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Seismic Design of Dams

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Synonyms

CSR; Design seismic parameters; Earthfill dams; Factor of safety; Granular and plastic soils; Limit equilibrium; Newmark sliding block; Nonlinear finite element analysis; Performance-based design; PWP model; Risk analysis; Soil liquefaction; SPT; Time history

Introduction

Design of new dams or safety evaluation of existing dams for seismic loads is standard practice and routinely required. In a broad term, dams can be classified into three types – concrete dam, rock-fill dam, and earthfill dam – and seismic design can be at a dam safety level and/or at a serviceability level.

This entry provides contents and discusses methods for design and analysis of earthfill dams at a dam safety level where the ultimate limit state is applied for the highest level of design earthquake loads. For dam safety, dam failure is the primary concern; for serviceability consideration, the dam should remain functional and any damages should be easily repairable under this level of earthquake loads.

Dam Performance in Past Earthquakes

Damages to dams under earthquakes can result from ground shaking, soil liquefaction, ground cracking, ground displacements (lateral spreading and settlement), and in extreme cases surface rupture along an earthquake-active fault. Examples of recent big and devastating earthquakes include the 2008 Sichuan earthquake (a crustal earthquake with a magnitude of M8.0 occurred in China) and the 2010 Chile earthquake (a subduction earthquake with M8.8 occurred off the coast of central Chile).

Historical data show that dams around the world have performed well and satisfactory and that the probability of a dam failure under strong ground motion shaking is low. Nearly all well-built and well-compacted embankment dams can withstand moderate earthquake shaking with peak ground accelerations (PGA) greater than 0.2 g. Dams constructed of clay soils on clay, or on rock, or on overburden foundations resistant to liquefaction have withstood (with no apparent damages) extremely strong shaking with PGAs from 0.35 to 0.8 g.

Soil liquefaction, either in the dam fills or in the foundation, is the most damaging factor that affects the performance of dams under earthquakes. Dams built of sandy soils, especially hydraulic or semi-hydraulic fills, or built on foundations of loose (low density) sandy soils are highly

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susceptible to earthquake damage or vulnerable to failures due to the potential for soil liquefaction. A famous case history of such is the near failure of the Lower San Fernando Dam in California, USA, in a 1971 earthquake (Seed 1979).

Design Seismic Loads Based on a Risk Analysis

Public tolerance to seismic risk for the consequence of dam failure ultimately determines the adequacy of an existing dam or the criteria for the design of a new dam. For instance, at a specific location with the same tectonic setting or geological condition and thus the same earthquake hazard condition, a nuclear reactor facility may be designed using a PGA of 1.0 g for a very low probability earthquake event, a hydroelectric dam using a PGA of 0.7 g for a low probability earthquake event, a residential building using a PGA of 0.5 g for a median probability of happening, and a temporary bridge for construction traffic using a PGA of 0.3 g for a relatively high probability of occurring. This is mainly because the societal risk tolerance on a relative scale increases in order from nuclear radiation leak to flood from a dam breach, to collapse of a residential building, and to loss of a temporary structure.

The first factor contributing to seismic risk, such as from an earthquake event with a PGA of 0.7 g, is the occurrence rate of such earthquake which is often measured by annual exceedance probability (AEP). The other factor is the consequence as a result of an earthquake, which is ultimately measured in terms of loss of life and sometimes economic loss. At a conceptual level, seismic risk can be expressed as the product of seismic hazard probability and consequence, and it represents the probabilistic expectation of the consequence.

Ideally, seismic design of a dam should be based on a risk analysis, including calculation of the actual probability of a dam failure and its consequence. The adequacy of the dam would then be judged from the seismic risk (such as annual probability of single or multiple fatalities) that is acceptable to the society for the loss of life involved in a dam failure.

The risk-based approach to dam safety evaluation should balance the public risk and the limited societal resources available to manage the particular risk. As shown in Fig. 1, it is considered generally acceptable that the maximum level of societal risk to fatality is less than 10^{-3} per annum

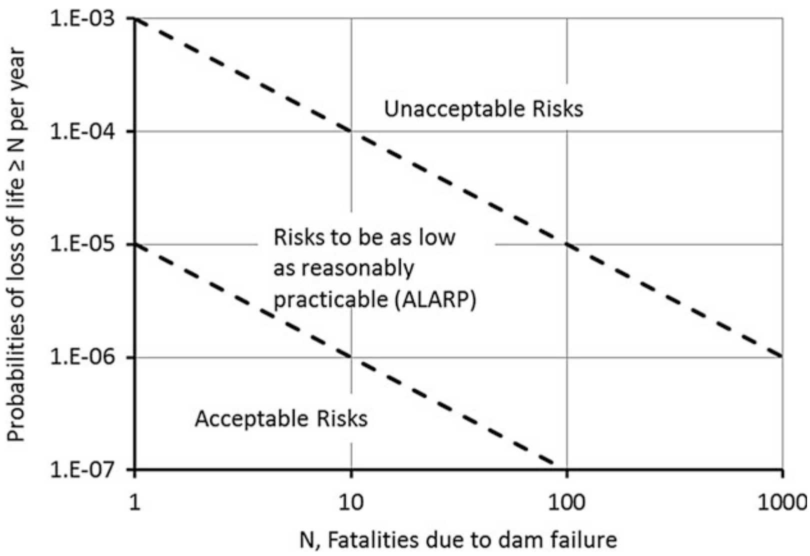

Fig. 1 An example relationship to manage societal risk for dam safety

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for loss of one life and the risk is less than 10^{-5} per annum when more than 100 lives would be lost in the event of a dam failure. The principle that the risks should be as low as reasonably practicable (ALARP) is generally followed in practice, and it is thus reasonable to use an annual probability of 10^{-5} and 10^{-6} for 100 and more fatalities.

In mathematical terms, seismic risk to loss of life (fatality) is calculated using the following two equations:

$$P_{\mathrm{Fatality}} = P_{\mathrm{Failure}} \times P_{\mathrm{Fatality/Failure}}$$
and
$$P_{\mathrm{Failure}} = P_{\mathrm{Earthquake}} \times P_{\mathrm{Failure/Earthquake}}$$

where

P_{Fatality} = unconditional probability of fatality from earthquake, i.e., seismic risk.
$P_{\mathrm{Earthquake}}$ = unconditional probability for an earthquake to occur, such as the one with a PGA of 0.7 g.
$P_{\mathrm{Failure/Earthquake}}$ = conditional probability of a dam failure in event of the earthquake.
$P_{\mathrm{Fatality/Failure}}$ = conditional probability of loss of life in the event of a dam failure.

The probabilities caused by all dam failure initiating events (failure modes) need to be aggregated or compounded in order to obtain the total probability to a dam failure. Failure modes (such as soil liquefaction) for earthquake loading should be identified in advance in order to perform a risk calculation. The risk calculation for a particular failure mode (P_{Failure}) is carried out using an event tree approach where significant events are sequenced in levels. Construction of an event tree for a dam requires special knowledge in geotechnical earthquake engineering and a comprehensive understanding of the dam.

