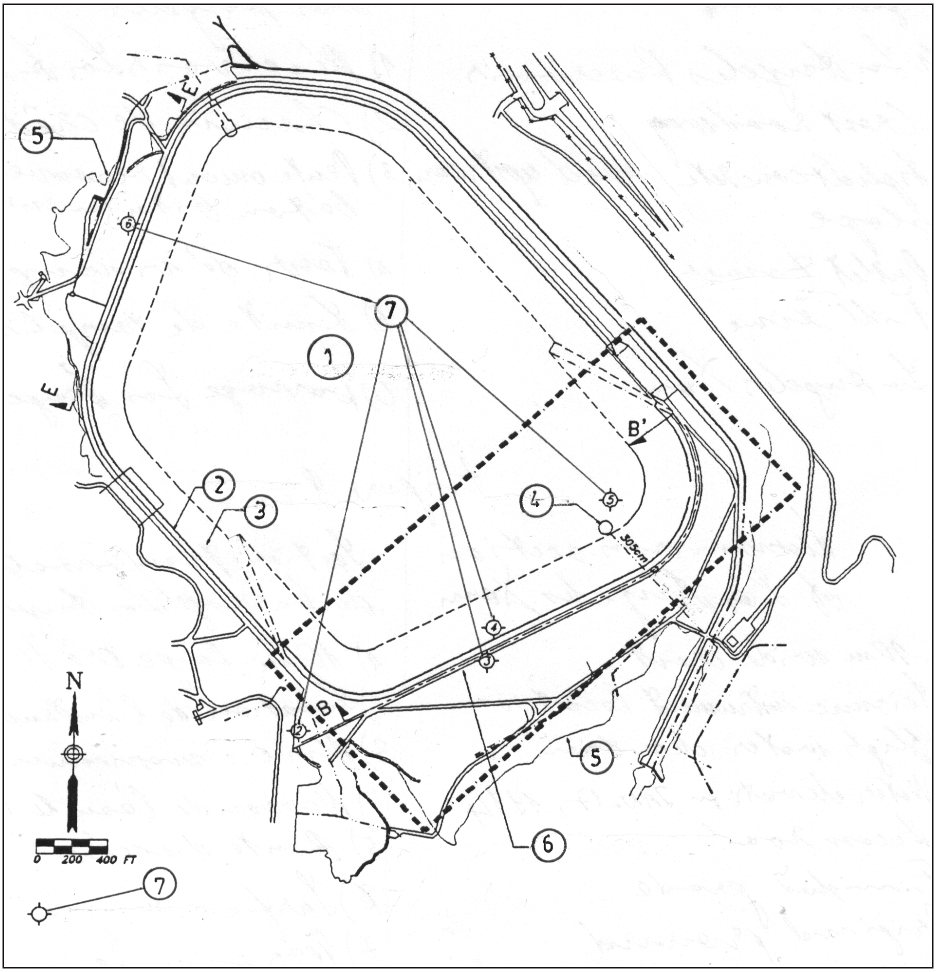


Fig. 9.1

Reservoir Layout of Los Angeles Dam

*Vue en plan du réservoir Los Angeles*

1) Los Angeles Reservoir
2) Crest Roadway
3) Asphalt Concrete Paved Upstream Slope
4) Outlet Tower
5) Fill Line
6) Los Angeles Dam
7) Seismic Instruments

*1) Réservoir Los Angeles*
*2) Chemin de crête*
*3) Talus amont revêtu de béton bitumineux*
*4) Tour de vidange*
*5) Limite du remblai*
*6) Barrage Los Angeles*
*7) Appareils de mesures sismiques*

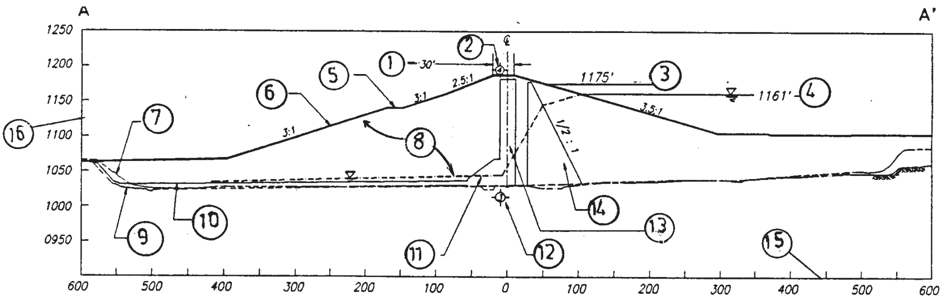


Fig. 9.2

Los Angeles Dam – Maximum Cross Section

*Barrage Los Angeles – Coupe transversale maximale*

| | |
|---|---|
| 1) 10 m Wide Crest | *1) Largeur en crête : 10 m* |
| 2) Seismic Instrument Location | *2) Emplacement de l'appareil de mesure sismique* |
| 3) High Water Elevation | *3) Niveau de crue* |
| 4) Water Elevation on January 17, 1994 | *4) Niveau de retenue, le 17 janvier 1994* |
| 5) Access Road | *5) Route d`accès* |
| 6) Finished Grade | *6) Surface réglée* |
| 7) Original Ground | *7) Terrain naturel* |
| 8) Compacted Embankment | *8) Remblai compacté* |
| 9) Bedrock Stripping Line | *9) Ligne de décapage du fond rocheux* |
| 10) Drainage Blanket | *10) Tapis drainant* |
| 11) Cobble Zone | *11) Zone de galets* |
| 12) Seismic Instrument | *12) Appareil de mesure sismique* |
| 13) Chimney Drain | *13) Drain cheminée* |
| 14) Clay Zone | *14) Zone d`argile* |
| 15) Distance (ft) | *15) Distance (pieds)* |
| 16) Elevation (ft) | *16) Cote (pieds)* |

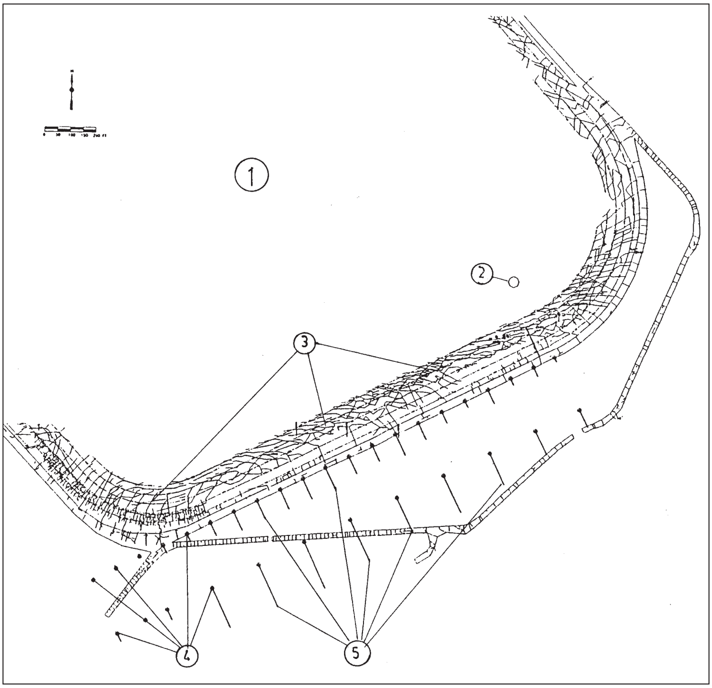


Fig. 9.3

Cracks and Transverse Movements Induced by Northridge Earthquake

*Fissures et mouvements transversaux induits par le séisme Northridge*

1) Los Angeles Reservoir
2) Outlet Structure
3) Earthquake Induced Cracks in Asphaltic Lining
4) Measuring Points
5) Vectors of Transversal Movements

*1) Réservoir Los Angeles*
*2) Tour de vidange*
*3) Fissures causées par le séisme dans le masque en béton bitumineux*
*4) Points de mesure*
*5) Vecteurs des mouvements transversaux*

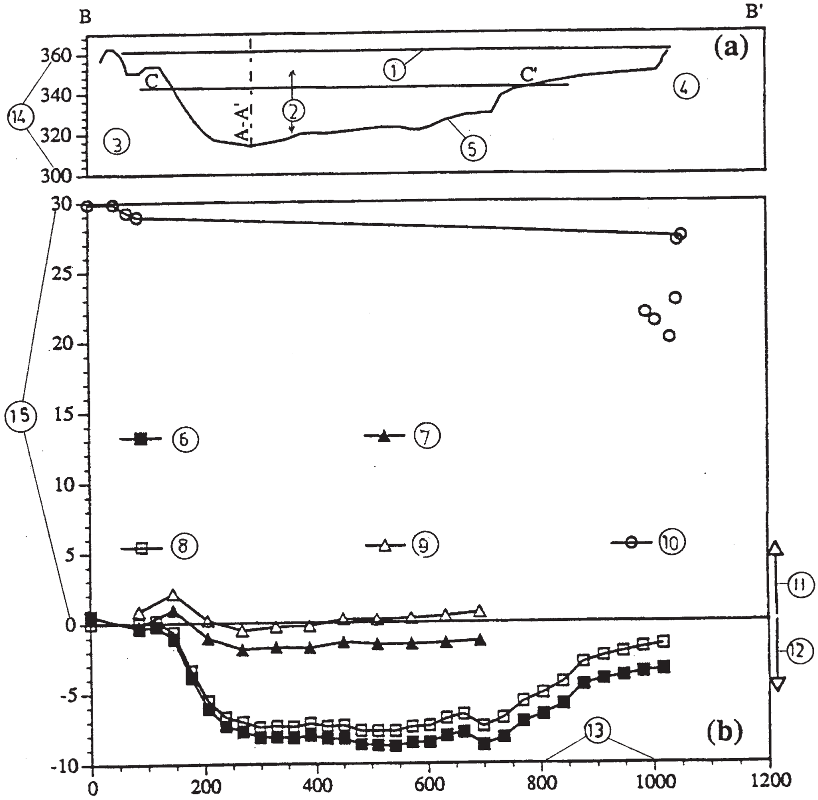


Fig. 9.4

Los Angeles Dam – Profile of Crest Settlements

*Barrage Los Angeles – Profil des tassements de crête*

1) Crest line
2) Downstream slope
3) West Abutment
4) East Abutment
5) Bedrock Surface
6) Uncorrected BB' Settlements
7) Uncorrected CC' Settlements
8) Corrected BB' Settlements
9) Corrected CC' Settlements
10) Tectonic Settlements
11) Uplift
12) Settlement
13) Distance (m)
14) Bedrock depth (m)
15) Settlement (cm)

*1) Ligne de crête*
*2) Talus aval*
*3) Appui ouest*
*4) Appui est*
*5) Surface du fond rocheux*
*6) Tassements non-corrigés de la ligne BB'*
*7) Tassements non-corrigés de la ligne CC'*
*8) Tassements corrigés de la ligne BB'*
*9) Tassements corrigés de la ligne CC'*
*10) Tassements tectoniques*
*11) Soulèvement*
*12) Tassement*
*13) Distance (m)*
*14) Profondeur du fond rocheux (m)*
*15 Tassement (cm)*

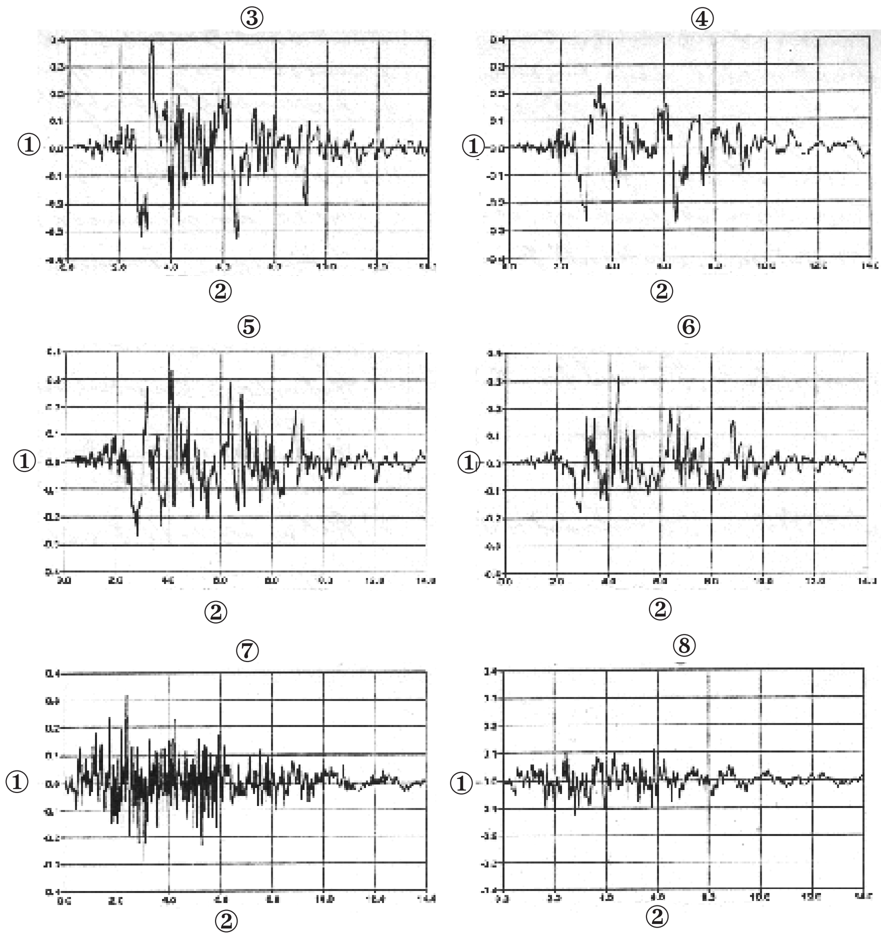

Fig. 9.5

Los Angeles Dam – Recorded Foundation and Abutment Motions

*Barrage Los Angeles – Mouvements enregistrés en fondation et aux appuis*

| | |
|---|---|
| 1) Acceleration (g) | *1) Accélération (g)* |
| 2) Duration of Shaking (seconds) | *2) Durée des secousses (secondes)* |
| 3) LAD Station 2. Abutment Transverse | *3) Station LAD 2. Transversale à l'appui* |
| 4) LAD Station 3. Foundation Transverse | *4) Station LAD 3. Transversale à la fondation* |
| 5) LAD Station 2. Abutment Longitudinal | *5) Station LAD 2. Longitudinale à l'appui* |
| 6) LAD Station 3. Foundation Longitudinal | *6) Station LAD 3. Longitudinale à la fondation* |
| 7) LAD Station 2. Abutment Vertical | *7) Station LAD 2. Verticale à l'appui* |
| 8) LAD Station 3. Foundation Vertical | *8) Station LAD 3. Verticale à la fondation* |

# 10. LOS LEONES DAM, CHILE

Los Leones is one of the largest conventional dams designed to impound mine tailings. It is a compacted earth and rockfill dam, built to Stage III at the time of preparation of this write-up. It impounds copper mine tailings and a shallow "clear water" reservoir. In Stage IV, the dam will be raised to about 200 m above streambed.

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

## LOS LEONES DAM

Los Leones Dam is owned and operated by Codelco Chile, Division Andina. It is located in the Andes Mountains of Central Chile (see Fig. 10.1) and within an area of strong seismic activity. The dam impounds copper mine tailings and a shallow clear fluids pond. Before construction, the Los Leones River was diverted upstream from the reservoir through a 3.5 km long tunnel, designed to pass the 1000-year flood. The tailings flow from the concentrator, located at El. 2800 m , through a 15 km long pressure conduit discharging at the upstream end of the reservoir, and fill the glacial valley of the Los Leones River. Excess clear pond fluids and watershed runoff spill through a multi-port inclined intake structure and an outlet tunnel.

