EARTHQUAKE ENGINEERING FOR CONCRETE DAMS

ANALYSIS, DESIGN, AND EVALUATION

ANIL K. CHOPRA

WILEY Blackwell

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Analysis, Design, and Evaluation

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Cover Photograph: Mauvoisin Dam, a 250-m-high arch dam, is located in the Swiss Alps. It is perhaps the best instrumented dam in the world

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This book is dedicated to the memory of my mentors:

Ray W. Clough

Joseph Penzien

Nathan M. Newmark

Anestis S. Veletsos

Emilio Rosenblueth

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PART III: DESIGN AND EVALUATION

Preface

Concerns about the seismic safety of concrete dams has been growing during the past few decades, partly because the population at risk in locations downstream of major dams continues to expand, but also because it is increasingly evident that the design concepts used at the time most existing dams were built were inadequate. During this time span the knowledge of the complex nature and intensity of ground motions has been increasing rapidly, as thousands of recordings have now been accumulated. It is now widely recognized that ground motions intense enough to cause structural damage should be anticipated at many dam sites, and it has become apparent that the seismic designs of most dams did not fully recognize the hazard.

The structural damage sustained by Koyna Dam during an earthquake in 1967 was of profound significance to the development of earthquake engineering for concrete dams. A modern dam, designed according to analysis procedures and design criteria that represented "standard" practice worldwide at the time, had been damaged by ground shaking that was intense, but by no means extreme. It was clear that the design forces had little resemblance to how the dam responded during the earthquake. The experience at Koyna Dam was a watershed event in the sense that it dispelled the myth – at that time – among many engineers that these massive dams are immune to earthquake damage, and motivated the development of dynamic analysis procedures for concrete gravity dams, eventually revolutionizing earthquake engineering for all types of concrete dams.

As a result, earthquake analysis and design of concrete dams has progressed from static force methods involving the use of seismic coefficients, to procedures that now recognize the dynamics of dam–water–foundation systems. It is the story of this progress that is presented in this book.

This book provides a comprehensive, integrated view of this progress currently scattered in hundreds of research publications. It was conceived as a reference book for graduate students, researchers, and professional engineers. It should help graduate students study the subject before embarking on their own research on earthquake engineering for concrete dams. Researchers in this field should gain new insights and improved understanding of the subject. Professional engineers should develop a better understanding of the limitations of the various methods of dynamic analysis used in practice, and become familiar with modern methods that overcome these limitations.

The book is organized into three parts: I. Gravity Dams; II. Arch Dams; and III. Design and Safety Evaluation. The objectives of Parts I and II are to (i) develop response spectrum analysis and response history analysis procedures for concrete dams; (ii) develop an understanding of the dynamics of dams, leading to identification of system parameters that influence their dynamic response; (iii) demonstrate the effects of dam–water–foundation interaction on earthquake response; and (iv) identify factors that must be included in earthquake analysis of concrete dams. In Part I, these topics are presented in the context of two-dimensional models, which may be appropriate for gravity dams. In Part II, they are presented for three-dimensional models, applicable to all types of dams; arch, buttress, and gravity. The objectives of Part III are to (i) examine critically the definitions of design earthquakes according to various regulatory bodies and professional organizations; (ii) present modern methods for selecting ground motions; and (iii) illustrate application of dynamic analysis procedures to the design of new dams and safety evaluation of existing dams.

The book provides a comprehensive view of the subject with many references to the published literature. However, Parts I and II are based primarily on the research of several doctoral students at the University of California, Berkeley, who graduated in the year noted:

- Partha Chakrabarti, 1973
- John F. Hall, 1980
- Gregory L. Fenves, 1984
- Ka-Lun Fok, 1985
- Liping Zhang, 1990
- Han-Chen Tan, 1995
- Arnkjell Løkke, 2018

and on the work of visiting researcher,

- Jinting Wang (2008).

This book has been influenced by my own research experience in collaboration with my doctoral students, and by my experience in consulting on many projects worldwide. Over the period 1970–1995, my research on earthquake engineering for concrete dams was supported by the National Science foundation and U.S. Army Corps of Engineers.

I remain grateful to the University of California at Berkeley for the privilege of serving on its faculty.

– Anil K. Chopra

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I am grateful to several individuals who helped in preparation of this book:

- Dr. Arnkjell Løkke, whose Ph.D. research is the basis for Chapter 11, prepared many of the figures in Chapters 2–7.
- Dr. N. Simon Kwong participated in the preparation of Chapter 13 in several ways; he generated numerical results, developed figures, provided advice, and reviewed drafts. He also reviewed the first part of Chapter 12 and developed its figures.
- Professor Jinting Wang provided information on seismic design of Dagangshan Dam. Based on original reports in Chinese, he prepared the first draft of Section 14.4.
- Professor Pierre Léger advised on a broad range of topics in several ways. He was generous in sharing his vast knowledge over many long conversations and in providing many publications; his command of the published literature was truly impressive. He also reviewed Chapters 11 and 12.
- Dr. Robin K. McGuire reviewed the first part of Chapter 12 and advised on seismic risk analysis concepts and methods.
- Larry K. Nuss advised on several practical aspects of design and evaluation of concrete dams, and provided publications by several government agencies.
- Claire M. Johnson, a partner from beginning to end, prepared and edited the text, assembled the manuscript, and helped in too many ways to enumerate here.
- Clare Romeo served as the copy editor and proofreader.

