HAWAII DAM SAFETY GUIDELINES:

SEISMIC ANALYSIS &

POST-EARTHQUAKE INSPECTIONS

Circular C131

DEPARTMENT OF LAND AND NATURAL RESOURCES
Engineering Division

HAWAII DAM SAFETY GUIDELINES:

SEISMIC ANALYSIS &

POST-EARTHQUAKE INSPECTIONS

Circular C131

Prepared by

Horst G. Brandes, Ph.D., P.E.
Applied Geosciences, LLC

DEPARTMENT OF LAND AND NATURAL RESOURCES
Engineering Division

Honolulu, Hawaii
April 2004

LINDA LINGLE
Governor

BOARD OF LAND AND NATURAL RESOURCES

PETER T. YOUNG	Chairperson
TIMOTHY E. JOHNS	Member-at-Large
TOBY MARTYN	Member-at-Large
KATHRYN W. INOUYE	Oahu Member
GERALD L. DEMELLO	Hawaii Member
TED K. YAMAMURA	Maui Member
LYNN P. MCCRORY	Kauai Member

DEPARTMENT OF LAND AND NATURAL RESOURCES

PETER T. YOUNG, Chairperson
DAN DAVIDSON, Deputy

ENGINEERING DIVISION

ERIC T. HIRANO, Chief Engineer

PREFACE

This document provides general guidance for dam owners and engineers for the inspection of dams following an earthquake and for the evaluation of existing dams with regard to their performance during and after seismic shaking. An earthquake may cause failure of or severe damage to a dam and its appurtenant structures. In the event of damage, immediate action may be necessary to prevent further weakening or catastrophic failure of the structure. Accordingly, all dam operators should be carefully instructed in the procedures to be followed if an earthquake should occur that produces motions of intensity sufficient to cause damage. To reduce the risk associated with such earthquakes, it is incumbent upon owners to assess the safety of their dams by means of a proper seismic response investigation. All dams should be capable of withstanding anticipated seismic motions without failure that may result in loss of life or extensive property damage.

Verification of the seismic adequacy of a particular dam structure requires consideration of all pertinent factors, including those that are not normally considered in standard methods of analysis, such as consequences of failure (generally determined from a breach analysis), operating practices and pool levels, age of the dam, availability and detail of design plans and subsequent alterations, adequacy of emergency action plans and experience of dam operator, among others.

The focus of these guidelines is on evaluation and inspection of existing earth embankment dams, which are the predominant type in Hawaii. Other types of jurisdictional dams, such as concrete-lined dams, rock fill dams and arch dams are occasionally constructed. Many elements of these guidelines will apply to these types of structures as well and they are also expected to conform to sound engineering principles and current state of the practice for their type of design. It is recommended that owners of such structures confer with DLNR personnel regarding appropriate standards.

These guidelines, which consider current engineering practice and local experience, are not intended to represent a rigid set of criteria for design, inspection and analysis, and are not a substitute for experience and sound judgment. DLNR recognizes that certain requirements contained herein may be mitigated by site conditions, operating practices, design details, and other factors specific to the dam in question. Compliance with these guidelines will not guarantee a safe structure or prevent failure since each dam and location are unique. However, assessment of individual dams by competent seismic engineers with experience in dams and compliance with these guidelines can substantially reduce the risk of failure and catastrophic consequences.

Table of Contents

SEISMIC ANALYSIS OF EARTHFILL DAMS

1. INTRODUCTION

Dams in seismic regions need to be evaluated to make sure that they can withstand earthquakes while protecting public safety, life and property. Most of the dams in Hawaii are several decades old and some are in excess of 100 years in age. The majority of them were designed with scant attention paid to seismic considerations. Hence, the need for evaluating the ability of dams to perform satisfactorily during an earthquake cannot be overemphasized, especially in light of present awareness that earthquake risks in parts of Hawaii are very high (Section 3).

The behavior of dams and their foundations under earthquake loading (Section 2) is an extremely complex problem that is not yet fully understood. However, much progress in seismic dam engineering has occurred in the last 35 years, particularly following the near breaching of the Lower San Fernando dam during the 1971 San Fernando earthquake. If anything, progress in understanding underlying principles and formulating rational methods of analysis has accelerated in recent years. Additional developments are sure to follow. It is therefore essential that seismic investigations be conducted by knowledgeable seismic engineers following the state-of-the-practice in the profession. Comprehensive reviews of current practice are presented by Idriss (2002), Brandes (2002), Bureau (2002), Finn (2001) and Kramer (1996), among others.

Seismic evaluation of an existing dam requires an assessment of site-specific geological and seismological conditions to determine seismic potential and associated ground motions (Section 4), a field and laboratory testing program to characterize the distribution and properties of the soils (Section 7), and an analysis of the effects of earthquake shaking on the dam structure and its appurtenances (Sections 8 through 10). Procedures to be followed in carrying out a seismic investigation are outlined in Sections 5 and 6. Specific requirements are stated where appropriate and are supplemented by commentary and observations on current practice. Since a lack of consensus remains on a number issues, particularly relating to stability and deformation analysis procedures, detailed specifications are dispensed with in those cases in favor of discussions that are useful in planning and carrying out a seismic response investigation. Emphasis is placed on the fact that no two dams or settings are alike and that different procedures may be called for depending on specific circumstances. Concrete dam structures present a unique set of peculiarities and are not specifically addressed by these guidelines.

2. EFFECT OF EARTHQUAKES ON EMBANKMENT DAMS

Earthquakes impose additional loads on embankment dams over those experienced under static conditions. Typically, such loading is of short duration, cyclic and involves motion in the horizontal and vertical directions. Earthquakes can affect embankment dams by causing any of the following (ANCOLD, 1998):

- Settlement and cracking of the embankment, particularly near the crest of the dam, along with piping erosion through cracks
- Reduction of freeboard due to settlement, which may, in the worst case, result in overtopping of the dam
- Instability of the upstream and or downstream slopes of the dam
- Differential movement between the embankment, abutments and spillway structures, increasing the likelihood of leakage and piping failure
- Liquefaction or loss of shear strength of saturated granular soils in the embankment or its foundations due to increase in pore pressures induced by the cyclic loading of the earthquake
- Differential movements or instability on faults or other low strength seams or defects passing through the dam foundation
- Overtopping of the dam in the event of large tectonic movement in the reservoir basin, or by seiches induced upstream
- Overtopping of the dam by waves due to earthquake induced landslides into the reservoir from the valley sides
- Damage to outlet works passing through the embankment leading to leakage and potential piping erosion of the embankment

The potential for such problems depends on:

- The seismicity of the area in which the dam is sited, and the assessed design earthquake
- Local foundation and topographic conditions
- The type and detailed construction of the dam
- The water level in the dam at the time of the earthquake

There are four main issues to consider in analyzing the adequacy of a particular dam to resist earthquakes:

- The general design of the dam, particularly the provision of filters with good drainage capacity, to prevent internal erosion of the dam and foundation. Many of the older dams in Hawaii can not be considered 'well-built' and are often lacking in design information. Special care should be taken in evaluating their seismic response.
- The stability of the embankment during and immediately after the earthquake
- Deformations induced by the earthquake (settlement, cracking) and dam freeboard.
- The potential for liquefaction of saturated sandy and silty soils in the foundation, and possibly in the embankment, and how this affects stability and deformations during and immediately after the earthquake.

3. EARTHQUAKE HAZARD IN HAWAII

Earthquakes in Hawaii constitute a major hazard to dams and other structures. The largest event on record, the 1868 Southern Hawaii earthquake, had a Richter magnitude of 7.9 and an intensity of XII on the Mercalli scale. It resulted in catastrophic damage to property, loss of life, and a large tsunami. Other large earthquakes in the last 150 years are listed in Table 1. Historic seismic activity in the State clearly indicates that the possibility of future large earthquakes is real and quite significant. Indeed, the probability of major earthquakes in parts of Hawaii is comparable to that in the most seismically prone regions in the U.S., i.e. Southern California, Alaska and the New Madrid seismic area.

Nearly all of the seismicity in the Hawaiian archipelago occurs on or very near the Island of Hawaii and is associated, directly or indirectly, with active volcanic processes and flexing of the lithosphere beneath the volcanic edifice. Seismic activity decreases significantly with distance from the Island of Hawaii. Distant earthquakes are attributable to stresses caused by the weight of the islands and to normal intraplate seismicity, although their frequency-magnitude relationship is not well established (Klein and Koyanagi, 1989; Klein et al., 2001). It is worth noting that offshore earthquakes are somewhat more numerous toward he northwest, with activity diminishing along the island chain (Klein and Koyanagi, 1989). Clearly though, the earthquake threat is largest on the Island of Hawaii. However, this does not mean that seismic hazards can be discounted on the other islands. Seismic waves can travel significant distances to produce hazardous shaking conditions elsewhere, albeit at a reduced intensity. The overall seismic hazard along the chain of islands is reflected in the seismic zone assignments for the State of Hawaii shown in Figure 1.

Table 1. Large Earthquakes in Hawaii

Date	Location	Maximum Intensity	Magnitude
1868	Southern Hawaii	IX	7.0
1868	Southern Hawaii	XII	7.9
1929	Hualalai	VIII	6.5
1951	Kona	VIII	6.9
1973	North of Hilo	VIII	6.2
1975	Kalapana	VIII	7.2
1983	Kaoiki	IX	6.7
1989	Kalapana	VII	6.2

Source: USGS Bulletin 2006

Three general sources for earthquakes have been identified on the Island of Hawaii. Shallow earthquakes occur at a rate of thousands per year, but they are usually very small and do not constitute a significant threat, except perhaps for structures placed in the immediate vicinity of the source. Shallow earthquakes are due to magma movement close to the surface and often occur prior to an eruption. These smaller events take place beneath Kilauea, Mauna Loa, Hualalai, Kohala, and Loihi. Deeper and larger earthquakes occur underneath the flanks of Kilauea, Mauna Loa and Hualalai, and particularly in the rift zone area south of Kilauea and Mauna Loa. Magma intrusion and lateral spreading of lava flows through the complex of calderas and rifts causes compression and fracturing of the volcanic flanks and their progressive movement in a seaward direction. Focal depths of near-vertical and strike-slip earthquakes in the Kaoiki and Hilea seismic zones range from about 4 km to 11 km. Large events also occur near the interface between the top of the prevolcanic seafloor and overlying lava flows, along a decollement plane that is some 9 km deep near the coastline and dips gently toward the north. On an even larger scale, it has been suggested that prehistoric catastrophic events may have occurred that involved breaking and sliding into the ocean of large sections of the island (Klein et al., 2001). Maps identifying the principal fault and rift zones across the State have been published by the USGS (1996) and the American Association of Petroleum Geologists (1974), whereas the seismicity of the principal seismic sources are described in Klein et al. (2001) and in Klein and Koyanagi (1989).

4. SITE SPECIFIC GROUND MOTIONS FOR ANALYSIS

For both deterministic and probabilistic seismic hazard analyses, the maximum earthquake that each seismic source is capable of generating must be estimated. Typically, these estimates are based on the physical characteristics of the source, as well as the maximum historical seismicity. Once the earthquake potential of faults in the region has been evaluated, it is possible to select a design or evaluation earthquake (or earthquakes) to analyze the seismic hazard associated with a particular dam.

Terminology

The following definitions are adopted in these guidelines:

- <u>Maximum Credible Earthquake (MCE)</u>
  The MCE is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. Each active fault or tectonic province identified in the regional and local settings of the dam will be associated with one MCE. Hence, it is possible for several MCEs to affect a single dam site.

- <u>Controlling Maximum Credible Earthquake (CMCE)</u>
  The CMCE is the most critical of all the MCEs capable of affecting the dam. It

is determined after successively assuming that each MCE would occur along its associated fault, or within its associated tectonic province, at a location closest to the dam with postulated capability of generating the event. The MCE that would result in the most severe consequences for the dam considered represents the CMCE.

- Maximum Design Earthquake (MDE)
  The MDE will produce the maximum level of ground motion for which the dam should be designed or analyzed. It will be required at least that the reservoir impounding capacity of the dam be maintained when subjected to that seismic load. Where causative faults can be identified the MDE can often be determined as an appropriate level of the MCE (or CMCE), depending upon the consequences of failure. Conversely, it can be derived by a combination of deterministic and probabilistic approaches.

Minimum Criteria for Design Earthquakes

The MDE to be used in a deterministic approach will depend on the risk class assigned to the particular dam. The classification scheme in Table 2 should be used for seismic analyses purposes.

Dams should be analyzed, at a minimum, for the MDE criteria listed in Table 3, without release of the reservoir. If the dam is situated in very close proximity to a seismic fault, due consideration needs to be given to near-field effects such as fault directivity. In that case, separation of near-field effects from historical information to quantify the earthquake of interest is difficult and a deterministic approach should be used. On the other hand, for locations beyond the range of applicability of particular attenuation relationships, or when dealing with source zones, a probabilistic approach should be favored. Given the lack of confidence in current attenuation relationships beyond the Island of Hawaii, and given the possible incremental role that other diffuse seismic sources play at long distances from the zones of active volcanism, a probabilistic approach should be favored for all islands east of Hawaii, perhaps with the exception of eastern Maui and Kahoolawe.

Table 2a. Weighting Factors for Risk Classification of Dams

	RISK			
	Extreme	High	Significant	Low
Capacity (Ac-ft)	> 5000 (6)	5000 – 500 (4)	500 – 50 (2)	< 50 (0)
Height (ft)	> 120 (6)	120 – 60 (4)	60 – 25 (2)	< 25 (0)
Evacuation Requirements (No. of persons)	> 1000 (12)	1000 – 100 (8)	100 – 10 (4)	< 10 (0)
Potential Downstream Economic Damage	High (12)	Moderate (8)	Significant (4)	None (0)

Weighting factors are indicated in parenthesis.

Total Risk Factor = **Capacity** Risk Factor + **Height** Risk Factor
+ **Evacuation Requirements** Risk Factor
+ **Potential Downstream Economic Damage** Risk Factor

Table 2b. Risk Classes*

Total Risk Factor	**Risk Class**
(0-6)	I **(Low)**
(7-18)	II **(Significant)**
(19-30)	III **(High)**
(31-35)	IV **(Extreme)**

* It should be noted that other factors, such as the availability or lack of construction and maintenance records, processed instrumentation and surveillance records, the level of effort expended in previous safety evaluations, and new or planned downstream development, may affect the risk associated with a particular structure. Such factors, however, cannot be easily quantified but should be taken into consideration in selecting an appropriate risk class.

Tables 2a and 2b modified from USCOLD guidelines (1999)

Table 3. Minimum Criteria for Design Earthquakes*

RISK CLASS[a]	MAXIMUM DESIGN EARTHQUAKE (MDE)	
	DETERMINISTICALLY DERIVED	PROBABILISTICALLY DERIVED AEP[b]
Extreme, High (III, IV)	CMCE[c]	1/10,000[c]
Significant (II)	50% to 100% of CMCE[d, e]	1/1,000 to 1/10,000[e]
Low (I)	50% of CMCE or less[f]	1/100 to 1/1,000[f]

[a]See Table 2 for risk classification

[b]*AEP* = Annual Exceedance Probability

[c]An appropriate level of conservatism should be applied to the factor of safety calculated from these loads to reduce the risk of dam breaching to tolerable values.

[d]*MDE* firm ground accelerations and velocities can be taken as 50% to 100% of *CMCE* values, depending on the overall consequences of failure.

[e]For consequences approaching Risk Class I (Table 2b), particularly if no incremental fatalities would result from failure, an earthquake corresponding to 50% of the *CMCE* (or one corresponding to an *AEP* of 1/1,000) may be appropriate. On the other hand, as the Risk Factor approaches Risk Class III, an *MDE* close to the *CMCE* should be chosen.

[f]If no incremental loss of life would result from failure, the *MDE* for Class I dams can be taken as less than 50% *CMCE* (or *AEP* exceeding 1/1,000), in proportion to the potential economic risk that failure of the dam would pose.

* Typically, dams in Risk Classes III or IV will require a detailed method of analysis and the use of acceleration time histories, especially if such dams are also located in areas of high seismic intensity. Simpler methods of evaluation, that use response spectra or peak ground motion parameters may be acceptable for dams of significant or low risk ratings.

Table modified from Canadian Dam Safety Guidelines (CDA, 1999)

Selection of Seismic Evaluation Parameters

Seismic parameters represent one or several ground motion-related variables or characteristics, such as peak ground acceleration (PGA), peak velocity or displacement, response spectra, acceleration time histories, or duration. They can be obtained using deterministic or probabilistic seismic hazard analysis (PSHA) procedures and, preferably, both.

Deterministically-Derived Parameters. Seismic evaluation parameters that characterize the MDE depend on a number of factors, including site classification, physical properties and thickness of foundation materials, near-field and directivity effects, distance from the zone of rupture, fault type, magnitude of event, and tectonic framework. Also of importance are the direction of fault fracture propagation and topography of the dam site. Preferably, seismic evaluation parameters should be specified using site-dependent considerations, making use of existing knowledge and actual observations that pertain to earthquake records obtained from sites with similar characteristics. In particular, attenuation characteristics that are in use elsewhere in the U.S. should not be applied blindly to Hawaii due to differences in earthquake focal depths, transmission paths, and tectonic settings.

PGA, despite recognized shortcomings such as a lack of predictability in the near-field, or the fact that it often occurs at high frequency (of little significance to most dams), remains frequently used to rate the seismic exposure of a given site. Empirical relationships are used to estimate PGA, or for that matter a host of other peak motion parameters. A number of these expressions have been reported and new ones are being developed at a rapid pace. As already noted, care should be taken to study the background material for each of these to assure that they are applicable to the regional and site geology under consideration. Specific PGA attenuation relationships have been developed for the Island of Hawaii by Munson and Thurber (1997) and by Klein (1994).

It is important to distinguish if seismic parameters predicted by attenuation relationships take into account the effect of surface soil layers or not since soft deposits such as ash soils can alter the bedrock motions dramatically. For example, the Munson and Thurber (1997) expression allows for either rock or ash conditions. However, soil depth is not a variable in these relationships even though it is of great consequence when evaluating surface ground motions.

The duration of shaking is a significant seismic evaluation parameter, as it has been shown to be directly related to the extent of damage, especially in the case of embankment dams. This is even more critical when the foundation or the embankment contain soils that are prone to accumulate excess pore pressures during an earthquake. Local conditions may affect the expected duration of earthquake shaking and should be considered on a case-by-case basis.

When site-specific response spectra that characterize the MDE are called for in an investigation, they can be determined from either generalized spectral curves, or more accurately, from spectra based upon attenuation relationships. The latter are the preferable ones and should be used when feasible, particularly for large dams where large events are expected. Since no such relationships exist for Hawaii at this time, it may be helpful to consider the average of a number of attenuation relationships for spectral coefficients developed for provinces with similar characteristics and compare them to the response spectrum of a natural earthquake, scaled to the same PGA. Historically though, generalized spectral shapes have often been used for dam safety evaluation purposes. For example, Seed, Ugas and Lysmer (1974) developed normalized spectral shapes applicable to rock and soil conditions that have been adapted for use in the 1994 UBC code (Figure 2). These results were deemed appropriate for earthquake magnitudes near 6.5, but were considered very conservative for magnitudes less than 6.0. The curves are based on data specific to California and may not be appropriate for Hawaii. These and other generalized spectral shapes should not be used as the principal means for defining site-specific ground motions for analysis of dams in Hawaii. However, they may be useful in the interpretation of seismic motions obtained by other means.