The dam was built across a relatively narrow gorge, where the river has eroded thick banks of randomly deposited streambed alluvium, glacial till, and landslide materials. The embankment was designed to be built in four stages as a rolled earthfill, with a planned final crest at El. 2138 m. The Stage I dam was started in October 1978. Final completion to the top of Stage IV (Stage IV-B) is expected to occur in December 1997, with a final dam height of 160 m at centerline and 198 m at the downstream toe. At completion of Stage IV, the reservoir will have a capacity of 148 $hm^3$ and will impound 222 million metric tons of tailings, representing 20 years of mining activities. Los Leones Dam will reach a final volume of 12 million cubic meters.

The dam is a conventional, compacted, earthfill embankment. It has an upstream core built of morainic deposits, an inclined chimney drain and filter and transition zones on the downstream side of the core, and a downstream drainage blanket. The downstream shell is built of compacted earthfill, borrowed from alluvial fans in neighboring canyons. Near the toe, a secondary drainage system collects seepage from the underlying aquifer and paleostreams in the dam foundation. The chimney drain slopes toward upstream and is connected to a drainage blanket, placed on top of the foundation surface. The drainage blanket underlies part of the upstream– and all of the downstream half of the dam. The dam section is shown on Fig. 10.2.

Laboratory testing of the embankment materials was conducted in Chile. The tests included moisture, density, gradation, compaction and two series of triaxial compression tests on isotropically consolidated, drained specimens (TX/ICD tests) of materials representative of the core (Zone A) and shell (Zone B). The effective friction angle of the embankment materials ranged from 35.5 to 46 degrees, depending on the level of confinement (the friction angle decreases at higher confining pressures). The core materials (Zone A) have a cohesion measured at between 0.20 and 1.60 metric tons/$m^2$. The shell materials are cohesionless. The permeability of the core materials was measured at 0.5 x $10^{-5}$ cm/s.

The tailings were estimated to have an effective cohesion of 0.2 tons/$m^2$ and a friction angle of 28 degrees, based on strength data published in the literature for Chilean copper tailings. Their measured coefficient of permeability, or 0.5 x $10^{-6}$ cm/s, is one order of magnitude smaller than the coefficient of permeability of Zone A. This is indicative that Zone A acts as a drain with respect to the less pervious tailings. Saturation of parts of Zone A and the upstream Zone B only occurs when clear water is impounded above the solid tailings.

## THE MARCH 3, 1985 EARTHQUAKE

The March 3, 1985 earthquake (Ms 7.8) was centered near the coast of central Chile. At least 180 people were killed and 2500 injured. Extensive damage occurred in the cities of San Antonio, Valparaiso, Viña del Mar, Santiago and Rancagua. The earthquake was felt in Chile along a stretch extending 2000 km from Copiaco to Valdivia. A few modern structures, including reinforced concrete buildings in Reneca and Viña del Mar, suffered significant damage. Damage to adobe structures was extensive. A portion of the port of Valparaiso experienced over 41 cm of lateral spreading as a result of liquefaction. Bridge damage, in the form of subsidence and spreading of approach fills and pier and span collapses, was observed. Minor damage occurred at industrial facilities. Two tailings dams failed, including the Veta de Agua tailings impoundment, near the town of El Cobre, and the Cerro Negro Dam near the town of the same name.

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

Strong motion acceleration records were obtained at both the base and crest of the dam, with a peak horizontal acceleration of 0.13 g at the toe (PGA) and 0.21 g at the crest (PCA). Overall duration of felt shaking exceeded 100 seconds, with a bracketed duration exceeding 40 seconds. Fig. 10.3 shows the significant phase of the 1985 time histories and the large amplifications from the dam crest response. Fig. 10.4 shows the crest response spectrum. Of significance is a very large peak spectral amplification ratio, which was almost five at crest level, and the relatively short period (0.55 s) at which it occurred, which confirmed the overall stiffness of the dam materials.

## BACK-CALCULATED RESPONSE

The recorded 1985 base motion provided seismic input to analyze the Stage II dam and calibrate a numerical model of final Stage IV-B. This model is shown on Fig. 10.5. The recorded crest motion was used to compare calculated and recorded responses of the Stage II dam.

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

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

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

Maximum calculated settlement was less than 0.5 cm, in acceptable agreement with observed performance (no measurable deformations). Both calibration methods provided consistent results. Calculated peak spectral acceleration and Arias intensity were within 15 percent of the reference values. The comparison between calculated and recorded response spectra, see Fig. 10.4, was satisfactory. The calibration analysis provided a basis for the development of a numerical analysis model for the design of the final raising of the dam.

## CONCLUSION

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

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

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

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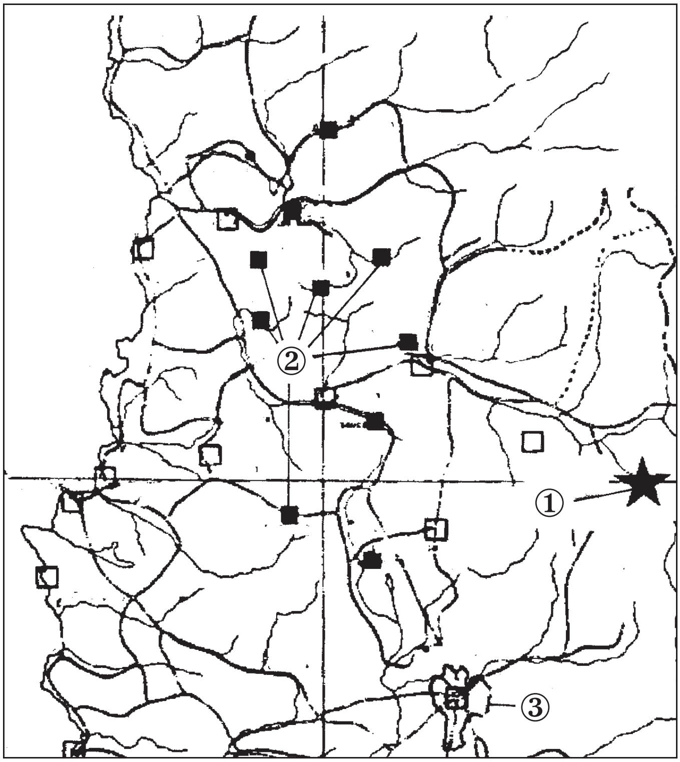


Fig. 10.1

Location of Los Leones Dam

*Situation du barrage Los Leones*

1) Los Leones Dam
2) Various Tailings Dams
3) Santiago de Chile

*1) Barrage Los Leones*
*2) Différents barrages de stériles*
*3) Santiago du Chili*

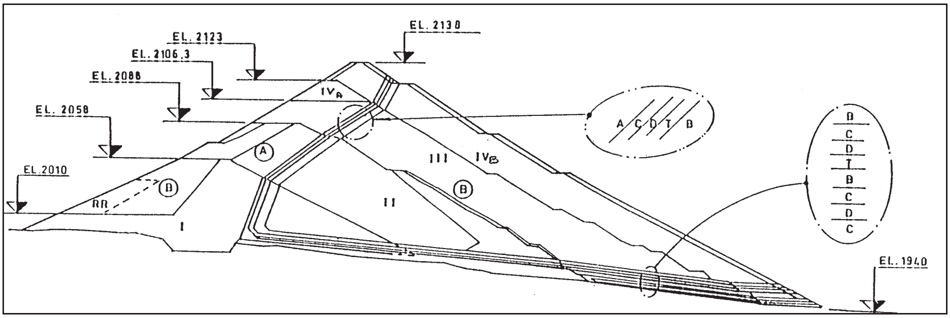


Fig. 10.2

Los Leones Dam – Embankment Zoning Details

*Barrage Los Leones – Détails du zonage des remblais*

| | |
|---|---|
| A) Upstream Morainic Core | *A) Noyau amont morainique* |
| B) Upstream and Downstream Fill | *B) Remblai amont et aval* |
| C) Filter | *C) Filtre* |
| D) Drain | *D) Drain* |
| T) Transition | *T) Zone de transition* |
| RR) Riprap | *RR) Riprap* |
| I to IV) Stages of Construction | *I à IV) Étapes de construction* |

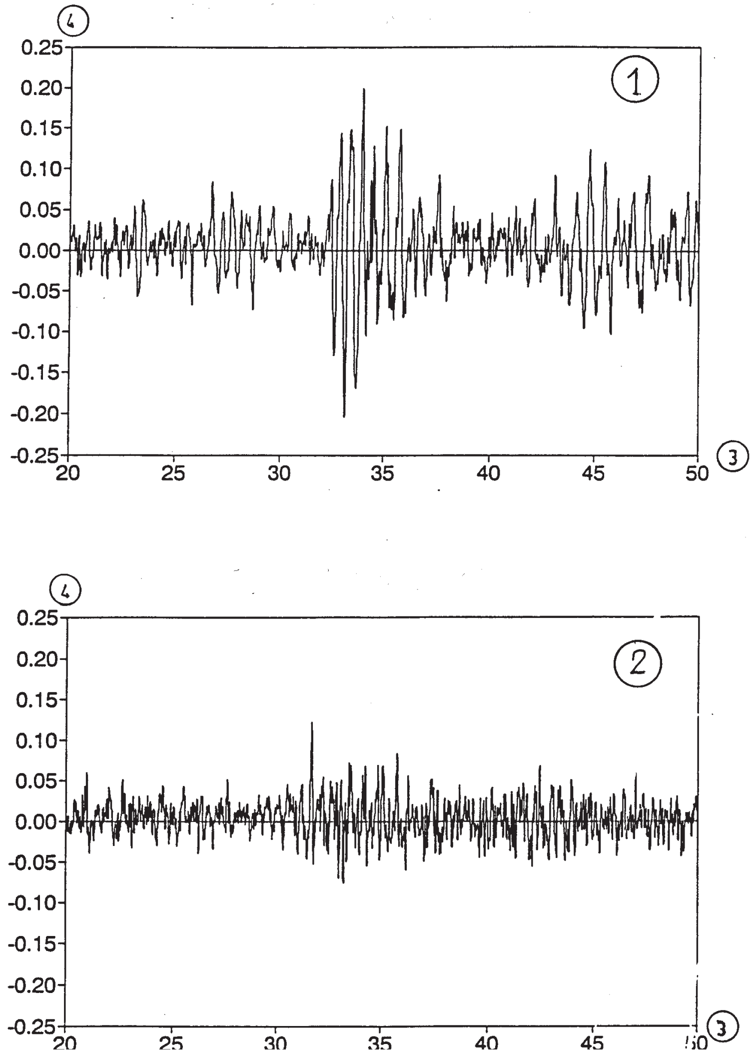


Fig. 10.3

Los Leones Dam – Crest and Base Motions

*Barrage Los Leones – Mouvements en crête et au pied*

1) Crest Motion (Horizontal)
March 3, 1985 Earthquake
2) Base Motion (Horizontal)
March 3, 1985 Earthquake
3) Time (seconds)
4) Acceleration (g)

*1) Mouvement en crête (horizontal)*
*Séisme du 3 mars 1985*
*2) Mouvement au pied (horizontal)*
*Séisme du 3 mars 1985*
*3) Temps (secondes)*
*4) Accélération (g)*

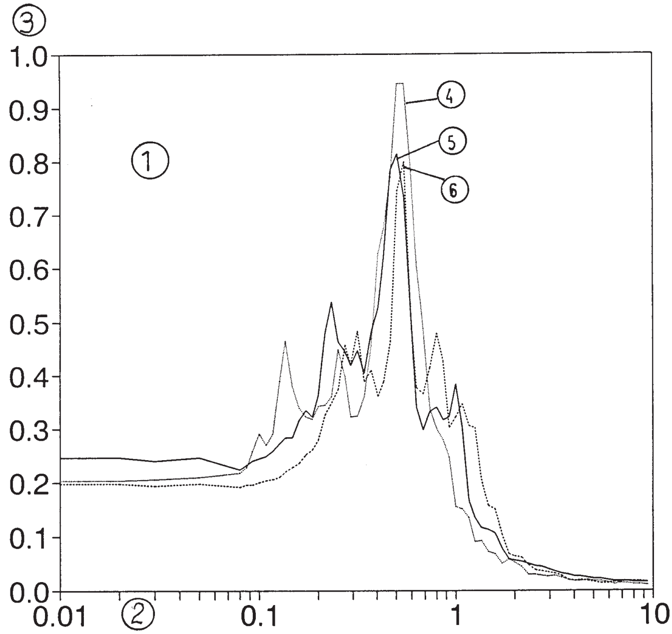


Fig. 10.4

Los Leones Dam – Crest Response Spectrum Comparison (March 3, 1985 EQK)

*Barrage Los Leones – Comparaison des spectres de réponse de la crête (Séisme du 3 mars 1985)*

| | |
|---|---|
| 1) Damping 5 % | *1) Amortissement 5 %* |
| 2) Undamped Natural Periods (seconds) | *2) Périodes naturelles non amorties (secondes)* |
| 3) Acceleration (g) | *3) Accélération (g)* |
| 4) Recorded Values | *4) Valeurs mesurées* |
| 5) Calculated Values for Dry Dam | *5) Valeurs calculées pour le barrage sec (sans percolation)* |
| 6) Calculated Values for Dam Wet due to Pond Seepage | *6) Valeurs calculées pour le barrage avec percolation* |

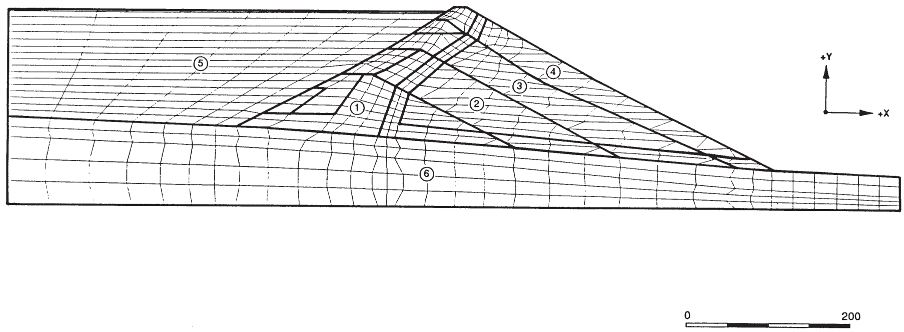


Fig. 10.5

Los Leones Dam – Finite Difference Model

*Barrage Los Leones – Modèle aux différences finies*

| | |
|---|---|
| 1) Stage I Dam | *1) Étape I du barrage* |
| 2) Stage II Dam | *2) Étape II du barrage* |
| 3) Stage III Dam | *3) Étape III du barrage* |
| 4) Stage IV Dam | *4) Étape IV du barrage* |
| 5) Tailings | *5) Stériles* |
| 6) Foundation | *6) Fondation* |

# 11. MASIWAY DAM, PHILIPPINES

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

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

## MASIWAY DAM

The dam regulates releases from the 107 m high Pantabangan Dam, which is located three miles upstream. The layout of the dam and appurtenant facilities is shown on Fig. 11.1. Masiway Dam is an 25 m high zoned earthfill dam with a central clay core, see Fig. 11.2. It has a crest length of 427 m and crest width of 10 m. The shells consist of alluvial material and conglomerate. The upstream shell of the dam consists of random alluvium, with a permeability of less than $10^{-3}$ cm/s and slopes at 2.3:1 (horizontal to vertical). The downstream shell slopes at 2:1 (h to v). Filter zones were placed on both sides of the core, and a chimney drain and a horizontal drainage blanket are located below the downstream shell.

Other project features include a concrete chute service spillway with three radial gates, an unlined, 79 m wide auxiliary spillway with a fuseplug, an intake and semi-outdoor 12 MW single– unit powerhouse. The Masiway Hydroelectric Project was placed in service in 1981. On July 16, 1990, the day of the earthquake, the reservoir elevation was El. 128 m. The reservoir elevation on August 6, 1990 was El.125.99 m.

## THE JULY 16, 1990 EARTHQUAKE

On July 16, 1990, the heavily populated Island of Luzon, Philippines, was shaken by a large earthquake (M 7.7). The earthquake affected an area over 52 000 square km. At least 1700 people were killed and perhaps 1000 were missing. At least 3500 persons were severely injured. Over 4000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake and many of the better hotels were damaged.