1
Introduction

1.1 EARTHQUAKE EXPERIENCE: CASES WITH STRONGEST SHAKING†

As far as can be determined, no large concrete dam with full reservoir has been subjected to extremely intense ground shaking. The closest to such an event was the experience at Koyna (gravity) Dam (Figure 1.1.1), with the reservoir nearly full, during the 1967 earthquake (Chopra and Chakrabarti 1973). Ground accelerations recorded at the dam site during a nearby earthquake of magnitude 6.5 had a peak value of 0.38 g in the stream direction and strong shaking lasted for 4 sec. Significant horizontal cracking occurred through a number of taller non-overflow monoliths at or near the elevation where the downstream face changes slope; however, the dam continued to retain the reservoir even though the water level was 25 m above the cracks (Figure 1.1.2). A similar experience had occurred in 1962 at Hsinfengkiang (buttress) Dam (Figure 1.1.3) during a magnitude 6.1 earthquake in close proximity. Although not recorded, ground motions were probably quite intense causing cracking at 16 m below the crest (Figure 1.1.4); the dam continued to retain the reservoir though the water level was 3 m above the cracks.

A few dams have withstood very intense ground shaking with little or no damage because of their unusual design or low water level. Perhaps the strongest shaking experienced by a concrete dam to date was that at Lower Crystal Springs Dam, a 42-m-high curved gravity structure (Figure 1.1.5) dam with nearly full reservoir, located within 350 m of the San Andreas fault that caused the magnitude 7.9 1906 San Francisco earthquake. Built with interlocking concrete blocks, the dam was undamaged, even though its reservoir was full. However, the earthquake resistance of this dam greatly exceeds that of typical gravity dams due to its curved plan and a cross section that was designed thicker than normal in anticipation of future heightening, which was never completed; a section view of this dam is shown in Figure 1.1.6.

† The first part of this section is adapted from a National Research Council report (1990).

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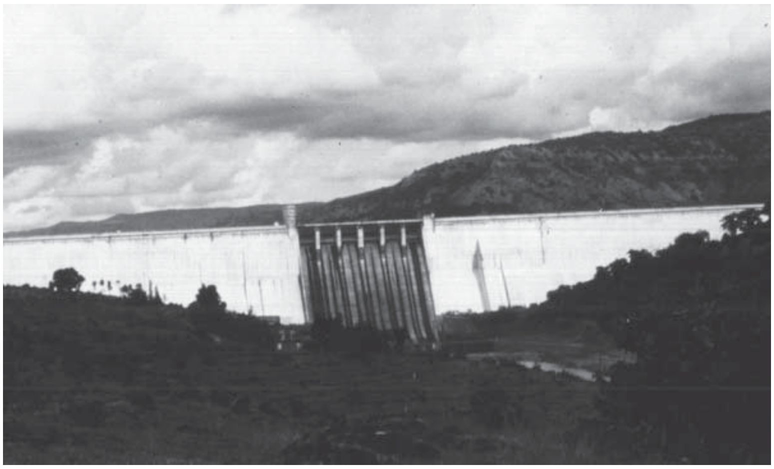


Figure 1.1.1 Koyna Dam, India, constructed during 1954–1963; this dam is 103 m high and 853 m long.

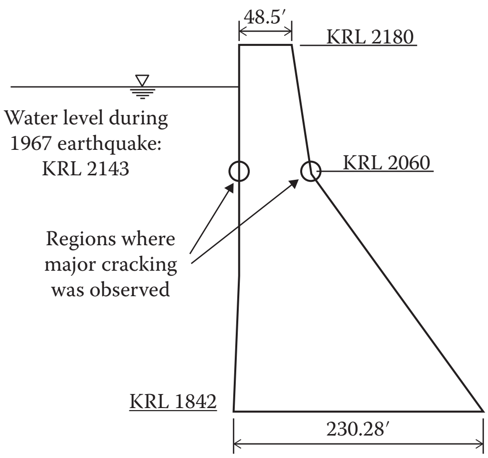


Figure 1.1.2 Cross section of Koyna Dam showing water level during 1967 earthquake and regions where principal cracking at the upstream and downstream faces was observed. Source: Adapted from National Research Council (1990).