Vertical peak and spectral accelerations are usually considered less critical for embankments that are not too close to sources of earthquakes. They are more important for concrete dams and concrete appurtenances. When necessary, they can be determined by scaling horizontal accelerations, along with corresponding frequencies in the case of spectral values, using factors in the range of 1/2 to 2/3. Again, it would be preferable to rely on attenuation relationships developed specifically for vertical accelerations.

While definition of seismic parameters by peak values and spectral shapes is sufficient for some dam applications, in other cases a time history analysis may be required. This could be the case for high hazard dams when induced stresses approach the strength of the dam or foundation materials, or when it is necessary to consider the inelastic behavior of the embankment. When it is appropriate to perform time-dependent response analysis, it is recommended that several sets of acceleration time histories be used to represent the MDE. The set should be chosen so that sufficient energy content is considered at all frequencies of importance to the dam. A comprehensive number of records are especially critical when conducting nonlinear dynamic response analysis. The pattern of permanent deformations that results from such an analysis is often very sensitive to the specific time histories chosen, particularly when they contain a near-source fling with large positive or negative amplitude. It is also recommended that both normal and reverse polarities be tried successively, for each set of acceleration time-histories, to ensure that no potential critical condition has been overlooked.

Ideally, all horizontal and vertical acceleration time histories to be used for dam response analysis would be represented by natural accelerograms, consistent with the specified spectral requirements, and obtained for site conditions similar to those present at the dam site. In most cases however, and particularly in Hawaii, strong ground motion records do not cover the whole range of conditions to be simulated. Hence, natural records must often be scaled, modified or supplemented by artificial motions in order to represent a specific earthquake size or seismotectonic environment. The need to alter real records is particularly acute when large magnitudes are postulated, e.g., in the case of the MDE. A number of procedures are available to develop or modify time histories for the purpose of dam analysis. Response-spectrum-compatible (RSC) time histories can be obtained using either response-spectrum matching time history adjustments, source-to-site numerical model time-history simulation, multiple natural time-history scaling, or by connecting accelerogram segments from different earthquakes. Of these procedures, the first one is the method of choice and the last one should be avoided whenever possible. Obtaining reliable RSC time histories should be trusted only to experienced seismic engineers or seismologists.

Acceleration spectra or design time history of ground motions must take into account amplification effects that may occur in the foundation and embankment soils specific to the location and design of the particular dam. Soil amplification of ground motions can be estimated crudely by the methods proposed by Makdisi and Seed (Section 9). More reliable estimates usually involve the use of numerical linear or nonlinear wave propagation models. Numerical models are usually necessary when the surface topography is not level or when the dam is located in a large sedimentary basin.

Finally, time histories should be specified at time intervals of 0.02 seconds or less in order to accurately capture all the features of the record. Also, applying 'free-field' motions to the base of a foundation-dam system may be inaccurate due to the interaction between the dam body and the underlying foundation. However, in most cases this will err on the conservative side.

Probabilistically-Derived Parameters. A PSHA involves relating a ground motion parameter and its probability of exceedance at the site, for a specified duration of time (such as the operating life of the dam). The value of the ground motion parameter to be used for the seismic evaluation is then selected after defining a probability level, applicable to the dam and site considered, and acceptable to all parties. This typically involves a thorough mathematical and statistical process that takes into account local and regional geologic and tectonic settings, as well as applicable historic and geologic rates of seismic activity. The results are typically expressed in terms of PGA, PGV or spectral amplitudes at successively specified periods. Fortunately, much of this work has already been done by the USGS, which has produced maps for the entire U.S. Appendix A shows maps for Hawaii depicting PGA with 2% and 10% probability of exceedance in 50 years, as well as PGA spectral accelerations for similar probabilities at 5% critical damping.

For seismic analysis of dams in Hawaii, the annual exceedance probabilities (AEP) indicated in Table 3 should be used to define input motion representing the MDE, depending on the applicable risk class.

Deterministic vs Probabilistic Approaches. When adequate information is available on the tectonic framework, attenuation relationships and site-specific soil conditions, deterministic approaches to specify the MDE have often been specified. This has been the case in the Western U.S. until recently. In places where few strong motion records exist and where the correlation between active faults and strong ground motion shaking is difficult to ascertain, such as in the eastern U.S. and east of the Island of Hawaii, PSHA provides an invaluable aid to judgment in defining the MDE. In such cases the probabilistic approach leads to mathematically more rigorous estimates. In addition, if the MCE has a very long return period, deterministic estimates may lead to overly conservative ground motions for dams that have lifetimes considerably less than the recurrence period of the MCE. Probabilistic estimates rely on similar information as deterministic approaches, i.e. tectonics framework, attenuation relationships and site-specific soil conditions. In addition, the recurrence times for earthquakes of various magnitudes are required. In places such as the Eastern U.S., and to a certain extent in Hawaii, such recurrence intervals are less certain than for example in the Western U.S., making probabilistic estimates less certain.

Whereas in the past deterministic approaches have been favored in dam engineering, there has been a gradual shift to probabilistic methods for determining ground motion parameters. Section 4 outlines specific recommendations for use in Hawaii. Regardless of the philosophy to be followed, an adjunct PSHA will always provide useful insight and will help to better assess how reasonable the selected motions are. One can then apply judgment to avoid that excessively conservative procedures be used either concurrently or at successive steps of a dam safety evaluation.

Risk-Based Approach

In this method, the adequacy of a dam is considered by calculating the probability of breaching of the dam and the expected incremental loss of life due to the dam breaching. The acceptability of the dam is then tested by comparison of the calculated values with societal and individual risk criteria. In the approach, the concept of the MDE is not applicable since one must consider the risk arising from the full range of possible earthquake events. It is an appealing concept since the approaches discussed previously do not account for the conditional probability of dam breaching, so that if an MDE (or other selected earthquake) event were to occur, this does not necessarily mean that the dam would actually be breached. The risk-based approach is relatively new, and the methods for assessing dam breaching probabilities are still being developed, and can be time consuming and costly. Although some agencies such as the Australian National Committee on Large Dams, BC Hydro in Canada, and others in the U.S. are moving in this direction, the risk-based method is not endorsed herein since it does not yet represent the state-of-the-practice. However, it may be of value for high hazard dams, particularly if consideration needs to be given to the risk of breaching from the sum of floods, earthquakes, and other causes.

5. MINIMUM SCOPE OF SEISMIC ANALYSIS

The following criteria, which are based on nearly a century of observations of dam performance during earthquakes, outline the scope of earthquake studies that are to be conducted for dams in the State of Hawaii.

Seismic concerns can be dismissed by seismic engineer when all of the following apply:

1. The dam and foundation are not susceptible to liquefaction;

2. The dam is well-built (densely compacted), and peak accelerations at the base of the dam are 0.2g or less; or the dam is constructed of clay soils, is on clay or rock foundations, and peak accelerations are 0.35g or less;

3. The slopes of the dam are 3 horizontal to 1 vertical or flatter;

4. The static factors of safety of the critical failure surfaces involving the crest (other than the infinite slope case) are greater than 1.5 under loading conditions expected prior to an earthquake; and

5. The freeboard at the time of the earthquake is a minimum of 15 percent of the embankment height for dams on the Island of Hawaii (not less than 7 feet), and 10 percent for dams everywhere else (not less than 5 feet). Fault displacement and reservoir seiches should be considered as separate problems.

Conduct deformation and liquefaction assessments when:

1. The above criteria are not met.

6. SEISMIC ANALYSIS PROCEDURE

An assessment of the seismic adequacy of a dam should follow the following steps:

6.1 Develop the seismicity for the analysis as described in Section 4.

6.2 Design a preliminary exploration and testing program consistent with seismic loading, site geology, dam size, and existing data, necessary to carry out steps 6.3 and 6.4 (see Section 7).

6.3 Use the criteria in Section 5 to determine if seismic concerns can readily be dismissed or whether they should be addressed by more detailed analyses.

6.4 If seismic concerns cannot be dismissed out of hand, decide on the type of seismic investigation that is needed. Identify liquefiable soil units and their liquefaction potential (see Section 8).

6.5 Design a second phase exploration and testing program to focus on aspects that will potentially cause stability problems. A method of analysis considered most appropriate should be selected at this point and the exploration and testing program should be designed to provide the necessary data for such an analysis (Section 7)

6.6 For dams susceptible to liquefaction follow steps 6.7 through 6.10. For dams not susceptible to liquefaction follow steps 6.11 through 6.14.

Dams susceptible to liquefaction (Sections 8 and 9):

6.7 Evaluate earthquake induced pore pressures in the materials susceptible to liquefaction.

6.8 Perform a post-liquefaction stability analysis using limit equilibrium methods and the pore pressures determined in the previous step. For materials that have liquefied, use an appropriate residual strength.

6.9 If the factor of safety from step 6.8 is less than 1.0, perform a deformation analysis. The degree of sophistication used to predict displacements should correspond to the anticipated magnitude of the deformations and to the risk rating of the dam (Table 2). In particular, dams in risk categories III and IV merit approaches that are much more rigorous than those in categories II and I.

6.10 Evaluate stability and deformation results and determine remediation measures required to assure that the dam performs satisfactorily under design seismic loading.

Dams not susceptible to liquefaction (Section 9):

6.11 Perform a pseudo-static stability analysis using the seismic coefficients in Table 4. If the calculated factor of safety is larger than required, the earthquake performance of the dam is acceptable and no deformation analysis will be required.

6.12 If the factor of safety is less than required, perform a Newmark-type deformation analysis using the Makdisi and Seed (1978) approach.

6.13 Should the results of the Makdisi and Seed analysis be unacceptable and further analysis is warranted, perform a seismic deformation analysis using a fully validated dynamic code. Again, the degree of sophistication for such an analysis should be commensurate with the magnitude of the anticipated motions and the risk posed by the dam.

6.14 Evaluate stability and deformation results and determine remediation measures required to assure that the dam performs satisfactorily under design seismic loading.

Table 4. Minimum seismic coefficients and factors of safety for stability calculations

Horizontal Seismic Coefficient k_h	**Factor of Safety**[1] ***FS***	**Comments**[2]
0.20 (Hawaii County) 0.15 (elsewhere)	1.10	*FS* assumes a 10% strength reduction

[1] In determining the *FS* for pseudo-static analysis, a search for the critical failure surface shall be made, rather than simply applying pseudo-static coefficients to the failure surface from the static analysis.

[2] The minimum *FS* should be increased in proportion to the anticipated loss in strength. For example, a 20% reduction in strength would require a minimum *FS* = 1.20.

In general, deformations of 0 to 5 feet are considered sustainable provided the deformation does not exceed 10% of the dam height and does not seriously compromise freeboard. Deformations of 5 to 10 feet are considered serious. It should be noted that as deformations approach 10 feet, predictions become less reliable and computations may fail to properly describe the behavior of abutments and the development of embankment cracking and slumping. Freeboard, crest width, zoning, remaining freeboard, and embankment slopes should all be considered before deciding on whether a 5 to 10 foot deformation is acceptable. Deformations greater than 10 feet are considered to be in nearly the same class as embankments with post-seismic factors of safety less than unity. Structural modifications will in all likelihood be required (modified from Babbitt and Verigin, 1996).

7. EXPLORATION AND TESTING

Exploration and testing programs are necessary to determine the physical qualities of the foundation and embankment and their engineering properties. Development of an investigation program for seismic purposes should be guided by site geology, seismicity, and dam characteristics. A preliminary investigation may be conducted simply to determine whether seismic concerns exist, whereas a follow-up investigation may focus on providing specific information to carry out a particular seismic analysis. The following tools are available for exploration and testing and should be considered when preparing a site investigation program:

Exploration

- Surface Trenching – Trenching can reveal depth of soil and degree of rock weathering, and the characteristics and variability of materials near the surface of the foundation. Trenching is also valuable for investigating near-surface faults and prominent jointing features.

- Drilling – Drilling is the most common exploration method for identifying soil layers and geotechnical material properties. The objective of a well-designed drilling program is to characterize the soil profile through the embankment and foundation. It is important that drilling be extended to sufficient depth to provide the information necessary to address stability, deformation, and ground motion propagation concerns in and beneath the dam. Although dam embankments are usually constructed so that they are relatively uniform in their long direction, a sufficient number of borings should be drilled to identify non-uniform conditions in the foundation and near the abutments. Also, a sufficient number of samples must be obtained and tests performed to produce confidence in the interpolation between drill holes. Attention should be paid to obtaining high quality specimens when undisturbed conditions are essential for laboratory testing. Drilling procedures should minimize disturbance to the embankment and foundations soils. Upon completion of drilling, all holes should be properly sealed with a suitable bentonite-cement grout mixture.

- Standard Penetration Testing (SPT) – This test, which is routinely conducted as part of a drilling program, is the most common type of in-place test. Its usefulness derives from widely accepted correlations between driving resistance (blow counts) and density and strength. It is also commonly used to assess liquefaction potential and estimate residual strengths in sandy deposits. However, the results are dependent on equipment calibration and operator diligence. It is therefore important that drilling equipment, procedures and quality control be rigorously monitored (refer to Table 5).

- Cone Penetrometer Testing (CPT) – The CPT offers advantages over the SPT in terms of productivity. A CPT push usually takes only a fraction of the time necessary to drill a hole to the same depth. It provides a continuous log of the subsurface and is therefore particularly useful for defining the extent and depth of soil and rock units. It is also used for assessing liquefaction potential. Its main drawbacks are that no samples are retrieved and that refusal can be encountered at large depths and in bedrock or very stiff soil or gravel units.

- Becker Hammer Testing (BHT) – This test has been used in the past to assess liquefaction potential of gravelly soils through correlation of results with the SPT. Results have not always been satisfactory. The BHT test has not found application in Hawaii yet.

- Seismic Surveys – Data collected during reconnaissance seismic surveys can be of great value in mapping soil and rock units, both in the vertical and horizontal direction. Some configurations, such as in a cross or downhole test, can be used to measure shear wave velocity, and hence shear modulus. In fact, this is the preferred method to determine shear modulus for ground motion analysis.

Active faults within the foundation of the dam have the potential to cause damaging displacement of the structure. Faults within the proximity of the dam need to be identified during the exploration program and assessed for activity. However, faulting as a result of earthquakes is difficult to predict and reliable analysis procedures that account for hazards associated with local faults do not exist. A competent geologist should be employed if faults nearby are suspected to help establish their relevance to the safety of the dam. These guidelines do not address the issue of nearby faults, which invariably requires a case-specific approach.

Laboratory Testing

Laboratory tests are conducted for soil classification purposes, to judge the general quality of embankment and foundation soils, and to provide strength and deformation characteristics for modeling purposes. The greatest challenge associated with strength and stress-strain testing is that of obtaining high-quality, undisturbed specimens that are representative of conditions in the field. In addition, most test configurations that use undisturbed specimens have limitations in their ability to replicate field stress and strain paths. Nonetheless, a carefully crafted laboratory testing program that includes a sufficient number of tests can provide important insight and parameters on seismic soil behavior. The following types of tests are the most relevant for dam behavior under earthquake loading.

- Triaxial Shear Tests – Monotonic tests are conducted to determine undrained soil strengths and to provide parameters to describe dilative or contractive stress-strain behavior. Cyclic tests are used to predict strength under cyclic loading conditions. However, there are significant limitations with the triaxial test resulting from the difficulty of obtaining truly representative specimens and from the inability to closely duplicate field stress paths and failure modes. Also, testing of coarse materials is not practical due to sample and equipment size restrictions.

- Direct and Simple Shear Tests – Both monotonic and cyclic variations of this test exist. The test determines strength and deformation characteristics, albeit under a different set of load and deformation constraints than in the triaxial test. In sophisticated versions of the testing system, such as the NGI simple shear device, it is possible to conduct drained, undrained or partially drained tests. The major advantage of this type of test is that cyclic loads are applied in a horizontal shear mode, thus better simulating horizontal seismic ground motions that propagate vertically from bedrock below.

- Soil Classification Tests – These types of tests are relatively simple but enormously useful for preliminary assessment of the seismic vulnerability of dams and their foundations. For example, clayey soils are usually not prone to generation of excess pore pressure and attendant liquefaction phenomena, although they may still be susceptible to strength degradation, whereas the presence of non-plastic silts and coarser fractions under saturated conditions should be of concern.

- Moisture-Density – These tests are used to judge the general quality of soils in the embankment and foundation. Low-density soils, such as hydraulic fills and loose alluvial foundation deposits can be very contractive, low-strength, and susceptible to large deformations. In general, embankments that are compacted to at least 95% of the maximum standard Proctor density (ASTM D 698), or at least 90% of the maximum modified Proctor density (ASTM D 1557), or at least 70% relative density if Proctor testing is not appropriate, should be relatively resistant to large deformations for peak bedrock accelerations up to at least 0.20 g.

8. LIQUEFACTION ANALYSIS

Perhaps the most critical issue relating to the effect of earthquakes on dams is weather liquefaction of the dam or foundation may occur, and if so, what the consequences may be. Historically, liquefaction has been the major cause of dam failures due to earthquakes. Liquefaction is used to include all phenomena giving rise to a loss of shearing resistance and to the development of excessive strains as a result of transient or repeated disturbance of saturated cohesionless soils. Large displacements are associated with flow failures, lateral spreading, settlement and soil boils.

Soils Susceptible to Liquefaction

Saturated sands, silty sands, silts and gravelly sands are prone to liquefaction. Although the presence of fines reduces the susceptibility of liquefaction, recent experience indicates that soils with small amounts of clay may liquefy under certain conditions. However, there is no widespread consensus on standard criteria for assessing the liquefaction potential of fine-grained soils. Some agencies have adopted the Modified Chinese criteria, or slightly modified versions of it, which suggests that soils are potentially liquefiable only if:

	Clay content	< 15%	(where clay size < 0.005 mm)
and	Liquid limit	< 35%	
or	Water content	> 0.9 * liquid limit	

Other soil characteristics also play a role in determining liquefaction susceptibility, including particle shape, gradation characteristics, mineralogy and depositional environment.

Assessment of Liquefaction Potential

The most widely accepted, simplest and most practical method of assessing whether there is potential for liquefaction involves the use of liquefaction charts, which represent the cumulative experience from field observations throughout the world of whether liquefaction did or did not occur at specific sites along with the level of ground motions associated in each case. These charts can be used to determine what combinations of shaking intensity and soil resistance are likely to result in liquefaction. Cyclic resistance ratio (*CRR*) curves on these charts represent limiting conditions that determine if liquefaction will occur. The intensity of earthquake shaking is expressed in terms of the average effective cyclic stress ratio (*CSR*) of Seed and Idriss (1971):

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65 \frac{a_{\max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \qquad (1)$$

where τ_{av} is the equivalent shear stress amplitude and $a_{\max}$ the peak horizontal acceleration at ground surface generated by the earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are the total and effective vertical overburden stresses, respectively, and r_d is a nonlinear stress reduction coefficient that varies with depth (NCEER, 1997). Soil resistance to liquefaction is evaluated using any of a number of in situ tests, including the standard penetration test (SPT), the cone penetration test (CPT), shear wave velocity measurements (V_s), and the Becker penetration test.