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

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

**Dam.** This 25 m high embankment dam suffered extensive damage. The upstream shell slumped up to two meters horizontally and one meter vertically. Locations of observed cracks and directions of movement are shown on Fig. 11.3. All the principal cracks were parallel to the dam axis. The largest of these cracks extended along most of the crest access road, about 1.5 m from the centerline, and to a depth of about 1.7 m, as observed in test pits. Major cracks were also observed at between 1 and 3.3 m below the crest elevation, especially along the upstream slope. The shell ravelling appeared to approach the natural angle of repose of the constituent materials at several locations.

On the upstream side of the crest, several aligned sinkholes provided further evidence of movement of the upstream shell along the core. The dam settled by almost one meter along the left spillway training wall. Based on observations and reports, settlements gave the impression of being of the same order of magnitude over the entire length of the embankment. A subsequent report (Swaisgood and Au-Yeung (1991) indicated overall settlements between 13 cm to over 1 m, see Fig. 11.4. The difference in behavior between the upstream and downstream shell pointed to the probable occurrence of liquefaction in the upstream shell.

**Spillways.** There was little observable damage to the spillways, except longitudinal cracking of maximum width of about 10 cm along the entire length of the fuseplug. The dam owner reported that the spillway remained fully operational after the earthquake. Slope failures were also observed along the training dike which connects the left abutment of the main dam and the right extremity of the fuseplug.

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

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

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

## CONCLUSION

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

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

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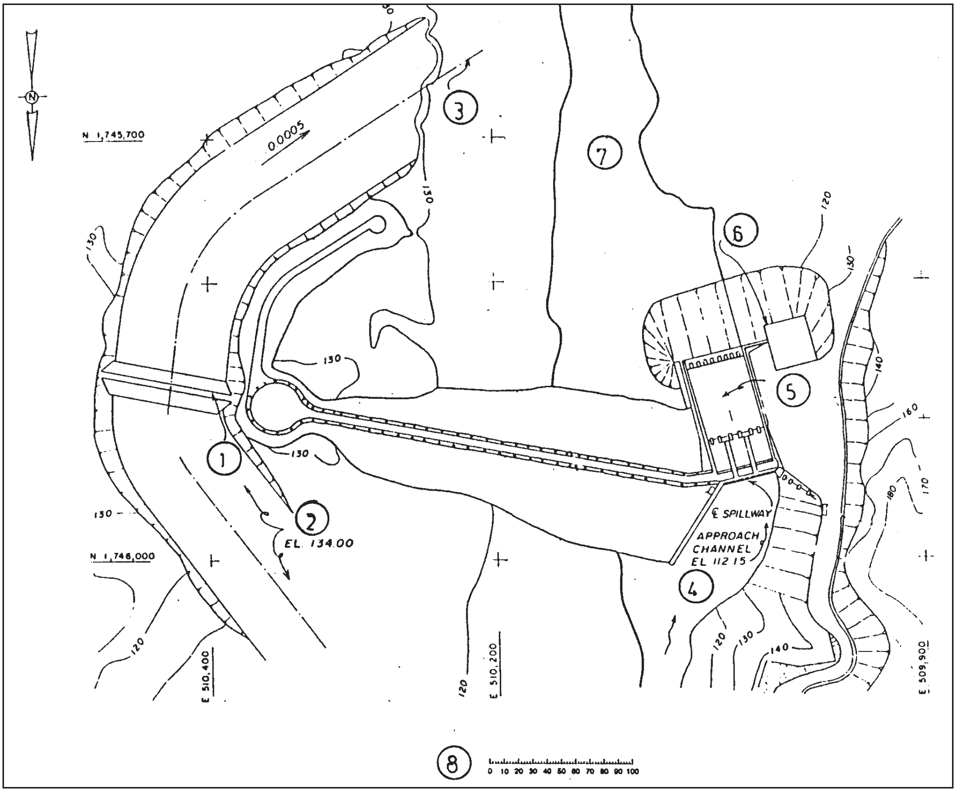


Fig. 11.1

Masiway Dam – Plan View

*Barrage Masiway – Vue en plan*

1) Fuse Dike with Crest Elevation 130
2) Approach Channel
3) Outfall of Auxiliary Spillway
4) Spillway Approach Channel
5) Spillway and Stilling Basin
6) Power House
7) Pampanga River
8) Scale (100 m)

*1) Digue fusible avec niveau de crête 130*
*2) Canal d'entonnement*
*3) Canal de décharge de l'évacuateur auxiliaire*
*4) Canal d'entonnement de l'évacuateur de surface*
*5) Évacuateur de crue et basin d`amortissement*
*6) Usine*
*7) Rivière Pampanga*
*8) Échelle (100 m)*

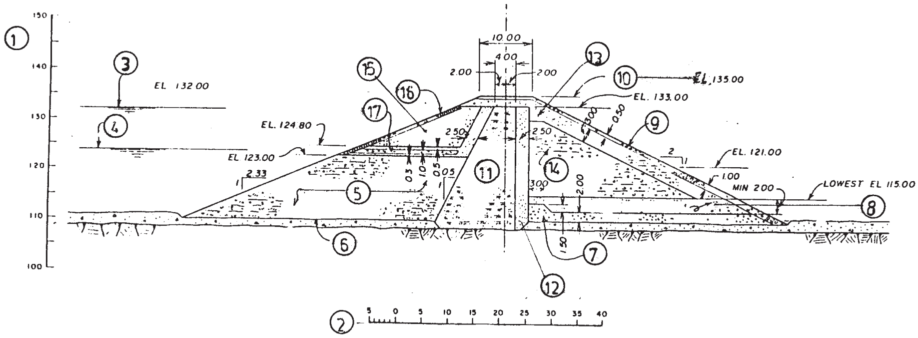


Fig. 11.2

Masiway Dam – Maximum Section

*Barrage Masiway – Coupe transversale maximale*

| | |
|---|---|
| 1) Elevation in m | *1) Cote en m* |
| 2) Scale in m | *2) Échelle en m* |
| 3) Maximum Flood Level | *3) Niveau maximal de crue* |
| 4) Minimum Operating Level | *4) Niveau minimal d'exploitation* |
| 5) Random Alluvium | *5) Alluvions tout-venant* |
| 6) Alluvium | *6) Alluvions* |
| 7) Select Alluvium | *7) Alluvions sélectionnées* |
| 8) Random Alluvium | *8) Alluvions tout-venant* |
| 9) Cobble Riprap | *9) Riprap de galets* |
| 10) Crest of Dam | *10) Crête du barrage* |
| 11) Impervious Core | *11) Noyau étanche* |
| 12) Sand Filter | *12) Filtre de sable* |
| 13) Random Alluvium | *13) Alluvions tout-venant* |
| 14) Conglomerate Fill | *14) Remblai de conglomérats* |
| 15) Select Alluvium | *15) Alluvions sélectionnées* |
| 16) Cobble Riprap | *16) Riprap de galets* |
| 17) Gravel | *17) Gravier* |

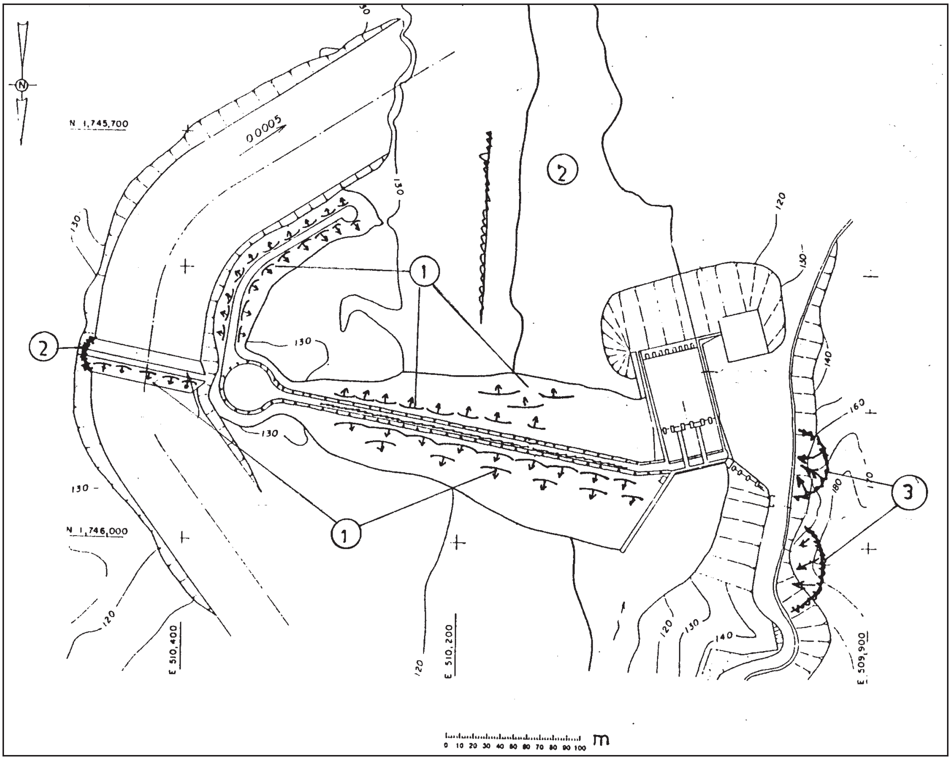


Fig. 11.3

Masiway Dam – Earthquake Damage

*Barrage Masiway – Dégâts d'origine sismique*

| | |
|---|---|
| 1) Slump Scraps Showing Direction of Moving | *1) Effondrements montrant la direction du mouvement* |
| 2) Cracks with Small Amount of Movement | *2) Fissures avec faible mouvement* |
| 3) Major Slide | *3) Glissements importants* |

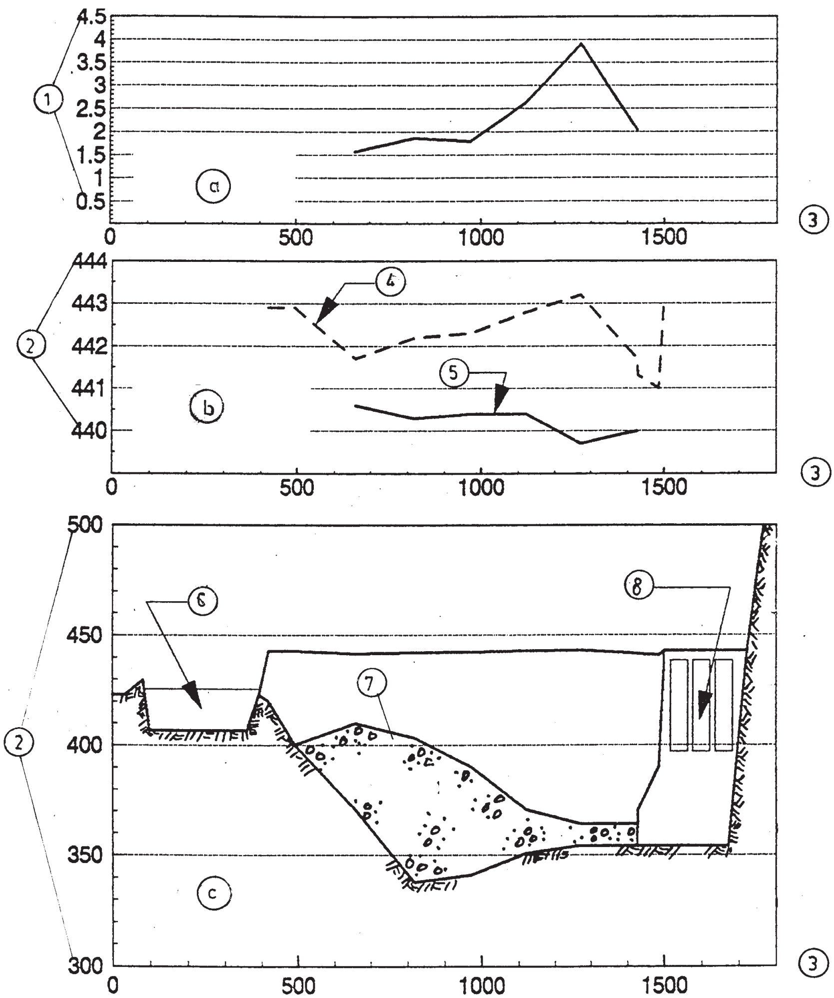


Fig. 11.4

Masiway Dam – Crest Settlement

*Barrage Masiway – Tassement de la crête*

a) Percent Settlement
b) Crest Settlement
c) Centerline Profile – Looking Downstream
1) Settlement as Percent of Dam and
   Alluvium Height
2) Elevation (ft)
3) Horizontal Distance (ft)
4) Before Earthquake
5) After Earthquake
6) Fuse Dike
7) Alluvium
8) Spillway

*a) Tassement en  pourcentage*
*b) Tassement de la crête*
*c) Profil le long de l'axe (vue vers l'aval)*
*1) Tassement en pourcentage de la hauteur du*
   *barrage et des alluvions*
*2) Cote (pieds)*
*3) Distance horizontale (pieds)*
*4) Avant le séisme*
*5) Après le séisme*
*6) Digue fusible*
*7) Alluvions*
*8) Évacuateur de crue*

# 12. MOCHIKOSHI TAILINGS DAM, JAPAN

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

## THE HOZUKIZAWA DISPOSAL POND

The following descriptions of the impoundment and the earthquake effects are a summary taken from Ishihara (1984). The Hozukizawa disposal pond at Mochikoshi was constructed in a bowl-shaped depression on top of a mountain by sealing off the periphery with three dikes (Fig. 12.1). A detailed plan view is shown on Fig. 12.2. The site originally consisted of a weathered deposit of tuff with cobble inclusions, deposited by a series of neighboring volcanic eruptions.

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

The mine's milling operation to extract gold was conducted in a processing concentration plant located beside the Mochikoshi River (Fig. 11.1). The tailings were pumped as a slurry through discharge pipes up 600 m to the disposal pond, which was located on top of the mountain. The slurry was delivered either to the top of Dike No. 1 or Dike No. 2, and discharged toward the pond through three pipes at each dike location. The dikes were successively raised by placing local volcanic soils at a rate of approximately 2 m per year by the upstream method of tailings dam construction (the downstream slope is maintained fixed, while the crest is raised in the upstream direction).

Cross sections before and after the failures of dikes No. 1 and No. 2 are shown on Fig. 12.3 and 12.4. These cross sections include the logs of borings drilled after the failures. Typical gradations of the dike and tailings materials are shown on Fig. 12.5 and 12.6, respectively.

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

The dikes were constructed of a mixture of the weathered tuffs and volcanic ashes that covered a widespread area of the mountains in the vicinity. These materials are a mixture of gravel, sand and silt, as shown by the wide range of the grain size distribution curves shown on Fig. 12.5. The wet unit weight of these soils ranged between 14 and 19 $kN/m^3$ ; the natural water content was 30 to 60 percent; and the void ratio was 1.1 to 2.6.

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

## THE IZU-OHSHIMA-KINKAI EARTHQUAKE OF 1978

On January 14, 1978, a destructive earthquake (M 7.0) shook the southeastern area of the Izu Peninsula, about 120 km southwest of Tokyo, Japan (Fig. 12.7). The epicenter of this event was located about 15 km off the east coast of the peninsula. The main shock was followed for about a week by a series of aftershocks, with epicenters moving gradually in a westerly direction (Fig. 12.9). The two largest aftershocks, including one of magnitude 5.8, took place at 7:30 a.m. and 7:36 a.m. on January 15, 1978. Their epicenters were located approximately in the middle of the Izu Peninsula, close to the Mochikoshi tailings dam site.

Most of these events were estimated to have had a focal depth of about 10 km. The strong ground shaking produced by the earthquake was recorded at several stations, but outside the area of highest intensity shaking. A survey of the overturning of tombstones in many cemeteries in the epicentral area was used to obtain an approximate estimate of the distribution of shaking intensity. Estimated contours of equal peak horizontal accelerations are shown on Fig. 12.8 (Ohashi, et al., 1978).

