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Seismic Performance Assessments of Embankments

Kemal Onder Cetin(1)*, Anil Yunatci(2), Bilal Umut Ayhan(3)

1) Middle East Technical University, Department of Civil Engineering, Ankara, Türkiye, +(90) (312) 210 2418 (kemalondercetin@gmail.com, ocetin@metu.edu.tr)

2) GeoDestek, Ankara, Türkiye

3) Istanbul Bilgi University, Department of Civil Engineering, Istanbul, Türkiye

Abstract Steps followed during seismic design and performance assessments of embankments include: i) seismic hazard assessments, ii) pseudo-static stability evaluations and the determination of seismic coefficient k, iii) allowable permanent deformations. For earthfill and rockfill dams and embankments, there exists a consensus regarding the selection of a 50 % probability of exceedance in 100 years hazard level for operation basis earthquake levels. The dam subjected to this shaking level is expected to behave elastically or almost elastically. However, for dams classified under "high" and "very high" risk, international codes suggest a seismic assessment which adopts a 10,000-year return period hazard level for safety evaluation earthquake, whereas national guidelines at regions of higher seismicity often recommend ground motions having lower return period (T_r= 2475 years) in practice. For transport infrastructure embankments and retaining structures, until recently, 475-year return period ground motions have established the design practice. However, after AASHTO (2006), 1,000-year return periods have started to be adopted as the basis for design. The corresponding seismic coefficient is applied to the critical block using the least favorable combinations to model earthquake shaking in both horizontal and vertical directions. The use of a predetermined constant seismic coefficient as a fraction of maximum acceleration, e.g.: half of maximum acceleration, is commonly proposed in practice, nevertheless it is shown that selection criteria shall be based on allowable displacement, critical block geometry, earthquake moment magnitude (or duration), and other relevant factors. It was noted that the seismic coefficient commonly used in outdated literature, corresponding to 0.5 of the maximum acceleration, aims to target permanent displacements in the order of 1.5-2.5 cm. A wide range of permissible permanent displacement values is defined in the literature. For dams, the stability of the dam body was concluded not to be significantly jeopardized when permanent displacements are below 1-1.5 meters (3-5 feet). For transport infrastructure embankments or natural slopes, permissible permanent displacement values range from 5 to 10 cm.

Keywords geostructures, earthfill and rockfill dams, retaining structures, seismic hazard assessment, deterministic analysis, seismic coefficient (k)

Introduction

The engineering evaluations for dynamic response of geo-structures such as natural or engineered slopes, soil-rock fills, transport infrastructure embankments and dams involve four assessment steps, as shown in Tab. 1. Firstly, design basis seismic hazard levels are identified to quantify the demand. The second step involves identifying their dynamic responses under seismic hazard levels. The third step assesses whether the response characteristics including induced-stresses, -strains, and -displacements, align with acceptable performance levels. If the desired performance level is exceeded, the evaluation proceeds to discuss improvement measures for existing structures or revisions of the design for the ones to be constructed.

Table 1 Seismic Performance Assessment Steps of Embankments and Slopes

Definitions
1. The Assessment of Seismic Hazard Level
2. Seismic Response Analyses: Pseudo-static vs. Dynamic
3. The Assessment of Seismically induced Deformations and Displacements: Allowable Deformation/Displacement Criteria
4. Mitigation Solution: If Needed

In this manuscript, current widely used engineering criteria for evaluating seismic hazard levels will be briefly discussed. As part of response assessments, the historical development of semi-static (pseudo-static) analyses, which serve as the initial stability assessment stage in the design under seismic loads, will be addressed. Special attention will be given to the discussion of estimating the seismic coefficient, "k", as functions of permanent deformations and displacements. Due to the complexity of the subject, discussions about evaluations related to dynamic response analyses will be excluded herein. A compilation of permissible deformation criteria available in the literature will be presented. The discussions on rehabilitation, and design revision measures, which constitute the fourth assessment stage, will also be deferred to another study. Following a brief overview of the historical advancements in the evolution of seismic design methods for embankments and slopes, the manuscript will provide discussions aligned with the outlined scope.