Using “soil liquefaction” as an example failure mode, an event tree for dam failure could consist of earthquake acceleration (Level 1), contributing earthquake magnitude (Level 2), liquefaction analysis method (Level 3), soil liquefaction capacity (Level 4), dam crest settlement magnitude (Level 5), reservoir water level (Level 6), and dam damage level (Level 7). For illustration purpose, at Level 1 for earthquake acceleration, it may have four scenarios with a PGA of 1.0, 0.7, 0.5, and 0.3 g, respectively, and each PGA scenario has its probability of occurrence (AEP). At Level 3 assuming two models for liquefaction analysis, the weighting factors (e.g., 0.55 and 0.45) would be assigned to each model to make a total weighting of 1.

In many cases where a dam consequence class is available after completion of a life safety model and analysis, it can be conservatively assumed that the conditional probability ($P_{\mathrm{Fatality/Failure}}$) equals to 1.

Although in recent years the risk-based seismic dam safety evaluation is increasingly used in dam safety management, it is not widely used in engineering design or seismic safety evaluation due to the limited ability to perform such a complex risk-based analysis. As described below, the standard-based or traditional approach is more commonly used in seismic design.

Standard-Based Method and Design Earthquake AEP

The standard-based method is a semi-probabilistic method that defines the seismic hazard using the probabilistic approach, but does not explicitly calculate the probability of a dam failure and its

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consequence. Without any quantifiable risk calculation, this is an empirical or experience-based approach to risk evaluation and management. The consequence, i.e., conditional fatalities in the event of a dam failure, is indirectly evaluated by using a dam consequence class scheme (FEMA 1998; CDA 2007). In some countries the highest consequence class is "Extreme" when potential (expected) fatalities are 100 and more.

With this approach, a seismic hazard level (AEP) is selected according to the consequence class of a dam so that the selected AEP conforms to the societal acceptable risk level. For a dam under an "Extreme" class, it would be reasonable to target a dam failure probability of 10^{-5} to 10^{-6} per annum, and an earthquake with an AEP of 10^{-5} or higher could be adequate, taking into account the satisfactory post-earthquake performance of dams around the world, i.e., the low conditional probability of a dam failure under strong shaking. For background information, the nuclear industry uses the 84th percentile spectra at AEP of 10^{-4} or approximately median-mean spectra at a probability of 10^{-5} per annum. With less stringent safety criteria than the nuclear industry, dam safety design can adopt a design earthquake with an AEP of 1/10,000 (i.e., 10^{-4}) based on the mean spectra. The mean is the expected value given the epistemic uncertainties, and the mean hazard value typically such as in Canada varies from 65th to 75th percentiles in the hazard distribution.

The design earthquake AEP for an "Extreme" class dam can vary from one region to another or from one country to another depending on the risk tolerance ability, and it is normally jointly selected by the dam owner and the government regulatory agency. In some developing countries with lower societal reliability than the developed countries, an earthquake AEP of 1/1,000 is used for dam safety design. On the other hand, even in developed countries such as Canada, an earthquake AEP of 1/2,475 is used for building safety design as a result to balance public risk and economic cost.

Once a seismic hazard level is selected, conventional analyses such as limit equilibrium or finite element analyses are conducted using seismic loads corresponding to the selected AEP. The results (such as stresses, ground displacements, or stability in a dam or its foundation) and consequences are evaluated deterministically using standards, specifications, and design codes. The potential for a dam failure is evaluated by comparing deterministically the resulting stresses and displacements with ultimate stability and established failure criteria. The evaluations of results are primarily done by empirical evidence, past experience, and engineering judgment.

Seismic Hazard Evaluations

A ► deterministic seismic hazard analysis (DSHA) is normally conducted for each known earthquake source to determine site ground motion parameters such as PGA, response spectra at 50th and 84th percentile, magnitude of earthquake, and duration of strong shaking. This would include assessment of potential seismic hazards from earthquake activities along local or regional known faults for the life of a dam.

A ► probabilistic seismic hazard analysis (PSHA) is usually conducted to determine design seismic parameters at various AEP levels, in additional to DSHA where known active faults or subduction zones are identified. A PSHA normally consists of **identification** of earthquake sources by developing a source model and the earthquake occurrence rate for each source zone with the understanding of local geology and regional past earthquake history, **application** of ground motion prediction equations (Earthquake Spectra 2008) that are appropriate to the region (e.g., hardness and shear wave velocity of the bedrock) and the types of seismic sources, and **determination** of

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earthquake response spectra at various levels of probability by the integration of hazard contributions over all earthquake magnitudes and distances for all seismic source zones.

The response spectra, either from DSHA or from PSHA, are normally defined for a series of discrete natural periods or frequencies (such as 0.1, 0.2, 0.5, 1, 2, 5, 10, and 33 Hz; the frequency for PGAs can range from 33 to 100 Hz) that are used in the hazard calculation, and they are always associated with an uncertainty level (e.g., median, mean, or 84th percentile).

A seismic hazard analysis determines the potential intensity of seismic loads that could hit a damsite; and it always proceeds to, but does not always relate to, the design of a new dam or performance evaluation of an old dam for earthquake loading. However, seismic design of dams uses results of a seismic hazard analysis. Figure 2 shows typical response spectra from DSHA and PSHA at the most commonly used damping ratio of 5 %.