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

The failure of Dike No. 1 was triggered by the main shock of the Izu-Ohshima-Kinkai Earthquake. A cross-section through the collapsed dam is presented on Fig. 12.3. Dike No. 1, which was the largest (28 m high and 7.3 m wide at crest level) collapsed almost concurrently with the strong phase of the earthquake shaking. An attendant at the pond, who happened to be stationed at a house on the left bank, came out immediately upon perceiving an unusually high level of shaking and watched the failure. According to his account, within about ten seconds after the main shock, the front face of the dike bulged, and a breach occurred in the upper part of the embankment, near the left abutment. It was followed by a huge mass of tailings slimes rushing down the valley with a loud roar, toppling trees and scouring the valley floor in the process. When the rushing slimes reached the Mochikoshi River, they hit masonry walls on the opposite river bank, surging up to a height of about 10 m and leaving near

30-cm thick deposits over the road beside the river. The slimes flowed down into the Mochikoshi River, leaving 1.0 to 1.9 m thick sediments in the river bed along a distance of about 800 m from the point of confluence. The flowing slimes travelled further into the Kano River, and contaminated that river to a distance of about 10 km downstream.

The top portion of Dike No. 1 failed totally throughout a height of 14 m from the top level of the embankment down to the top elevation of the starter dam, as illustrated on Fig. 12.3. A volume of 80 000 $m^3$ of materials was released by the dike failure, of which 60 000 $m^3$ were tailings slimes and 20 000 $m^3$ were part of the dike-forming volcanic ashes.

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

At about 1:00 p.m. on that day, a caretaker standing on the opposite side of Dike No. 2 noticed a gradual sinking of the central part of the embankment. While running to the site, he watched the dike fail suddenly through a crest breach about 20 m wide, which led to the release of the impounded tailings sludge. Later on, the breach size increased to a crest width of 65 m and generated a number of cracks over the sloughing surface. A total volume of 3000 $m^3$, consisting of 2000 $m^3$ of tailings slimes and 1000 $m^3$ of dike materials, flowed down the valley a distance of about 240 m. A cross section of the failure surface, superimposed on the original dike section, is shown on Fig. 12.4.

## INSTRUMENTATION AND STRONG MOTION RECORDS

No strong motion records were obtained at the Mochikoshi tailings dam sites. Estimated peak ground accelerations in the general area are shown on Fig. 12.8. It may be seen that, at the Mochikoshi site, the peak ground acceleration was estimated to be approximately 0.25 g.

## CONCLUSIONS

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

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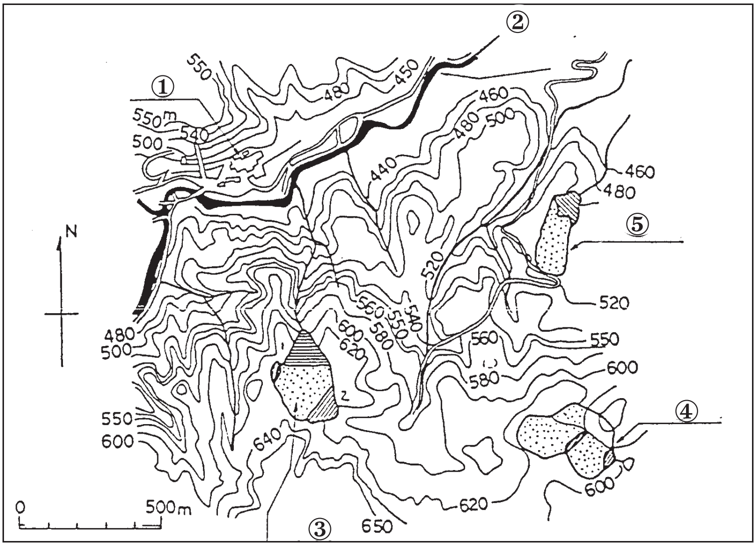


Fig. 12.1

Hozukizawa Tailings Pond of Mochikoshi Tailings Dam

*Bassin de dépôt de stériles Hozukizawa du complexe Mochikoshi*

| | |
|---|---|
| 1) Concentration Factory | *1) Usine de concentration* |
| 2) Mochikoshi River | *2) Rivière Mochikoshi* |
| 3) Hozukizawa Tailings Pond | *3) Bassin de dépôt de stériles Hozukizawa* |
| 4) Hirayama Pond | *4) Bassin Hirayama* |
| 5) Narasawa Pond | *5) Bassin Narasawa* |

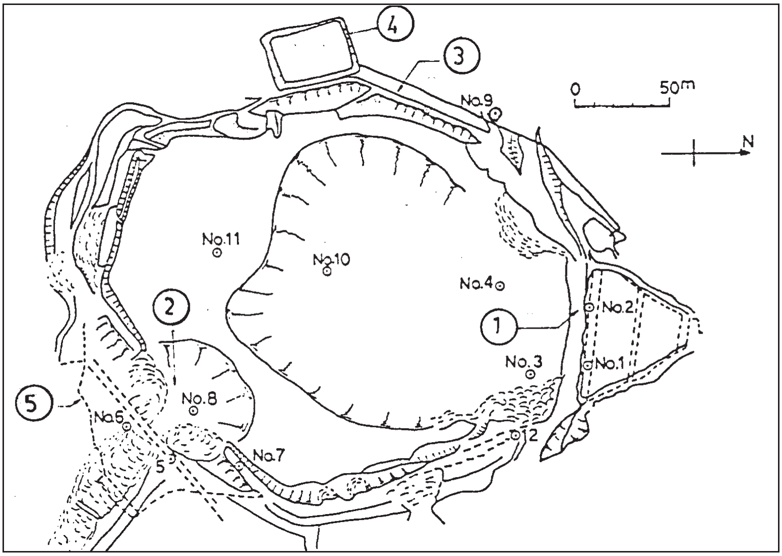


Fig. 12.2

Plan of Hozukizawa Disposal Pond

*Vue en plan du bassin de dépôt Hozukizawa*

1) No 1 Dike — *1) Digue No 1*
2) No 2 Dike — *2) Digue No 2*
3) No 3 Dike — *3) Digue No 3*
4) Pond — *4) Bassin*
5) Drainage — *5) Drainage*
⊙ Borings No 1 to No 11 — *⊙ Forages No 1 à No 11*

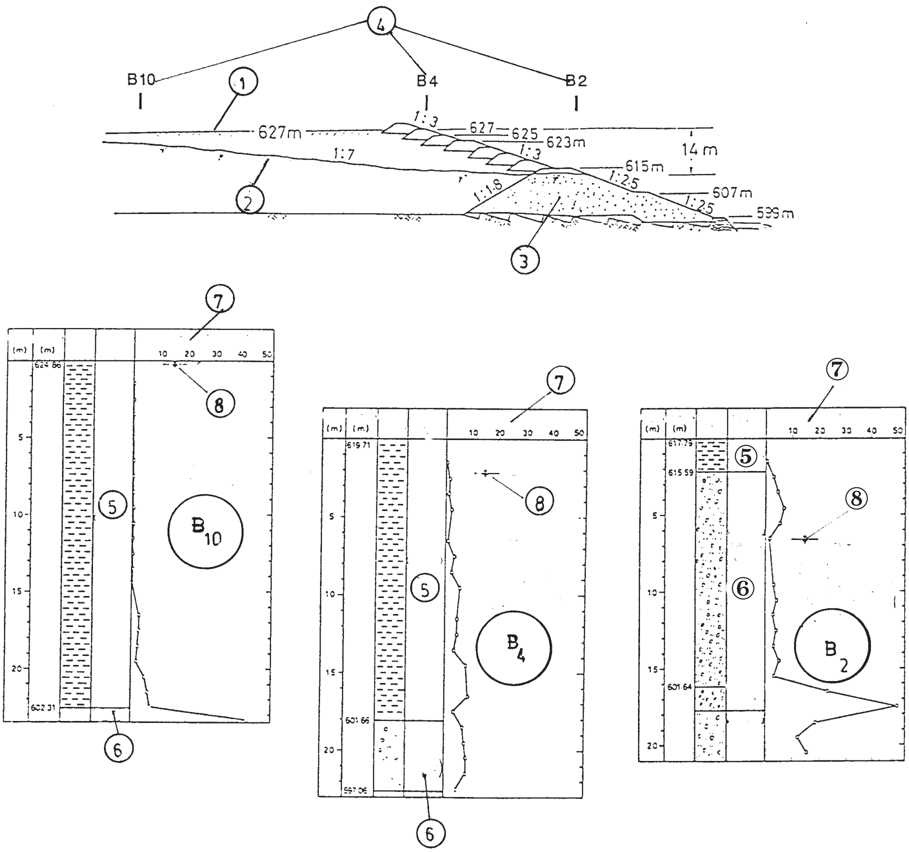


Fig. 12.3

Cross Section through Embankment No 1 Dike and Boring Logs

*Coupe transversale du remblai de la Digue No 1 et relevés de forages*

| | |
|---|---|
| 1) Before Failure | *1) Avant la rupture* |
| 2) After Failure | *2) Après la rupture* |
| 3) Volcanic Soil | *3) Sol volcanique* |
| 4) Boreholes | *4) Forages* |
| 5) Silt (Tailings) | *5) Silt (stériles)* |
| 6) Volcanic Ashes | *6) Cendres volcaniques* |
| 7) Blow Count | *7) Nombre de coups* |
| 8) Ground Water Level | *8) Niveau de la nappe phréatique* |

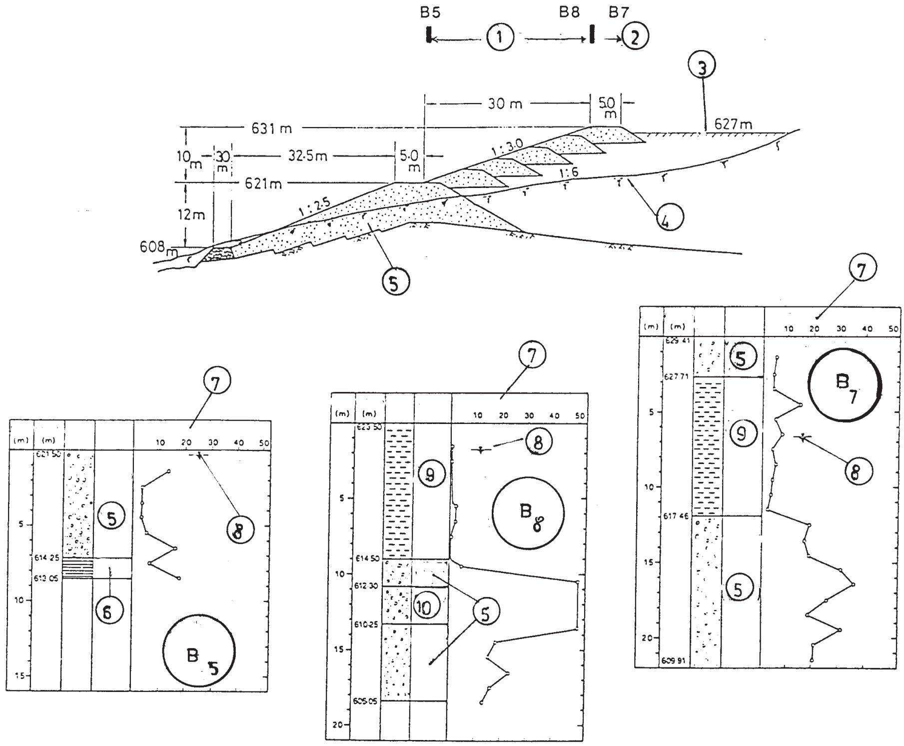


Fig. 12.4

Cross Section through Embankment No 2 Dike and Boring Logs

*Coupe transversale du remblai de la Digue No 2 et relevés de forages*

| | |
|---|---|
| 1) In Failure Area | *1) Dans la zone de rupture* |
| 2) Beyond Failure Area | *2) Au-delà de la zone de rupture* |
| 3) Before Failure | *3) Avant la rupture* |
| 4) After Failure | *4) Après la rupture* |
| 5) Volcanic Soil | *5) Sol volcanique* |
| 6) Clay (from Tuff) | *6) Argile (provenant de tuf)* |
| 7) Blow Count | *7) Nombre de coups* |
| 8) Groundwater Level | *8) Niveau de la nappe phréatique* |
| 9) Silt (Tailings) | *9) Silt (stériles)* |
| 10) Gravelly Tuff | *10) Tuf graveleux* |

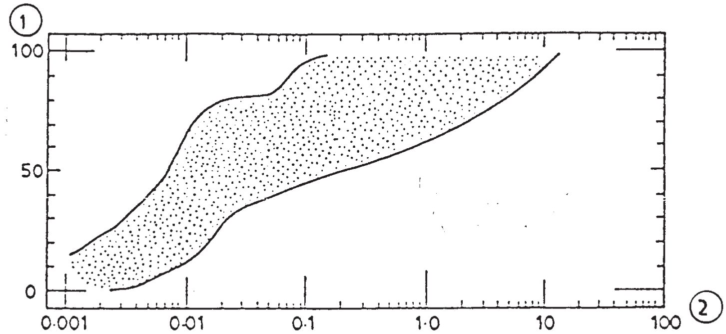


Fig. 12.5

Mochikoshi Tailings Dam – Distribution of Grain Sizes in the Dikes

*Barrage de stériles Mochikoshi – Granulométrie des matériaux des digues*

| | |
|---|---|
| 1) Percent Finer by Weight (%) | *1) Pourcentage plus fin en poids (%)* |
| 2) Grain Size (mm) | *2) Dimensions des grains (mm)* |

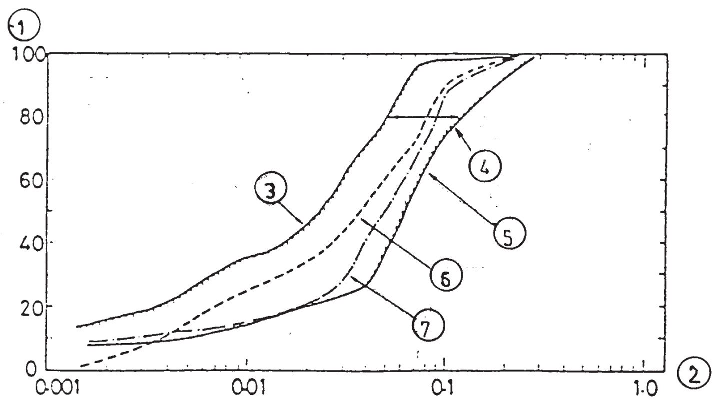


Fig. 12.6

Grain Size Distribution Curves of Tailings Materials from Mochikoshi

*Courbes granulométriques des stériles de Mochikoshi*

| | |
|---|---|
| 1) Percent Finer by Weight (%) | *1) Pourcentage plus fin en poids (%)* |
| 2) Grain Size (mm) | *2) Dimensions des grains (mm)* |
| 3) From Finest Layer | *3) Couche la plus fine* |
| 4) Approximate Range of Grain Sizes | *4) Plage approximative granulométrique* |
| 5) From Coarsest Layer | *5) Couche la plus grossière* |
| 6) Sample A (Near No 2 Dike) | *6) Échantillon A (près da la digue No 2)* |
| 7) Sample B (Near No 2 Dike) | *7) Échantillon B (près de la digue No 2)* |