Figure 1.1.3 Hsinfengkiang Dam, China. Completed in 1959, this dam is 105 m high and 440 m long.

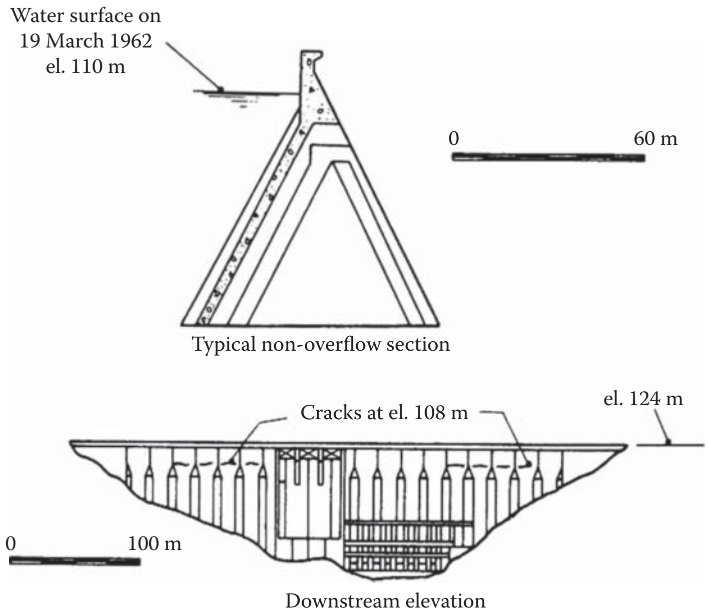


Figure 1.1.4 Cracking in Hsinfengkiang Dam, China, due to earthquake on March 19, 1962. Source: Adapted from Nuss et al. (2014).

Figure 1.1.5 Lower Crystal Springs Dam, California, USA. Built in 1888, this 45-m-high curved-gravity dam is located within 350 m of the San Andreas Fault, which is under the reservoir, oriented roughly parallel to the dam.

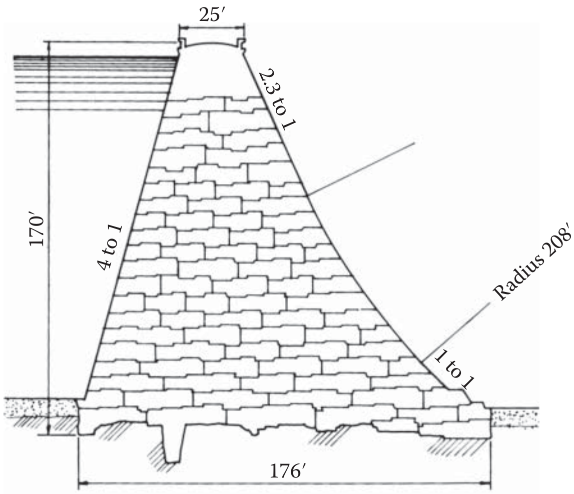


Figure 1.1.6 Section view of the Lower Crystal Springs Dam (adapted from Nuss et al. [2014] and Wieland et al. [2004]).

Another example of a concrete dam subjected to very intense shaking is the 113-m-high Pacoima (arch) Dam (Figure 1.1.7). During the 1971 magnitude 6.6 San Fernando earthquake, an accelerograph located on the left abutment ridge recorded a peak acceleration of 1.2 g in both horizontal components and 0.7 g vertical, with strong shaking lasting for 8 sec, suggesting that the excitation at the dam–foundation‡ interface – which was not recorded – must have been very intense. However, the only visible damage to the dam was a 3/8 in. opening of the contraction joint on the left thrust block and a crack in the thrust block. During the 1994 magnitude 6.7 Northridge earthquake, peak accelerations recorded ranged from 0.5 g at the base of the dam to about 2.0 g along the abutments near the crest. The damage sustained was more severe than in 1971. The contraction joint between the dam and the thrust block in the left abutment again opened, this time by 2 in. at the crest level (Figure 1.1.8), decreasing to 1/4 in. at the bottom of the joint (60 ft

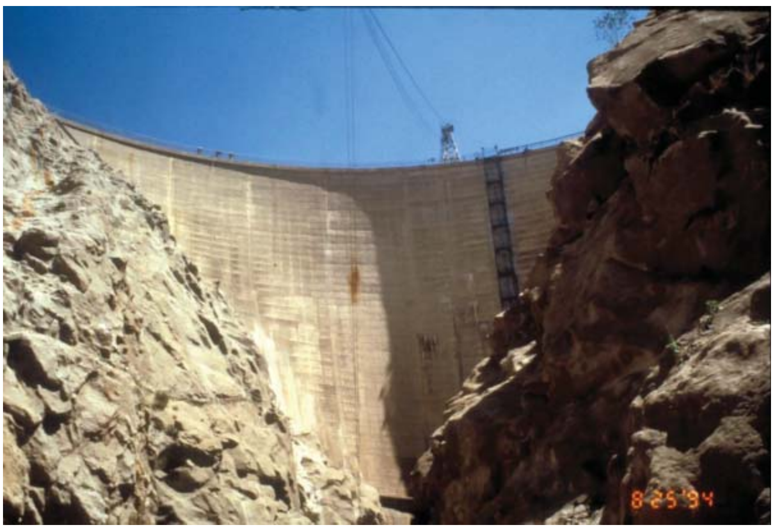


Figure 1.1.7 Pacoima Dam, California, USA. Completed in 1929, this dam is 113 m high and 180 m long at the crest.