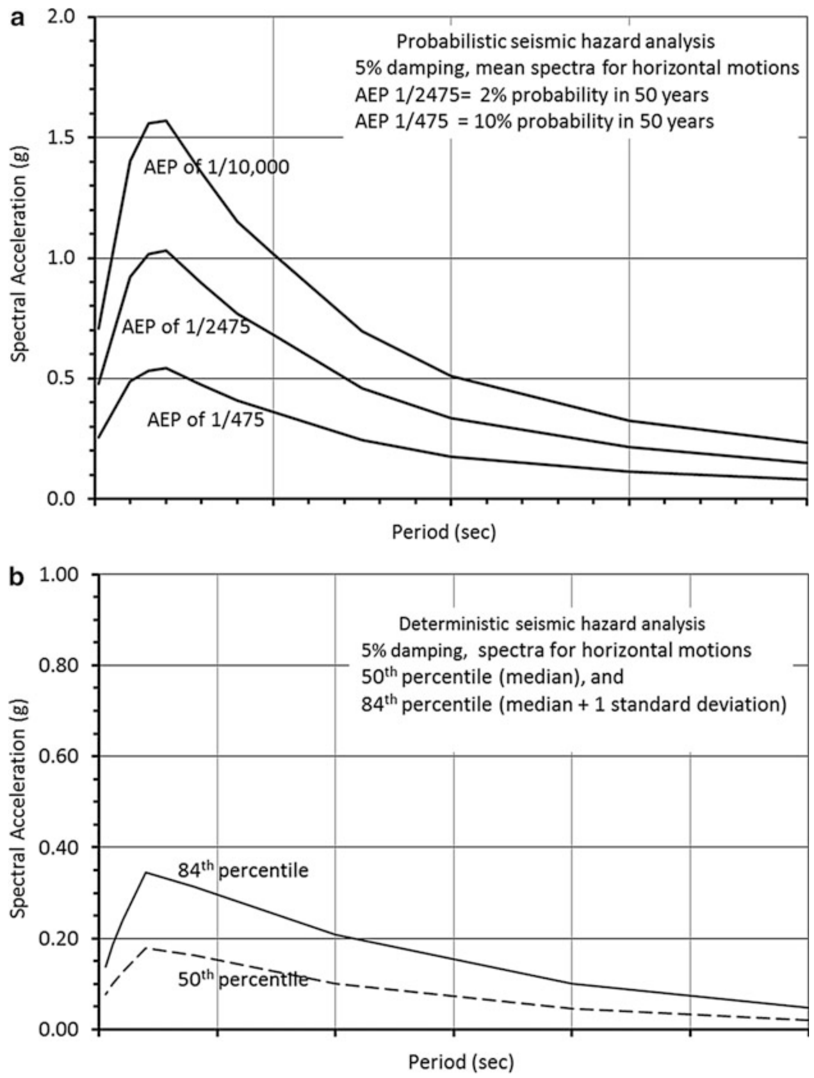

Fig. 2 Example results of seismic hazard analyses (**a**) probabilistic method and (**b**) deterministic approach

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Design Seismic Parameters

Seismic parameters for design of dams consist mainly of peak ground accelerations (PGAs), site-specific response spectra for horizontal and vertical accelerations, magnitude and site-source distance for the design earthquake, duration of strong shaking, ground motion time histories as required for dynamic analyses, and fault displacements in rare situations when a dam is (or to be) on an active fault.

Design Response Spectra and PGAs

Response spectra and PGAs from DSHA can be used directly in seismic design, and they are conventionally computed at an 84th percentile value in the hazard distribution, i.e., one standard deviation (using $\varepsilon = 1$ in ground motion prediction equations) above the predicted median value. In some cases for seismic faults with low rate of activities, the 50th percentile values (the median) are used in design.

When seismic hazard is evaluated using a probabilistic approach, response spectra and PGAs from PSHA are used for seismic safety design of dams, using the results for the selected AEP (e.g., mean spectra with AEP of 1/10,000).

In current practice, the vertical ground motions do not seem to have much effect on the performance of earthfill dams and thus are not normally included in design analysis.

Design Earthquake Magnitude and Site-Source Distance

The PSHA results represent at a specific AEP level a composite of hazard contributions from earthquakes of all magnitudes and distances. The response spectra from a PSHA are also called the uniform (or equal) hazard response spectra (UHRS).

Deaggregation of the composite seismic hazard is performed to identify relative contributions of individual earthquakes or scenario earthquakes with various magnitudes and distances. For seismic design of a dam, representative earthquake magnitudes (M) and site-source distances (D) are obtained by deaggregation of the uniform hazard at natural periods that are significant and critical to the dam seismic response.

The representative M and D for the design earthquake are then used in seismic analysis where magnitude and duration of the earthquake are needed such as in the soil liquefaction analysis, or for selection of ground motion records needed in a time-history analysis.

Input Ground Motion Time Histories

The current trend in seismic design of dams is to conduct linear or nonlinear time-history analysis to obtain dynamic response of dam to earthquake loads. Time-history analysis of dam requires input ground motion time histories (acceleration, velocity, and displacement).

For dynamic analysis of earthfill dams, multiple ground motion records (five to eight commonly adopted) from past earthquake are selected, and each record is then linearly scaled to fit the design response spectra (such as a UHRS) over the range of natural periods that are appropriate for the dam. Using this method, an individual ground motion record is multiplied by a single scale factor to increase or decrease the magnitude of the motion and its spectrum, without modifying the shape of its spectrum or its frequency contents.

The selected ground motion records should be consistent with the seismic parameters and representative to the design earthquake in terms of magnitude, site-source distance, duration of strong shaking, tectonic setting and source mechanism, and consistency of site conditions between the recording station and the dam.

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In some cases when uniform scaling of recorded ground motions is unable to meet the requirements, the design earthquake ground motions would be obtained by modifying a recorded ground motion in the time or frequency domain.

General Approach to Seismic Design and Analysis

For dams under static conditions, limit equilibrium analyses for slope stability are normally conducted for long-term operating reservoir conditions (steady-state seepage) and for short-term reservoir drawdown conditions. In general, for satisfactory stability factor of safety (FS) would be minimum 1.5 for long-term conditions (FS $>$ 1.5) and 1.3 for short-term conditions (FS $>$ 1.3).

For dams under seismic loading, two primary modes of dam failure must be addressed, the overtopping failure caused by excessive settlement of dam crest and the internal erosion and piping failure caused by cracks in dams, damage of filter layer upstream, or drains downstream of the core zones.

Some of the common design measures to mitigate these failures include the following: remove or improve problematic foundation soils by ground treatment and adequately compact the dam fills, use wide core zones of plastic soils that are resistant to erosion, use well-graded wide filter zones upstream of the core, construct chimney drains downstream of the core to lessen soil saturation and reduce downstream seepage, and use dam crest and downstream slope details to provide protection of the dam in the event of an overtopping.

Seismic analyses appropriate to a dam and its foundation are conducted using design seismic parameters to provide adequate information on expected stresses and ground displacements for evaluation of expected performance of the dam. Seismic analyses (see Fig. 3) can include a soil liquefaction evaluation, a pseudo-static stability analysis, a Newmark sliding block deformation analysis, a post-earthquake static stability analysis, and a finite element dynamic analysis for computing permanent ground deformations.