Historical progress in seismic design and performance evaluations

Embankments, including dams and transport infrastructures, have been designed with consideration for earthquake loads since the 1930s. In fact, embankments and slopes were among the first structures to be designed with seismic loading criteria compared to other engineering structures. The first dynamic analysis of an earthfill dam was conducted by Mononobe et al. in 1936. The dam body was represented by an infinitely long symmetric triangular linear elastic section resting on a rigid foundation (Mononobe et al., 1936; Seed and Martin, 1966). In this method, seismic forces acting on the dam body are determined by the product of the weight of the critical block and the seismic coefficient "k." The typical seismic coefficient values used at that time ranged from 0.10 to 0.15. During these analyses, when the safety factor drops below 1.0, the embankment is inaccurately classified as "close to failure." Ambraseys (1960) cautioned against relying on a predetermined constant seismic coefficient, advocating for the selection of the seismic coefficients based on seismic response analyses. Although semi-static analyses account for the embankment's inertia and the hydrodynamic effects of water loads on stability, they fall short in analyzing the combined interaction of the embankment body mass, foundation site conditions, water bodies, and earthquake shaking, and thus fail to reflect these interactions in the results.

In summary, the early practice in 1960s has evolved into multidimensional dynamic response analyses considering the multiple earthquake levels and using the soil-rock constitutive models based on effective stress. These advancements have shifted the approach from engineering judgement and experience-based assessment to pseudo-static analyses, and ultimately, two- or three-dimensional dynamic response analysis in current practice. Determining the seismic hazard (earthquake scenario) levels that serve as the foundation for these analyses has also evolved and has now reached its partially mature state. The next section will provide a brief summary of this evolution.

Seismic hazard assessment framework

There exists a considerable amount of randomness in the source properties of the earthquake process, namely the magnitude, location and time of occurrence. Variation of ground motion intensity along the distance from source to the site of interest also contains randomness. The basic logic behind probabilistic seismic hazard assessment is assigning a probability distribution for every random variable in the process. The probabilistic seismic hazard methodology, as described in Cornell (1968) integrates all the probabilities from each random variable, to come up with the rate of exceedance for a selected value of ground motion intensity.

Probabilistic seismic hazard analysis has often been compared and contrasted with deterministic methods of hazard calculations. While the scope of this paper is beyond flourishing this discussion; it is considered beneficial to briefly make a reasonable interpretation, in order to make a proper selection among the methods. McGuire (2001) discusses the role of probabilistic and deterministic methods in decision making purposes, in which the study distills to the major conclusion stating that both probabilistic and deterministic methods have a role in seismic hazard and risk analyses performed for decision-making purposes. These two methods can complement one another to provide additional insights to the seismic hazard or risk problem. Depending on the seismotectonic setting, indicating whether the total hazard is to be controlled by a single source, a fault segment with in-depth characterized properties or the performance criteria of the structure as well as the project scope, one method may have priority over the other. In many applications, use of hazard disaggregation to gain insight about the dominating set of scenarios based on the results obtained using the probabilistic framework is a common approach in practice.

Deterministic approach

The deterministic approach fundamentally covers the identification of each active earthquake source (faults), assigning a scenario earthquake magnitude, which is a function of the geometrical characterization of the associated fault segment anticipated to undergo independent rupture, closely or loosely supported by recorded seismicity data, also often coupled with closest source to site distance. The resulting magnitude – distance pair, along with additional input parameters controlling a varying number of effects, including but not limited to source mechanism and local site effects, forms the input for the empirical strong ground motion parameter estimation models. These intensity prediction models also contain their own model uncertainties and the median ground motion obtained from the independent earthquake scenario, or 84% (median+1 standard deviation in normal distribution) for embankment dams in the high and highest risk class can be calculated as a basis for design. The scenario that produces the largest value among all studied sources, assuming mutually exclusive and totally independent ruptures, is selected as the design ground motion level. This approach is called deterministic because the earthquake magnitude, source to site distance and the number of standard deviations to be added to the median estimate are considered as the only possible combination of variables in the calculations. For those interested in performance of critical structures, the question that comes to mind is whether a higher hazard (shake) will occur than what is obtained with the selected criteria in the deterministic scenario.

When adequacy of the median moment magnitude obtained from the fault rupture geometry, or the level of danger posed by ground movements with 1 standard deviation added to the median value is questioned by the designer, a "worst case scenario" is sought; in which applying a higher number of standard deviations, in the order of "2" on the calculated median ground motion intensity levels is brought into discussion. While the worst-case scenarios may indicate progress to a higher level of confidence, it often introduces conditions in which the project evolves into an unmanageable status in terms of fiscal constraints and engineering practice in seismically active regions, while not rationally quantifying the risk of failure of the structure and justifying target performance levels. Ground motion intensity levels, exceeding median + 2 standard deviation estimates at a particular spectral period (either peak components or spectral) has been frequently addressed after large magnitude events, which evokes the discussion of establishing performance-based criteria coupled with a fully probabilistic framework in which uncertainties due to ground motion intensity estimation and structural demands can be seamlessly blended and quantified.