‡ The word "foundation" denotes the rock that supports the dam.

below the crest), at which point a large crack extended down diagonally through the lower part of the thrust block to meet the foundation (Figure 1.1.9). The good performance of the dam can be attributed primarily to the low water level – 45 m below the dam crest – at the time of both earthquakes. Additional information is available in Scott et al. (1995).

Figure 1.1.8 Two-inch separation between Pacoima Dam Arch (left) and the thrust block (right) on the left abutment (Scott et al. 1995).

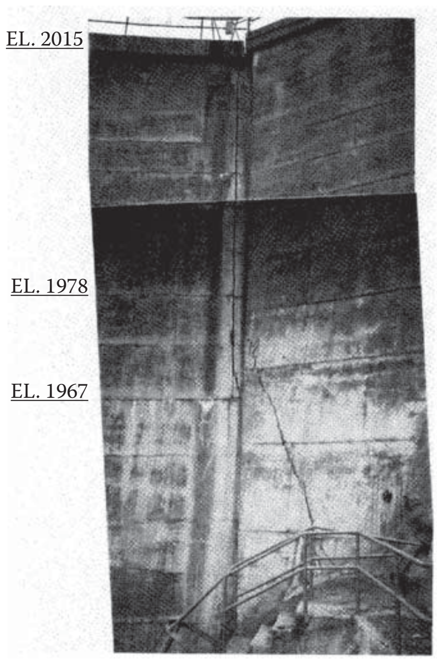
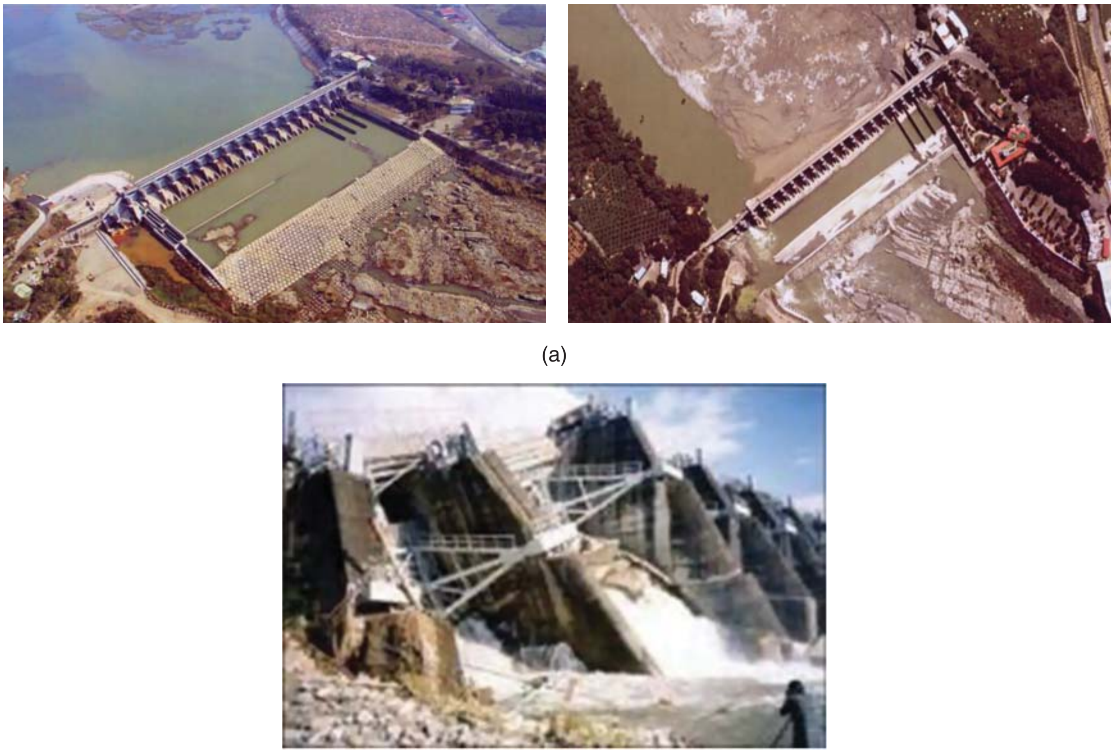


Figure 1.1.9 Crack at the joint between the Pacoima Dam arch and the thrust block and diagonal crack in the thrust block (Scott et al. 1995).