SPT Chart (Figure 3)

Liquefaction resistance based on *SPT* results can be determined with reference to Figure 3. This chart is for an earthquake of magnitude 7.5. Corrections for earthquakes of different magnitude, as well as for high overburden pressures and sloping ground surface, are discussed later. The raw measured penetration number, N_m, is corrected to a standard set of conditions so that field values can be compared in a meaningful way. N_m is adjusted to correspond to 60% of the maximum potential free fall energy of a U.S. safety hammer operated by rope and pulley. A correction factor, C_E, is applied to the penetration number for other combinations of hammers and hammer release modes. Although values are suggested for use below, it is preferable to directly measure the energy ratio at each site where the *SPT* is used. Adherence to ASTM D-1686 procedures during field testing should yield consistent energy ratios. In granular soils, N is also affected by the effective overburden pressure σ'_{vo}, hence it is corrected to a standard stress of 95.6 kN/m^2 (1 ton/ft^2) through the use of the factor C_N. Corrections to N values are also made for borehole diameter (C_B), rod length (C_R), and sampling method (C_S):

$$(N_1)_{60} = C_N N_m C_E C_B C_R C_S \qquad (2)$$

$(N_1)_{60}$ represents the corrected standard penetration value to be used with the SPT liquefaction chart. Recommended values of C_E, C_B, C_R, and C_S are listed in Table 5. A number of different expressions exist for the overburden correction factor C_N. One of the most common ones is given by Liao and Whitman (1986):

$$C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}} \qquad (3)$$

where P_a equals 1 U.S. ton/ft^2, or 96 kPa, and is given in the same units as the overburden stress σ'_{vo}. Values of C_N should not exceed about 2.0 for very shallow deposits.

Table 5. SPT Corrections

Correction factor	Variable	Term	Value
Energy ratio[1]	Donut hammer	C_E	0.5 – 1.0
	Safety hammer		0.7 – 1.2
	Automatic-trip donut-type hammer		0.9 – 1.3
Borehole diameter	65 mm – 115 mm (2.5 in – 4.5 in)	C_B	1.0
	150 mm (6 in)		1.05
	200 mm (8 in)		1.15
Rod length	3 m – 4 m (10 ft – 13 ft)	C_R	0.75
	4 m – 6 m (13 ft – 20 ft)		0.85
	6 m – 10 m (20 ft – 33 ft)		0.95
	10 m – 30 m (33 ft – 100 ft)		1.0
	> 30 m (100 ft)		< 1.0
Sampling method	Standard sampler	C_S	1.0
	Sampler without liner		1.1 – 1.3

[1]The preferred method is to measure the energy ratio repeatedly at each site.

CPT Chart (Figure 4)

Resistance of soil deposits to liquefaction can also be determined from tip resistance measured during a cone penetration test (CPT). As opposed to the SPT, the CPT provides continuous profiles of penetration resistance. This represents a distinct advantage for interpreting the stratigraphy of soil deposits since the location of soil boundaries and the presence of thin soil layers can be detected much more accurately. CPT results are generally more consistent and repeatable than those from any of the other tests used for liquefaction assessment. In addition, the CPT test is conducted under much lower rates of penetration than the SPT, hence it is inherently more suitable for providing empirical estimates of soil behavior parameters, including those associated with seismic response. However, CPT criteria for evaluating liquefaction resistance are not as widely accepted yet as those associated with the SPT. The reason for this is that the database upon which the liquefaction criteria in the corresponding CPT chart (Figure 4) are based on is not nearly as extensive as it is for the SPT chart. There is no doubt that as new field performance data becomes available and the correlations in Figure 4 are revised, the CPT approach will find a wider acceptance in engineering practice

The CPT liquefaction assessment chart in Figure 4 is used to determine the CRR for a magnitude 7.5 earthquake and for clean sands with less than 5% of fines. The dashed curves show approximate cyclic shear strain potential, γ_e, to emphasize that shear strain and the potential for ground deformation increase as the resistance to penetration decreases. The raw cone penetration resistance, q_c, is corrected for overburden stress and normalized to q_{c1N}, as follows:

$$q_{c1N} = C_Q \frac{q_c}{P_a} \tag{4}$$

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}} \right)^n \tag{5}$$

where P_a equals 1 U.S. ton/ft^2, or 96 kPa, and is given in the same units as the overburden stress σ'_{vo}. A maximum value of 2.0 is applied to the normalizing factor C_Q at shallow depths. The exponent n has a value that depends on soil type and ranges from about 0.5 for clean sands to about 1.0 for clayey-type soils. More details on this parameter and the influence of soil type on q_{c1N} are given in Robertson and Wride (1997). An additional correction to cone penetration resistance for the presence of thin layers of sand or stiff clay, bounded on either side by softer soil, is discussed by Robertson and Fear (1995).

Shear Wave Velocity Chart (Figure 5)

Shear wave velocity V_s can be used as an index to liquefaction resistance since both V_s and CRR reflect, albeit in different proportions, a soil deposit's void ratio, effective confining stress, stress history and geologic age. Although shear velocity can be determined with great accuracy in the field by various downhole, crosshole, seismic CPT, and surface wave spectral analysis techniques, it reflects small strain behavior, whereas liquefaction involves large strain phenomena. Shear wave velocity measurements should be conducted in conjunction with a limited number of borings when assessing liquefaction resistance so that soil intervals that would appear to classify as liquefiable by the V_s criteria do not include non-liquefiable, soft, clay-rich soils. Similarly, weakly cemented soils may be susceptible to liquefaction even though they classify as non-liquefiable based on measured values of V_s (Andrus and Stokoe, 1997). Shear wave velocity is normalized in a similar manner to the SPT and CPT procedures described above:

$$V_{S1} = V_S \left(\frac{P_a}{\sigma'_{vo}} \right)^{0.25} \qquad (6)$$

where again P_a equals 1 U.S. ton/ft^2, or 96 kPa, and is given in the same units as the overburden stress σ'_{vo}. The liquefaction chart in Figure 5 should only be used for Holocene-age uncemented soils. A dashed CRR curve is shown for values of V_s less than 100 m/s to indicate that essentially no field performance data exists at lower velocities to substantiate the indicated trend. It should also be noted that the above chart should be used with caution for Hawaiian calcareous soils since preliminary experimental results suggest that it may be unconservative.

Magnitude Scaling Factors and Other Corrections

The CRR curves in the SPT, CPT and V_s charts correspond to an earthquake of magnitude 7.5. Seed and Idriss (1982) suggested the use of magnitude scaling factors (*MSF*) for earthquakes of magnitude other than 7.5. These factors are used to shift the CRR base curve vertically according to:

$$CRR_{cor} = CRR_{7.5} \times MSF \qquad (7)$$

The range of recommended MSF values is shown in Figure 6.

The chart-based empirical approach to liquefaction assessment has become widely used in engineering practice. It was originally intended for relatively shallow and level deposits. In order to expand the range of conditions for which the approach can be applied, correction factors were developed by Seed (1983) for large static overburden stresses and for large shear stresses on potential failure planes, specifically for use in conjunction with liquefaction assessment of embankment dams. When applicable, these factors are used as follows:

$$CRR_{cor} = CRR_{7.5} \times MSF \times K_\sigma \times K_\alpha \qquad (8)$$

Very few liquefaction observations have been made where large initial overburden or shear stresses played a role. Therefore, these factors have been developed primarily from the results of extensive laboratory testing.

The liquefaction resistance of a soil increases with increasing confining pressure. Cyclic triaxial compression tests show that the resistance to liquefaction, as measured by the CSR, decreases nonlinearly with increasing confining pressure. To account for this effect at overburden pressures larger than 100 kPa (depths larger than about 10 to 15 m), the use of the factor K_σ in Figure 7 is recommended. However, caution needs to be exercised when using these factors because there is a lack of field data to substantiate the reported values.

Where a sloping ground surface exists, potential failure surfaces are subjected to gravitational shear stresses prior to earthquake shaking. The effect of an initial static shear stress, τ_{st}, on subsequent liquefaction resistance can be considered by normalizing such a stress by the effective vertical stress, i. e. $\alpha = \tau_{st} / \sigma'_{vo}$. This ratio is zero for a level ground surface. In general, it has been found that the presence of a static shear stress on the potential failure surface tends to increase the cyclic resistance, in terms of the CSR necessary to trigger liquefaction, as long as the soil is relatively dense and the confining pressure is low. On the other hand, loose soils and some soils under high confining pressure have a lower liquefaction resistance when an initial static shear stress is present. To account for non-zero shear stresses in failure surfaces that are inclined, use of the factor K_α may be warranted (Figure 8). It should be noted though that there is no consensus on whether the values in Figure 8 are appropriate and they should be used with caution. Liquefaction analysis of sloping ground conditions is a complex problem that is still not well understood.

It has been observed that liquefaction resistance of soil deposits increases with age. Young Holocene sediments are more prone to liquefaction than older Holocene sediments. Pleistocene and earlier deposits are even more resistant. However, this trend has not been quantified due to a lack of data and an incomplete understanding of the effects of age on liquefaction resistance. The liquefaction assessment procedures described above should only be used for Holocene deposits that are not more than a few thousand years old. In some cases it may be possible to balance the effect of a decreasing value of K_σ with depth with a parallel increase in liquefaction resistance that could be expected due to increasing age with depth. Such an approach should only be used with great care and certainly would only be reasonable for natural deposits where age effects increase uniformly with depth.

Residual Shear Strength

One of the effects of seismic shaking, particularly for cohesionless soils, is a loss of shearing resistance that may occur depending on volume change characteristics of the soil, drainage conditions and confining stress. Since any post-seismic stability analysis requires knowledge of soil strength, it is important to be able to predict what the residual or steady state strength is. A number of laboratory and field procedures have been proposed. Laboratory methods were developed by Castro and Poulos (1977) and Poulos et al. (1985). Conversely, Seed (1986) and others back-analyzed a number of flow sides and proposed a correlation with SPT penetration number that is often used as a preliminary estimate (Figure 9). In other cases the normalized residual undrained shear strength is correlated to either corrected SPT or CPT values (Ishihara, 1994; Finn, 1993). Additional details are given in the original publications and in Brandes (2002). There is a diversity of views on what strength to use, particularly at low SPT values. All that can be recommended is that the available methods be reviewed and judgment be used with regard to the particular conditions at hand. One procedure that is relatively comprehensive and merits some consideration is that of Stark and Mesri (1993).

9. SEISMIC STABILITY AND DEFORMATION ANALYSES

Current practice in seismic dam engineering uses any of a number of methods of analysis to evaluate the stability of embankment dams. These methods range from simple limit equilibrium procedures to highly sophisticated dynamic numerical modeling techniques. As already mentioned, the type (or types) of analyses depends on a number of factors, including the risk class of the dam, the magnitude of design ground motions, and the findings from preliminary phases of the investigation. The discussion that follows is intended to provide a brief description of the major methods of analysis, although it is by no means exhaustive.

Pseudo-Static Analysis

In the past, assessment of the stability of earth structures usually involved the pseudo-static method. In this procedure, the effect of earthquake ground motions is represented by constant horizontal and vertical inertial forces and stability is assessed in terms of a factor of safety following standard limit equilibrium principles. The inertial forces are assumed to act through the centroid of the failed mass of weight W:

$$\begin{aligned} F_h &= k_h W \\ F_v &= k_v W \end{aligned} \tag{9}$$

where k_h and k_v are dimensionless horizontal and vertical pseudo-static coefficients.

Most modern commercial limit equilibrium slope stability programs allow for this type of analysis. The difficulty arises in selecting appropriate values of k_h and FS. Since k_h represents the inertial shaking effects, it is reasonable to assume that it should be related in some fashion to the peak horizontal acceleration. In general, slope deposits are compliant to various degrees and the maximum acceleration only occurs over a very short period of time. Therefore, in practice k_h is taken as a fraction of the maximum acceleration, although considerable judgment is required in selecting appropriate values. The values of k_h and FS recommended for use in Hawaii are listed in Table 4.

The pseudo-static method of analysis, despite its earlier popularity, suffers from a number of severe limitations. It attempts to represent complex dynamic behavior in terms of a pair of pseudo-static forces. These inertial forces are assumed to be constant and acting in the downslope direction only, when in reality ground motions are variable, cyclic and undergo continuous direction reversals. Also, the implicit assumption is that the soil is uniformly rigid-perfectly plastic, in effect so that stability can be expressed in terms of a single factor of safety. This represents a gross simplification. Back-analysis of dam failures due to earthquake shaking have indicated that in many cases failure occurred despite estimated factors of safety larger than 1.0. Today, it is generally accepted that pseudo-static analysis is not an accurate tool to assess seismic stability, particularly in cases where significant excess pore pressures may accumulate or where strength degradation is in excess of about 15%.

The pseudo-static method can provide a crude index of stability, but it should only be used as a screening tool to determine whereas additional analysis should be conducted (Section 6). Methods based on evaluation of permanent deformations, such as those described in the following sections, are increasingly being used for seismic slope stability analysis and are to be preferred over the pseudo-static procedure.

Newmark and Makdisi-Seed Deformation Analysis

Displacements associated with time-varying inertial forces can be estimated, to a first degree, with the procedure proposed by Newmark (1965), which represents an extension of the pseudo-static approach. Newmark likened the failure mass to a rigid block sliding on an inclined plane. Failure, and attendant deformations, would be initiated whenever the static and inertial forces exceed the yield resistance along the interface. Displacements accumulate during intervals where the acceleration exceeds the yield strength and are computed by double integration of the accelerogram in excess of the yield value.

The Newmark procedure has been found to give reasonable results if the yield acceleration is evaluated accurately. However, the assumption of a rigid sliding mass makes this procedure appropriate only for relatively stiff soils where the motion is of low frequency. It should not be applied where embankments or their foundations are susceptible to liquefaction or strain weakening because it will significantly underestimate displacements.

The Makdisi and Seed (1978) approach is based on Newmark's method, but modified to allow for the dynamic response of the embankment from a range of ground motions representing earthquakes of various magnitudes. Displacements are calculated by comparing the acceleration at depth to the corresponding yield acceleration by means of a Newmark-type sliding block analysis. The procedure involves use of a set of charts (Figure 10), which requires knowing the peak crest acceleration, the predominant period of the embankment, and the yield acceleration for the potential sliding mass. The yield acceleration is typically taken as 80% of the undrained shear strength of the soil.

The Makdisi-Seed approach is widely used and accepted among practicing engineers. However, it is based on a limited set of case studies and, strictly speaking, should only be applied to dams with seismic motions corresponding to earthquake magnitudes in the range of 6.5 to 8.5 where the PGA at the base of the dam is at least 0.2g. As was the case with Newmark's approach, it should not be used for dams susceptible to liquefaction or strain weakening in the embankment or its foundation.

Post-Liquefaction Equilibrium and Deformation Analysis

Most dams that have suffered failure due to earthquake shaking have included saturated sandy materials in the embankment or the foundation. Liquefaction is a major contributory factor in these failures. In order to assess post-liquefaction stability of embankments, two types of analysis are often performed: 1) limit equilibrium analysis, and 2) deformation analysis. The limit equilibrium analysis is performed to calculate a post-liquefaction factor of safety, and the deformation analysis is conducted to obtain a global picture of liquefaction-induced deformations.

A post-liquefaction limit equilibrium analysis consists of a) identifying soils in the embankment or foundation that are susceptible to liquefaction, b) evaluating excess pore pressures induced by the earthquake in the liquefiable materials, and c) conducting a limit equilibrium stability analysis using the pore pressures calculated in the previous step. Estimates of earthquake-induced pore pressures can be obtained through a program of laboratory testing and/or empirical procedures such as the ones proposed by Martin and Seed (1978) and Seed and de Alba (1986). In many cases, it will be sufficient to assume that all potentially liquefiable zones reach the residual undrained shear strength, with clayey zones retaining full undrained strength and rockfill zones retaining full drained strength. In this case it is not necessary to evaluate the induced pore pressures, and it is sufficient to use the static pore pressures at the beginning of the earthquake.

Prediction of post-liquefaction deformations are increasingly becoming an essential part of a post-liquefaction analysis since they provide a better picture of the overall performance of the dam and they allow the engineer to make a better judgment as to the extent of remedial measures that may be required. Displacements can be interpreted in terms of performance criteria such as the allowable loss of freeboard and tolerable deformations, as long as the limitations of accuracy of the methods being used is kept in mind.

Estimates of potential liquefaction-induced deformations can be obtained by performing a static deformation analysis using appropriate estimates of pore pressures induced by the earthquake and corresponding residual strengths for the liquefied soils. The analysis involves initializing the embankment and foundation to pre-earthquake stress conditions, and then incorporating the earthquake-generated excess pore pressures and residual strengths that simulate post-liquefaction conditions, followed by a static deformation analysis. This type of uncoupled analysis generally leads to conservative estimates of post-liquefaction deformations since it does not allow for dissipation of the excess pore pressures with time. More accurate estimates can be obtained using fully and semi-coupled dynamic analyses as discussed in the following section.

Dynamic Deformation Analyses

Today, numerical modeling techniques such as the finite element method are routinely used as investigative tools in assessing the dynamic response of dams to earthquake loading. The codes used in seismic dam engineering practice can be divided into those based on equivalent-linear (EQL) response analysis and those based on fully non-linear (NL) analysis. A brief discussion of each approach, along with a listing of some of the more frequently used codes, follows.

EQL analysis is essentially an elastic analysis developed to approximate non-linear behavior of soils under cyclic loading. Total stress and strain response are calculated using iterated shear moduli and damping coefficients compatible with the average strain induced within each element of the model. 1-D response is often obtained with the program SHAKE91, whereas 2-D response can be calculated with codes such as FLUSH, SuperFLUSH, DYNDSP, QUAD4M, and QUAKE/W. 3-D response can be computed with TLUSH, or it can be approximated with a suitable 2-D code by stiffening so that the fundamental period of the modeled section matches that of the 3-D dam. Following the response analysis, induced stresses can be compared with stresses causing liquefaction, or displacements can be computed using the calculated acceleration histories by means of the Newmark method. After such analyses, it is desirable to perform conventional stability computations of the upstream and downstream slopes of the dam using computer programs such as STABL, UTEXAS4 or SLOPE/W.

However, given the elastic nature of EQL analyses, these codes do not directly model the inelastic nature of soils that is responsible for permanent deformations. EQL codes tend to predict stronger response that actually occurs and they become less reliable when the ground motions become very demanding.