Probabilistic Approach

The basic methodology of probabilistic seismic hazard assessment (PSHA) involves calculating the frequency of specified level of ground motion (either their peak or spectral values) will be exceeded at the location of interest. In a PSHA, the annual rate of events (annual rate of exceedance), ν, that produces a ground motion intensity parameter, let's pick spectral acceleration, Sa, that exceeds a specified level, z, at the site is calculated. The inverse of ν corresponds to the definition of return period. The calculation of the annual frequency of exceedance "ν" involves, i) the rate of earthquake of various magnitudes, ii) rupture dimensions of earthquakes, iii) the location of the earthquakes relative to the site, and iv) attenuation of the ground motion from the earthquake rupture to the site. The general expression for a selected "linearly characterized" source is presented in Equation 1.

$$v_i(Sa > z) = N_i(M_{min}) \int_{RL=0}^{RL_{max}} \int_{E_x=0}^{1} \int_{m=M_{min}}^{M_{max}} \int_{\varepsilon=\varepsilon_{min}}^{\varepsilon_{max}} f_{RL_i}(m, RL) f_{E_{x_i}}(m, E_x) f_{m_i}(m) f_{\varepsilon}(\varepsilon)\, P(Sa > z|m, r(RL, E_x), \varepsilon)\, dRL\, dE_x\, dm\, d\varepsilon \quad [1]$$

$P(Sa>z|m,r(RL,E_x),\varepsilon)$ is the probability of exceedance of the specified ground motion level for the given magnitude and distance, $f_i(m)$ and $f_i(r)$ are the probability density functions for the magnitude and distance for that source. The integration is carried out for every possible magnitude value between M_{min} and M_{max}, and source to site distance values corresponding to the magnitude of interest. $N_i(M_{min})$ is the annual rate of earthquakes having magnitudes greater than or equal to M_{min}. In Equation 1, ε is the number of standard deviations above or below the median ground motion, $f_\varepsilon(\varepsilon)$ is the probability density function for ε; which is essentially the standard normal distribution. RL is the rupture length, and E_x is the location of rupture along the fault length, "0" and "1" representing both ends of the fault. Unlike area source idealization, site to source distance is now a function of rupture dimension and location of rupture along the fault. More complicated forms of the hazard integral are possible by introducing additional variables to be randomized. Fig. 1 illustrates the basic steps involved in PSHA.

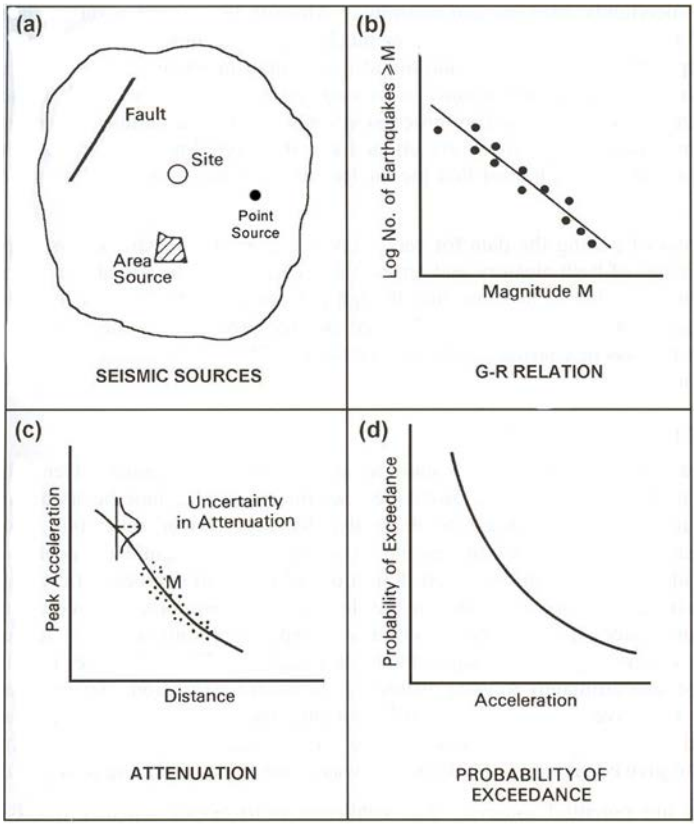

Figure 1 Basic steps of PSHA (Modified from Finn et al., 2004)

Whether calculated by deterministic or probabilistic methods, the response (performance) expected from the dam, or slopes in general varies under different calculated seismic hazard levels. In order to point out these differences, the seismic hazard scenarios frequently referred to in national and international specifications and the performance levels that the embankment or slope is expected to provide within the scope of these scenarios will be summarized in the next section.