(a)

(b)

Figure 1.1.10 Shih-Kang Dam, Taiwan, (a) before and after the Chi-Chi earthquake; (b) close-up of damaged bays. Completed in 1977, this gated spillway is 21 m high and 357 m long. (a) Two photos courtesy of C.-H. Loh, National Taiwan University, Taiwan. (b) Photo courtesy of USSD.org.

Shih-Kang Dam in Taiwan (Figure 1.1.10) – a 70-ft (21.4-m)-high, 18-bay gated spillway – located directly over a branch of the Che-Lung-Pu fault that caused the 1999 magnitude 7.6 Chi-Chi earthquake represents the first known dam failure during an earthquake. However, this failure was caused primarily by fault rupture, not ground shaking, although it was very intense, as indicated by the peak ground acceleration of 0.5 g recorded at a location 500 m from the dam. During the Chi-Chi earthquake the branch fault ruptured, with a vertical offset of 29 ft (9 m) and a horizontal offset diagonal to the dam axis of about 23 ft (7 m). As a result, bays 16–18 incurred extensive damage, but the damage to the other bays was surprisingly little; spillway piers sustained cracking, simply supported bridge girders came off their bearings, and six gates were inoperable after the earthquake.

It is clear from the preceding observations that concrete dams can be significantly damaged by ground shaking due to earthquakes. They are not as immune to damage as had commonly been presumed prior to the 1967 experience at Koyna Dam. This fact is now universally recognized, and there is much interest in the earthquake performance of concrete dams.

1.2 COMPLEXITY OF THE PROBLEM

The ability to evaluate the effects of earthquake ground motion on concrete dams is essential in order to assess the safety of existing dams, to determine the adequacy of modifications planned to improve existing dams, and to evaluate proposed designs for new dams to be constructed. However, the prediction of performance of concrete dams during earthquakes is one of the most complex and challenging problems in structural dynamics because of the following factors:

1. Dams and the impounded reservoirs† are of complicated shapes, as dictated by the topography of the site (see Figures 1.2.1 and 1.2.2).
2. The response of a dam is influenced greatly by the interaction of the motions of the dam with the impounded water and the foundation, both of which extend to large distances. Thus the mass, stiffness, material damping, radiation damping of the foundation (see Section 1.6), and the earthquake-induced hydrodynamic pressures must be considered in computing the dynamic response.
3. During intense earthquake motions, vertical contraction joints may slip or open; concrete may crack; and separation and sliding may occur at lift joints in concrete, dam–foundation interface, and fissures in foundation rock. These phenomena are highly nonlinear and extremely difficult to model realistically.
4. The response of dams is affected by variations in the intensity and frequency characteristics of the ground motion over the width and height of the canyon; however, this factor cannot be fully considered at present for lack of instrumental records to define the spatial variations of the ground motion.

Considering all these factors, analytical and computational procedures to determine the response of dam–water–foundation systems subjected to ground shaking are presented in this book. A substructure method for linear analysis of two-dimensional (2D) models, usually appropriate for gravity dams, is the subject of Chapters 2–6; and of three-dimensional (3D) models – required for arch dams, buttress dams, and gravity dams in narrow canyons – is the subject of Chapter 8. The Direct Finite-Element Method (FEM) for nonlinear analysis of 2D or 3D dam–water–foundation systems is presented in Chapter 11.

Figure 1.2.1 Olivenhain Dam, California, USA. Completed in 2003, this is a 318-ft-high roller-compacted concrete dam with a crest length of 2552 ft.

† "Reservoir" is the place of storage, not the fluid itself.

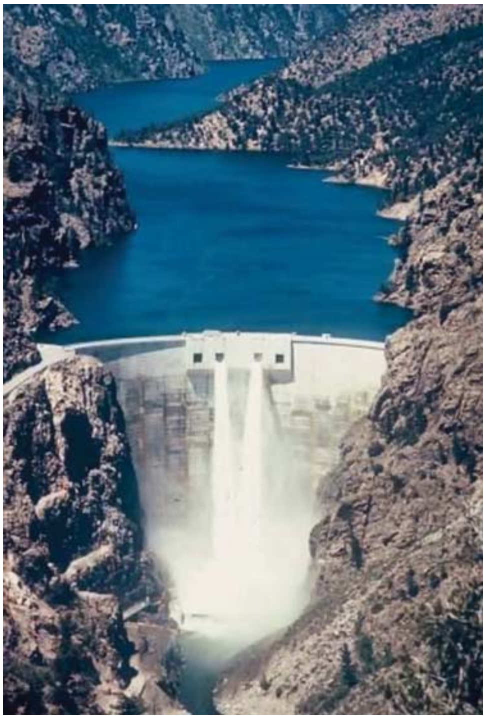


Figure 1.2.2 Morrow Point Dam, Colorado, USA, a 465-ft-high single-centered arch dam.