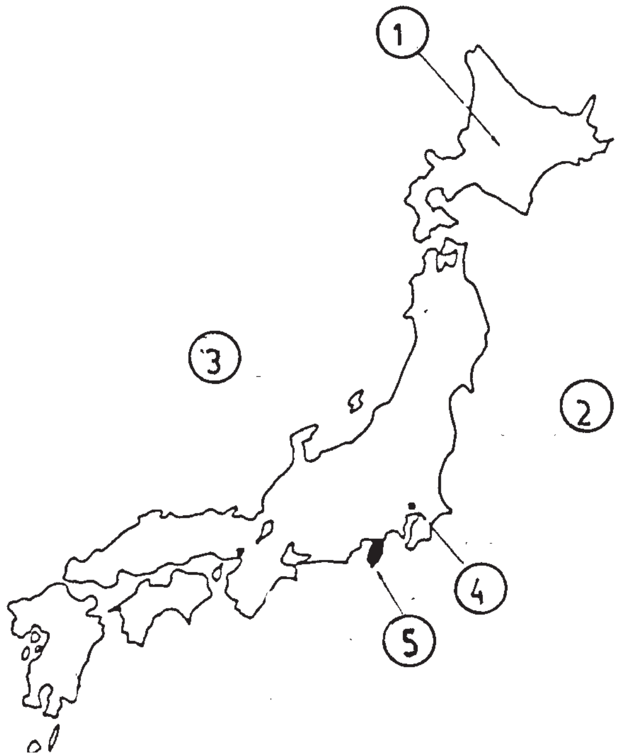


Fig. 12.7

Mochikoshi Tailings Dam – Location of Izu Peninsula

*Barrage de stériles Mochikoshi – Situation de la presqu'île Izu*

1) Hokkaido
2) Pacific Ocean
3) Japan Sea
4) Tokyo
5) Izu Peninsula

*1) Hokkaido*
*2) Océan Pacifique*
*3) Mer du Japon*
*4) Tokyo*
*5) Presqu'île Izu*

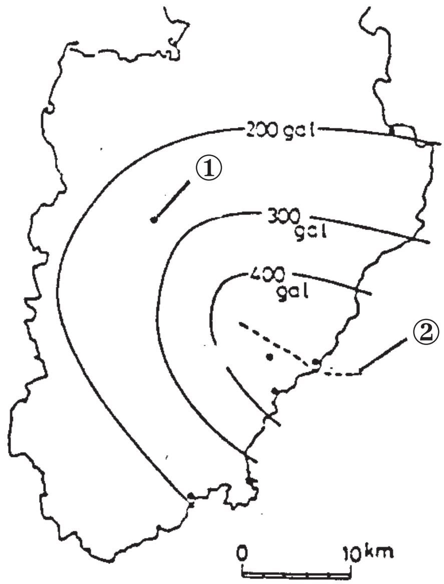


Fig. 12.8

Mochikoshi Tailings Dam – Contours of Equal Shaking Intensity in Terms of Estimated Maximum Horizontal Accelerations

*Barrage de stériles Mochikoshi – Lignes d'égale intensité des secousses sismiques en termes d'accélérations horizontales maximales estimées*

| | |
|---|---|
| 1) Mochikoshi Location | *1) Situation de Mochikoshi* |
| 2) Inatori Fault | *2) Faille Inatori* |

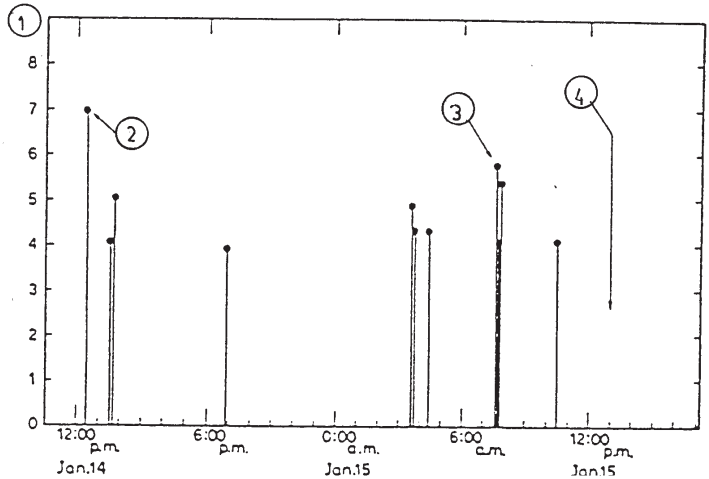


Fig. 12.9

Mochikoshi Tailings Dam – Occurence of Aftershocks with Magnitude Greater than 4.0, Following the Main Shock Izu-Oshima-Kinkai

*Barrage de stériles Mochikoshi – Répliques de magnitude supérieure à 4,0, enregistrées après la secousse principale du séisme Izu-Oshima-Kinkai*

1) Earthquake Magnitude
2) Main Shock M = 7.0
3) Aftershock M = 5.8
4) Failure of Dike No 2

*1) Magnitude des séismes*
*2) Secousse principale M = 7,0*
*3) Réplique M = 5,8*
*4) Rupture de la Digue No 2*

# 13. PANTABANGAN DAM, PHILIPPINES

On July 16, 1990, a large earthquake (M 7.7) struck the Philippines Islands. Pantabangan Dam, owned by the country's irrigation and power administration, the Philippines National Power Corporation, is one of six dams that were located within a short distance from the epicenter. The main dam and Aya Creek Dam, which also forms part of the Pantabangan complex, settled a maximum of about 28 and 20 cm, respectively. Minor cracks were observed on the crests, at the contacts between those dams and their abutments. Estimated peak ground acceleration at the site was 0.65 g. The excellent performance of the Pantabangan project was attributed to the low reservoir level that prevailed at the time of occurrence of the earthquake.

## PANTABANGAN DAM

The Pantabangan Project was placed in service in 1977. The Pantabangan impoundment has three components: Aya Creek Dam on the southeast; a low intermediate saddle dam; and the main Pantabangan Dammain dam on northwest, see Fig. 13.1. All three dams are zoned earthfill dams. Each of the embankments has a central impervious clayey core with outer shells of alluvial material and weathered conglomerate, a vertical or near-vertical filter and chimney drain, and a horizontal drainage blanket. The maximum sections of the two principal embankments are shown on Fig. 13.2. The impervious core of the main dam is substantially larger at its base than that of the Aya Creek embankment.

Maximum height of the largest embankment is 107 meters and the crest length is 732 m. Slopes are 2.5:1 (horizontal to vertical) upstream, and 2.2:1 (h to v) downstream. The downstream face of Pantabangan Dam is protected with select coarse alluvial material, while most of the upstream slope is faced with a reinforced concrete slope protection.

Aya Creek Dam is approximately 430 m long and has a maximum height of 100 m. Its slopes are 3:1 and 2.2:1 (h to v), upstream and downstream, respectively. Both of the upstream and downstream faces are protected with select coarse alluvial material.

Other project features include a concrete chute spillway located in rock on the left abutment of the Aya Creek Dam, two intakes towers and two concrete-lined outlet tunnels, 7 m in diameter (originally used as diversion tunnels), a low level outlet, and a surface powerhouse. The spillway chute is 270 m long and terminates in a flip bucket.

## THE JULY 16,1990 EARTHQUAKE

On July 16, 1990, the heavily populated Island of Luzon, Philippines was shaken by a large earthquake (M 7.7). The earthquake affected an area over 52 000 square km.

At least 1700 people were killed and perhaps 1000 were missing. At least 3500 persons were severely injured. Over 4000 homes and commercial or public buildings were damaged beyond repair. The most serious damage occurred in soft soils regions such as the Central Plains town of Gerona, the river delta town of Agoo and eastward of the City of Baguio, a mile high within the Cordillera Mountains. The transportation system was severely disrupted. Baguio, a popular resort, was devastated by the earthquake. Many of the better hotels were damaged.

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

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

**Dams.** The upstream side of the crest of Pantabangan Dam settled a maximum of 26 cm at the maximum section. The settlement decreased proportionately toward the abutments. Settlement profiles are presented on Fig. 13.3. A transverse crack was found in the asphalt pavement of the crest road, at the contact between the embankment and the left abutment. No increase in seepage through the dam was reported.

The only evidence of distress in the saddle dam consisted of diagonal cracks on the paved roadway over the crest, near its left abutment. The cracks were obviously produced by tensile stresses induced by differential settlement, due to the presence of a ridge or a change of geometry in the foundation of the left abutment of the dam. Aya Creek Dam experienced an average settlement of 20 cm near its maximum section, see Fig. 13.3. A thin crack was found along the crest roadway at the contact between the embankment and the left abutment. No seepage increase was reported.

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

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

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

## CONCLUSION

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

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

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

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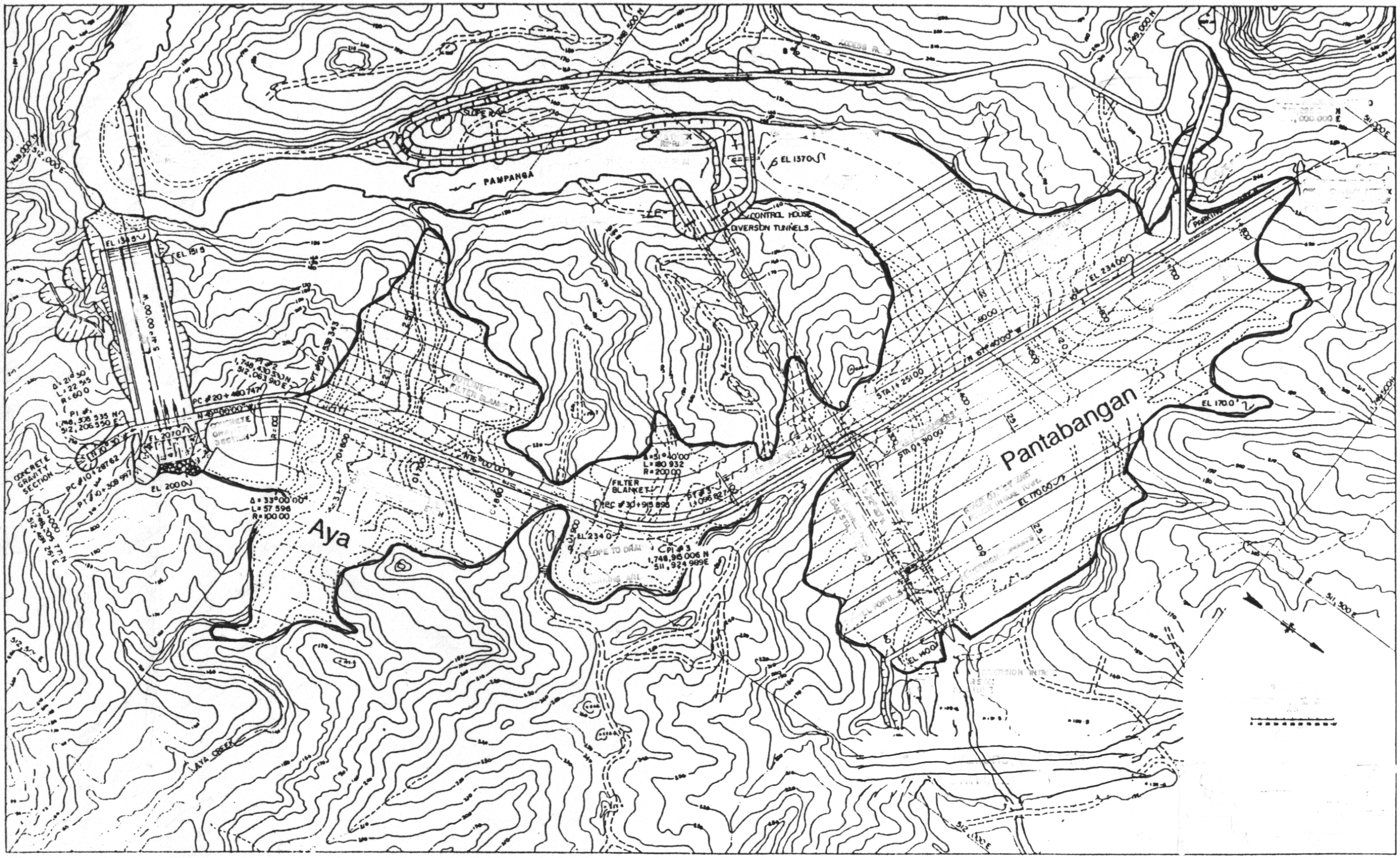


Fig. 13.1

Plan View of Pantabangan and Anya Dams

*Vue en plan des barrages Pantabangan et Aya*

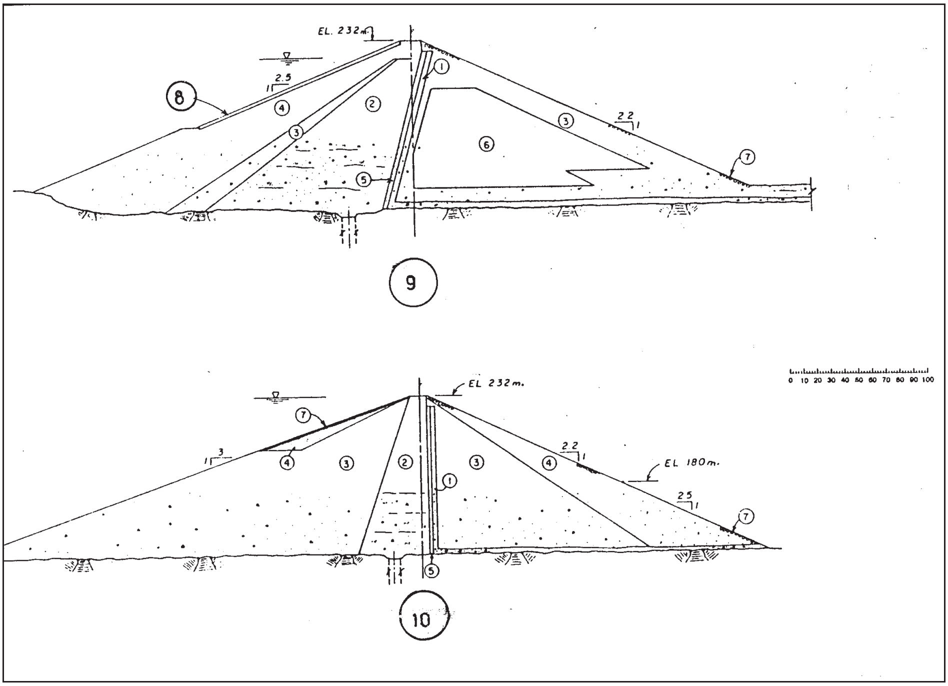


Fig. 13.2

Pantabangan and Aya Dams – Maximum Sections

*Barrages Pantabangan et Aya – Coupes transversales maximales*

| | |
|---|---|
| 1) Gravel Filter | *1) Filtre de gravier* |
| 2) Impervious Core | *2) Noyau étanche* |
| 3) Random Alluvium | *3) Alluvions tout-venant* |
| 4) Select Alluvium | *4) Alluvions sélectionnées* |
| 5) Sand Filter | *5) Filtre de sable* |
| 6) Conglomerat Random Fill | *6) Remblai de conglomérats tout-venant* |
| 7) Cobble Riprap | *7) Riprap de galets* |
| 8) Concrete Slope Protection | *8) Protection du talus en béton* |
| 9) Pantabangan Dam Maximum Section | *9) Coupe transversale maximale du barrage Pantabangan* |
| 10) Aya Dam Maximum Section | *10) Coupe transversale maximale du barrage Aya* |