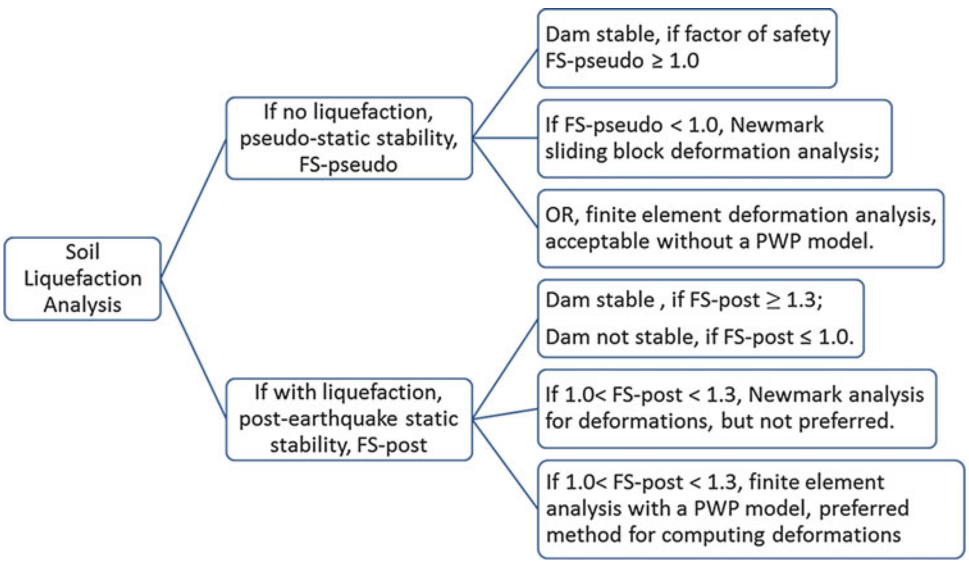

Fig. 3 Types of seismic analyses based on liquefaction susceptibility (PWP = excess pore water pressures from earthquake loads)

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Evaluation of Soil Liquefaction

Liquefaction of Sands Under Cyclic Loads

Sand liquefaction is a fluidization process of saturated sand mass subject to cycles of shear stress. Under shaking, it can be easily observed that loose sands in a dry container will experience volume contraction and settle to a more compact state. In a saturated condition, immediate volume change of the sand would not occur because water in the pore does not drain quickly enough under the rapid earthquake loading. Instead, the potential for volume change translates into a quick increase in excess pore water pressure (PWP) in the sand mass. Liquefaction occurs when the PWP exceeds a threshold value that the pore water effectively suspends sand particles. Sand boiling to ground surface is a surficial expression of liquefaction of sands in the ground.

Liquefaction strength, or capacity, or resistance (more commonly used) of sands can be measured by laboratory testing using reconstituted samples prepared to a target relative density. It is found (see Fig. 4) that liquefaction resistance generally increases with relative density as expected, and under the same relative density, it decreases with more cycles of shear stress used in the testing.

Because liquefaction is triggered by multiple cycles of shear stresses, liquefaction resistance of soils is represented using both the cyclic shear stress level (τ_{cyc}) and the number of cycles for such uniform (constant amplitude) stresses to trigger a liquefaction failure. Cyclic stress ratio (CSR) refers to the ratio of the cyclic shear stress to the initial vertical normal stress (σ'_{v0}) at which the sample is under cyclic shearing, i.e., $CSR = \tau_{cyc}/\sigma'_{v0}$. For earthquake loading, the number of cycles is related to earthquake magnitude and duration. In engineering analysis, nonuniform cycles of shear stress from a M7.5 earthquake are artificially set equal to, in terms of net effect on soil, a shear stress level with 15 uniform cycles. Thus, liquefaction resistance of soil is typically characterized by CSR at 15 cycles or CSR_{15} as shown in Fig. 5.

Liquefaction Resistance of Sandy Soils

Except with ground-frozen sampling technique that is practically not applicable, liquefaction resistance of in situ sands is considered not measurable from laboratory testing.

The most widely adopted test for field evaluation of relative density for sands is the standard penetration test (SPT) with split-barrel soil sampling. A key result of SPT is SPT N value that is the number of blow counts for a penetration depth of 305 mm (1 ft.), and $(N_1)_{60}$ is defined as the N value corrected for the soil under an in situ effective vertical normal stress of about 100 kPa (1 atmospheric

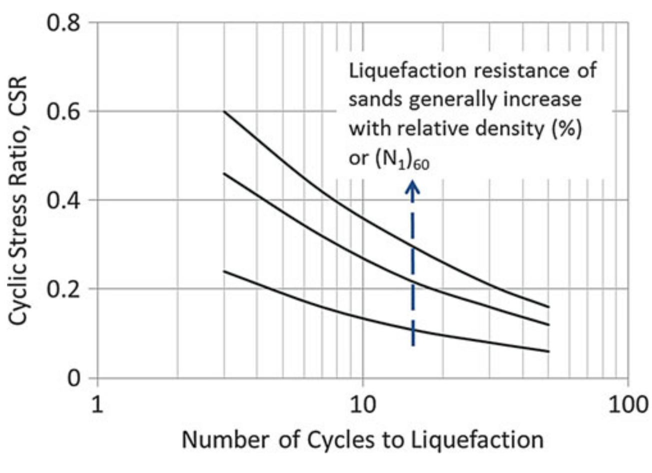

Fig. 4 Liquefaction resistance trend curves for clean sands (<5 % fines)

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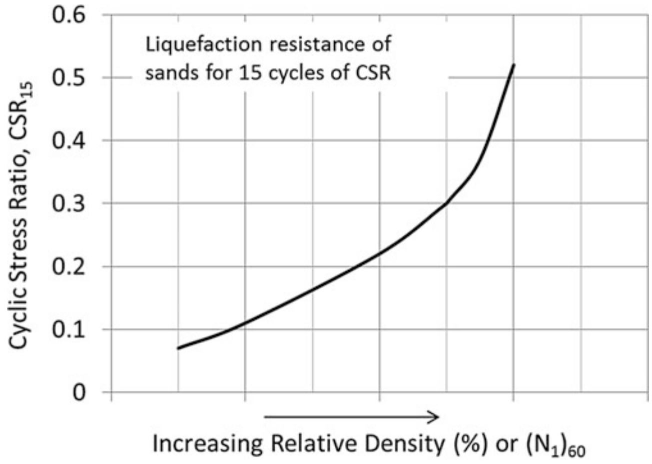

Fig. 5 An example liquefaction resistance curve for clean sands

pressure) and to the standard SPT hammer energy level (i.e., at 60 % of the theoretic maximum hammer energy). The general understanding is that the measured SPT N value is corrected downward (i.e., reduced) when the test is performed for soil at a depth with a confining stress higher than 100 kPa (e.g., at 300 kPa).

Current technology or methodology for evaluating liquefaction resistance of sandy soils was developed empirically by indirect studies of performance case histories of dams in historical earthquakes, where in some cases soil liquefaction was assessed to have occurred and in other cases liquefaction was believed not triggered.

Field soil liquefaction resistance of sands (particle size from 0.075 to 4.75 mm) has been derived from field performance of soils under various levels of historical earthquakes, and CSR_{15} from case history studies is plotted with in situ measured $(N_1)_{60}$ (Seed and Harder 1990) as an equivalence to the relative density of sand. Similar to the curve shown in Fig. 5, sands with $(N_1)_{60} > 30$ are considered to be dense and generally not liquefiable; sands with $(N_1)_{60} < 10$ are generally loose and highly susceptible to liquefaction.

Unless completely confined with no drainage path, gravels (particle size from 4.75 to 75 mm) are not so much vulnerable to soil liquefaction due to its high permeability that allows fast dissipation of excess pore water pressures. However, sandy soils containing some amount of gravels are liquefiable, and their liquefaction resistance can be tested in the field using a large penetration hammer such as the Becker Hammer Penetration Test (Youd et al. 2001).