Design basis seismic intensity levels

The Operating Basis Earthquake (O.B.E.), widely referred to in dam engineering, defines the level of strong ground motion that the embankments or dams are likely to be exposed to during its operational lifespan. Generally, considering that dams are typically designed for a lifespan of 100 years, there is a general consensus within the scope of O.B.E. evaluation that a level corresponding to a 50% hazard level over 100 years, equivalent to a strong ground motion event with a return period of 144 years, would be adequate (for example: ICOLD Bulletin 72, Revision 2010; FEMA 2005; Turkish Dam Agency, DSI Guide No: 1, 2012). The expected performance level from a dam structure exposed to O.B.E. level is defined as either the structure sustaining no damage or any damage being at sufficiently low levels as not to interrupt the operational activities of the structure. The prompt and economical repair of potential low-level damage is also among the expectations. Determining the O.B.E. level is more of an economically driven criterion for the operation rather than a safety-related assessment, so it may be governed by the preferences of investors than regulatory and approving public institutions (Wieland, 2005).

However, a more speculative scenario emerges when determining the hazard levels that form the basis for the design or safety of the structure. Within this context, in the literature, one encounters the maximum design earthquake (M.D.E.) and, more recently, definitions such as the safety-evaluation earthquake (S.E.E.) replacing this terminology in relation to the design process compatible with multiple hazard levels. When determining the safety-oriented earthquake level, a more complex scenario arises compared to the relatively consensual definition of the O.B.E., recommending the use of probabilistic evaluation levels ranging from the 475-year (10% in 50 years) design earthquake level to the 10,000-year (1% in 100 years) level. In addition to this differentiation, there are contradictions regarding which scenario will be considered if deterministic and probabilistic analysis results do not overlap. Some sources suggest selecting the lower of the deterministic and probabilistic hazard analysis results (Turkish Dam Agency, DSI Guide No: 1, 2012), while others suggest selecting the higher (ICOLD Bulletin 72, 2010).

Despite all this differentiation and complexity, it is clear that the new definitions have been introduced, along with efforts to classify embankment types, heights, and the dimensions of the impact in case of damage, somewhat assist in deciding the level of risk to be undertaken. However, especially when compared with national and international standards in determining safety-oriented hazard levels, serious differences and disagreements are observed. Particularly, for embankments classified under high and very high-risk groups, there is inconsistency between defining the safety-oriented earthquake (S.E.E.) level identified through probabilistic studies with earthquake scenarios corresponding to a recurrence period of 2,475 years and the recommendation of 10,000-year recurrence periods by the ICOLD Bulletin 72, 2010 revision. In regions, where seismic hazard levels are extremely high, determining S.E.E. levels corresponding to lower recurrence periods may sometimes support project feasibility by reducing maximum ground acceleration values calculated at levels compatible with 10,000-year recurrence periods to more reasonable levels of 1.5-2.0 g. However, the safety of these relatively lower recurrence levels needs to be debated.

At this stage, it is crucial to emphasize the necessity of establishing a common language between the project owner and the geoscientist or earthquake engineer. In this relationship, the party assuming the risk should clearly articulate the risk, and the decision on which safety criteria will be acceptable should not solely rest on the hazard assessor's mind. Today's accumulation of knowledge has evolved to enable decisions to be made based on the behavior (response) of the structure rather than solely on the forces that will affect the structure.

For conventional transport infrastructure embankments and slopes, the selection of design earthquake recurrence periods is less complex compared to hydraulic structures. In international literature, until 2006, recurrence periods of 475 years, corresponding to a 10% exceedance probability in 50 years, were used as the basis for design. After the AASHTO LRFD Bridge Design Specifications (2006), this recurrence period has been increased to the order of 1,000 years. This change has been accepted not only for bridges but also for key components of transportation systems, including highway embankments, fills, and retaining structures. In national specifications, however, earthquake scenarios corresponding to recurrence periods of 475 years have continued to be used as the basis of design for transport infrastructure embankments, fills, and retaining structures.