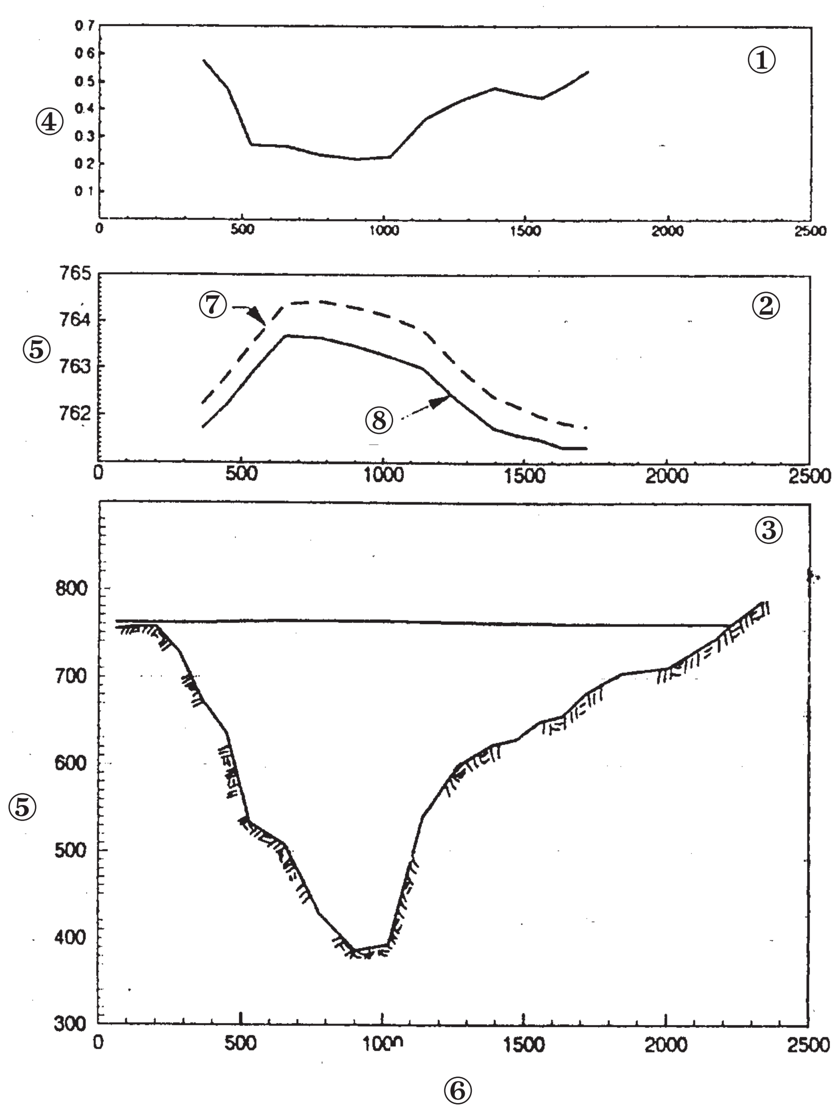


Fig. 13.3
Pantabangan Dam – Crest Settlements
*Barrage Pantabangan – Tassements de la crête*

| | |
|---|---|
| 1) Settlements as Percent of Dam plus Alluvium Height | *1) Tassements en pourcentage de la hauteur du barrage et des alluvions* |
| 2) Crest Settlements | *2) Tassements de la crête* |
| 3) Longitudinal Dam Profile Looking Downstream | *3) Profil longitudinal du barrage (vue vers l'aval)* |
| 4) Percent Settlements | *4) Tassements en pourcentage* |
| 5) Elevation (ft) | *5) Cote (pieds)* |
| 6) Horizontal distance (ft) | *6) Distance horizontale (pieds)* |
| 7) Before earthquake | *7) Avant le tremblement de terre* |
| 8) After earthquake | *8) Après le tremblement de terre* |

# 14. SEFID RUD DAM, IRAN

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

## SEFID RUD DAM

Sefid Rud Dam (sometimes referred to as Manjil Dam in the literature) was built from 1958 to 1962 as a 106 m high concrete gravity buttress dam. The dam plays a major role for irrigation purposes. About 50 percent of the rice production of Iran depends on stored water releases from Sefid Rud Dam. The dam has a maximum base width of about 100 m and a crest length of about 417 m. It is located at the confluence of the Ghezel-Owzan and Shah Rud rivers, and impounds a reservoir of about 1.8 billion cubic meters, with a tributary watershed area of about 58 000 $km^2$. The dam is composed of 23 buttresses spaced at 14 m center-to-center (Fig. 14.1 and 14.2). Webs have a constant thickness of 5 m. The buttresses were designed to act independently from each other. To avoid lateral movements near the right and, especially, the left abutment, buttresses Nos. 6 to 12 and Nos. 18 to 24 were keyed with a series of ground-supported lateral thrust slabs (Fig. 14.3). "Active" joints were provided at the downstream toe of buttresses Nos. 8 to 20 (Fig. 14.4). These joints originally included Freyssinet-type flat jacks, 2 m by 1 m in size, designed to improve the distribution of foundation stresses between upstream and downstream, and reduce the ratio between shear and normal loads at the foundation-bedrock interface. The joints were grouted once the buttresses presumably reached their final state of stress equilibrium, after a few years of operation.

The spillway is located in the gravity block near the left abutment and has a rated capacity of 2000 $m^3/s$. A powerhouse with an installed capacity of 87.5 MW was built at the toe of the dam. Five low-level outlets, two on the left abutment side and three on the right abutment side, control irrigation water releases. Those outlets are also used for sediment flushing. Two morning glory spillways in the left abutment provide an additional 1600 $m^3/s$ of outlet capacity.

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

## THE JUNE 21, 1990 EARTHQUAKE

Iran is located along the Mediterranean-Himalayan seismic belt, within the area where the Arabian and European tectonic plates collide. Historically, Iran has been a seismically very active region, where earthquakes of magnitude 6.0 or greater are frequent. The June 21, 1990 Earthquake was centered within the Maku Zanjan seismo-tectonic province, at the edge of the Alborz mountains, at latitude 36.96 N and longitude 49.41 E (Fig. 14.5). It devastated the two Iranian provinces of Gilan and Zanjan. The epicenter was about 200 km northwest of Tehran. The main shock was assigned magnitudes that ranged between 7.3 and 7.7. It was immediately followed by two large aftershocks (M 6.2 and M 6.5), and for months by numerous aftershocks, some of up to magnitude 5.9. The event was felt over an area larger than 600 000 $km^2$, see Fig. 14.6. Its focal depth was estimated at between 20 and 30 km.

Primary ground movements were interpreted to have occurred in the north-northwest direction, hence nearly parallel to the dam axis. Several faults, including the Rudbar and Harzevil fault zones, have been related to the occurrence of this earthquake. Immediately west of the dam, about 30 cm of strike-slip displacement and 50 cm of vertical thrust movement were observed (Fig. 14.7), confirming the compressional nature of the tectonic process.

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

The Manjil earthquake induced cracks at the horizontal lift joints in the upper part of the central buttresses of Sefid Rud Dam (Fig. 14.8). Those joints were located where the downstream slope of the webs experiences a change in slope. All 23 buttresses were cracked. The principal horizontal cracks ran across entire buttresses and caused some leakage along the downstream face of the dam. Except for buttress No. 5, at least one and as many as four major cracks occurred along each of the buttresses. Principal cracks were accompanied by major concrete spalling, and were up to 2 cm in width. No damage was reported in the lower part of the webs. Cracks were most frequent between El. 258.25 m and El. 264.25 m, and at the aforementioned change in web slope (El. 262.25 m). At the dam crest (El. 276.25 m), some of the concrete slabs of the roadway cracked and spalled, including longitudinal cracking along the downstream curbstone. The parapet wall at the top of buttress No. 11 failed and was tilted toward downstream. The guard house at the center of the dam crest was completely destroyed.

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

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

The dam was equipped with plumb lines through five of its buttresses, 38 inclinometer stations, and over 100 joint monitoring stations (see Fig. 14.1). The latter were installed to measure any relative movements of the vertical joints between buttresses and between buttresses and thrust blocks. Other instrumentation included uplift pressure cells, piezometers, weirs and seepage measuring devices, and concrete temperature monitoring systems. Unfortunately, no topographic survey control stations had been left in place that would have allowed the monitoring of the global position of the dam with respect to the valley walls.

One of the five plumb lines became non-operational as a result of the failure of fasteners holding its protective tubing. Maximum permanent relative horizontal displacement in the upstream downstream direction, measured at the top of the buttresses, was about 0.4 inch (10 mm). Hence, the earthquake caused non-recoverable movements of some buttress blocks, although of very small magnitude. Hysteresis loops of earthquake-induced crest displacements obtained at plumb lines showed a bi-directional amplitude of about 25 mm. Horizontal movements calculated from inclinometer readings were consistent with those measured from the plumb lines. Relative movements at joint level in directions parallel or perpendicular to the contraction joints largely exceeded the range of the measuring instruments at most of the recording stations. In general, each block of the dam moved toward downstream with respect to the block on its left. Cumulative displacements of between 45 and 70 mm were hand-measured. It was concluded that either the entire foundation experienced permanent downward movement between the left and right abutments, or that most dam buttresses became slightly tilted toward the left abutment (Fig. 14.9). Uplift water pressures were found to have strongly decreased after the earthquake, perhaps as a result of the closure of joints or of increased compressive forces across seepage paths. This finding was interpreted favorably with respect to the overall safety of the dam.

## EMERGENCY PROCEDURES AND POST-EARTHQUAKE REPAIRS

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

The primary purpose of long-term repair work was to stop leakage through the buttress blocks and restore shear strength in the cracked sections to assure monolithic action of the buttresses. First, all buttresses were water-tested at 200 kPa above hydrostatic pressure to assess the extent of cracking. It was found that about 80 of the cracks required treatment. For each of those cracks, epoxy-grouting (with Rodur), using an average of 20 boreholes per crack, was accomplished. Rodur can bond wet concrete surfaces in cracks and at low temperature. About 92 metric tons of grout were used. In addition to grouting, twelve post-tensioned VSL anchors with 1.00 MN capacity were installed through each of the buttresses. The average length of those anchors was 40 m, with a maximum inclination of 22 degrees with respect to the vertical. Bonded length of the anchors was 12 m. Overall, 234 anchor holes with a cumulative length of 9450 m were drilled, and 738 metric-tons of cement and 36 metric-tons of additives were used for tendon grouting. All repair work was completed within eight and a half months.

## CONCLUSION

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

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

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

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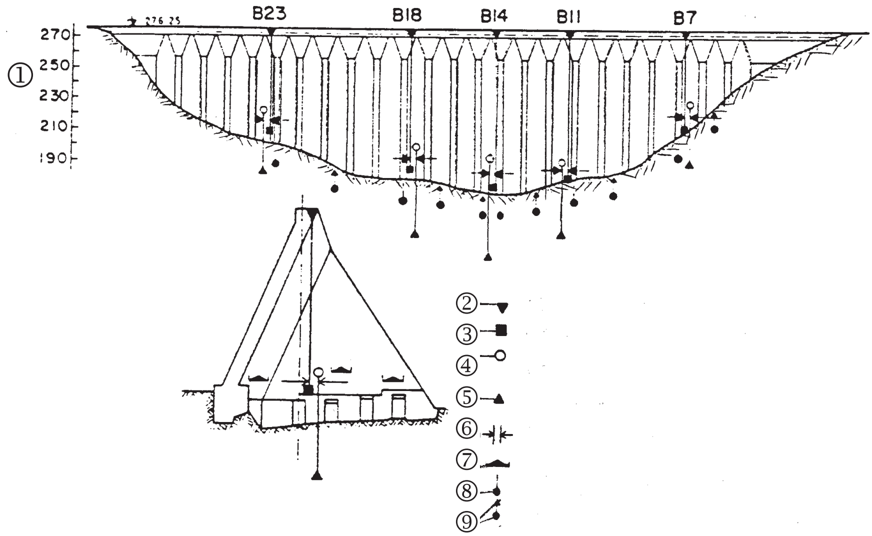


Fig. 14.1

Sefid Rud Dam – Longitudinal Section, Cross-Section, and Location of Monitoring Instruments

*Barrage Sefid Rud – Profil longitudinal, profil en travers, et emplacement des appareils d'auscultation*

| | |
|---|---|
| 1) Elevation (m) | *1) Cote (m)* |
| 2) Suspension point of direct plumb line | *2) Point de suspension du pendule direct* |
| 3) Plumb bob | *3) Masse du pendule direct* |
| 4) Float Vessel for Inverted Plumb Line | *4) Cuve du flotteur du pendule inversé* |
| 5) Anchoring | *5) Ancrage du pendule inversé* |
| 6) Reading Station | *6) Station de lecture* |
| 7) Clinometer | *7) Clinomètre* |
| 8) Uplift Pressure Measuring Gauge | *8) Appareil de mesure des sous-pressions* |
| 9) Piezometric Level Monitoring Well | *9) Puits de mesure des niveaux piézométriques* |

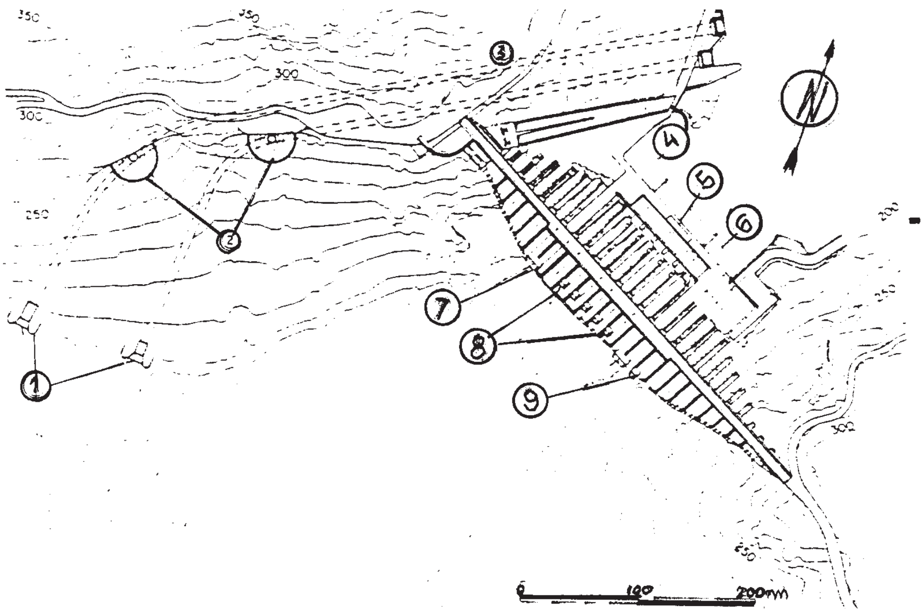


Fig. 14.2

General Layout of Sefid Rud Dam

*Disposition générale du barrage Sefid Rud*

1) Intakes of the Two Diversion Tunnels
2) Morning Glory Spillways into Diversion Tunnels
3) Orifice Spillways
4) Spillway Chute
5) Powerhouse
6) Platform of Switchyard
7) Bottom and Irrigation Outlet, Left Bank
8) Penstock Intakes
9) Bottom and Irrigation Outlet, Right Bank

*1) Prises des deux galeries de dérivation*
*2) Évacuateurs en tulipe reliés aux galeries de dérivation*
*3) Évacuateurs en charge*
*4) Coursier de l'évacuateur*
*5) Usine*
*6) Plateforme du poste électrique*
*7) Vidange de fond et prise d`irrigation, rive gauche*
*8) Prises des conduites forcées*
*9) Vidange de fond et prise d`irrigation, rive droite*

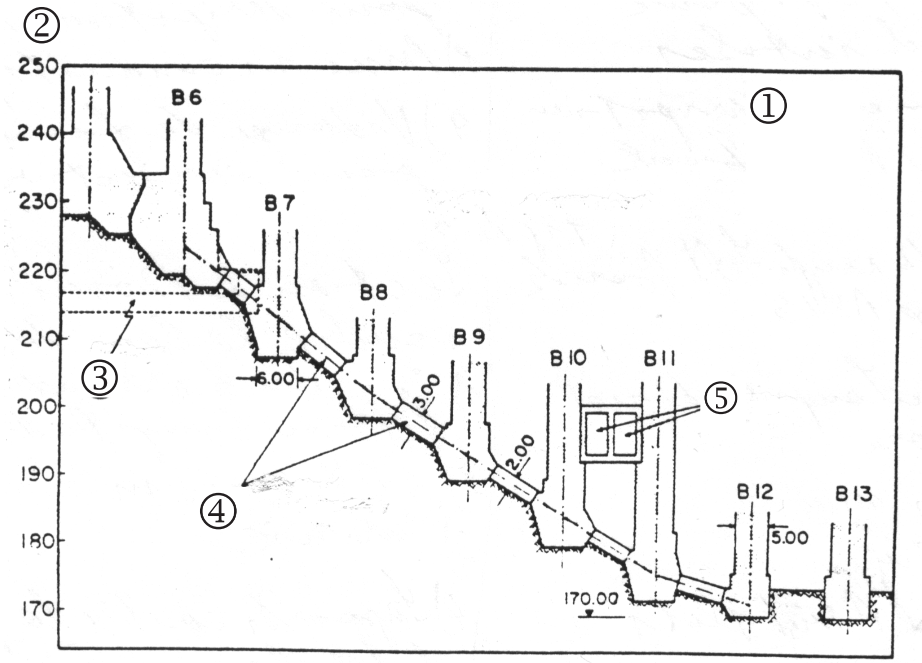


Fig. 14.3

Section through Left Bank along Dam Axis

*Coupe à travers la rive gauche, le long de l'axe du barrage*

| | |
|---|---|
| 1) Longitudinal Section | *1) Coupe longitudinale* |
| 2) Elevation (m) | *2) Cote (m)* |
| 3) Gallery | *3) Galerie* |
| 4) Lateral Thrust Slabs | *4) Semelles de butée latérale* |
| 5) Irrigation Channels | *5) Canaux d`irrigation* |