At a given relative density or $(N_1)_{60}$, sandy soils containing significant amount of fines (particle size <0.075 mm) are known to have higher liquefaction resistance than clean sands with no fines (Youd et al. 2001). Nonplastic silts, containing 100 % fines but with very low plasticity (such as plasticity index PI $<$ 5 %), are considered to behave under cyclic loading in a similar manner as sands. The liquefaction resistance of nonplastic silts is evaluated as sands with upward correction on fines content.

Liquefaction Resistance of Plastic Soils

A more descriptive terminology for liquefaction failure of plastic soils (silts or clayey silts), containing 100 % fines and with relatively high PI (such as PI $>$ 10 %), is "cyclic strain softening" which emphasizes more on the structural breakdown of the material under cyclic loading than on the buildup of excess pore water pressure. Evaluation of liquefaction resistance for plastic soils is an area that requires more research. In the 1980s and 1990s of the last century, soil index parameters

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(water content, liquid limit, PI, and fines content) were used as basis for liquefaction assessment, but the current trend method is to use results of laboratory cyclic tests (Finn and Wu 2013), especially for evaluation of more critical structures such as for dam safety.

Liquefaction resistance of in situ plastic soils can be measured directly by laboratory testing on in situ Shelby samples. Soil samples in a cyclic direct simple shear test are normally first consolidated to its pre-consolidation pressure (the highest historic pressure experienced by the soil or σ'_p) and then sheared by cycles of τ_{cyc} under σ'_{v0} and initial shear stress (τ_{st}) similar to the in situ stresses of the sample.

As compared to sands where the relative density or $(N_1)_{60}$ is the single parameter, there are more parameters and factors affecting and contributing to the liquefaction resistance for plastic soils (see Fig. 6). The curves for plastic soils are generally flatter than curves for sands; that is, the resistance decreases less with increasing number of cycles. The two key factors are the over-consolidation ratio ($OCR = \sigma'_{v0}/\sigma'_p$) and PI. Liquefaction resistance of plastic soils generally increases with increasing OCR and PI but decreases with increasing τ_{st}.

As a general observation, plastic soils having $CSR_{15} > 0.4$ (for 15 cycles) are considered to be hard, insensitive to earthquake loading, and unlikely to experience much strength loss. Plastic soils having CSR_{15} near or less than 0.15 may be sensitive to shaking and can experience significant loss of strengths in a strong earthquake.

Earthquake-Induced CSR

For liquefaction analysis, seismic load in a soil element is represented by time history of shear stresses (τ_{dyn}) imposed by earthquake shaking, and they can be calculated from a site response analysis using the input ground motions at the damsite and dynamic properties of dam fills and foundation soils.

Soil dynamic properties for each of the soil zones in the dam and its foundation primarily consist of low-strain shear modulus (G_{max}) and the damping characteristics (Seed et al. 1986). The low-strain shear modulus (G_{max}) is commonly computed from shear wave velocity (V_s) that can be measured by seismic downhole or crosshole surveys. Shear modulus would reduce from G_{max} as shear strain increases; reduction of soil shear modulus (stiffness) with increasing strains is more significant for gravels than for sands and more for sands than for clays. Soil damping ratio would be less than 5 % at low strain, but it increases with increasing strain. The maximum damping ratio is about 25 % for gravels and sands and in the order of 20 % for clays and silts.

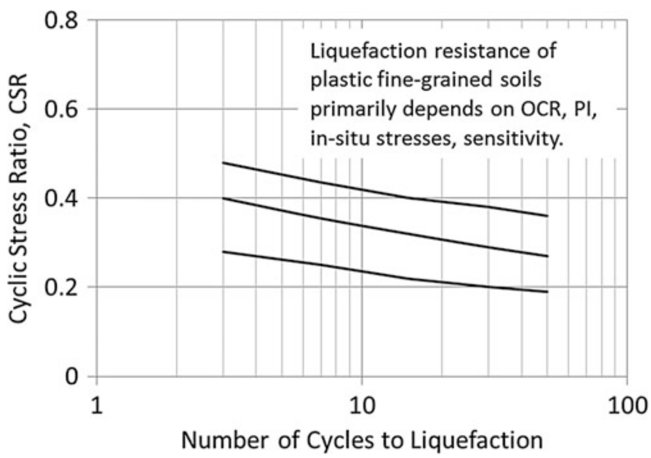

Fig. 6 Liquefaction resistance trend curves for plastic soils, generally flatter than sands

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A seismic site response analysis (▶ Site Response: 1D Time Domain Analyses) is commonly performed in a total stress analysis, without including PWP effect, using a 1D soil column analysis for a low dam or using a 2D finite element dynamic analysis to model the geometric effect for a high dam. A site response analysis, either 1D or 2D model, can be conducted using the equivalent linear method (Idriss et al. 1974) or a true nonlinear method (Finn et al. 1986) for the simulation of shear modulus degradation (decrease) and damping ratio increase with the increase of shear strain. A true nonlinear approach is considered more appropriate when the ground shaking level is high, such as with a PGA of 0.4 g or higher.

Factor of Safety Against Liquefaction

Upon completion of a site response analysis, cyclic stress ratio ($CSR = \tau_{dyn}/\sigma'_{v0}$) in each soil zone is calculated by converting the nonuniform cycles of shear stresses from the earthquake to equivalent cycles of uniform shear stresses (Wu 2001) and then normalized to the in situ vertical normal stress (σ'_{v0}).

The liquefaction resistance (CSR_{15}) of soils is determined, for sands using $(N_1)_{60}$ from SPT and for plastic soils using liquefaction resistance curves from laboratory cyclic shear tests.

A factor of safety against soil liquefaction for each soil zone is calculated to be the ratio of liquefaction resistance of soil and seismic shear stress from earthquake, i.e., $FS_{LIQUEFACTION} = CSR_{15}/CSR$. A computed $FS_{LIQUEFACTION}$ near or less than 1.0 indicates triggering of soil liquefaction, and $FS_{LIQUEFACTION} > 1.5$ means not liquefiable.

Pseudo-Static Limit Equilibrium Stability Analysis

A pseudo-static analysis is a limit equilibrium method which includes additional seismic inertia forces in a conventional static slope stability analysis. Seismic coefficients (k_h for horizontal and k_v for vertical) are often used in such analysis, and they are normally taken as the peak ground accelerations (PGAs) as a fraction of the gravity acceleration. The seismic inertia forces are represented by k_hW and k_vW in horizontal and vertical direction, respectively, where W is the weight of a sliding block. Due to the nature of seismic shaking, shear strengths of soils for rapid loading conditions are conventionally used for characterization of soil resistance to earthquake loadings.

In a scenario that soils in a dam and its foundation would not develop significant PWP from shaking, a pseudo-static factor of safety greater than 1 (FS-pseudo >1) is a very strong indication that there would be little or no damage to the dam from an earthquake.