Assessing seismic coefficient, k in a performance-based design framework

Semi-static analysis methods, which are still widely used in the preliminary evaluation stage of embankment engineering analyses today, were the sole analysis method used in the seismic design of many existing ones built in the 1960s and earlier. As shown in Fig. 2, in this analysis method, the seismic forces acting on the dam body are determined by the product of the weight of the critical block, W, and the seismic "k" coefficient. The seismic coefficient is applied to the critical block using the least favorable combinations to model earthquake shaking in both horizontal and vertical directions. Typical seismic coefficient values used during that time ranged from 0.10 to 0.15. If the safety factor fell below 1.0, the embankment was inaccurately classified as "close to failure." Ambraseys (1960) suggested that the seismic coefficient, k, should be determined based on seismic response analyses and highlighted the limitations of using a fixed k value. Despite considering the inertia of the embankment mass and hydrodynamic forces (if there is any), pseudo-static analyses are insufficient for assessing the combined interaction of the embankment body mass, site conditions, earthquake, and reservoir, in case of dams, and may therefore not accurately represent their effects.

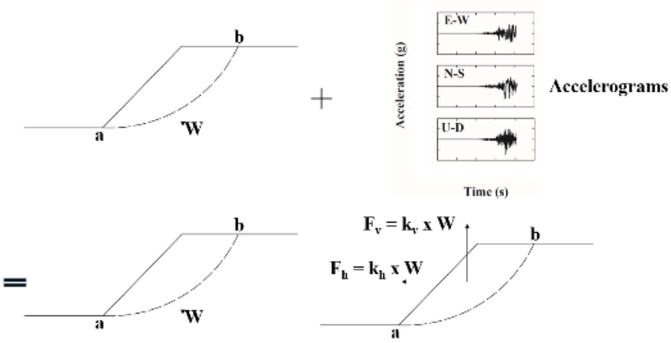

Figure 2 Pseudo-static analysis for embankments and slopes

Seed and Martin (1966) emphasized the need to select the seismic coefficient, k, based on dynamic response analyses and considering the stiffness of the material constituting the embankment body, rather than choosing fixed values. They determined the acceleration and stress variations in the embankment using the

Mononobe sliding beam method and viscoelastic response analyses. This allowed the stresses occurring on the sliding planes of the dam to be converted into equivalent seismic coefficients that varied across the dam body and over time. It was observed in all examples studied during that period that the seismic coefficient did not exceed a value of 0.4. However, no relationship was established between the maximum ground acceleration and the seismic k coefficient. Marcuson (1981) pointed out the relationship between the seismic coefficient and maximum ground acceleration, suggesting that an appropriate semi-static k coefficient should be selected at levels ranging from one-third to one-half of the calculated maximum ground acceleration considering the amplification or reduction effects consistent with local soil conditions in the embankment area. Similarly, after conducting numerous analyses using the Newmark method, Hynes-Griffin and Franklin (1984) concluded that if the safety factor exceeded 1.0 in semi-static stability analyses using a seismic k coefficient selected at one-half of the maximum ground acceleration/g, the expected deformations in embankment dams would be minimal and acceptable. Additionally, there is a limitation on the use of semi-static analyses in dam types where an increase in pore water pressure in the foundation soils of the embankment body may occur during an earthquake, disregarding a potential strength loss of up to 15-20% during earthquakes.

Despite their widespread use, semi-static analysis methods are now limited to the preliminary design stage due to their inherently irrational goal of converting repetitive motion, such as earthquakes, into equivalent static loads, resembling empirical structures. Often, when the safety factor falls below 1.0 during analysis, inaccurate conclusions are drawn, such as the critical dam mass being renewed and sliding completely downhill. However, under a repetitive and constantly changing (transient) load like an earthquake, inertia forces cannot always be applied in the same direction, changing with every millisecond of the earthquake. Thus, a rigid block that becomes unstable at any moment during the earthquake will stabilize again when the direction of the inertia force changes or diminishes due to the earthquake continuing. Furthermore, a block sliding downhill may move upward due to a subsequent earthquake in the opposite direction, potentially partially compensating some of the permanent deformations that occurred downhill in the previous seconds of the earthquake. Based on these observations, Newmark (1965) proposed that earthquakes' effects on dams should be examined in terms of deformations rather than minimum safety factors or equivalent seismic coefficients. Newmark and derivative methods were developed based on the assumption that when the shear stresses acting on the sliding plane exceed the shear strength, the critical mass block begins to slide. The acceleration level at which sliding is triggered is called the yield acceleration level. The displacement of the rigid block can be obtained by taking the double integral of the acceleration-time data above the yield level. The validity of the method has been confirmed using recorded displacement data from the La Villita Dam in Mexico, which is frequently exposed to high seismic levels (Elgamal et al., 1960). It should be noted that the Newmark method can be applied to dam bodies consisting of low-sensitivity fill materials that do not produce significant strength loss (less than 20% loss) under earthquake loads and do not produce significant pore water levels.