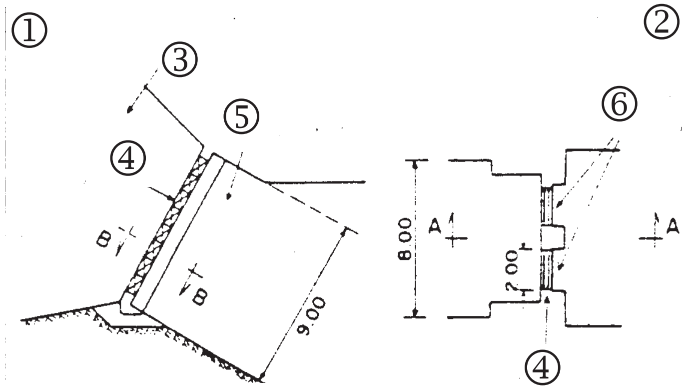


Fig. 14.4
Details of Active Construction Joint
*Détails d'un joint de construction actif*

1) Section A-A
2) Section B-B
3) Buttress
4) Flat Jacks
5) Foundation Block
6) Concrete Supports

*1) Coupe A-A*
*2) Coupe B-B*
*3) Contrefort*
*4) Vérins plats*
*5) Bloc de fondation*
*6) Supports en béton*

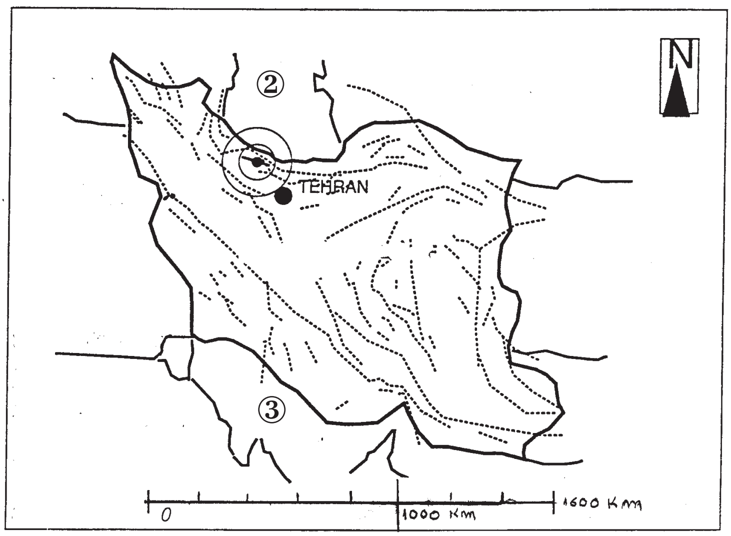


Fig. 14.5
Tectonics of Iran and June 21, 1990 Epicenter
*Tectonique de l`Iran et épicentre du séisme du 21 juin 1990*

1) Islamic Republic of Iran
2) Caspian Sea
3) Persian Gulf

*1) République Islamique d'Iran*
*2) Mer Caspienne*
*3) Golfe Persique*

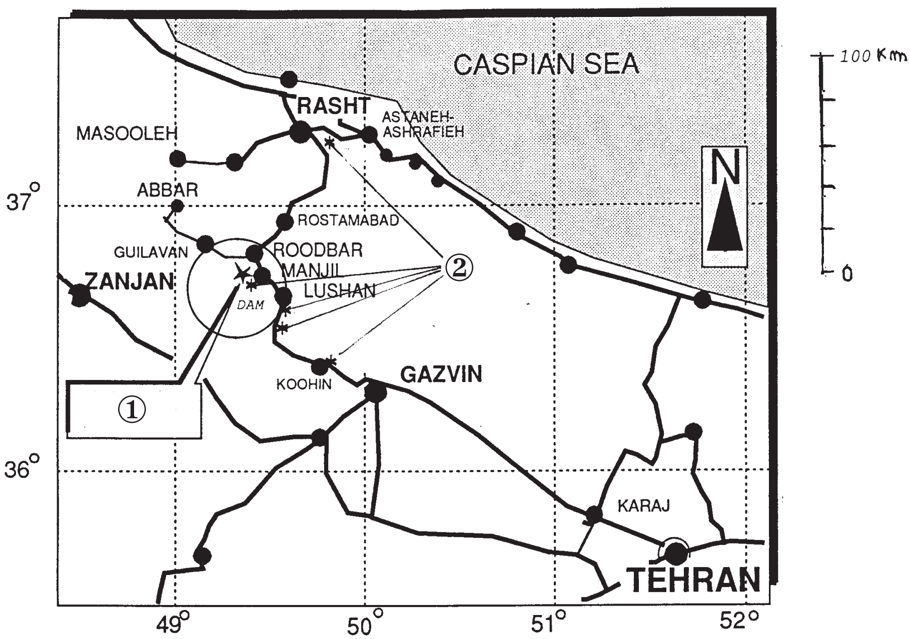


Fig. 14.6

Affected Areas near Epicenter of Manjil Earthquake

*Zones affectées proches de l`épicentre du séisme Manjil*

1) Epicenter, June 21, 1990 — *1) Épicentre, 21 juin 1990*

2) Damaged Areas — *2) Zones endommagées*

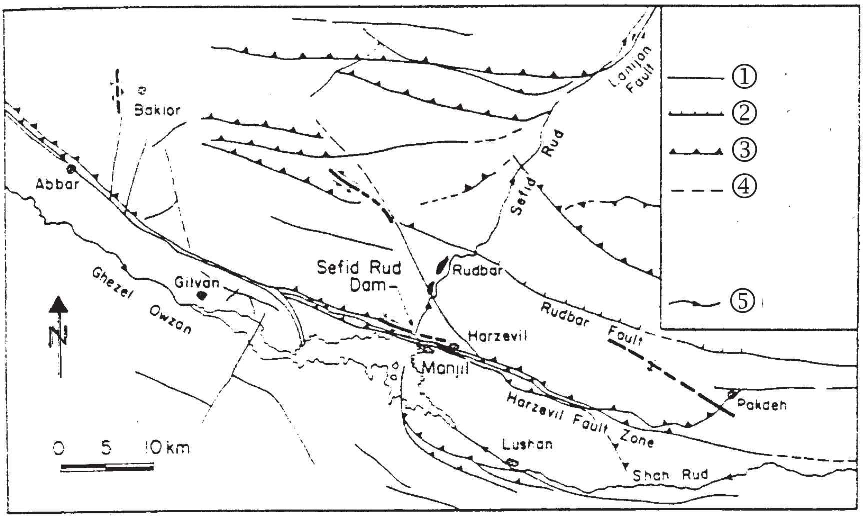


Fig. 14.7

Identified Fault Rupture Zones

*Zones de rupture de failles identifiées*

1) Fault Trace
2) High Angle Reverse Fault
3) Thrust Fault
4) Identified Fault Rupture Caused by 1990 Manjil Earthquake
5) River

*1) Traces des failles*
*2) Faille inverse raide*
*3) Faille avec chevauchement*
*4) Rupture de faille identifiée et causée par le séisme Manjil, 1990*
*5) Rivière*

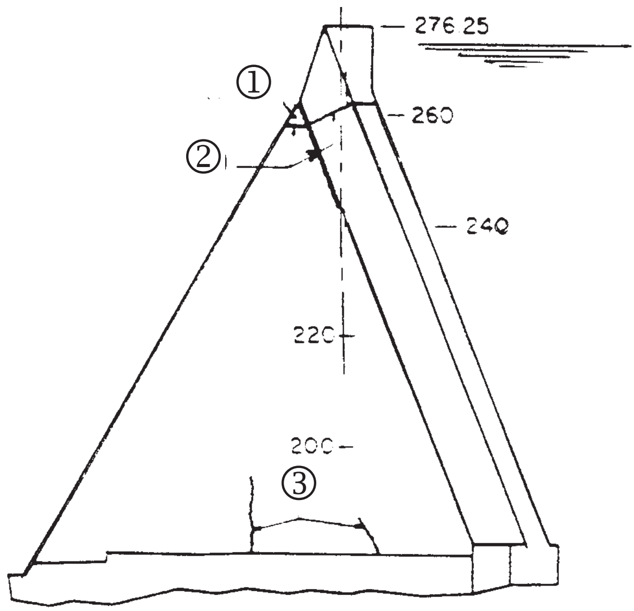


Fig. 14.8

Left Abutment Side, Buttress 15 – Typical Crack Mappings

*Contrefort 15, côté rive gauche – Traces des fissures caractéristiques*

| | |
|---|---|
| 1) Wedge | *1) Coin* |
| 2) Cracks Caused by Earthquake | *2) Fissures causées par le séisme* |
| 3) Old Cracks | *3) Fissures anciennes* |

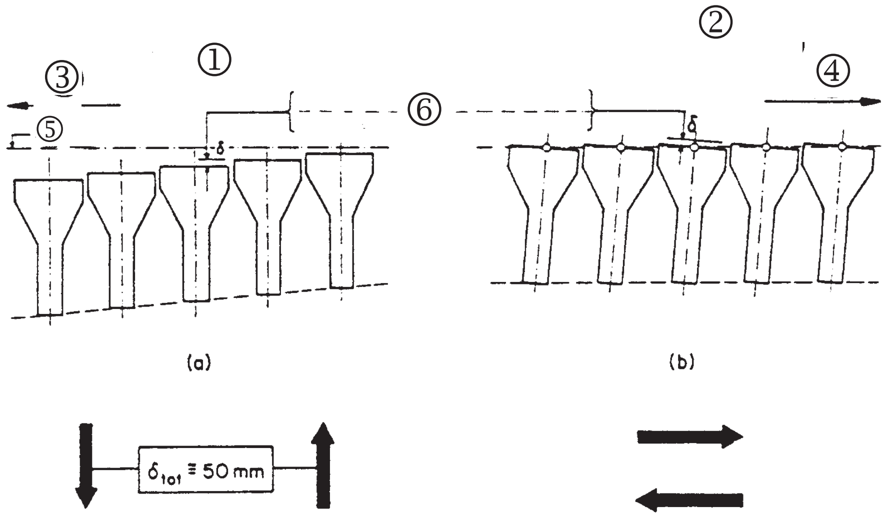


Fig. 14.9
Interpretation of Buttress Movements
*Interprétation des mouvements du contrefort*

1) Mechanism No 1
2) Mechanism No 2
3) Right Bank
4) Left Bank
5) Original Dam Axis
6) More or Less Persistent Offset between Buttresses

*1) Mécanisme No 1*
*2) Mécanisme No 2*
*3) Rive droite*
*4) Rive gauche*
*5) Axe initial du barrage*
*6) Décalage plus ou moins persistant entre les contreforts*

# 15. SHEFFIELD DAM, CALIFORNIA, USA

On June 29, 1925, a magnitude 6.3 earthquake occurred in the vicinity of the City of Santa Barbara, California. The earthquake resulted in 12 deaths and substantial damage. The earthquake was felt over an area of at least 130 000 square km. The epicenter was located about seven miles northwest of Sheffield dam, a 219-m-long embankment, 7.6 m high. The dam, which was composed of silty sand and sandy silt, failed during the earthquake. The failure released about 114 million litres of water, which temporarily flooded the lower part of the city to a depth of about one or two feet before discharging into the sea. Peak acceleration at the site was estimated in 1968 at 0.15 g, but was probably at least 0.25 g, based on estimates provided by more recent attenuation relationships.

## SHEFFIELD DAM

Sheffield Dam was built in 1917 in a ravine north of the City of Santa Barbara. A representative section through the embankment at its maximum height is shown on Fig. 15.1 (Seed et al., 1968). The 219-m-long embankment had a maximum height of 7.6 m. The body of the dam was composed of silty sand and sandy silt, compacted by routing the construction equipment over the fill. The upstream slope was faced with a 1.2-m-thick clay blanket, which was extended 3 m into the foundation to serve as a cutoff. The clay blanket was overlain with a 13 cm concrete facing.

The foundation soils consisted of terrace alluvium, 1.2 to 3 m thick, overlying sandstone bedrock. The alluvium was mainly silty sand and sandy silt containing cobbles varying from 8 to 16 cm in diameter, with some thin layers of clayey sand and gravelly sandy clay. It was reported that the upper 0.3 to 0.45 m of foundation topsoil were somewhat looser than the underlying deposits and that there had been no formal stripping of the upper soil layers prior to construction of the embankment (Seed et al., 1968; U.S. Army Corps of Engineers, 1949).

Seed et al. (1968) reported that seepage had been noted near the downstream slope and in the area beyond the toe before the earthquake. Seepage around and beneath the cutoff was reported to have resulted in saturating the lower part of the embankment and the foundation (Willis, 1925). At the time of the earthquake, the depth of water in the reservoir was about 4.6 to 5.5 m.

## THE JUNE 29, 1925 SANTA BARBARA EARTHQUAKE

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

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

The Sheffield Reservoir formed by the dam was capable of impounding a maximum of about 170 million litres of water. At the time of the earthquake, the depth of water in the reservoir was only about 4.6 to 5.5 m, so that the failure released about 114 million litres of water which temporarily flooded the lower parts of the city to a depth of about 0.3 or 0.6 m before discharging into the sea.

There were no eye-witnesses when the failure occurred. However, after inspecting the damage, O’Shaughnessy (1925) reported that "a great mass of the center, about 90 m in length, slid downstream perhaps 30 m". Herbert Nunn (1925), City Manager of the City of Santa Barbara, wrote: "After examination by several prominent engineers, the conclusion has been reached that the base of the dam had become saturated, and that the shock of the earthquake....had opened vertical fissures from base to top; the water rushing through these fissures simply floated the dam out in sections". Willis (1925) reported: "The foundations of the dam had become saturated and the rise of the water as the ground was shaken formed a liquid layer of sand under the dam, on which it floated out, swinging about as if on a hinge".

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

## INSTRUMENTATION AND STRONG MOTION RECORDS

Sheffield Dam was not instrumented at the time of occurrence of the earthquake. In their 1969 reanalysis of the dam failure, Seed, Lee and Idriss used empirical correlations between peak ground accelerations and Rossi-Forell intensity to estimate the peak ground acceleration (PGA) in the vicinity of the Sheffield Dam and assigned it a value of 0.15 g. They estimated the duration of significant shaking at between 15 and 18 seconds. In view of more recent knowledge, modern estimates of mean PGA for a site located only 11.2 km from the epicenter of a magnitude 6.3 earthquake would be of the order of 0.25 g, based on a weighted average of five recently published well accepted PGA attenuation equations. Actual PGA may have been higher, since 0.25 g represents a mean estimate.

## CONCLUSIONS

Sheffield Dam, a 7.6-m-high compacted silty sand and sandy silt embankment built on a similar foundation, was shaken by a magnitude 6.3 earthquake and failed completely. This is one of the rare known cases of complete failure of a dam as a result of earthquake loading.