The state-of-the-art in seismic response prediction of dams involves the use of NL analysis, particularly with the development in recent years of fully-coupled, effective stress codes. These programs are particularly useful when loss of strength, large deformations, and liquefaction are of concern. Another advantage is that the same numerical model can be used for pre-earthquake static conditions, dynamic shaking, and post-earthquake response analysis. In fully coupled codes the soil is treated as a two-phase medium, consisting of solid and water phases and excess pore pressures follow directly from the computed volume changes. Cyclic and residual pore pressures are dependent on the particular soil model adopted, which usually consists of an elasto-plastic model with kinematic hardening, or a boundary surface model with hardening. These models are very complex and they require soil parameters that are often difficult to determine. Fully coupled models have also not been fully validated to date and they can present numerical difficulties.

As a compromise, semi-coupled and uncoupled codes employ semi-empirical relationships such as the ones proposed by Martin et al. (1975) and Seed et al. (1983) to relate cyclic shear strains and stresses to pore pressures. This makes them less theoretically rigorous, but places fewer restrictions on the type of soil models that can be used. Semi-coupled and uncoupled codes are generally less complex and computationally demanding. The soil parameters that are required can often be obtained through simpler field or laboratory procedures.

Typical of the fully-coupled codes are DYNAFLOW, DYNARD, SWANDYNE and SUMDES. Among the semi-coupled codes are DESRA-2, DSAGE, TARA-3, TARA-3L, FLAC, FLAC3D and GEFDYN.

10. APPURTENANT STRUCTURES

In addition to the dam, appurtenant structures such as spillways, intakes, outlets, penstocks, gates, navigation locks and associated equipment should also be able to withstand earthquake loading well enough to protect public safety, life and property. The most important consideration is that failure of an appurtenant structure should not lead to loss of control of the reservoir after the earthquake. Regardless of the method of analysis selected, the final evaluation of seismic safety should be based on engineering judgment and experience with similar structures, keeping in mind that each structure and its immediate environment are unique and may not be duplicated elsewhere.

Appurtenant structures should be evaluated depending upon their importance. Critical structures are those whose failure or damage could lead to failure and/or damage of the main dam and/or other appurtenant structures. Failure may result in uncontrolled releases of water from the reservoir and/or in an inoperable structure, unable to make releases to protect the dam against failure. Water regulating structures, such as gates and valves, which are related to the safety of dam, may need to be able to perform their intended functions after an earthquake. Critical structures should be evaluated for the MDE (Table 3). Non-critical structures can be evaluated for seismic events less than the MDE, commensurate with economic or other risks posed by their failure.

It is important to understand that ground motions for evaluation of appurtenant structures may be quite different from those used for the dam itself, depending on location and support conditions for each structure. In general, amplified motions at the base of the structure need to be determined using appropriate ground motion propagation methods.

There are two general methods of analysis that can be used with appurtenant structures: the pseudo-static method and the more rigorous dynamic method. A dynamic analysis should be considered if (1) the structure has a fundamental frequency less than 33 Hz, or (2) the structure's cost of construction would justify a 'refined' dynamic analysis. A response spectrum approach is particularly suitable for linear analyses, whereas a time-history approach is preferred for non-linear conditions, where the response of the structure needs to be characterized more accurately. Due consideration also needs to be given to hydrodynamic loads when the structure is in contact with water and to soil-structure interaction effects, if appropriate.

Appurtenant structures should be evaluated according to the seismic provisions of the local building code with respect to all aspects not addressed herein. Suitable material and damping properties need be selected, particularly keeping in mind that many of the appurtenant dam structures in Hawaii are quite old.

11. REFERENCES

American Association of Petroleum Geologists (1974). Geological Map of Alaska and Hawaii, Circum-Pacific Edition, Compiled by A.P. Bennison.

ANCOLD (1998). Guidelines for Design of Dams for Earthquake. Australian National Committee on Large Dams, pp. 122.

Andrus, R.D. and Stokoe, K.H. (1997). Liquefaction Resistance Based on Shear WaveVelocity. *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils.* Salt Lake City, NV, pp. 89-128.

Babbitt, D.H. and Verigin, S.W. (1996). General Approach to Seismic Stability Analysis of Earth Embankment Dams. *California Division of Safety of Dams,* Report.

Brandes, H.G. (2002). Chapter 7: Geotechnical and Foundation Aspects. In: *Earthquake Engineering Handbook*, W.-F. Chen and C. Scawthorn (Eds.). Boca Raton, FL: CRC Press LLC, pp. 7.1-7.61.

Bureau, G.J. (2002). Chapter 26: Dams and Appurtenant Facilities. In: *Earthquake Engineering Handbook*, W.-F. Chen and C. Scawthorn (Eds.). Boca Raton, FL: CRC Press LLC, pp. 26.1-26.47.

Castro, G. and Poulos, S.J. (1977). Factors Affecting Liquefaction and Cyclic Mobility. *Journal of Geotechnical Engineering Division, ASCE*, Vol. 106(GT6), pp. 501-506.

CDA (1999). CDA Dam Safety Guidelines, Canadian Dam Safety Association, pp. 118.

Finn, W.D.L. (1993). Seismic Safety Evaluation of Embankment Dams. *Proceedings, International Workshop on Dam Safety Evaluation*, Vol. 4, pp. 91-135.

Finn, W.D.L. (2001). Earthquake Engineering. In: *Geotechnical and Geoenvironmental Engineering Handbook*, R.K. Rowe (Ed.). Boston, MA: Kluwer Academic, pp. 615-659.

Harder, L.F. and Boulanger, R. (1997). Application of K_{sigma} and K_{alpha} Correction Factors. *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils.* Salt Lake City, NV, pp. 167-190.

Idriss, I.M. (2002). Behavior of Embankment Dams During Earthquakes. *DVD*, Interagency Committee on Dam Safety, U. S. Bureau of Reclamation, Federal Emergency Management Agency.

Ishihara, K. (1994). Evaluation of Residual Strength of Sandy Soils. *Proceedings, XIII International Conference on Soil Mechanics and Foundation Engineering*, Vol. 5, pp. 175-181.

Klein, F.W. and Koyanagi, R.Y. (1989). The Seismicity and Tectonics of Hawaii. In: *The Eastern Pacific Ocean and Hawaii*, in: J. Winterer, D. Hussong and R. Decker (Editors), Vol. N, The Decade of North American Geology, Geological Society of America, pp. 238-252.

Klein, F.W. (1994). Seismic Hazards at Kilauea and Mauna Loa Volcanoes, Hawaii. *U.S. Geological Survey Open-File Report* 94-216, 96 pp.

Klein, F.W., Frankel, A.D., Mueller, C.S., Wesson, R.L. and Okubo, P.G. (2001). Seismic Hazard in Hawaii: High Rate of Large Earthquakes and Probabilistic Ground-Motion Maps. *Bulletin of the Seismological Society of America*, 91(3), pp. 479-498.

Kramer, S.L. (1996). Geotechnical Earthquake Engineering. Upper Saddle River, NJ: Prentice-Hall.

Liao, S.S.C. and Whitman, R.V. (1986). Overburden Correction Factors for SPT in Sand. *Journal of Geotechnical Engineering, ASCE*, Vol. 112(3), pp. 373-377.

Makdisi, F.I. and Seed, H.B. (1978). Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. *Journal of the Geotechnical Engineering Division, ASCE*, Vol. 104(GT7), pp. 849-867.

Martin, G.R., Finn, W,D.L. and Seed, H.B. (1975). Fundamentals of Liquefaction Under Cyclic Loading. Journal of the Geotechnnical Engineering Division, ASCE, Vol. 101, pp. 423-438.

Martin, P.P. and Seed, H.B. (1978). MASH – A Computer Program for the Nonlinear Analysis of Vertically Propagating Shear Waves in Horizontally Layered Soil

Deposits. *Report No. UCB/EERC-78/23*, Earthquake Engineering Research Center, University of California at Berkeley.

Munson, C.G. and Thurber, C.H. (1997). Analysis of the Attenuation of Strong Ground Motion on the Island of Hawaii. *Bulletin of the Seismological Society of America*, Vol. 87, No. 4, pp. 945-960.

NCEER (1997). Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Summary Report. T.L. Youd & I. Idriss (Eds.). *Report NCEER-7-0022*, NCEER, State University of New York, Buffalo, NY.

Newmark, N.M. (1965). Effect of Earthquakes on Dams and Embankments. *Geotechnique*, Vol. 15(2), pp. 139-160.

Poulos, S.J., Castro, G. and France, J.W. (1985). Liquefaction Evaluation Procedure. *Journal of Geotechnical Engineering, ASCE*, Vol. 111(6), pp. 772-792.

Roberston, P.K. and Fear, C.E. (1995). Liquefaction of Sand and its Evaluation. Proceeedings, *1st International Conference on Earthquake Geotechnical Engineering*, Tokyo, Keynote Lecture.

Robertson, P.K. and Wride, C.E. (1997). Cyclic Liquefaction and its Evaluation Based on the SPT and CPT. *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*. Salt Lake City, NV, pp. 41-88.

Seed, H.B. and Idriss, I.M. (1971). Simplified Procedure for Evaluating Soil Liquefaction Potential. *Journal of the Soil Mechanics and Foundations Division, ASCE*, Vol. 97(SM9), pp. 1249-1273.

Seed, H.B., Ugas, C. and Lysmer, J. (1976). Site-Dependent Spectra for Earthquake-Resistant Design. *Bulletin of the Seismological Society of America*, Vol. 66, pp. 221-243.

Seed, H.B. and Idriss, I.M. (1982). Ground Motions and Soil Liquefaction During Earthquakes. *Monograph No. 5*, Earthquake Engineering Research Institute, University of California at Berkeley.

Seed, H.B. (1983). Earthquake Resistant Design of Earth Dams. *Proceedings, Symposium on Seismic Design of Earth Dams and Caverns*, pp. 41-64.

Seed, H.B., Idriss, I.M. and Arango, I. (1983). Evaluation of Liquefaction Potential Using Field Performance Data. *Journal of Geotechnical Engineering, ASCE*, Vol. 109(3), pp. 458-482.

Seed, H.B., Tokimatsu, K., Harder, L.F.Jr. and Chung, R.M. (1985). The Influence of STP Procedures in Soil Liquefaction Resistance Evaluations. *Journal of Geotechnical Engineering, ASCE*, Vol. 111(12), pp. 1425-1445.

Seed, H.B. (1986). Design Problems in Soil Liquefaction. *Journal of Geotechnical Engineering, ASCE*, Vol. 113(8), pp. 827-845.

Seed, H.B. and De Alba, P. (1986). Use of the SPT and CPT Tests for Evaluating the Liquefaction Resistance of Sands. Proceedings, In-Situ 86, Conference on the Use of In-Situ Tests in Geotechnical Engineering, *ASCE Geotechnical Special Publication No. 6.*

Seed, R.B. and Harder, L.F. (1990). SPT-based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength. Proceedings, *H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 351-376.

Stark, T. and Mesri, G. (1993). Undrained Shear Strength of Liquefied Sands for Stability Analysis. *Journal of Geotechnical Engineering, ASCE*, Vol. 118(11), pp. 1727-1747.

USCOLD (1999). Updated Guidelines for Selecting Seismic Parameters for Dam Projects. United States Committee on Large Dams, Committee on Earthquakes.

USGS (1996). Geologic Map of the Island of Hawaii. Compiled by E.W. Wolfe and J. Morris, *USGS Map I-2524-A*, Miscellaneous Investigation Series, Sheets 1-3.

Youd, T.L. and Noble, S.K. (1997). Magnitude Scaling Factors. *NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*. Salt Lake City, NV, pp. 191-200.

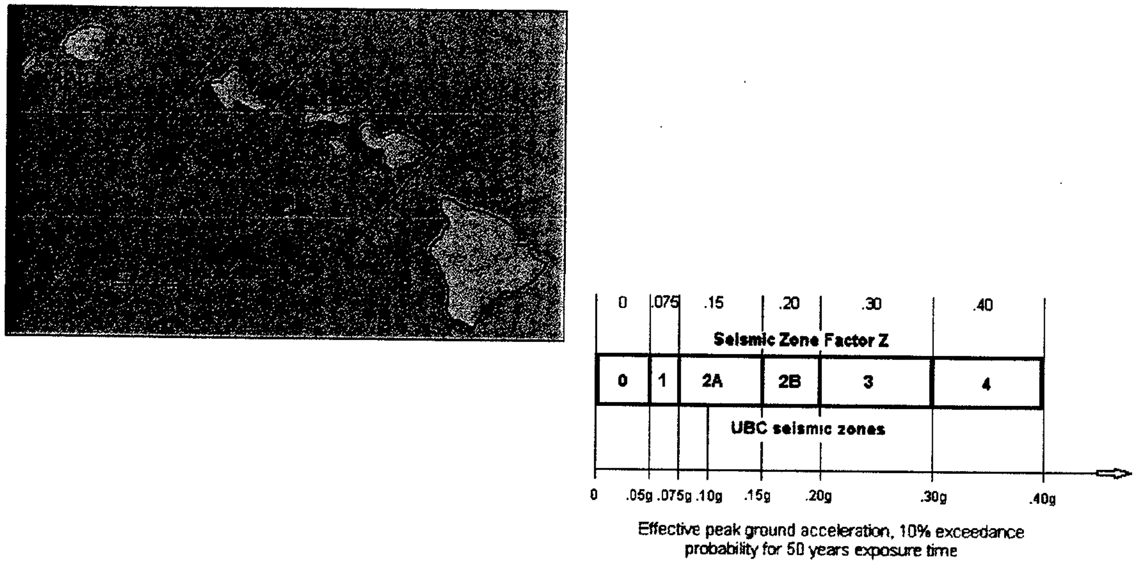


Figure 1. 1997 Hawaii seismic zone assignments (USGS)

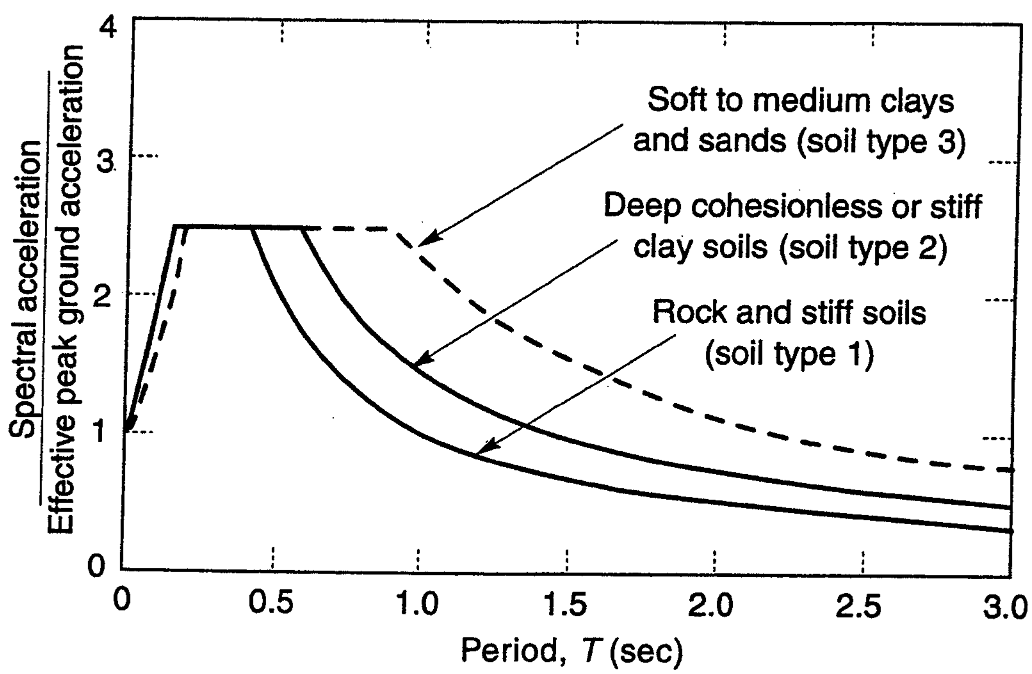


Figure 2. Normalized spectral shapes for various types of soil and rock (1994 Unified Building Code)

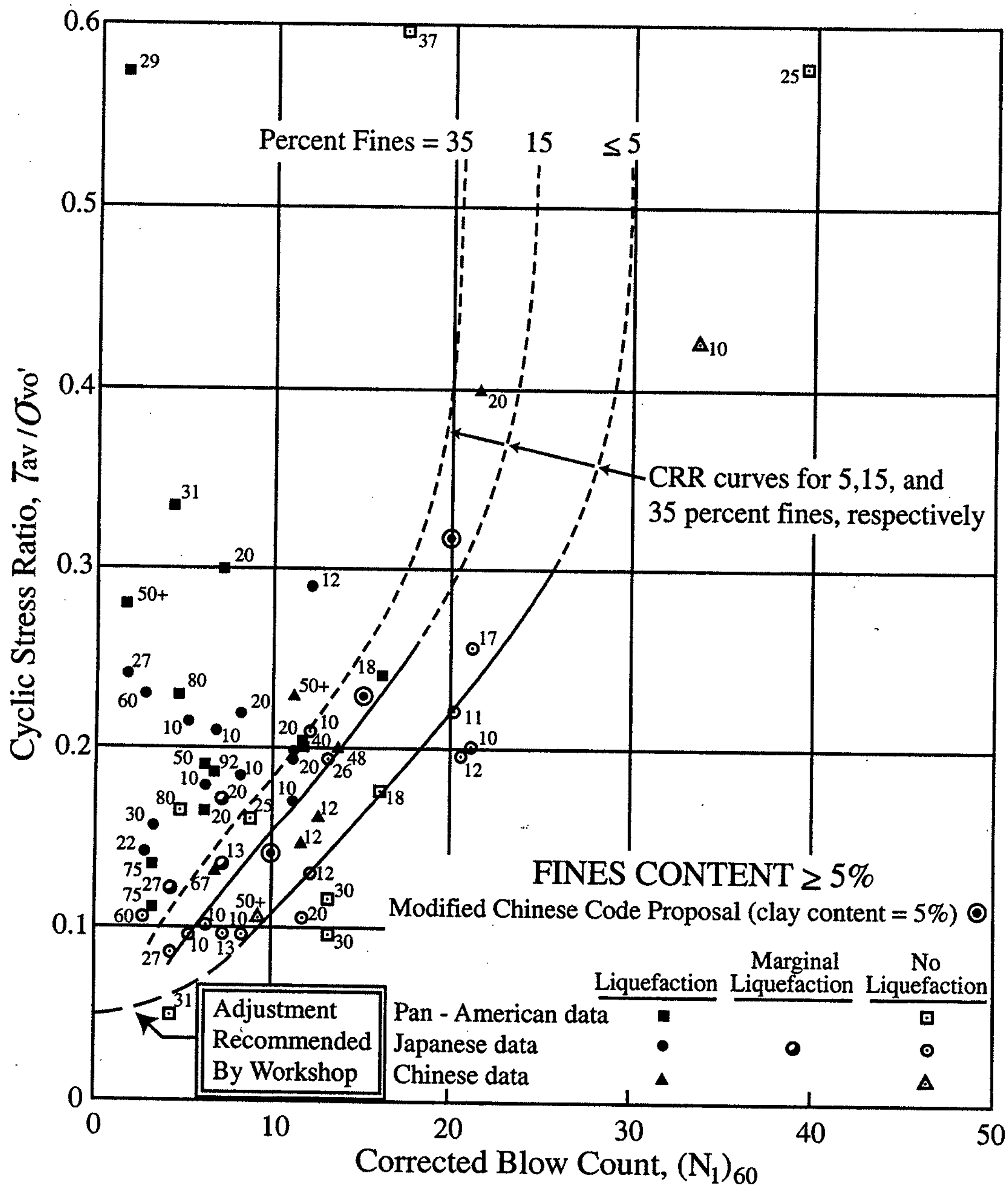


Figure 3. Liquefaction resistance based on the SPT (Youd and Noble, 1997; after Seed et al., 1985)