Makdisi and Seed (1977) demonstrated through numerical analyses, which, by today's standards, might be considered limited in number but represented serious efforts given the technology available at the time, that the seismic coefficient, k, could be determined through the interaction of dam height, depth of the critical block, maximum crest acceleration, and allowable displacement parameters. As shown in Fig. 3, after determining the maximum crest acceleration, the maximum seismic coefficient value, k_{max}, is found using a ratio determined by dividing the critical block depth, "y," by the dam height, denoted as 'h.' Then, the ratio obtained by dividing the seismic coefficient value, k_y, which triggers sliding and produces a safety factor of 1.0 in semi-static analyses, by k_{max}, is used to determine the expected post-seismic permanent horizontal displacement levels in the dam.

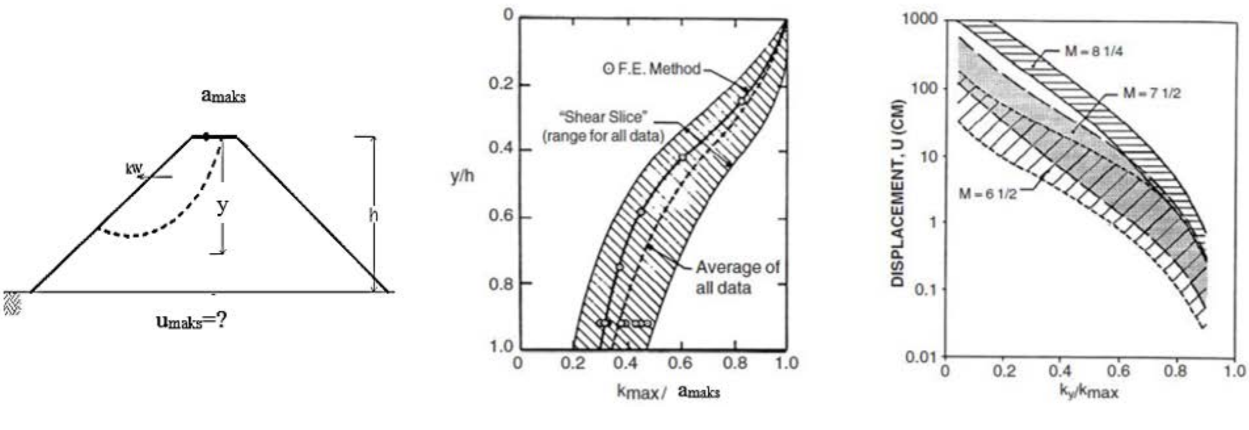

Figure 3 Assessing seismic coefficient, k, as per Makdisi and Seed (1977)

Makdisi and Seed's analysis method relies on several key input parameters, one of which is determining the maximum crest acceleration value. As presented by Kavruk (2003) and elsewhere, accelerations intensify at the crest of embankments to levels 1.5-4.0 times higher compared to the maximum rock ground accelerations. Taking into account that this amplification varies widely depending on factors such as dam rigidity, height, and seismic record characteristics, it is assumed that the maximum rock ground acceleration will amplify by a factor of about 2.0. With this assumption, seismic coefficient values corresponding to for circular failure surfaces passing from the toe of embankments, allowable displacement, moment magnitude, and y/h values for a safety factor of 1.0 can be obtained, as presented in Tab. 2. As shown in this simple calculation sequence, the primary parameters influencing the selection of the seismic coefficient are typically the allowable displacement criterion, earthquake moment magnitude, and depth of the critical sliding block. While selecting the seismic coefficient at levels around 0.4-0.6 of the maximum rock ground acceleration, as suggested in some specifications and design guidelines, may generally be somewhat compatible or on the safer side, as demonstrated in Tab. 2, this approach may occasionally result in unsafe outcomes for shallower embankments.

Table 2 The kg/$a_{max\text{-}rock}$ ratios varying with permanent lateral displacement, the moment magnitude of the event and y/h

Permanent Displacement (cm)	Moment Magnitude, M_w					
	6.50		7.50		8.25	
	y/h=1	y/h=0.6	y/h=1	y/h=0.6	y/h=1	y/h=0.6
2.5	0.41	0.64	0.44	0.69	0.53	0.83
10	0.24	0.38	0.33	0.52	0.45	0.70
30	0.12	0.19	0.24	0.37	0.38	0.60
60	0.08	0.13	0.17	0.27	0.33	0.52

The main purpose behind summarizing the equivalent seismic coefficients based on the work of Makdisi and Seed (1977) as presented in Tab. 2, is not to necessarily recommend the use of these values in preliminary quasi-static stability analyses, but to emphasize the necessity of selecting the equivalent seismic coefficient based on allowable displacement, critical block geometry, earthquake moment magnitude (or duration), and other relevant factors. After considering increased permissible (acceptable) permanent displacement levels, it becomes feasible to analyze and design the embankment with lower equivalent seismic coefficients. It should be noted that the seismic coefficient commonly used in outdated literature, corresponding to 0.5 of the maximum ground acceleration, aims to target permanent displacements at approximately 1.5-2.5 cm levels.