However, a pseudo-static factor of safety less than 1 (FS-pseudo <1) does not necessarily represent dam instability or unsatisfactory performance; instead due to the transient nature of earthquake motions, the dam would undergo some deformations for the short time interval when the ground acceleration exceeds the yield acceleration. In such case seismic ground deformations are computed and used for performance assessment.

It is not possible to predict failure of a dam or estimate seismic ground deformations by a pseudo-static analysis, and therefore other types of more comprehensive analysis are required to provide a more reliable basis for seismic design of dams.

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Newmark Sliding Block Seismic Deformation Analysis

Newmark (1965) pointed out that for seismic loading permanent ground deformations of a dam, in addition to the pseudo-static factor of safety, should be considered to be tolerable or not for dam performance assessment. The Newmark deformation method assumes that deformation of a dam is modeled by a rigid block sliding on an assumed failure surface under the design ground motions at the damsite.

Sliding deformation is assumed to occur whenever the acceleration of a sliding block exceeds the yield acceleration, which is the horizontal acceleration that results in a factor of safety of 1 in a pseudo-static slope stability analysis, and the sliding stops when the acceleration falls below the yield acceleration. Mathematically, the Newmark sliding block displacements are computed by double integration of acceleration time history on the sliding block for the portion of accelerations that exceeds the yield acceleration (Fig. 7).

Although it has been widely used at a time when there is lack of direct methods for computing seismic ground deformations, the Newmark sliding block model is generally recognized to be not a very good representation of how embankment dams deform under earthquake, especially for dams with low factors of safety. Due to its simplicity, the method may provide a range of expected ground deformations, but it would neither give distribution of ground displacements in the dam nor provide direct calculation of dam crest settlement.

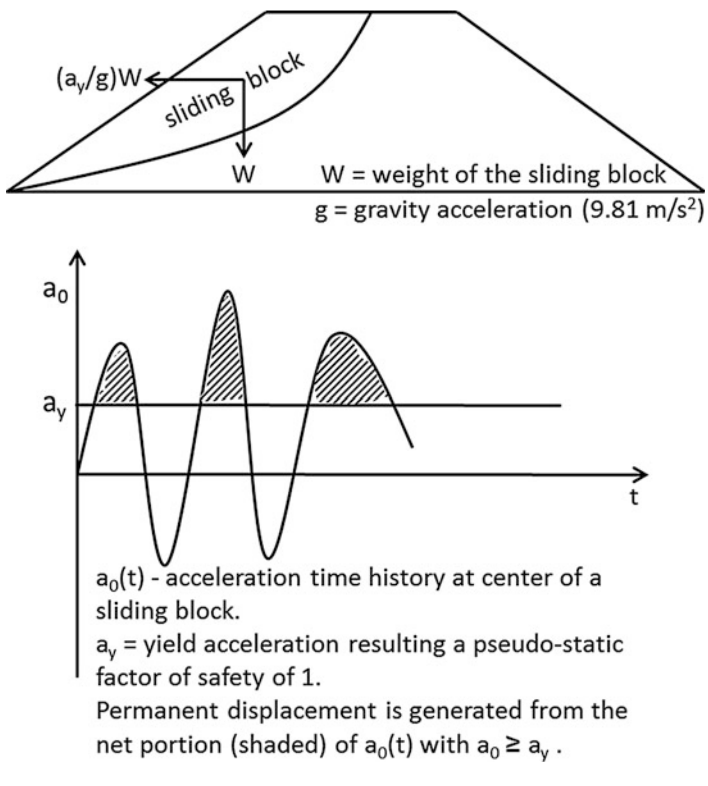

Fig. 7 Newmark sliding block deformation calculation diagram

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Post-Earthquake Static Stability Analysis

A direct consequence of soil liquefaction is a great reduction of shear strength, in particular for loose and contractive sands (Fig. 8), for which the shear strength may become a small fraction of its static and drained value due to the buildup of excess pore water pressures and fluidization. In design practice, shear strength of liquefied soil is also described as residual strength and sometime as post-liquefaction strength.

As a frictional material and under static loading, loose sand can have a friction angle possibly ranging from 30° to 35°; however, the post-liquefaction strength of the same loose sand may decrease to an equivalent friction angle of 5° or perhaps less (Seed and Harder 1990), indicating the degree of shear strength reduction due to liquefaction.

Residual strength of plastic soils after initial strain softening could decrease to 50–80 % of its pre-earthquake static value, and after sufficient shaking disturbance, it can eventually decrease to the remolded strength value. Depending on sensitivity, remolded strength of silts and clayey silts may range from about 25 % of its peak static strength for low-sensitivity soils to only 10 % for high-sensitivity soils.

A static limit equilibrium analysis, without seismic loads but using the post-earthquake soil strength, is conducted in order to assess the post-earthquake static stability of the dam. The analysis would use residual strength for liquefied soils and reduced strength in soils that do not liquefy but with the buildup of excess pore water pressures from the design earthquake shaking.

A factor of safety less than 1 from a post-earthquake static stability analysis (FS-post <1) is a strong indication of dam instability and a potential failure during or immediately after the design earthquake. As such, the design for a new dam would be improved and ground strengthening for an existing dam would be required, in order to increase the post-earthquake dam stability and meet the dam safety requirements.

If FS-post >1 is calculated, ground deformations from the design earthquake are often computed for dam performance evaluation, preferably using a nonlinear finite element dynamic analysis. The Newmark sliding block deformation analysis should be avoided for computing deformations of dams involving soil liquefaction.

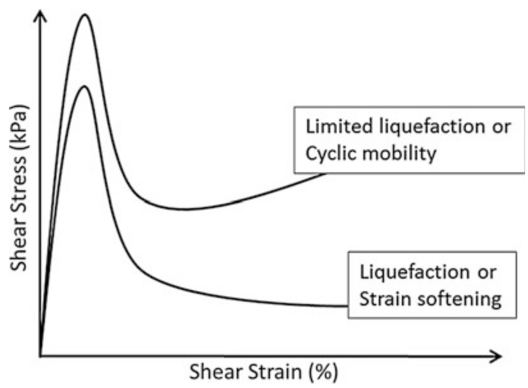

Fig. 8 Shear stress – strain response indicating great strength loss after liquefaction and some strength loss with cyclic mobility

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Seismic Deformations by Finite Element Dynamic Analysis

A more rigorous and direct method for computing earthquake-induced ground deformations, with and without soil liquefaction, is a nonlinear finite element dynamic analysis (Finn et al. 1994a, b) that overcomes the uncertainties associated with the approximation embedded in the pseudo-static stability calculation and in the Newmark deformation analysis. With PWP calculations built into effective stress soil models, the finite element method has the capability to simulate the coupled effects from dynamic site response, generation of excess pore water pressures, soil liquefaction, and post-liquefaction behavior, and this is a trend method for quantitative evaluation of earthquake-induced ground deformations.