1.3 TRADITIONAL DESIGN PROCEDURES: GRAVITY DAMS

1.3.1 Traditional Analysis and Design

Concrete gravity dams have traditionally been designed and analyzed by very simple procedures (U.S. Army Corps of Engineers 1958; Bureau of Reclamation 1965, 1966). Earthquake effects were treated simply as static forces and were combined with the hydrostatic pressures and gravity loads. In representing the effects of horizontal ground motion – transverse to the axis of the dam – by static lateral forces, neither the dynamic response characteristics of the dam–water–foundation system nor the amplitude and frequency content of earthquake ground motion were recognized. Two types of static lateral forces were included. Forces associated with the weight of the dam were expressed as a product of a seismic coefficient – which was typically constant over the height, with a value between 0.05 to 0.10 – and the weight of the portion of the dam being considered. Water pressures, in addition to the hydrostatic pressure, were specified as the product of the seismic coefficient and a pressure coefficient that was based on assumptions of a rigid dam and incompressible water. Finally, interaction between the dam and the foundation was not considered in computing the aforementioned earthquake forces.

The traditional design criteria required that an ample safety factor be provided against overturning, sliding, and overstressing; in particular, compressive stresses should be less than one-fourth of the compressive strength. Usually tension was not permitted, and even if it was, the allowable tension was so small that the possibility of cracking of concrete was not considered.

1.3.2 Earthquake Performance of Koyna Dam

Koyna Dam in India was designed by the traditional static analysis procedure using a seismic coefficient of 0.05. Even though a "no-tension" criterion was satisfied in the design procedure, as mentioned earlier, the earthquake of December 11, 1967 caused significant horizontal cracks in the upstream and/or downstream faces of a number of the taller non-overflow monoliths near the elevation at which there is an abrupt change in slope of the downstream face (Figure 1.1.2).

To understand why the damage occurred, the dynamic response of the tallest non-overflow monolith to the recorded ground motion was computed assuming linear behavior. The results indicated large tensile stresses on both faces, with the greatest values near the elevation where the slope of the downstream face changes abruptly. These computed stresses (shown in Figure 1.3.1), which exceeded 600 psi on the upstream face and 1000 psi on the downstream face, were about two to three times the estimated tensile strength – 350 psi – of the concrete at that elevation. Hence significant cracking consistent with what was observed after the earthquake could have been anticipated. A similar analysis of the overflow monoliths indicated that cracking should not have occurred there, which is also consistent with the observed behavior.

1.3.3 Limitations of Traditional Procedures

It is apparent from the preceding discussion that the dynamic stresses that develop in gravity dams bear little resemblance to the results obtained from traditional static design procedures. In the case of Koyna Dam, no tensile stresses were expected when designing the dam for earthquake forces based on a seismic coefficient of 0.05, uniform over the height; however, the earthquake caused significant tensile cracking in the dam. This discrepancy is the result of using too small a seismic coefficient and not recognizing the amplification of acceleration over the height of the dam.

The typical design seismic coefficients, 0.05–0.10, are much smaller than the ordinates of design spectra for intense earthquake motions in the range of vibration periods for concrete

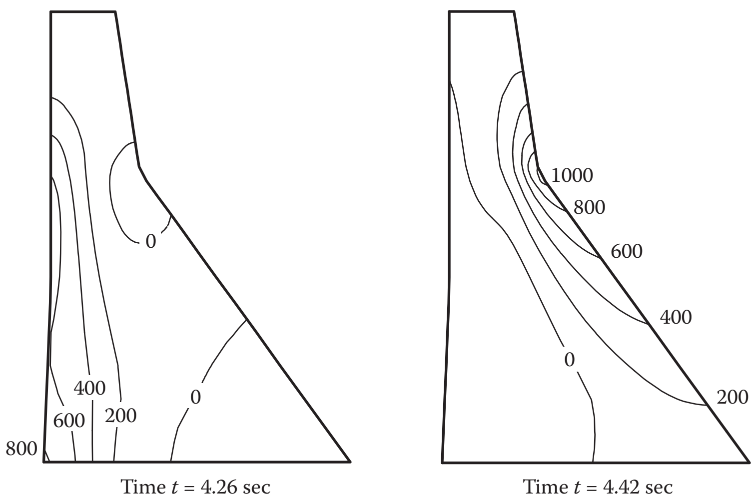


Figure 1.3.1 Maximum principal stresses in Koyna Dam at selected time instants due to transverse and vertical components of ground motion recorded during the December 11, 1967 earthquake; initial static stresses are included.