Semi-empirical models for permanent slope displacements

As part of performance assessment of seismically induced natural and embankment slopes, significant research efforts have been focused on assessing permanent displacements. Newmark's sliding block (NSB) concept (Newmark, 1965) has been frequently used by engineers to assess the performance of slopes under earthquake loading. As one of the first attempts, Ambraseys and Menu (1988) proposed a predictive model using 50 strong ground motion recordings. Their equation tried to predict the NSB displacement using the ratio between yield acceleration and PGA. After this, there have been various researchers (Yegian et al., 1991; Jibson, 1993; Ambraseys and Srbulov, 1994; Ambraseys and Srbulov, 1995; Crespellani et al., 1998) who tried to estimate the NSB displacement with various functional forms including additional ground motion intensity measures. However, these models were still utilizing a database with a limited number of ground motions. Therefore, the variability (i.e., standard deviation) in their predicted means was still high. To decrease such variability, Jibson et al. (1998) proposed a model using a relatively larger dataset consisting of 555 strong ground motion recordings from 13 events. However, in this time, aleatory variability in the predictions is increased. Since then, several predictive models have been proposed, some of which are presented in Tab. 3.

These predictive models link the intensity of shaking with permanent displacements, supporting the earlier conclusion that seismic coefficient k is governed by permanent displacements allowed for embankment or natural slopes. This necessitates discussions on allowable permanent displacements for slopes.

Allowable permanent displacements criteria for slopes

First and foremost, it should be emphasized that defining permissible displacements for embankments necessitates considering at least two different earthquake scenarios. Expectations for the operational earthquake level revolve around the embankment being able to continue its operational activities without interruption following this earthquake scenario. In this case, it is evident that the seismic response of the dam body should remain within elastic limits. Considering typical modulus reduction relationships based on unit deformation for materials like sand, clay, and rock, it can be noted that the elastic or nearly elastic limits correspond roughly to shear strains (unit deformations) in the ranges of 10^{-4} to 10^{-3} % for sands and clays, and 10^{-3} to 10^{-2} % for rocks. Similarly, for rock, the elastic behavior limit can be selected as 10^{-3} %. From hydraulic structures point of view, when considering the safety earthquake scenario, it is important to remember that the expectation is for the dam not to lose its ability to retain water. Therefore, the issue to be discussed is the determination of permissible permanent displacements that correspond to this scenario. A wide range of permissible permanent displacement values is defined in the literature under the safety earthquake scenario. Although related to factors such as dam height, zoning, filter thickness, and the strength and stiffness behavior of materials, it can be said that the stability of the dam body will not be significantly threatened when permanent displacements are below 1-1.5 meters (3-5 feet).