A well-known analysis procedure for the finite element approach is the one developed by Finn et al. (1986) using soil data from centrifuge tests and then verified using soil data and field measured deformations of the Upper San Fernando Dam in California, USA, in a 1971 earthquake (Wu 2001), and recently extended to include a dilative silt model (Finn and Wu 2013). This analysis procedure is highly regarded by practicing engineers and referenced for seismic design of dams by the US Federal Emergency Management Agency (FEMA 2005).

The Finn-Wu finite element procedure is simple and well suitable for practical engineering analysis. This analysis method adopts Mohr-Coulomb criteria for modeling shear strength of soils and uses a nonlinear hyperbolic model to simulate hysteresis response of soils under cyclic loads. Excess pore water pressures (PWP) caused by earthquake loads can also be computed using effective stress models with calculation of factor of safety against liquefaction for each soil element. In addition, the procedure provides two options for applying input ground motions: (1) ground accelerations applied at the model's rigid base and (2) outcropping ground velocities applied at the model's viscous and elastic base. The elastic base option is often used when the model base stiffness is not rigid relatively to the model body that prevents incident seismic waves from effectively reflecting back to the model body.

As an example of Finn-Wu finite element procedure, factors of safety against liquefaction computed from a dynamic analysis using a subduction earthquake input motion are shown in Fig. 9 (Finn and Wu 2013). The results showed that almost the entire saturated zone (below the water table shown in blue) under the downstream slope of the dam would undergo large cyclic strains with $FS_{LIQUEFACTION} < 1$. A computed deformed mesh of the dam, with soil material zones shown in colors, immediately after the earthquake using the same subduction ground motion is shown in Fig. 10, indicating a deep-seated ground deformation pattern and significant lateral movement and settlement of the downstream dam crest. The original dam surface is outlined in blue in the figure.

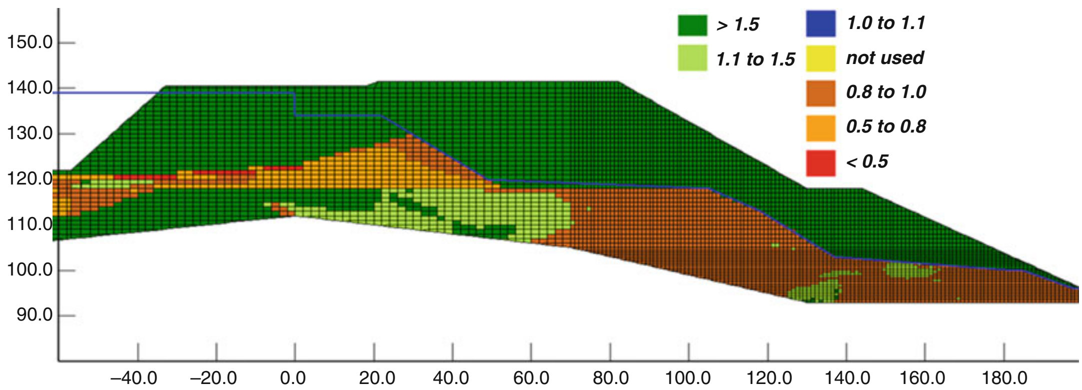

Fig. 9 Computed $FS_{LIQUEFACTION}$ shown in colors for an example finite element model of an earthfill dam (undeformed, partial model, dimension in m)

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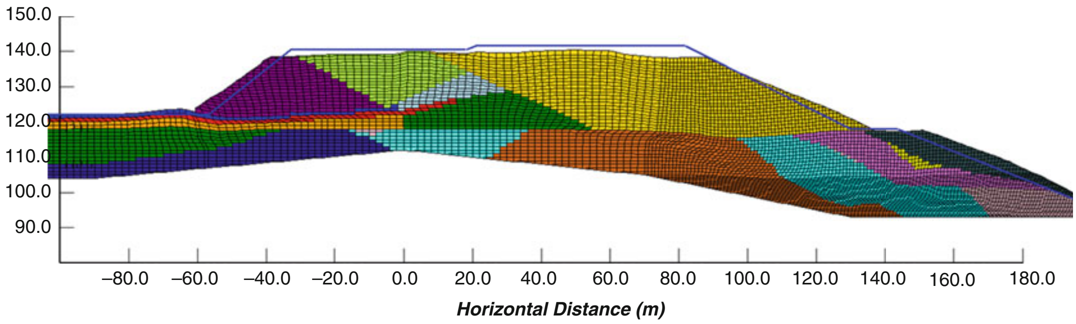

Fig. 10 Deformed dam cross section after a subduction earthquake (displacement scale factor 1.0, elevation for vertical axis in m)

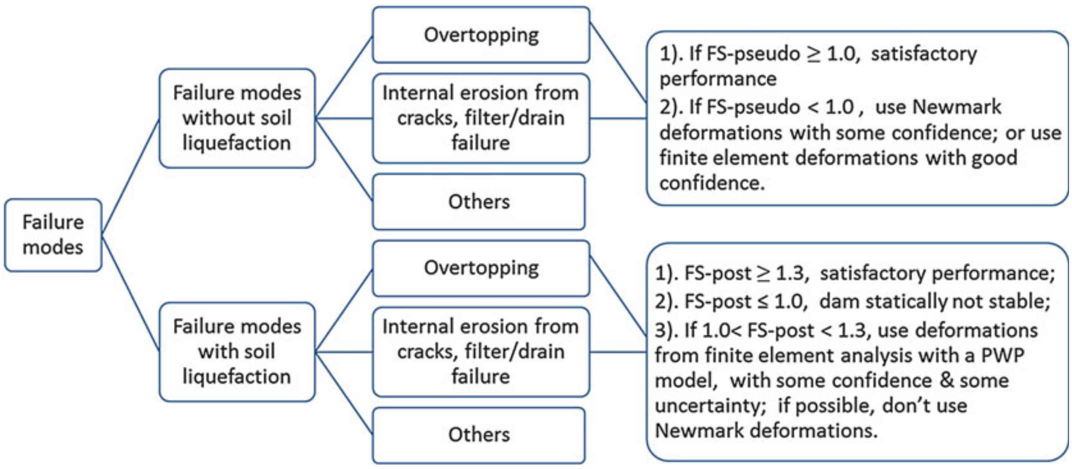

Fig. 11 Evaluation of dam performance for critical failure modes under earthquake

Seismic Performance Evaluation

In some cases, expected performance of dam can be assessed to either clearly safe or clearly unsafe, based on results of simple limit equilibrium stability analyses. In majority cases, however, dam performance is assessed from estimated ground deformations and strains. Ultimately, the expected ground deformations and the associated ground strains, either from a Newmark analysis or a nonlinear finite element analysis, should not exceed what the dam can safely withstand without catastrophic release of the reservoir water due to overtopping or internal erosion. In the process, uncertainties associated with the calculated factor of safety or with the computed ground deformation need to be understood to achieve the confidence on the expected performance. In some cases, the consequences of misjudging uncertainty levels would also be taken into considerations.