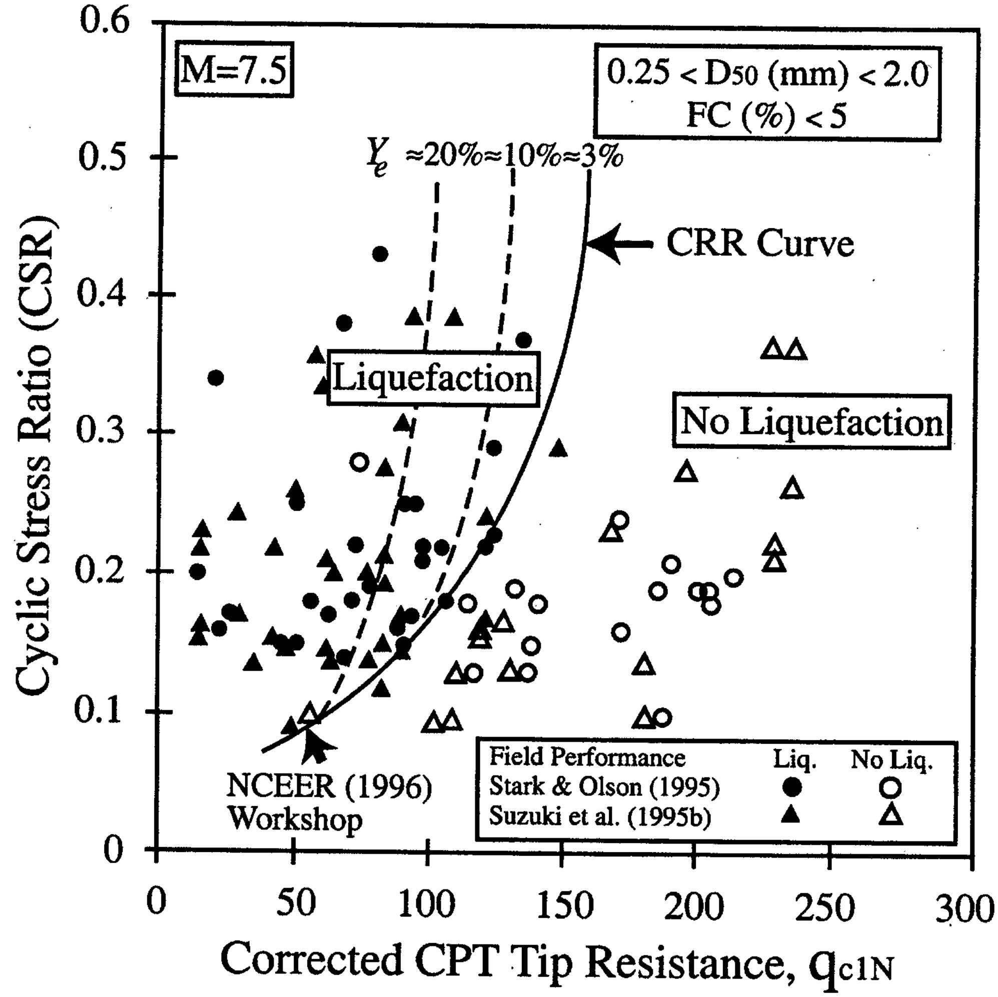


Figure 4. Liquefaction resistance based on the CPT (Robertson and Wride, 1997)

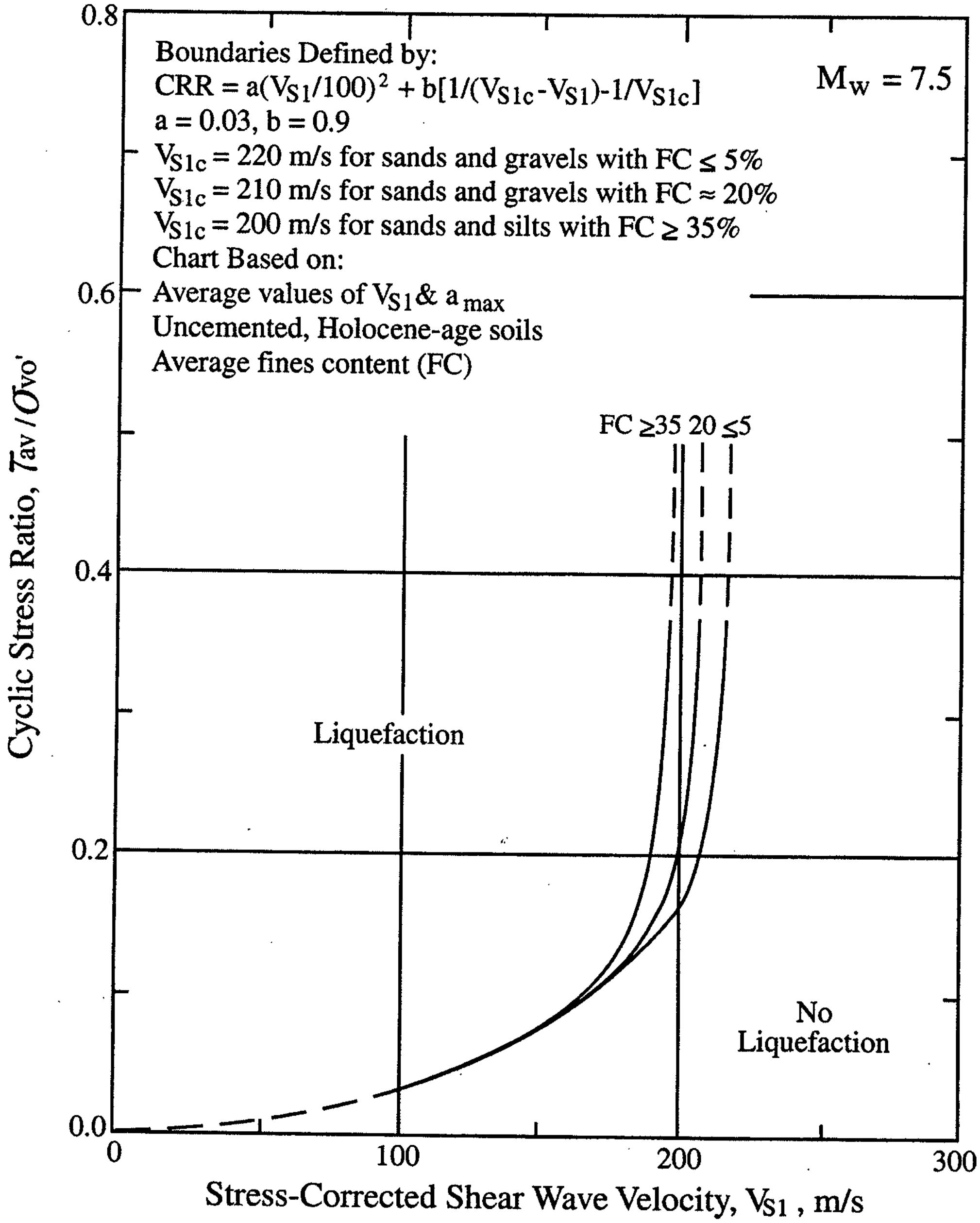


Figure 5. Liquefaction resistance based on shear wave velocity (Andrus and Stokoe, 1997)

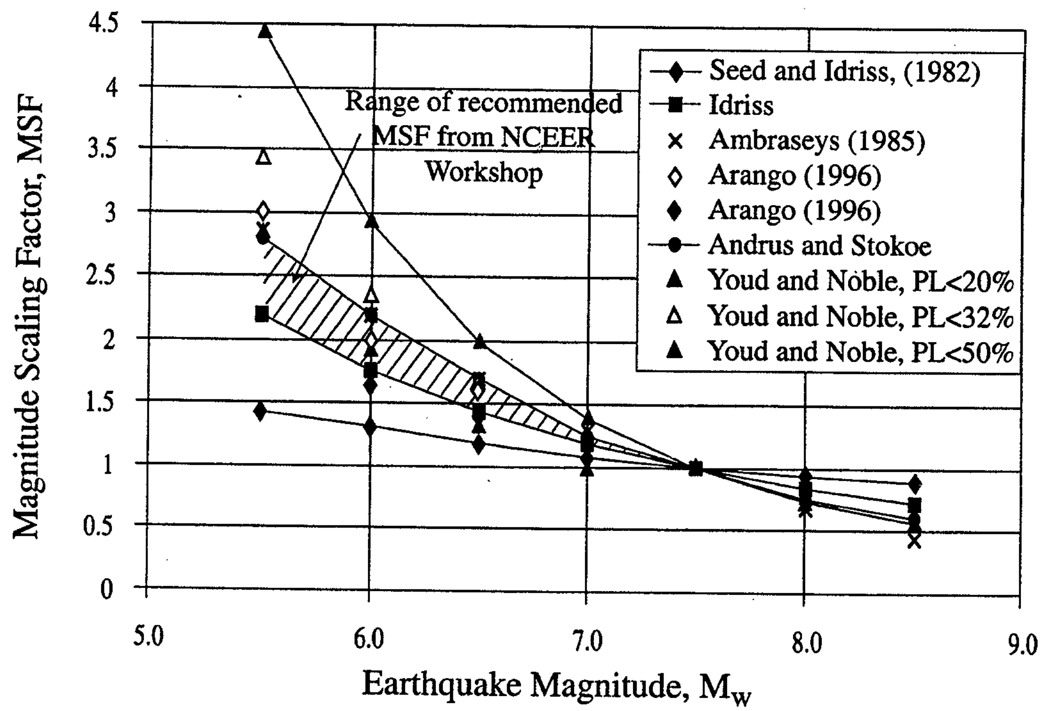


Figure 6. Range of MSF values (Youd and Noble, 1997)

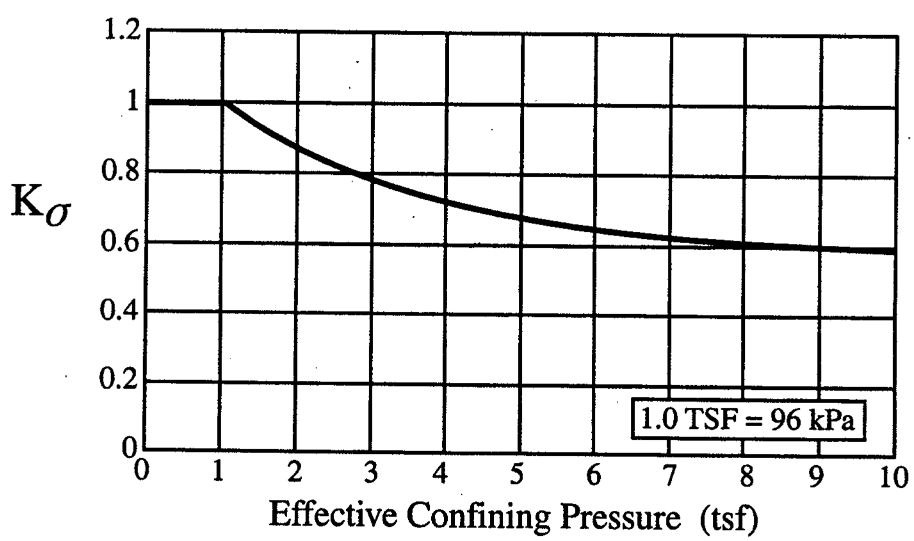


Figure 7. Correction for effective confining pressure for silty sands and gravels (Harder and Boulanger, 1997)

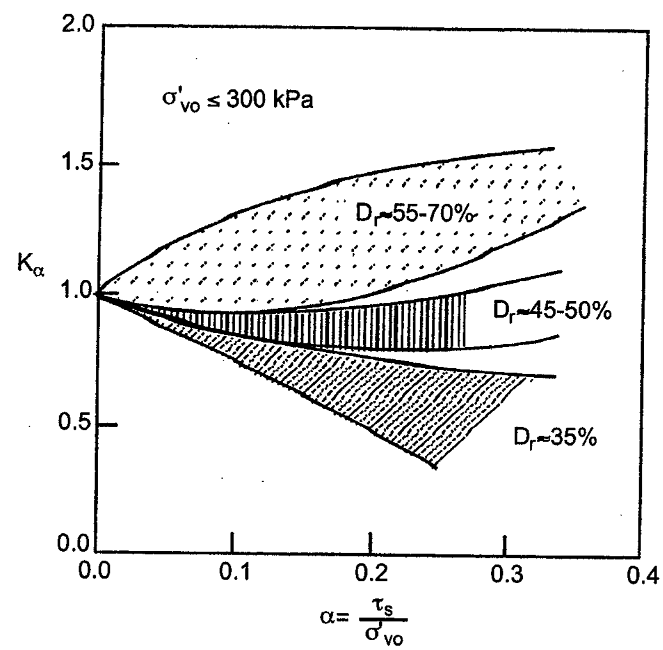


Figure 8. Effect of initial static shear stress on liquefaction potential (Harder and Boulanger, 1997)

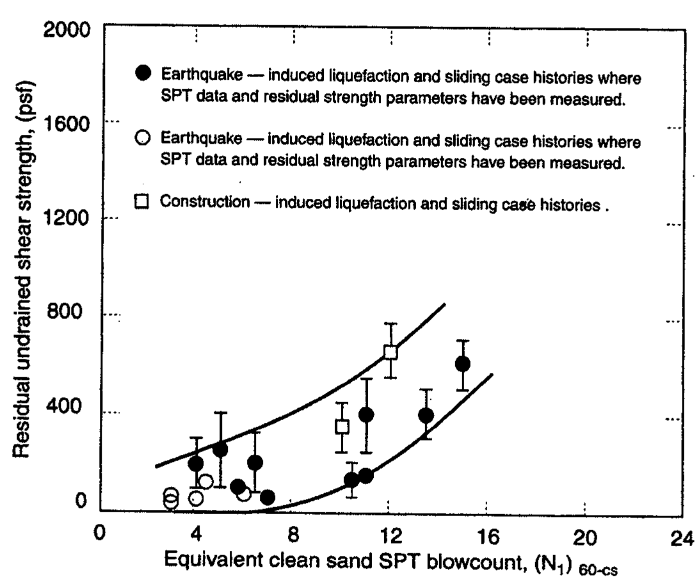


Figure 9. Residual shear strength as a function of SPT (Seed and Harder, 1990)

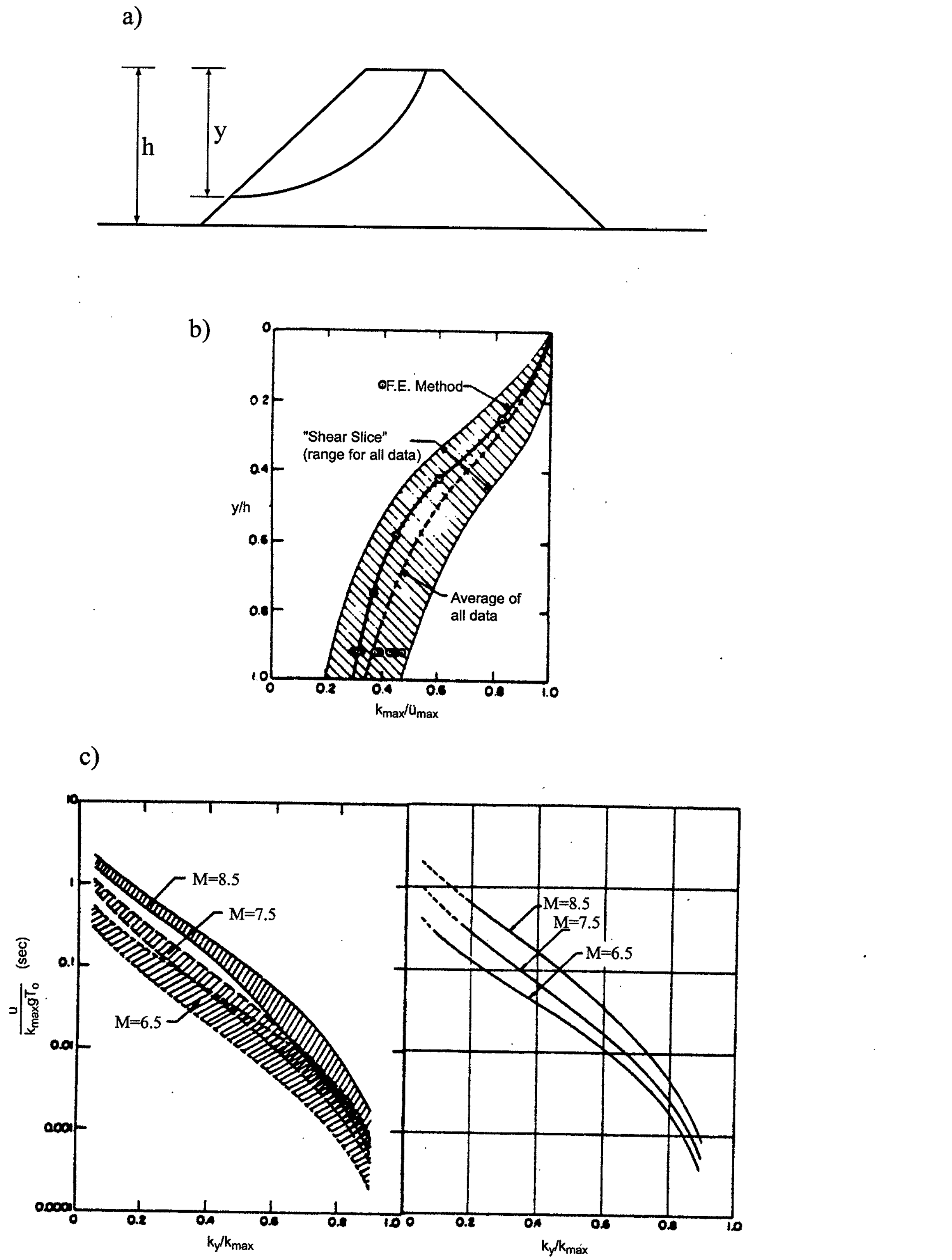


Figure 10. Simplified procedure to estimate permanent horizontal displacements for earth dams

INSPECTION OF DAMS AFTER EARTHQUAKES

1. INTRODUCTION

This section provides a guide for the inspection of dams following an earthquake. There are two phases of the inspection procedure:

(1) An immediate inspection by the dam operator (dam tender), and

(2) Follow-up inspections by dam engineering professionals

Inspection immediately following an earthquake is most crucial to decisions regarding continued operation of the structure. Follow-up inspections provide more detailed information on structure performance for the design of any needed repairs and provide insight to structural performance under seismic loading.