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

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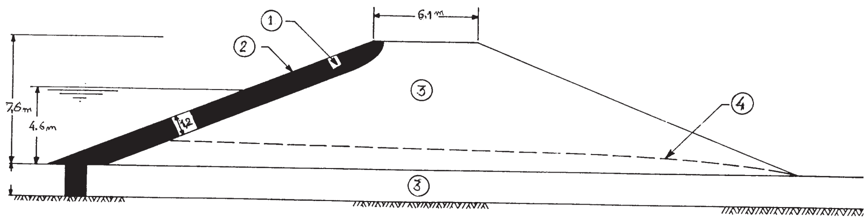


Fig. 15.1

Sheffield Dam – Cross Section through Embankment

*Barrage Sheffield – Coupe à travers le remblai*

| | |
|---|---|
| 1) Clay Blanket | *1) Tapis d'argile* |
| 2) Concrete Facing | *2) Revêtement en béton* |
| 3) Sandy Silt to Silty Sand | *3) Silt sableux et sable silteux* |
| 4) Piezometric surface | *4) Surface piézométrique* |

# 16. VERMILION DAM, CALIFORNIA, USA

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

No visible damage resulted at this modern, well-compacted, earthfill dam constructed on top of up to 82 m of coarse, dense alluvium that had been deposited by several advances and retreats of the Mono Creek Glacier during Pleistocene times. Repeated surveys of benchmarks, however, showed that settlements had occurred, the maximum crest settlement having been about 5 cm. No appreciable increase in seepage was reported.

## VERMILION DAM

Vermilion Dam (Lake Thomas A. Edison) is located on Mono Creek, a tributary of the South Fork of the San Joaquin River, on the western slope of the Sierra Nevada in Fresno County, CA. Vermilion Dam is a 50-m-high, zoned, compacted, sandy earthfill embankment, 1290 m long, owned and operated by Southern California Edison Company (Edison). Its slopes (Fig. 16.1) are 2.25 to 3:1 (horizontal to vertical) upstream and 2.0 to 2.5:1 (h to v) downstream. Its crest is at an elevation of 2331.9 m above sea level.

Lake Thomas A. Edison provides storage of about $154.10^6 m^3$. Final design of Vermilion Dam was prepared in 1952 and construction was completed in 1954. A detailed paper describing the unusual and complex foundation conditions was prepared by Terzaghi & Leps (1960). The foundation of the Dam is its most interesting feature. It was reported as consisting of highly varied layers and lenses of fluvial and glacio-fluvial silts, sands, gravels and boulders, all of which have been formed, reworked, and consolidated by several advances and retreats of the Mono Creek Glacier. The sediments are from 30 to 80 m thick. They are underlain by granodiorite of the Sierra Nevada batholith.

Because of the glacial preloading of the foundation and the generally coarse, granular texture of the thick glacio-fluvial deposits, there was assurance regarding the structural competence of the foundation soils. The problem in relation to creating a safe dam on the site had been to minimize and control the exit of the probable foundation seepage. It is apparent from over 40 years of operating experience that underseepage has been adequately controlled. The maximum seepage flow has not exceeded about 170 lit/s and has remained stable. This was achieved both by constructing an extensive, impervious, rolled fill blanket from the core of the dam upstream along the reservoir bottom for distances of up to 430 m, together with upstream cutoff trenches to a shallow but discontinuous impervious stratum of varved silt, and by generous provision of a deep, filtered, toe drain, together with gravity-discharge, relief wells.

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

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

## SEISMICITY

The California Division of Mines and Geology (1991) has reported that, since 1978, the Bishop-Mono Lake area has been one of the most seismically active regions in California, with local magnitudes ranging as high as 6.5. A map of pertinent regional faults is shown on Fig. 16.2. Faults of primary capability with regard to Vermilion Dam are in a zone located 20 to 40 km to the east and northeast of the dam.

## THE MAY 27, 1980 EARTHQUAKE

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

## EARTHQUAKE EFFECTS AND OBSERVED PERFORMANCE

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

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

While a subsequent examination of the Dam failed to find any visible damage, a resurvey of monuments on the surface of the Dam was carried out and carefully reviewed. The survey data indicated that the maximum settlement, occurring at the maximum dam section, was about 5 cm. Fig. 16.4 illustrates the chronology of maximum crest settlement since construction of the dam was completed in 1954.

## INSTRUMENTATION AND STRONG MOTION RECORDS

At the time, there was a minimal amount of seismic instrumentation on the westerly slope of the Sierra Nevada, except at Vermilion Dam. The greatest concentration of such instrumentation had been placed east of the Sierra, in the known, seismically active area between Bishop and Mono Lake, CA. At Vermilion Dam, in addition to Edison' s SMA-2, there was an array of strong motion instruments which had been placed by the California Division of Mines and Geology (CDMG) as part of the SMIP. The array was located on the dam crest, on berms and at the toe. Unfortunately, the SMIP array malfunctioned in May 1980, and no data were recorded. On the east side of the Sierra, however, many records were obtained. They were published by the CDMG (1980). Material regarding peak recorded acceleration attenuation at various distances from the 1980 epicenters is displayed on Fig. 16.3, up to epicentral distances of 30 km. The available data appear to check reasonably well with published attenuation equations, such as the Leps-Jansen chart (1984).

## CONCLUSIONS

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

## ACKNOWLEDGMENTS

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

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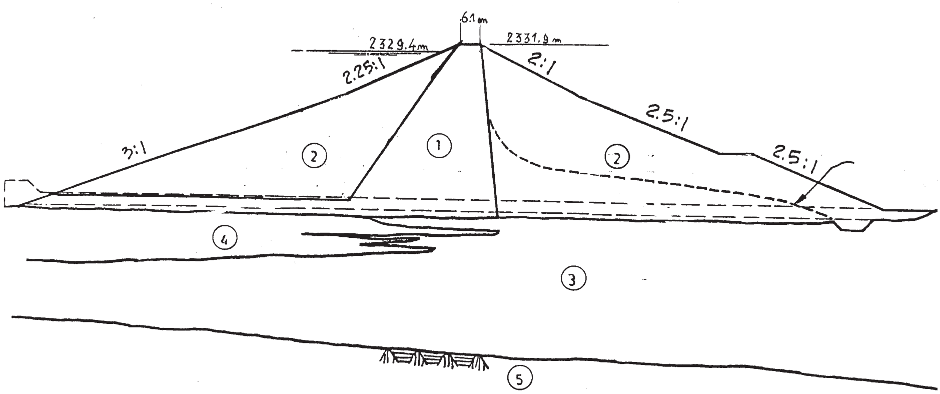


Fig. 16.1

Vermilion Dam – Cross Section

Barrage Vermilion – Coupe transversale

| | |
|---|---|
| 1) Silty Sand | *1) Sable silteux* |
| 2) Gravely Sand | *2) Sable graveleux* |
| 3) Glaciary Deposit | *3) Dépôt glaciaire* |
| 4) Varved silt | *4) Varve silteuse* |
| 5) Bedrock | *5) Fond rocheux* |

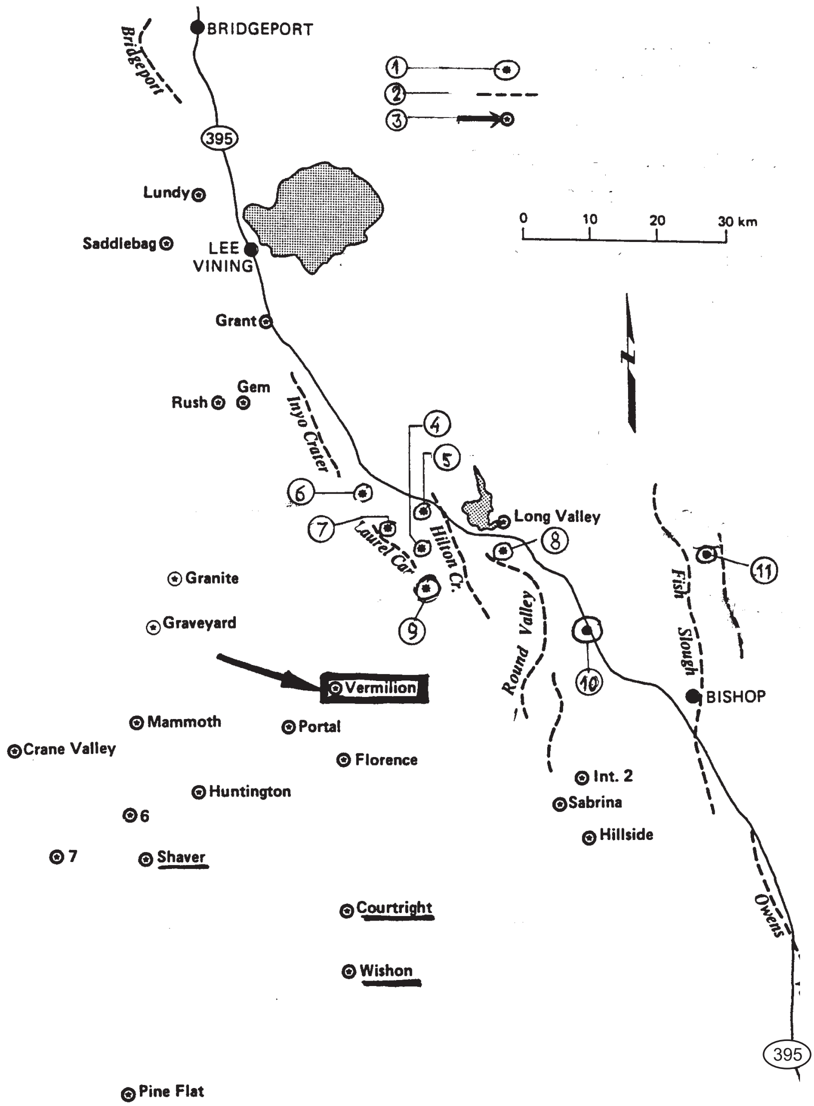


Fig. 16.2

Major Seismic Events concerning Vermilion Dam

Événements sismiques majeurs concernant le barrage Vermilion

| | |
|---|---|
| 1) Accurately Located Epicenters | *1) Épicentres localisés avec précision* |
| 2) Fault | *2) Faille* |
| 3) Dam | *3) Barrage* |
| 4) M 6.4, 5/25/80, 12 :44 p.m. | *4) M 6.4, 25/5/80, 12h44 p.m.* |
| 5) M 6.3, 5/25/80, 9 :33 a.m. | *5) M 6.3, 25/5/80, 9h33 a.m.* |
| 6) M 6.0, 5/25/80, 9 :49 a.m. | *6) M 6.0, 25/5/80, 9h49 a.m.* |
| 7) M 5.8, 9/30/81, 4 :53 a.m. | *7) M 5.8, 30/9/81, 4h53 a.m.* |
| 8) M 5.7, 10/04/78 | *8) M 5.7, 04/10/78* |
| 9) M 6.3, 5/27/80, 7 :51 a.m. | *9) M 6.3, 27/5/80, 7h51 a.m.* |
| 10) M 6.2, 11/23/84 | *10) M 6.2, 23/11/84* |
| 11) M 6.1, 7/02/86 | *11) M 6.1, 02/7/86* |

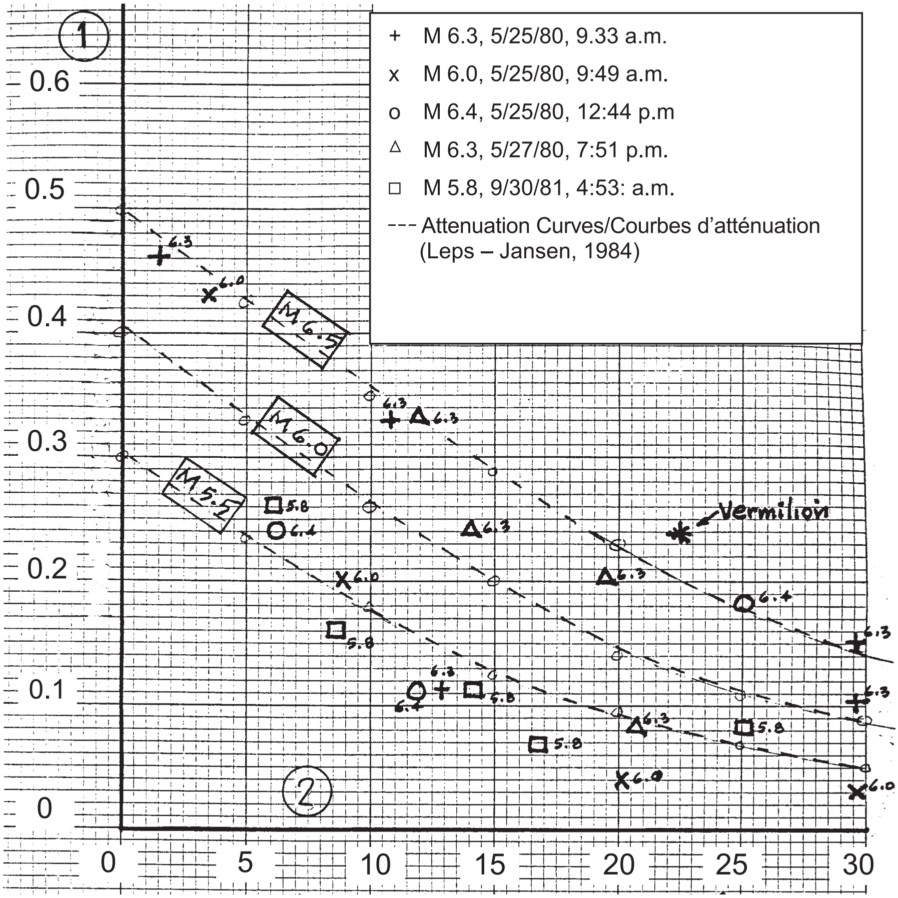


Fig. 16.3

Estimated PGA Data

*Accélérations de pointe du sol – Données estimées*

1) Peak Ground Acceleration (g) — *1) Accélérations de pointe du sol (g)*

2) Epicentral Distance (km) — *2) Distance de l'épicentre (km)*


Fig. 16.4

Vermilion Dam - Chronological Plot of Crest Settlement

*Barrage Vermilion - Graphique chronologique des tassements de crête*

| | |
|---|---|
| 1) Initial Benchmark Elevation | *1) Cote initiale du repère* |
| 2) May 1980 Earthquakes | *2) Séismes de mai 1980* |
| 3) Elevation (m) | *3) Cote (m)* |

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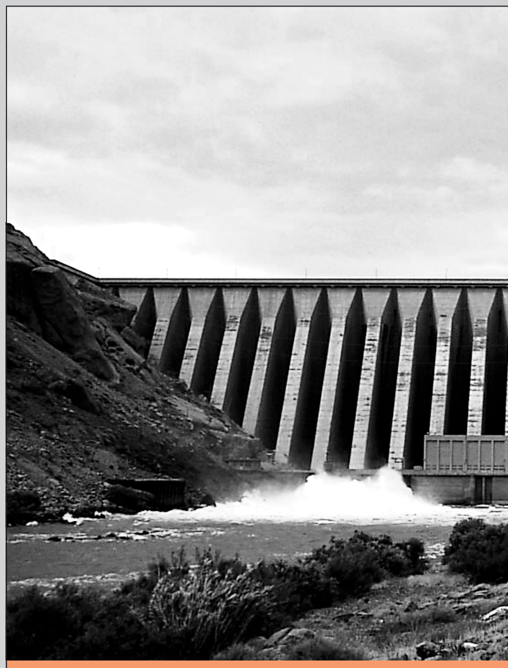

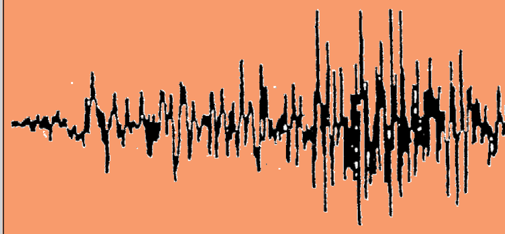

DESIGN FEATURES OF DAMS TO RESIST SEISMIC GROUND MOTION
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