Two key factors contributing to uncertainties in predicting the expected performance of dam under earthquake are related to soil liquefaction. The first is "Will liquefaction be triggered?", and the second is "What is the residual strength after soil liquefaction?" Ideally, a risk analysis would be beneficial to quantify the uncertainties. However, state of practice is mostly based on a deterministic approach for dam performance assessment, such as using analysis steps outlined in Fig. 11.

Overtopping of a dam in earthquake can be caused either by slope instability (failure) or by excess ground deformations (settlement) at dam crest. If post-earthquake stability analyses indicate a factor of safety well above 1 such as FS-post $>$ 1.3, historical dam performance experience in earthquakes

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would indicate that the dam will have limited or small deformations and will perform satisfactorily. When liquefaction is not involved, factor of safety greater than 1 with seismic force in a pseudo-static analysis (FS-pseudo > 1) would also indicate that the dam will perform well in a design earthquake.

On the other hand, confidence in dam safety decreases and probability of a dam failure due to overtopping or internal erosion increases when a post-earthquake factor of safety (FS-post) near or less than 1.0 is calculated using residual strengths for liquefied soils. A FS-post less than 1.0 would indicate slope instability under post-earthquake static conditions, and a dam slope failure can occur immediately after the earthquake.

In general, when a wedge or circular sliding surface has a low post-earthquake factor of safety, the deformations along the slip surface would be large or sometime excessive. In these cases, ground deformations calculated preferably from a finite element dynamic analysis are used to assess the potential of a dam failure by overtopping, or internal erosion, or loss of soils in piping.

Engineering judgment is carefully applied in assessing the level of uncertainties in design parameters and analytical methodology and thus the confidence level in the use of calculated deformations. When soil liquefaction is not an issue, deformations estimated from a Newmark sliding block analysis would prove to be adequate for many cases. For dams with liquefaction issues, seismic design or evaluation would use deformations computed from a finite element dynamic analysis, which has adequate soil models for simulation of soil liquefaction and post-liquefaction behavior and also can compute large-strain ground deformations. Even using a finite element approach, computed deformations for dams without involving soil liquefaction would be more reliable (with less uncertainties) than those involving liquefaction.

Summary

This entry describes basic principles and methodology used in seismic analysis and design for earthfill dams.

The risk-based approach for dam safety evaluation is introduced to indicate that seismic design criteria are governed by societal tolerance of seismic risk which consists of both seismic hazard and its consequence. In engineering practice, a full risk analysis is not commonly performed due to its complexity; instead, consequence of a dam failure in an earthquake is often represented using a dam consequence class scheme. A design level of seismic hazards is then selected from the consequence class for a dam. Seismic hazard of a damsite can be evaluated on a probabilistic approach. Once design seismic parameters are determined using the selected seismic hazard level, seismic analysis and design are conducted using a deterministic approach where seismic demands are compared with the ultimate capacity of soils.

State-of-practice methodology for evaluation of soil liquefaction is described, which include measurement of liquefaction resistance of soils by laboratory tests, assessment of liquefaction resistance from in situ tests in the field, soil parameters that impact the liquefaction resistance, and typical values of liquefaction resistance. Liquefaction potential of a soil element is evaluated by comparing the seismic shear stress from earthquake with the liquefaction resistance of the soil.

Seismic analyses discussed in this entry include pseudo-static stability analysis, Newmark sliding block deformation analysis, post-earthquake static stability analysis, and finite element dynamic analysis for computing permanent ground deformations.

It is pointed out that soil liquefaction is the key factor contributing to uncertainties in predicting the expected performance of dam under earthquake, which are based on results of limit equilibrium

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stability analyses and in majority cases from estimated ground deformations. Engineering judgment should be applied in assessing the level of uncertainties in design parameters and analytical methodology and thus the confidence level in the use of calculated factors of safety or ground deformations.

Cross-References

► Deterministic Seismic Hazard Analysis
► Probabilistic Seismic Hazard Analysis
► Site Response: 1D Time Domain Analyses

References

Canadian Dam Association (CDA) (2007) Dam safety guidelines. CDA, Toronto, Canada

Earthquake Spectra (2008) Special issue on the next generation attenuation project. Earthq Spectra Spec Issue 24(1):1–341

Federal Emergency Management Agency (FEMA) (1998) Federal guidelines for dam safety: hazard potential classification systems for dams, FEMA 333, Oct 1998. Reprinted April 2004

Federal Emergency Management Agency (FEMA) (2005) Federal guidelines for dam safety: earthquake analyses and design of dams, FEMA 65, May 2005

Finn WDL, Wu G (2013) Dynamic analyses of an earthfill dam on over-consolidated silt with cyclic strain softening, Keynote lecture. In: 7th international conference on case histories in geotechnical engineering, Chicago, Apr 2013

Finn WDL, Yogendrakumar M, Yoshida N, Yoshida H (1986) TARA-3: a computer program to compute the response of 2-D embankments and soil structure interaction systems to seismic loadings, Department of Civil Engineering, University of British Columbia, Vancouver

Finn WDL, Ledbetter RH, Marcuson WF (1994a) Seismic deformations in embankments and slopes. In: Balasubramaniam AS et al (eds) Proceedings symposium on developments in geotechnical engineering from Harvard to New Delhi 1936–1994, Bangkok. Balkema Publication. Rotterdam, Netherlands. ISBN 90 5410 522 4, pp 233–264

Finn WDL, Ledbetter RH, Wu G (1994b) Liquefaction in silty soils: design and analysis. Ground failures under seismic conditions. ASCE Geotech Spec Publ 44:51–76

Idriss IM, Seed HB, Serff N (1974) Seismic response by variable damping finite elements. J Geotech Eng ASCE 100(1):1–13

Newmark NM (1965) Effects of earthquakes on dams and embankments. Geotechnique 15(2):139–160

Seed HB (1979) 19th Rankine lecture: considerations in the earthquake resistant design of earth and rockfill dams. Geotechnique 29(3):215–263

Seed RB, Harder LF (1990) SPT-based analysis of cyclic pore pressure generation and undrained residual strength. In: Proceedings of H. Bolton seed memorial symposium, vol 2. BiTech Publishers, Vancouver, pp 351–376

Seed HB, Wong RT, Idriss IM, Tokimatsu K (1986) Moduli and damping factors for dynamic analyses of cohesionless soils. J Geotech Eng ASCE 112(11):1016–1032

DOI 10.1007/978-3-642-36197-5_96-1

Wu G (2001) Earthquake induced deformation analyses of the Upper San Fernando dam under the 1971 San Fernando earthquake. Can Geotech J 38:1–15

Youd TL et al (2001) Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J Geotech Geo-Environ Eng ASCE 127(10):817–833 (21 authors)