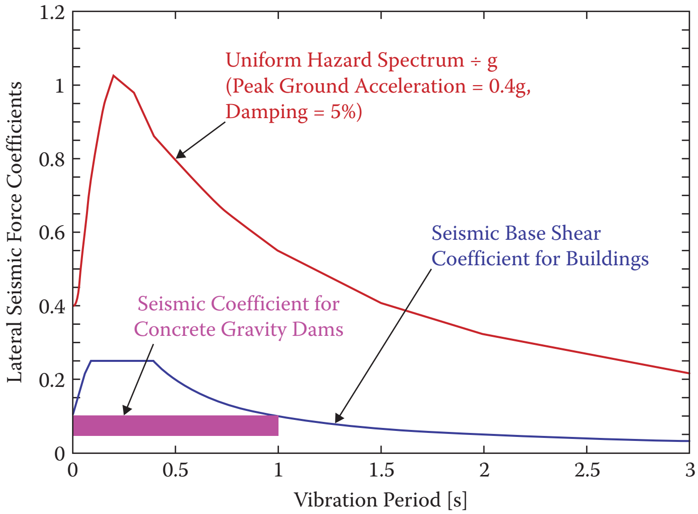


Figure 1.3.2 Comparison of uniform hazard spectrum and seismic coefficient for concrete dams and buildings. Source: Adapted from Chopra (1978).

gravity dams (Figure 1.3.2). Note that the seismic base shear coefficient values for dams are similar to those specified for multistory buildings. However, building code design provisions have been based on the premise that buildings should be able to: "(i) resist minor earthquakes without damage; (ii) resist moderate earthquakes without structural damage; and (iii) resist major earthquakes…without collapse but with some structural…damage." While these may be appropriate design objectives for buildings, major dams should be designed more conservatively, and this intended conservatism is reflected in the no-tension requirement imposed in traditional methods for designing dams. What the traditional methods fail to recognize, however, is that this requirement must be tied to the dynamic response of the dam that is influenced by its natural vibration periods and modes.

The effective modal earthquake forces may be expressed as the product of the weight of the dam per unit height and a seismic coefficient; its magnitude depends on the pseudo-acceleration spectral ordinate at the modal period and its height-wise distribution depends on the shape of the mode. The response of short-vibration-period structures, such as concrete gravity dams, is dominated by the fundamental mode of vibration, and the seismic coefficient varies over the dam height, as shown schematically in Figure 1.3.3b. In contrast, traditional analysis and design procedures ignore the dynamic amplification of response, as reflected in the response spectrum and the shape of the mode, and adopt a uniform distribution for the design coefficient (Figure 1.3.3a), resulting in an erroneous distribution of lateral forces and hence of stresses in the dam. The implications of these errors will be discussed in Chapter 7.

To eliminate these errors, it is imperative to consider the dynamics of the system subjected to realistic ground motions in estimating the earthquake response of concrete dams. In Chapters 2–6, such procedures for dynamic analysis of 2D models of gravity dams are developed. In Chapter 7, responses computed by these procedures are demonstrated to be consistent with motions of a gravity dam recorded during an earthquake and with the earthquake performance of Koyna (gravity) Dam.

The traditional design loadings for gravity dams include seismic water pressures in addition to the hydrostatic pressures, as specified by various formulas (U.S. Army Corps of Engineers 1958; Bureau of Reclamation 1966). These formulas differ somewhat in detail and in numerical

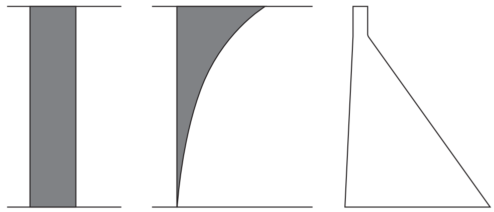


Figure 1.3.3 Distribution of seismic coefficients over dam height in traditional design and for the fundamental vibration mode. Source: Adapted from Chopra (1978).

values but not in underlying assumptions; they are all based on the classical results (Westergaard 1933; Zangar 1952) derived from analyses that assumed the dam to be rigid and water to be incompressible. One of these formulas specifies the seismic water pressure $p_e = cswH$, where c is a coefficient that varies from zero at the water surface to about 0.7 at the reservoir bottom, s is the seismic coefficient, w is the unit weight of water, and H is the total depth of water. For a seismic coefficient of 0.1, the additional water pressure at the base of the dam is about 7% of the hydrostatic pressure; and pressure values at higher elevations are even smaller. As a result, these additional water pressures have little influence on the computed stresses and hence on the geometry of the gravity section that satisfies the traditional design criteria.

On the other hand, earthquake-induced stresses in gravity dams are much larger when dam–water interaction arising from deformations of the dam and water compressibility effects are considered, as will be demonstrated in Chapters 2 and 6. It is apparent, therefore, that hydrodynamic effects are considerably underestimated because of assumptions implicit in traditional design forces.