The inspections are most meaningful if customized procedures are prepared for each individual dam. The general procedures described herein should be used as guidelines by professional individuals conversant with the design and operation of dams for their use in the preparation of inspection procedures for a specific dam. The procedures should list all of the features to be inspected, in an order believed to be most important and efficient. Communication links to designated offices regarding inspection findings should be a part of the plan. Aspects of the inspections are discussed below, and inspection checklists are given in Appendix C to aid in preparing detailed instructions.

Additionally, it is hoped that application of these guidelines and publication of inspection findings will provide dam designers with a wealth of information on dam performance during earthquakes. Dam designers have analytical techniques and materials testing procedures that are used to estimate the behavior of materials and the response of dams subject to seismic loadings. On the basis of these estimates, dams are designed and constructed. Verification of the adequacy of design analyses and construction methods, and the establishment of a high level of confidence in these methods, within both the profession and the general public, is necessary. Such verification will be greatly enhanced through complete and meaningful reporting on the behavior of dams and appurtenant structures during earthquakes.

2. INSPECTION IMMEDIATELY FOLLOWING AN EARTHQUAKE

Dam operators should be given instructions according to the following two sets of guidelines for actions to be taken immediately following an earthquake:

2.1 Guideline A

If an earthquake is felt at or near the dam, or has been reported to have occurred, with a magnitude of 4.0 or greater within a 25-mile radius, 5.0 or greater within a 50-mile radius, 6.0 or greater within a 75-mile radius, 7.0 or greater within a 125-mile radius, or 8.0 or greater within a 200-mile radius from the site, follow these procedures (The Hawaii Volcano Observatory maintains a map of earthquakes in Hawaii and their magnitudes that is updated within about 5 minutes of an earthquake at hvo.wr.usgs.gov/earthquakes/):

(1) Immediately make an overall inspection of the dam. Check the dam and abutments for sloughs, slides, cracks, displacements, settlements, sinkholes, springs or seeps, and other signs of distress.

(2) If the dam appears to be damaged to the extent that there is increased or new flow passing downstream, immediately attempt to report the conditions to the supervisory office or, if key personnel are not available, report directly to the headquarters office. If communications cannot be established with these personnel in a few minutes, implement a procedure such as outlined in Section (2.2).

(3) If visible damage has occurred but, in your best judgment, is clearly not serious enough to cause failure of the dam, make the following observations and contacts as quickly as possible:

a. Observe the nature, location, and extent of damage. Note the rate of any changing conditions. Note the reservoir and tail water elevations, and prevailing weather conditions.

b. If the reservoir inflow is abnormally reduced, inspect the upstream watercourses to determine if any earthquake caused landslide has blocked the inflow.

c. Make an estimate of the intensity of the earthquake using the Modified Mercalli Intensity (MMI) scale described in Appendix B.

d. Report all information to the supervisory office or, if key personnel are not available, report directly to any responsible agency. Be absolutely sure to state the dam name, your name, and extent of damage, when making a phone or radio report. When damage has occurred, it is extremely important that the one receiving your report understands your evaluation and description of the potential hazard at the dam. Decisions on further actions required must be promptly made by responsible professionals.

e. Reinspect the site of the damage and maintain communications with the key personnel previously receiving the report. Take photographs and record or make notes on observations. Use a digital camera, if possible, so that the photographs can be immediately sent to the supervisory office.

f. Be prepared to make additional inspections at any time, because of possible aftershocks.

g. If it has definitely been established that there is no impending dam failure, continue to step (4).

(4) Thoroughly inspect the following for damage using a customized list appropriate for the specific dam:

a. The crest and both faces of the dam for cracks, settlement, displacement, or seepage.

b. Abutments for slide movements, cracks, or new seepage/springs.

c. Drains and seeps for increased flow or stoppage of flow.

d. Spillway structures and gates for misalignment or structural distress.

e. Outlet works control house, tunnel, and gate chamber for cracks or spalling of concrete, displacement, or valve or gate misalignment.

f. Penstocks for leaks, misalignment, cracking or concrete spalling.

g. Powerplant facilities for cracks, spalling, tripped out generators, gate or valve distress, and for any indication of water passage failure.

h. Power supply and standby power unit, and other emergency operating equipment.

i. Visible reservoir and downstream areas for landslides, new springs and sand boils, and rockfalls around the reservoir and in downstream areas.

j. Canal headworks for cracks, spalling, or structural distress.

k. Other appurtenant structures for signs of distress.

l. Tunnels and conduits for silt, sand, gravel, rock, or concrete fragments being carried in the discharge stream.

(5) Report findings to the supervisory office or to other personnel in the headquarters office to whom you previously reported after the earthquake.

(6) If no apparent damage has occurred to the dam, embankment or appurtenant structures, make a "No Damage" report to the supervisory office.

(7) Continue to inspect and monitor the facilities for at least 48 hours or as instructed by the supervisory office because initially unobservable or delayed damage may subsequently become apparent.

(8) Some damage to structures may not be readily apparent during an inspection immediately following an earthquake. It is possible that settlement of structures, reactivation of old slides, or development of new slides or springs may not have occurred during ground shaking, but could appear after the initial inspection. A secondary inspection should be made 2 weeks to a month after the initial inspection. In timing a second inspection, consider weather conditions and activities that might obliterate evidence of damage.

(9) Information on the conditions of the structures and their performance in response to the earthquake may be obtained from readings on the instrumentation installed in the dam and foundation including pendulums, inclinometers, extensometers, survey monuments, piezometers, and seismographs. Data should be collected from all instrumentation as described in the instructions for reading the instruments. A schedule of very frequent readings should be followed for at least 48 hours after the earthquake.

2.2 Guideline B

If all communications from the dam are lost, and there is an evident potential danger for failure of the dam, use the following checklist as a guide:

(1) Quickly inspect the dam and abutments for damage.

(2) Evaluate the potential danger of failure, to the best of your ability.

(3) If failure is imminent, warning to downstream residents is essential. The provisions of the emergency action plan (EAP), if any, should be instituted without delay and the Hawaii Civil Defense should be contacted immediately, or as soon as communication is reestablished.

(4) If failure is imminent, all measures should be taken to rapidly reduce storage in the reservoir. Caution should be used in increasing discharge through the outlets works because the conduit may be sheared and increased flow could cause erosion of the structure or piping of the dam embankment materials.

It may be necessary to cut off flow through the outlet works, if possible, to avoid severe damage.

(5) Continue to attempt to establish or maintain contact with any supervisory office.

3. INITIAL ENGINEERING FOLLOW-UP INSPECTION

In the event that a dam operator reports that damage has occurred, or a dam has been severely shaken, qualified engineering personnel should be dispatched as rapidly as possible to the dam to make a technical evaluation of the extent of damage and the degree of hazard it presents. The members of such an inspection team should be familiar with the possible modes and causes of dam failures and associated structures, and should also be familiar with the main features of the project. Suggested checklists for use in this phase of the inspection are presented in Appendix C, and guidance for the inspection is given in this section.

3.1 Possible Modes and Causes of Dam Failures

The members of an inspection team must be aware of the modes of dam failures, both static and dynamic. Research and study of previous failures are required for the team members to reinforce their engineering understanding of why and how failures occur. Publications can be found in journals, conference proceedings, reports and books.

Weakness in a dam or foundation may take many forms. Some of the more common causes of static and dynamic dam failures, and examples of adverse conditions are discussed in this section (also see page 3). Adverse conditions that can lead to failure are categorized as follow:

Failure Category	Causes
Foundation instability	Liquefaction Slides Removal of solid and/or soluble materials by water Subsidence Differential movement Joint openings and/or grout curtain rupturing Movement on faults under or adjacent to dams
Defective spillways	Obstructions Damaged gates or hoists Slab, pier, or wall displacements Crushed or deteriorated concrete
Defective Outlets	Obstructions

	Damaged valves or hoists Damaged inlet/outlet structure walls, slabs or tower Crushed or deteriorated concrete
Concrete dam effects	High uplift Unanticipated uplift distribution Differential displacements and deflections Excessive seepage Overstressing, particularly at downstream toe, as may be evidenced by cracking or crushing of the concrete
Embankment dam defects	Liquefaction Slope instability Excessive seepage Removal of solid and/or soluble material Surface soil erosion caused by overtopping Embankment settlement producing inadequate freeboard for required spillway capacity Cracks or sinkholes
Reservoir margin defects	Slope instability Inherent weakness of natural barrier Sinkholes Landslide blockage of upstream watercourses

3.2.1 Foundation Deficiencies. These deficiencies are associated with the quality of the foundation materials or with the foundation treatment before, during and after construction of the dam. Differential settlement, slides, excessive pressures, weak seams or zones, and inadequate control of seepage are all potential failure mechanisms within a foundation. Foundations that have low shear strength or seams of weak material such as soft alluvium, smectite-rich silt and clay, highly weathered basalt, weakly cemented soil and gravel, or fault gouge can result in sliding of the foundation and embankment. Also, seams of pervious material in the foundations, which have no provisions for pressure relief, allow transmission of excessive uplift pressures and can cause sliding.

Seepage through foundations can cause piping of solid materials or the erosion of soluble materials by dissolution. Such removal of foundation material forms voids, which can increase until a portion of the remaining unsupported material collapses and failure of a section of the foundation occurs. Water can also cause a breakdown of some foundation materials such as dispersive soils or shales, or reduce the shear strength of the foundation rock, or of the dam/rock contact.

Some of these weaknesses can be identified by visual examination of the foundation environs. Visible cracks in a dam can be indicative of foundation movement. Visual evidence of piping usually consists of sediment carried away by seepage water. On the other hand, the washing of soluble material would require chemical analysis both of the reservoir and seepage waters, unless it is accompanied by marked changes in the color of the effluent.

3.2.2 Spillway and Outlet Works. Many adverse conditions such as obstructions to the flow, structural weakness, or faulty underdrains can be identified by visual examination. Structural failure in a conduit, tunnel, or other conveyance structure could obstruct the flow in the system, which would be evidenced by a reduction or unusual turbulence in the flow. Loss of the power source to operate facilities may also present operational conditions, which compromise the safety of the dam.

Spillways and outlets works controlled by gates and/or valves can only function properly if the gates and valves can be operated as intended. If a spillway or outlet works cannot be operated because of faulty gates, valves or operating equipment, the dam could be in danger of failure. Faulty operation of gates, valves, or operating equipment can result from settlement or shifting of the support structure, which could cause binding of gates or blockage by debris. If damage is suspected or likely, gates and/or valves should be operated soon after the earthquake to verify their operability.

Slides from the slopes above inlets can block approach channels. Slides could also damage intake structures and associated mechanical equipment such as gates, hoists and motors.

Cracking and movement of concrete structures may indicate distress. Floor slabs in a chute and stilling basin may be displaced by seismic activity and may change the drainage capabilities and cause excessive uplift.

3.2.3 Seepage. The main paths of seepage within a concrete dam are through contraction joints or along poorly bonded construction joints or lift lines. Cracks in mass concrete are also potential seepage paths. Formed drains installed in dams are designed to intercept the seepage and reduce the pressures from seepage, which could develop along lift lines, along the foundation contact or in cracks. Increased uplift at the foundation of the dam from percolation or seepage of water along underlying foundation seams or joint systems may be an indication of reduced effectiveness of the foundation grouting and the drainage system. If the uplift values are extreme or exceed the design assumptions, the stability of the dam may be reduced.

Uncontrolled seepage through the abutments or the foundation of a concrete dam can form pipes or voids, causing bridging of sections of an abutment or foundation, resulting in an undesirable concentration of stresses in the concrete.

Uncontrolled seepage through an embankment dam or foundation can result in excess pore pressures which weaken the soil mass and may cause springs, boils, or slope failures and/or the movement of soil particles to unfiltered exits, creating voids which can lead to a piping failure. The pipes or tunnels under an embankment also can cause the collapse of surrounding materials. This can lead to the formation of settlement cracks or, ultimately, to breaching of the embankment.

3.2.4 Defective or Inferior Materials. Low-density, saturated, cohesionless soils in an embankment or foundation can experience an increase in pore pressure and loss in shear strength when subjected to earthquake-induced shear stresses. Depending on a variety of factors, including material properties and in-place conditions, pre-earthquake stress conditions, and magnitude and duration of seismically induced stresses, the embankment or its foundation can exhibit instability, excessive vertical settlements, and loss in freeboard or cracking. Embankment dams constructed by hydraulic fill techniques have been found to be particularly susceptible to earthquake-induced damage because of the potential for liquefaction under earthquake loading.

Weak concrete due to poor aggregate, inadequate lift joint preparation and concrete that has deteriorated with age is especially vulnerable to damage.

3.2.5 Concrete Dam Overstressing. Overstressing in a concrete dam normally creates areas of distress and cracking that usually can be identified visually. Cracking, opening of lift lines or construction joints, changes in seepage and differential movements are all indications of potential overstressing. The overstressing may occur along the foundation because of differential or extreme foundation movements, or at any location in the mass concrete of the dam where stresses are excessive. The overstressing may be due to loading conditions such as earthquakes, temperature variations, contraction joint grouting pressures, foundation movements or excessive uplift pressures in the foundation along lift lines.

3.2.6. Reservoir Margin Defects. Landslides are the most prevalent form of instability affecting reservoir margins. The size of a landslide usually is the primary consideration when evaluating its safety aspects; however, a small landslide in a critical location could disable a spillway or outlet works and create an unsafe condition for the dam.

Landslides may dam watercourses into reservoirs. Subsequent overtopping of the landslide dams can cause them to fail rapidly and send surges to reservoirs threatening the dams impounding them and their appurtenant structures.

3.3 Post-Earthquake Inspection of Project Features

All project features should be inspected to determine whether there are any changes that may have been a result of the earthquake. Notes should be taken or observations recorded on a portable recorder. Sketches or videotapes may help to describe the nature and extent of any damages. Photographs should be obtained as soon as possible of any visible results from the seismic activity. These records will be invaluable in future comparisons to determine if there is additional distress developing in the structures. Photographing with a digital camera and locating observations and photographs using a geographic positioning sensor, if available, allows the information to be immediately transmitted and readily incorporated into incident reports. Measurements and readings should be taken of all instrumentation installed in the dam and foundation and in the immediate area, such as survey monuments and piezometers. Historical seepage measurements should be reviewed when available. Precise surveys, temporary strong-motion seismographs, and other instrumentation may be desirable to monitor structures and individual damage locations. Special steps need to be taken to ensure that records from the seismographs are properly extracted and transmitted to those responsible for their interpretation.

3.3.1. Embankment Dams. The external surfaces of an embankment dam can often provide clues to the behavior of the interior of the structure. For this reason, a thorough examination of all exposed surfaces of the dam should be made.

Surface displacement of an embankment often can be detected by visual examination. Sighting along the line of embankment roads, parapet walls, utility lines, guardrails, longitudinal conduits, or other lineaments parallel or concentric to the embankment axis can sometimes identify surface movements of the embankment. The crest should be examined for depressions and crack patterns that could indicate sliding, settlement, or bulging movements. The upstream and downstream slopes and areas downstream of the embankment should be examined for any sign of bulging, depressions, or other variance from smooth, uniform face planes. If a permanent system of monuments for measurement of movement exists, and if any movement is suspected, a resurvey should be made without delay.

Cracks on the surface of an embankment can be indicative of potentially unsafe conditions. Surface cracks are often caused by desiccation and shrinkage of materials near the surface of the embankment and/or differential settlement between zones; however, the depth and orientation of the cracks should be determined for a better understanding of their cause. Openings or escarpments on the embankment crest or slopes can identify slides and a close examination of those areas should be made to outline the location and extent of the slide mass. Surface cracks near the embankment/abutment contacts, as well as contacts with other structures can be an indication of settlement of the embankment and, if severe enough, a path for seepage can develop along these contacts. Therefore, these locations must be thoroughly examined. Backhoe trenches are often excavated to determine the depth of cracking. Pouring a suitable tracer fluid into cracks before trenching aids in delineating cracks in trench walls.

The downstream face and toe of the dam, as well as areas downstream from the embankment and abutments should be examined for wet spots, boils, depressions, sinkholes or springs which may indicate concentrated or excessive seepage through the dam or abutments. Any of these conditions may be in a developing mode and, if they worsen and are not corrected, ultimately could lead to failure of the embankment. Other indicators of seepage are soft spots, deposits from evaporating water and abnormal growth of vegetation. Seepage water should be examined for any suspended solids (turbidity) and, if dissolution is suspected, samples of the seepage and reservoir water should be collected for chemical analyses. Seepage should also be tested for taste and temperature to help identify its source. If saturated areas are located, they should be studied to determine if the wet spots are a result of surface moisture, embankment seepage, or derived from other sources. Wet areas, springs, and boils should be located accurately and mapped for comparison with future inspections. Seepage should be measured and monitored at increased frequency to ensure that an adverse trend does not develop which could lead to an unsafe condition.

Drainage systems should be inspected for increased or decreased flow and for any obstructions, which could plug the drains.

In addition to verifying anticipated embankment and foundation performance, instrumentation also can be an indicator of developing unsafe conditions. Readings should be made frequently if earthquake shaking has changed the historical steady state readings. Earthquakes can cause increases in piezometric levels by shaking, causing soil volume reduction and/or shearing, and indirectly by earthquake compacted soils transferring loads to stiffer soils.

3.3.2. Concrete and Masonry Dams. Concrete dams encompass a variety of structures, which include gravity, slab and buttress, multiple arch, and single arch dams. Masonry dams may be considered as gravity structures with many joints. Regardless of the type, all concrete dams are subject to the same basic considerations with respect to safety.

Concrete dams should be checked for indications of excessive stress and strain as well signs of instability. Most large concrete dams have survey points and/or plumb lines for regularly scheduled measurements of movement within the dam, the results of which can be plotted to determine the behavioral trend. There are obvious indications of movement that can be noted during an inspection. A masonry or gravity dam usually can be checked by sighting along the parapets or handrails from one abutment to the other. Each contraction joint or row of masonry locks should be examined for evidence of differential movement between adjacent blocks. The joints should be examined for evidence of excessive expansion or contraction and excessive movement. The foundation contacts should be examined for any evidence of differential movement between the dam and the foundation.

All cracks and spalls on dam faces and in galleries should be examined. Gravity dams are more likely to show new cracking in their upper parts and arch dams near their abutments and crest. Gallery cracks should be examined to see if they coincide with face cracks. Cracks and spalls noted during past inspections should be examined for any change of condition. New cracks and spalls should be noted and examined to determine the type, such as tension or crushing, and the reasons for their existence. They should be marked and measured so that any changes can be detected during subsequent inspections.

Seepage should be examined to determine the possible sources such as poor bond on lift lines, waterstop failure, structural cracks, and erosion of mortar. The quantity of seepage should be compared with previously observed quantities to determine if there has been any significant change in the flow for similar reservoir elevations.

Drain and weep holes should be checked to determine if they are open and functioning as designed. Drains in the foundation and the dam should be examined to determine if there have been significant changes in their flow.

3.3.3 Abutments and Foundation. The upstream portions of abutments and foundations are normally submerged within the reservoir. Therefore, physical examination is typically limited to the downstream abutment contacts and toe of dams. Grouting and drainage tunnels also may be available for inspection. Portions of foundations of appurtenant structures may be exposed for inspection.