As mentioned earlier, traditional analysis and design procedures ignore interaction between the dam and foundation. However, such interaction has very significant influence on the dynamics of the system, and, hence, on the earthquake-induced stresses. This will be demonstrated in Chapters 3 and 6.

Finally, the static overturning and sliding criteria that have been used in traditional design procedures for gravity dams have little meaning in the context of oscillatory response to earthquake motions.

1.4 TRADITIONAL DESIGN PROCEDURES: ARCH DAMS

1.4.1 Traditional Analysis and Design

Traditionally, the dynamic response of the system has not been considered in defining the earthquake forces in the design of arch dams. For example, the U.S. Bureau of Reclamation (1965) stated: “The occurrence of vibratory response of the earthquake, dam, and water is not considered, since it is believed to be a remote possibility.” Thus, the forces associated with the inertia of the dam were expressed as the product of a seismic coefficient – which was constant over the surface of the dam with a typical value of 0.10 or less – and the weight of the dam. Water pressures, in addition to the hydrostatic pressure, were specified in terms of the seismic coefficient and a pressure coefficient that was the same as for gravity dams, defined in Section 1.3.3. This pressure coefficient was based on assumptions of a rigid dam, incompressible water, and a straight dam. Generally, dynamic interaction between the dam and foundation was not considered in evaluating the aforementioned earthquake forces, but in stress analysis of arch dams the flexibility of the foundation sometimes was recognized through the use of Vogt coefficients (Bureau of Reclamation 1965).

The traditional design criteria required that the compressive stress not exceed one-fourth of the compressive strength or 1000 psi, and the tensile stress should remain below 150 psi.

1.4.2 Limitations of Traditional Procedures

As mentioned in Section 1.3.3 in the context of gravity dams, the seismic coefficient of 0.1 is much smaller than the ordinates of the pseudo-acceleration response spectra for intense ground motions (Figure 1.3.2). Thus the earthquake forces for arch dams also were greatly underestimated in traditional analysis procedures.

The effective earthquake forces on a dam due to horizontal ground motion may be expressed as the product of a seismic coefficient, which varies over the dam surface, and the weight of the dam per unit surface area. The seismic coefficient associated with earthquake forces in the first two modes of vibration of the dam (fundamental symmetric and anti-symmetric modes of a symmetric dam) varies, as shown in Figure 1.4.1. In contrast, traditional design procedures ignore the vibration properties of the dam and adopt a uniform distribution for the seismic coefficient, resulting in erroneous distribution of lateral forces and hence of stresses in the dam. A dynamic analysis procedure that eliminates such errors is developed in Chapter 8. Including dam–water–foundation interaction, this procedure is shown in Chapter 10 to produce seismic response results that are consistent with the motions of two arch dams recorded during earthquakes.

As mentioned in Section 1.3.3, the additional water pressures included in traditional design procedures for gravity dams are unrealistically small and have little influence on the computed stresses and hence on the geometry of the dam that satisfies the design criteria. This observation is equally valid for arch dams because the additional water pressures considered for arch dams are similar to those for gravity dams.

Demonstrated in Chapter 9 is the importance of two interaction mechanisms – which are ignored in traditional design – in the dynamics of arch dams. When dam–water interaction and water compressibility are properly considered, hydrodynamic effects result in significant increases in the earthquake-induced stresses in arch dams, more so than for gravity dams. Similarly, when dam–foundation interaction including foundation mass and radiation damping are properly considered, this interaction mechanism generally has a profound influence on the earthquake-induced stresses in arch dams just as in the case of gravity dams.

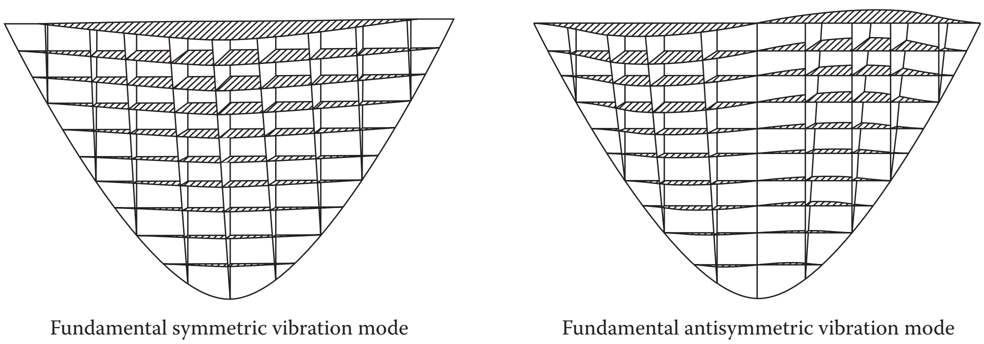


Figure 1.4.1 Distribution of seismic coefficients over the dam surface in the first two vibration modes of an arch dam. Source: Adapted from Bureau of Reclamation (1977).