Indications of harmful seepage may be either quite obvious or very subtle. Changes in measured flow from monitored drains, whether increases or decreases, are immediately suspect. Another indication of changes may be increased frequency of sump pump operation. The presence of suspended particles in seepage water is evidence that piping is taking place and is cause for immediate concern. Joint opening caused by earthquake shaking can rupture grout curtains. On the other hand, small increases in seepage (a few gallons per minute) are common and may be caused by minor opening of rock joints. Reduction of seepage back to pre-earthquake levels should start within a few hours after the earthquake.

When the possibility of dissolution of substances exists, samples of reservoir and seepage water should be collected for chemical analysis. Such analysis can identify the soluble material. If the rate of seepage can be determined, the rate of dissolution can be estimated.

3.3.4 Reservoir. The regions around the reservoir should be examined for indications of problems, which may affect the safety of the dam or reservoir. Landforms and regional geologic structures should be assessed. The region should be checked for subsidence indications such as sinkholes, depressions, and settlement of roads and road structures. The reaction of other structures on the same geologic formation may provide information on the possible behavior of the dam and its appurtenances. Whenever an inspection is made, the elevation of the reservoir should be recorded.

The reservoir basin surfaces should be examined for depressions, sinkholes, or erosion of natural surfaces or reservoir linings. The reservoir basin should also be inspected for excessive siltation, which can adversely affect the loading of the dam or obstruct the inlet channels to the spillway or outlet works.

Drainage basins in areas adjacent to but outside of the reservoir rim should be examined. Any new springs or seepage areas may indicate that reservoir water is passing through the reservoir rim. Such seepage may also cause land instability in these areas.

3.3.5 Landslides. Landslides, as used herein, include all forms of mass movement that can affect dams, appurtenances, reservoirs, or access routes. They include active, inactive, and potential slide areas, which can range from minor slope raveling to large volume movements. In addition to slide phenomena, inspections should also determine if there has been a toppling or sliding of intact rock blocks or masses. Those can occur not only in reservoirs but also in abutments of dams and above powerhouses. Landslides might form natural dams on tributary streams or cause waves on reservoir surfaces.

At least one professional team member should be knowledgeable about landslide causes, mechanisms, characteristics, symptoms, and treatment. Slide areas often can be identified by escarpments, leaning trees, hillside distortions, or misalignment of linear features.

3.3.6 Appurtenant Structures. All appurtenant structures that could affect the safe operation of the dam should be examined. These structures include spillways, outlet works, power outlets and power plants, and canal outlets. Those structures may include any or all of the following features:

Practically every hydraulic structure is served by approach and outlet channels composed of cut or fill slopes of soil or rock. Unlined spillways may have a concrete or solid rock control section to reduce seepage or erosion potential past the dam. Outlet works approach channels may be submerged and therefore may require special underwater investigation. Channel erosion protection adjacent to energy dissipation structures should be examined to determine if it is performing as designed. Special attention should be given to the possibility that the material may wash either out of the channel or back into the structure during operation.

The channels should have stable slopes and be free of sloughs, slides, and debris. They should be examined for evidence of sinkholes, boils, or piping. The channels should provide satisfactory clearance around intake and terminal structures so the structures can operate hydraulically as designed.

Concrete portions of spillways, outlet works, and power outlets all serve similar basic hydraulic and structural functions. Examination techniques and objectives therefore are similar. Tunnels and conduits should be examined for stress cracks, bulges, shifts in alignment, and excessive seepage. All passages, water and air, should be free of obstructions. Areas susceptible to collecting debris should be noted. Structures, especially towers and shafts, should be examined for evidence of differential settlement.

Outflows should be examined for turbidity and presence of soil, rock or concrete fragments, which may mean that the conduit has been breached and embankment or foundation material is being eroded. On the other hand, the source of turbidity may only be reservoir sediments that were stirred up by earthquake shaking. Repeated observations are usually necessary to identify the source of turbidity.

Fill adjacent to structures should be examined for subsidence or an increase of depth caused by soil movement, and contacts between the fill and the structures should be examined for evidence of piping. Cut or fill slopes adjacent to the structures should be examined for unstable conditions.

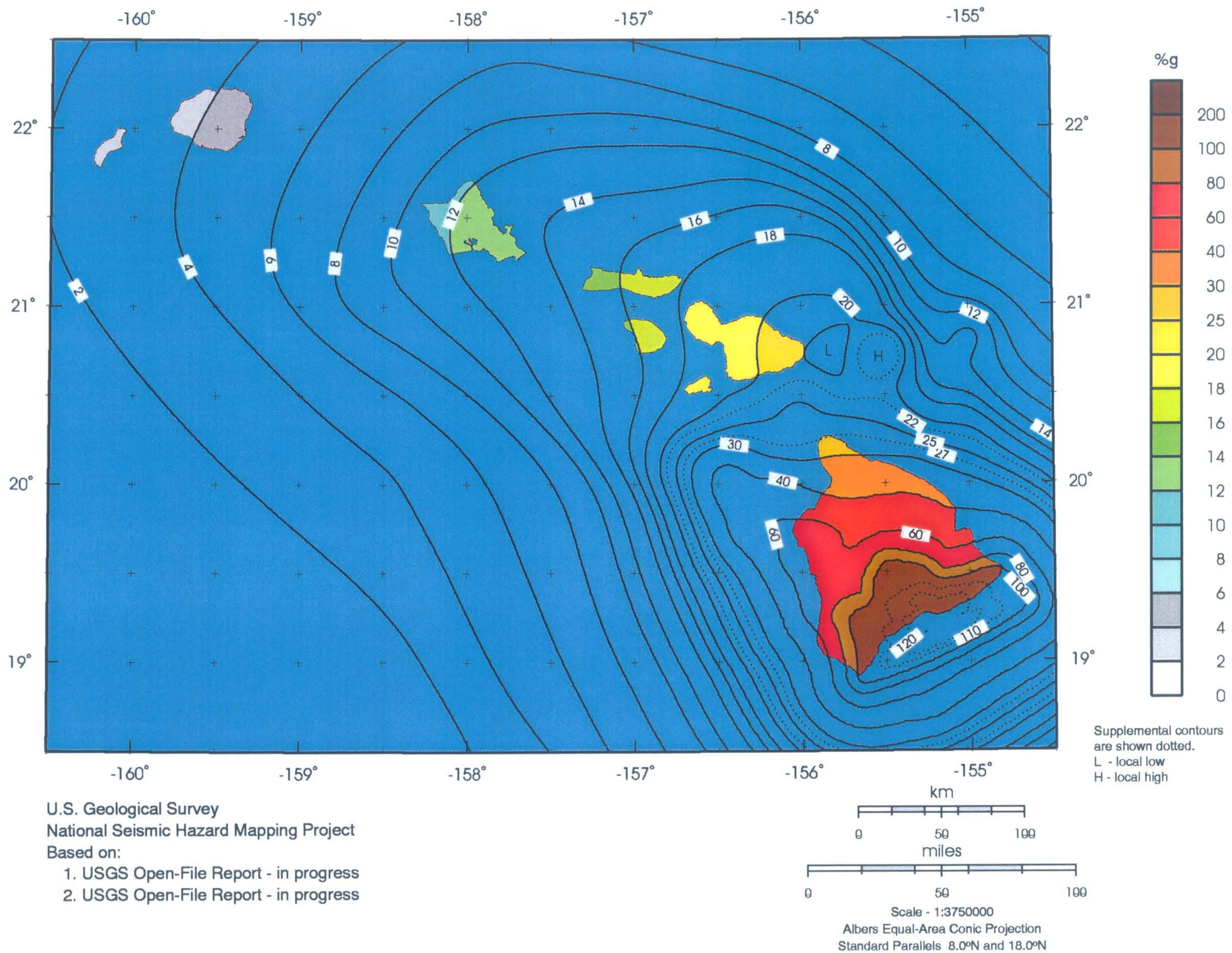
Bridges and hoist decks along with their structural members should be examined for condition and proper function. All guides for trashracks, gates, or other mechanical features should be in good condition. All drains should be open and show evidence of proper functioning. Stilling basin drain air vents should be examined to determine if the screens are in place and the vents are open.

Mechanical and associated electrical equipment should be operated through the full operating range to determine that the equipment performs satisfactorily. The equipment should be checked for proper lubrication and smooth operation without bin vibration, unusual noises, and overheating. The adequacy and reliability of the power supply also should be checked during operation of the equipment. Auxiliary power sources and remote control systems should be tested for adequate and reliable operation. All accessible equipment should be examined for damaged, loose, worn, or broken parts.

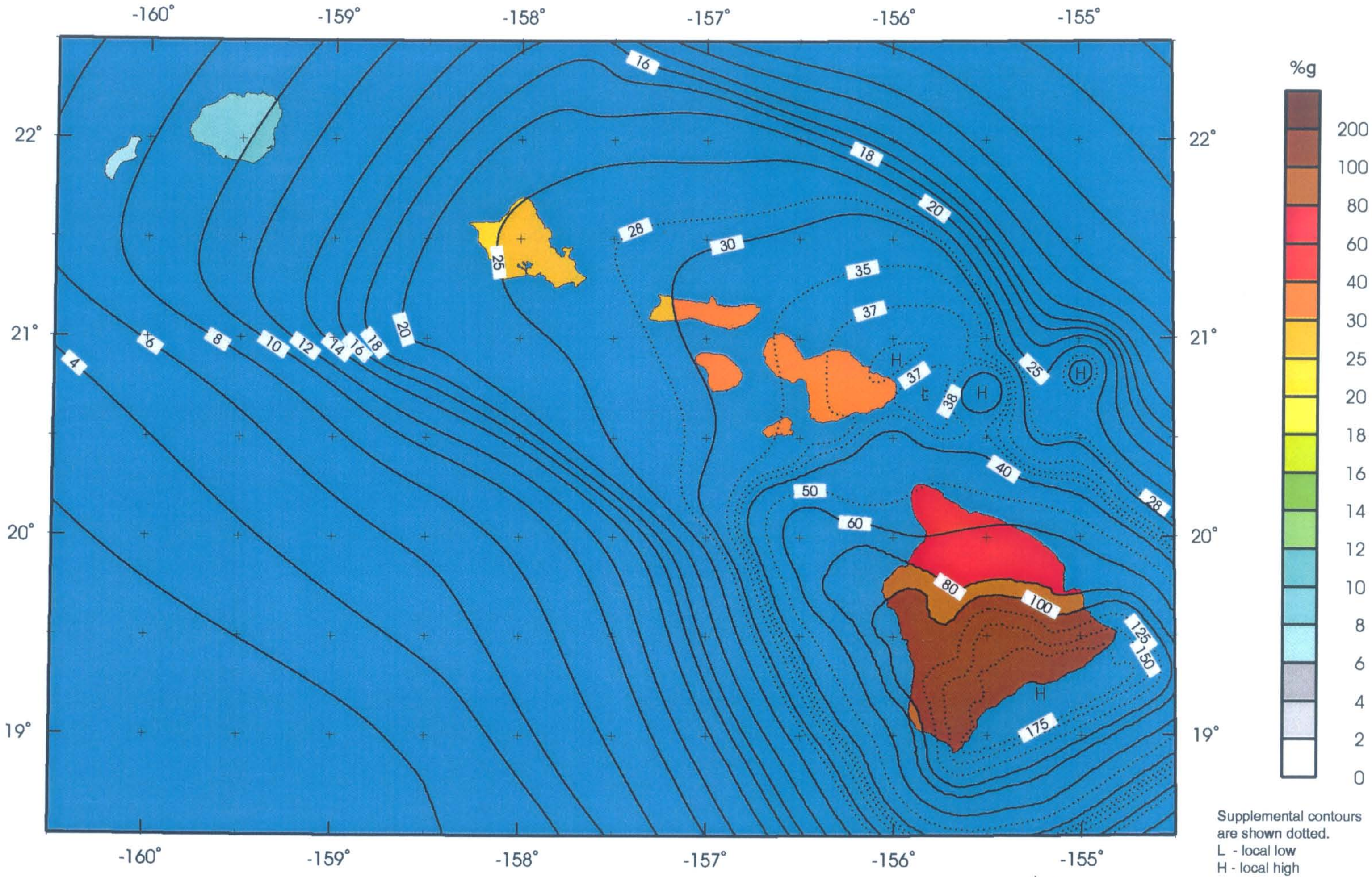
Wire ropes should be examined for broken wires, and wire rope or chain connections at gates should be examined for cracking and leakage. Hydraulic hoists and controls should be checked for oil leaks. Gate stems and couplings should be inspected for broken parts. Fluidways, leaves, metal seats, and seals of gates and valves should be examined for damage due to wear, misalignment, and leakage. Sump pumps should be examined and operated to verify reliability and satisfactory performance. Air vents for gates and valves should be checked to confirm that they are open and protected. Access ladders, walkways, and handrails should be examined for broken parts or other unsafe conditions.

APPENDIX A

PROBABILISTIC GROUND MOTIONS FOR STATE OF HAWAII



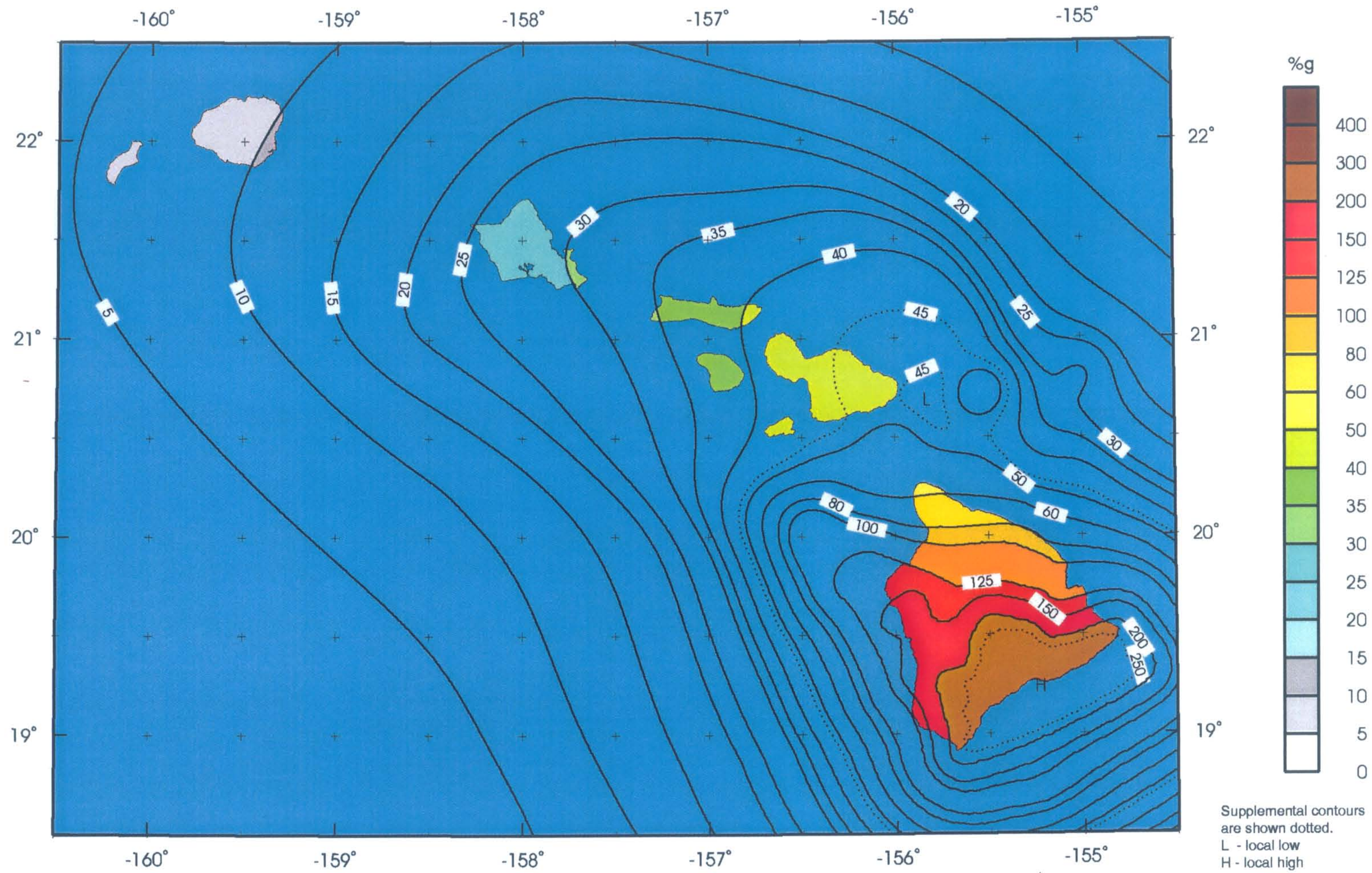
Horizontal Ground Acceleration (%g)
With 10% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity



U.S. Geological Survey
National Seismic Hazard Mapping Project
Based on:
1. USGS Open-File Report - in progress
2. USGS Open-File Report - in progress

km
0 50 100
miles
0 50 100
Scale - 1:3750000
Albers Equal-Area Conic Projection
Standard Parallels 8.0°N and 18.0°N

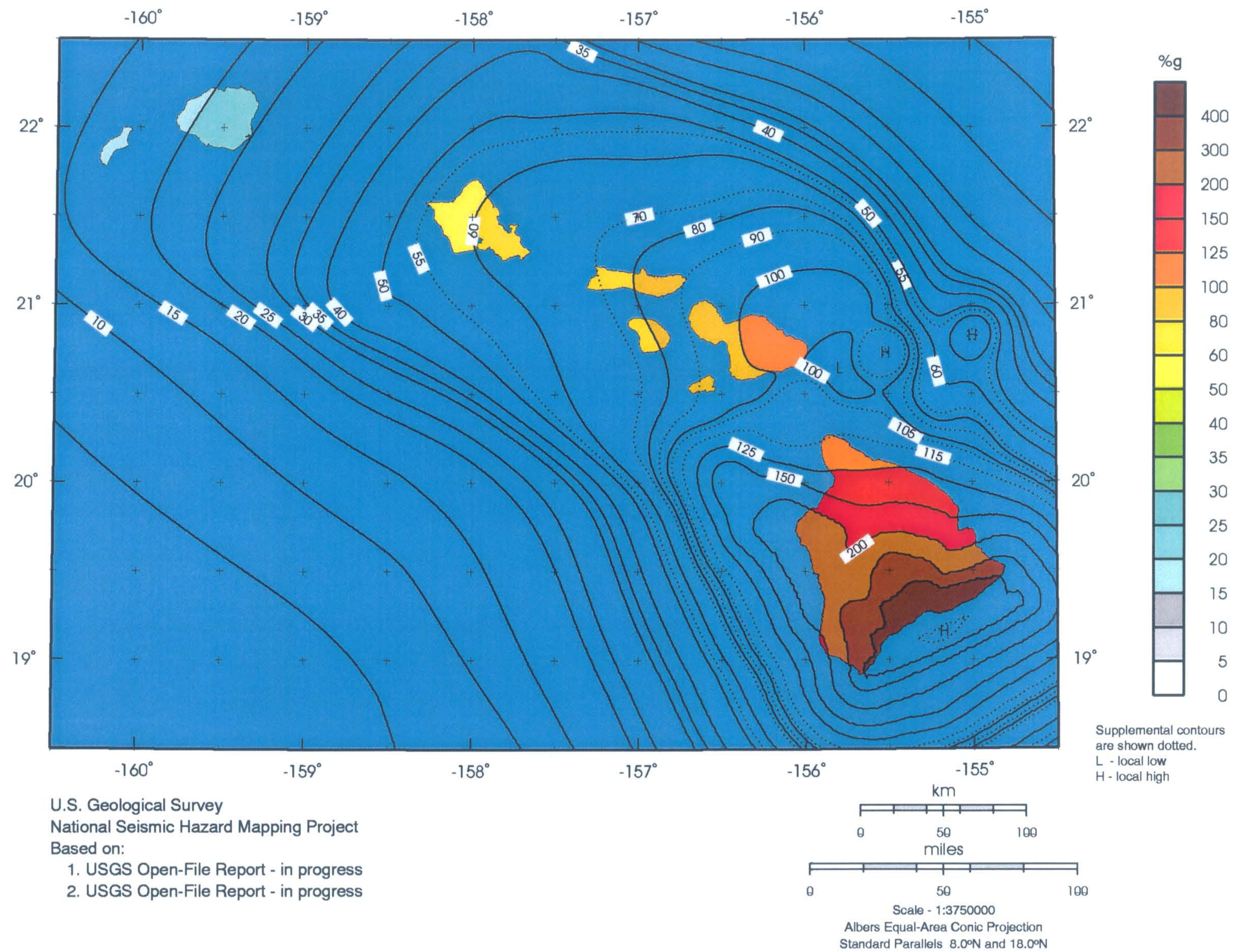
Horizontal Ground Acceleration (%g)
With 2% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity



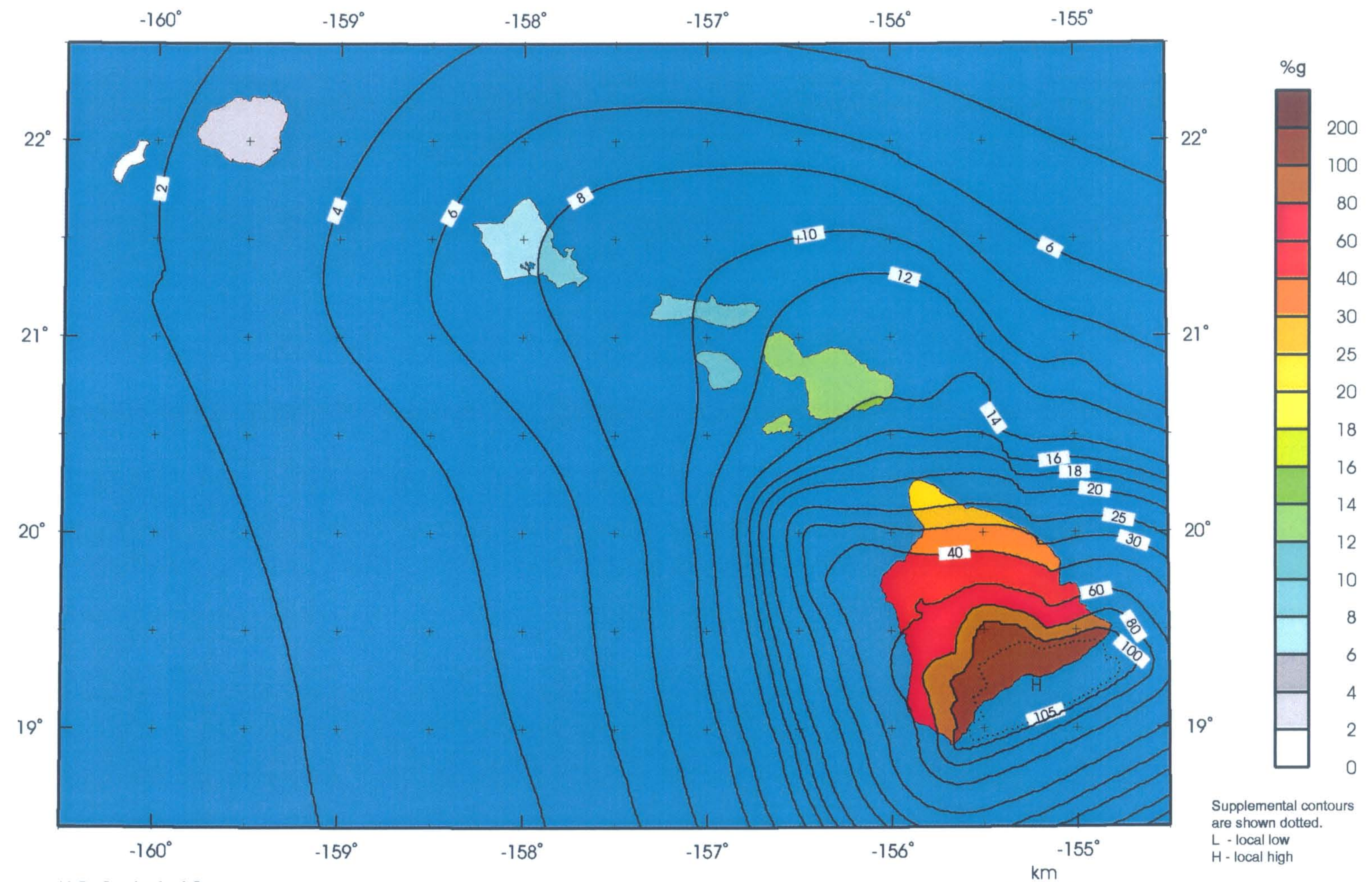
U.S. Geological Survey
National Seismic Hazard Mapping Project
Based on:
1. USGS Open-File Report - in progress
2. USGS Open-File Report - in progress

km
0 50 100
miles
0 50 100
Scale - 1:3750000
Albers Equal-Area Conic Projection
Standard Parallels 8.0°N and 18.0°N

Horizontal Spectral Response Acceleration (%g) for 0.2 Sec Period (5% of Critical Damping)
With 10% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity



Horizontal Spectral Response Acceleration (%g) for 0.2 Sec Period (5% of Critical Damping)
With 2% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity



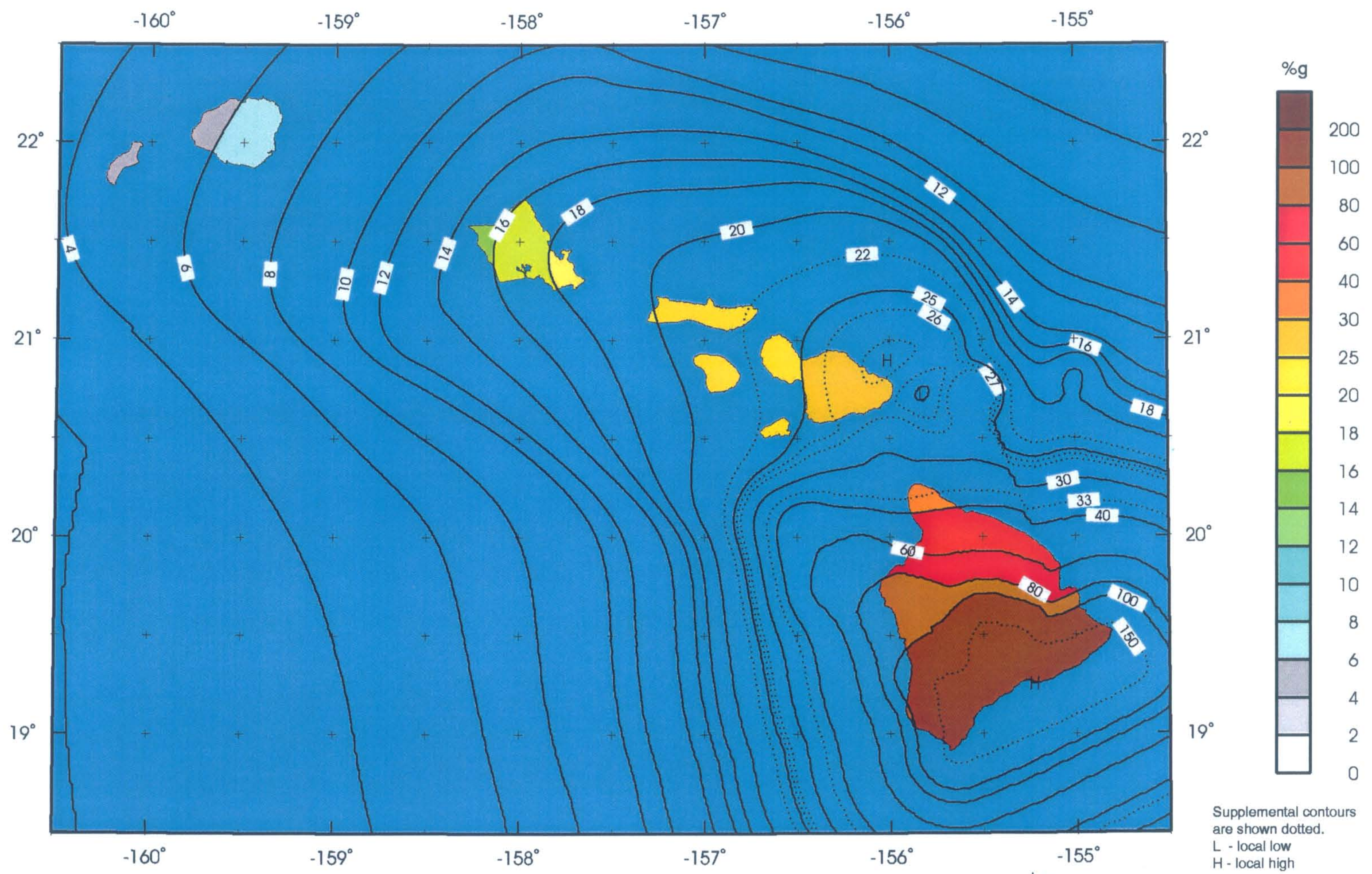
Supplemental contours are shown dotted.
L - local low
H - local high

U.S. Geological Survey
National Seismic Hazard Mapping Project
Based on:
1. USGS Open-File Report - in progress
2. USGS Open-File Report - in progress

km
0 50 100
miles
0 50 100

Scale - 1:3750000
Albers Equal-Area Conic Projection
Standard Parallels 8.0°N and 18.0°N

Horizontal Spectral Response Acceleration (%g) for 1.0 Sec Period (5% of Critical Damping)
With 10% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity



U.S. Geological Survey
National Seismic Hazard Mapping Project
Based on:
1. USGS Open-File Report - in progress
2. USGS Open-File Report - in progress

km: 0 50 100
miles: 0 50 100
Scale - 1:3750000
Albers Equal-Area Conic Projection
Standard Parallels 8.0°N and 18.0°N

Horizontal Spectral Response Acceleration (%g) for 1.0 Sec Period (5% of Critical Damping)
With 2% Probability of Exceedance in 50 Years
Firm Rock - 760 m/sec shear wave velocity

APPENDIX B

MODIFIED MERCALLI INTENSITY SCALE

Intensity	
I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of a truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably
V	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of tress, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved and a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, and walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving in motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed over banks.

XI Few, if any, masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.

XII Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

APPENDIX C

INSPECTION CHECKLISTS

The following inspection checklists should be adapted to each individual dam by reviewing design plans, specifications, engineering reports, and by discussing them with their operators. All observations should be fully described using words and sketches as appropriate. If an item is not applicable, it should be indicated so on the checklist. Also, records should state if nothing is found to be wrong or damaged, or if inspection was not done and the reason for it.

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR DAMS

GENERAL INFORMATION

Name of Dam: ____________________
____

State I.D.: ____________________

Type of Dam: ____________________

Date of Inspection: ____________________

Time of Inspection: ____________________

Inspection Team: ____________________
____
____
____
____
____

Description of earthquake as felt at site (refer to Mercalli Intensity Scale):

____
____
____
____
____
____
____
____
____
____
____

Operational Status at Time Of Inspection

Reservoir water elevation ______________
Outflow ______________
Weather conditions ______________
Water in storage ______________

Reservoir Rise During earthquake ___________

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

EMBANKMENT STRUCTURE

Upstream Face

- Slide movements ______
- Erosion - breaching ______
- Cracks ______
- Sinkholes ______
- Settlement ______
- Displacement ______
- Slope protection ______
- Debris ______
- Unusual conditions ______
- ______

Crest

- Surface cracking ______
- Settlement ______
- Lateral movement ______
- Camber ______

Downstream Face

- Slide movements ______
- Erosion - breaching ______
- Cracks ______
- Sinkholes ______
- Settlement ______
- Displacement ______
- Unusual conditions ______
- ______

Abutments

- Cracks, open joints ______

Abutments (continued)

- Erosion ______
- Sinkholes ______
- Slide movements ______
- Unusual conditions ______
- ______

Drainage/Inspection Adits

- Lighting, ventilation ______
- Total drain flow ______
- Individual drain flows ______ ______
- Cracks ______
- Seepage ______
- Joint offsets, openings, spalling ______
- Rockfalls ______

Seepage, Toe Drains, Galleries, Relief Drains

- Locations ______ ______ ______
- Estimated flows ______ ______ ______
- Change in flows ______ ______ ______
- Flow color ______ ______ ______
- Flow fines ______ ______ ______

Methods of flow measurements ______

Condition of measuring devices ______

Records

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

EMBANKMENT STRUCTURE

Performance Instruments

Piezometers ____________

Surface settlement points ____________

____

Internal movement devices ____________

____

Inclinometers ____________

____

Reservoir level gage ____________

____

Seismographs ____________

____

Other Observations

____

____

____

____

____

____

____

____

____

____

____

____

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

SPILLWAY

Approach Channel

- Debris ________
- Slides above channel ________
- Side slope stability ________
- Slope protection ________
- Log boom ________

Control Structure

- Slides above structures ________
- Debris ________
- Gates
  - Observed operation ________
  - Alignment ________
  - Anchorages ________
  - General condition ________
- Hoists or operating stems
  - Observed operation ________
  - Signs of movement ________
  - Anchorages ________
  - General condition ________
- Controls
  - Observed operation ________
  - Signs of movement ________
  - Anchorages ________
  - General condition ________
- Crest
  - Cracks or areas of distress ________
  - Signs of movement ________

Control Structure (continued)

Walls

- Movement (offsets) ________
- Cracks or areas of distress ________
- Settlement ________
- Joint spalling or opening ________
- Drains ________
- Backfill settlement ________

Apron

- Movement (offsets) ________
- Cracks or areas of distress ________
- Settlement ________
- Joint spalling or opening ________
- Drains ________

Bridge

- Pier alignment and Condition ________
- Structural condition of slab and beams ________
- Bearings alignment and condition ________

Cranes

- Observed operation ________
- General condition ________
- Structural distortion ________
- Anchorages ________

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

SPILLWAY (continued)

Chute

- Debris ____________
- Slides above chute ____________
- Walls
  - Movement (offsets) ____________
  - Cracks or areas of distress ____________
  - Settlement ____________
  - Joint spalling or opening ____________
  - Drains ____________
  - Backfill settlement ____________
- Floor
  - Movement (offsets) ____________
  - Cracks or areas of distress ____________
  - Settlement ____________
  - Joint spalling or opening ____________
  - Drains ____________
- Drainage gallery
  - Ventilation, lighting ____________
  - Misalignment ____________
  - Joint spalling or opening ____________
  - Cracking ____________
- Drains
  - Amounts of flow ______ ______
  - Locations of flowing Drains ______ ______

Stilling Basin

- Debris ____________
- Slides above basin ____________

Stilling Basin (continued)

- Walls ____________
  - Movement (offsets) ____________
  - Cracks or areas of distress ____________
  - Settlement ____________
  - Joint spalling or opening ____________
  - Drains ____________
  - Backfill settlement ____________
- Floor (if visible)
  - Movement (offsets) ____________
  - Cracks or areas of distress ____________
  - Settlement ____________
  - Joint spalling or opening ____________
  - Drains ____________
  - Erosion ____________

Outlet Channel

- Debris ____________
- Erosion ____________
- Slides above channel ____________
- Side slope stability ____________
- Slope protection ____________
- Vegetation or other obstructions ____________

Other Observations

____

____

____

____

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

OUTLET WORKS

Discharge
- Turbidity ______________
- Solids ______________

Approach Channel (if visible)
- Debris ______________
- Slides above channel ______________
- Side slope stability ______________
- Slope protection ______________

Intake Structure
- Slides above structure ______________
- Debris ______________
- Trash racks ______________

Conduit or Tunnel
- Ventilation, lighting ______________
- Change in seepage ______________
- Joint openings ______________
- Concrete spalling ______________
- Steel liner bulges ______________
- Rockfalls ______________

Valves or Gates and their Operators
- Observed operation ______________
- General condition ______________
- Signs of movement ______________
- Anchorages ______________

Valves or Gates and their Operators (continued)

Controls
- Observed operation ______________
- Signs of movement ______________
- Anchorages ______________
- General condition ______________
- Remote operation function ______________

Power Supply
- Primary ______________
- Emergency ______________

Cranes
- Observed operation ______________
- General condition ______________
- Structural distortion ______________
- Anchorages ______________

Bulkheads
- General condition ______________
- Seals ______________

Chute
- Debris ______________
- Slides above chute ______________
- Walls
  - Movement (offsets) ______________
  - Cracks, other distress ______________
  - Settlement ______________
  - Joint spalling or opening ______________
  - Drains ______________
  - Backfill settlement ______________

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

OUTLET WORKS (continued)

Chute (continued)

Floor

Movement (offsets) ______________

Cracks, other distress ______________

Settlement ______________

Joint spalling or opening ______________

Drains ______________

Stilling Basin

Debris ______________

Slides above basin ______________

Walls

Movement (offsets) ______________

Cracks, other distress ______________

Settlement ______________

Joint spalling or opening ______________

Drains ______________

Backfill settlement ______________

Floor (if visible)

Movement (offsets) ______________

Cracks, other distress ______________

Settlement ______________

Joint spalling or opening ______________

Drains ______________

Erosion ______________

Outlet Channel

Debris ______________

Slides above channel ______________

Side slope stability ______________

Slope protection ______________

Vegetation or other obstructions ______________

Other Observations

____

____

____

____

____

____

____

____

____

____

____

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

RESERVOIR

Landslides

Individual designations ______________

Locations ______________

Conditions ______________

Blocked Inflows ______________

Log Boom ______________

Other Observations

____

____

____

____

____

____

____

____

____

____

____

POST-EARTHQUAKE INSPECTION CHECKLIST
FOR EMBANKMENT DAMS

ACCESS ROADS

Roadways

Obstructions ____________

Condition of surface ____________

Bridges

Structural condition of deck slabs and beams ____________

Bearings alignment and condition ____________

Pier alignments and conditions ____________

Foundation conditions ____________

Other Observations

____

____

____

____

____

____

____

____

____

____

____