Table 3 A summary of some sliding block permanent displacement predictive models

Model	Eq. model	Functional form	Designated application	Number of records
Rigid	Newmark (1965)	$D_N = 3\frac{PGV^2}{PGA}\left(\frac{a_c}{PGA}\right)^{-1}$	Earth dams and embankments.	4
	Ambraseys and Menu (1988)	$log(D_N) = 0.90 + log\left[\left(1-\frac{a_c}{PGA}\right)^{2.53}\left(\frac{a_c}{PGA}\right)^{-1.09}\right]$	Ground and slopes	50
	Jibson (2007)	$log\left(D_{N(cm)}\right) = 0.215 + log\left[\left(1-\frac{a_c}{PGA}\right)^{2.341}\left(\frac{a_c}{PGA}\right)^{-1.438}\right]$	Natural slopes	2270
		$log\left(D_{N(cm)}\right) = -2.71 + log\left[\left(1-\frac{a_c}{PGA}\right)^{2.335}\left(\frac{a_c}{PGA}\right)^{-1.478}\right] + 0.424M$		
		$log\left(D_{N(cm)}\right) = 2.401 logI_a - 3.481 loga_c - 3.23$		
		$log\left(D_{N(cm)}\right) = 0.5611 logI_a - 3.833 loga_c - 1.474$		
	Saygili and Rathje (2008)	$log\left(D_{N(cm)}\right) = 5.52 - 4.43\left(\frac{a_c}{PGA}\right) - 20.93\left(\frac{a_c}{PGA}\right)^2 + 42.61\left(\frac{a_c}{PGA}\right)^3 - 28.74\left(\frac{a_c}{PGA}\right)^4 + 0.72ln\,(PGA)$	Natural slopes	2383
		$log\left(D_{N(cm)}\right) = -1.56 - 4.58\left(\frac{a_c}{PGA}\right) - 20.84\left(\frac{a_c}{PGA}\right)^2 + 44.75\left(\frac{a_c}{PGA}\right)^3 - 30.5\left(\frac{a_c}{PGA}\right)^4 - 0.64l\,n(PGA) + 1.55ln\,(PGV)$		
	Alfredo and Christian (2013)	$log(D_N) = -0.1 - 4.3\left(\frac{a_c}{PGA}\right) + \log\left(\frac{I_aT}{PGA}\right)$	-	-
	Bray and Travasarou (2007)	$log(D_N) = -0.22 - 2.83\ln(k_c) - 0.333(\ln(k_c))^2 + 3.04\ln(a_c) + 0.566\ln\,(k_c))\ln\,(a_c) - 0.244(\ln(a_c))^2 + 0.278(M-7)$	Earth and waste slopes	1376

It should be noted that permanent displacements exceeding 3 meters (approximately 10 feet) are considered unacceptable for embankment dams, and even in cases where displacements exceed these levels, the calculated values may not be reliable due to limitations in analysis methods and structural models. Permanent displacements between 1.5 and 3.0 meters constitute a gray area, indicating the need for careful analysis and potentially reassessment. It should be recognized that these assessments are general and rough recommendations in the literature, and it is possible to encounter examples of dam performance that do not conform to these limits, whether stable or unstable. Instead of a wholesale approach, it is imperative that the analysis and evaluation of results be conducted by specialized engineers tailored to the specific project.

For transport infrastructure embankment or natural slopes, the decision is less complex. Depending on the importance and intended use of the retaining structure, permissible permanent displacement values for slopes typically range from 5 to 10 cm.

Summary and conclusion

The analysis stages followed in determining the dynamic behavior of embankments and natural slopes consist of four analysis stages. The first stage involves determining the seismic hazard levels that will form the basis for the design or performance evaluations of the structure. The second stage deals with the calculation of the dynamic response behavior of the structure under these seismic hazard levels, and during this stage, the response characteristics (stress, strain, displacement, etc.) are examined to determine whether they are consistent with the expected or allowable performance levels, which constitutes the third stage. In the final stage, if the determined performance is not compatible with acceptable levels, improvement measures for existing geo-structures or revision measures for structures in the design phase are discussed.

In this manuscript, the first analysis stage focuses on summarizing the current national and international criteria for determining seismic hazard levels, highlighting their agreements and differences. The second stage covers a discussion on pseudo-static (or semi-static) analyses, which represent the initial stability assessment stage for geotechnical structures under seismic loads in their historical development process. Particularly, emphasis is placed on the seismic coefficient "k," which is a fundamental input parameter for pseudo-static analyses. Due to the complexity of the subject and the page limit of the paper, evaluations related to dynamic response analyses have been deferred for another study. In the scope of the third evaluation stage, a compilation of available permissible deformation criteria from national and international literature has been made, addressing not only the compatibility and discrepancies between these criteria but also their shortcomings. Justified by the same reasoning for excluding dynamic response analyses from the paper, the fourth evaluation stage concerning improvement and section revision measures has also been reserved for another study. A detailed discussion on seismic hazard assessment framework, selecting the design basis seismic intensity levels, and corresponding seismic coefficient k values, was provided. Currently available semi empirical models for assessing permanent seismic slope displacements are evaluated. Followings are the major findings and conclusions of these discussions:

- The deterministic seismic hazard assessment involves the identification of active earthquake sources (faults), assigning a scenario earthquake magnitude, which is a function of the geometrical characterization of the associated fault segment anticipated to undergo independent rupture.
- The resulting earthquake magnitude – distance pair forms the input for the empirical strong ground motion parameter estimation models.
- The median ground motion obtained from the independent earthquake scenario, or 84% (median+1 standard deviation in normal distribution) for critical embankments located in the high and highest risk class are selected as design basis scenario.
- In probabilistic seismic hazard assessment (PSHA), the frequency of specified level of ground motion (either their peak or spectral values), exceeding certain thresholds are assessed for the location of interest.
- The annual rate of events (annual rate of exceedance), ν, that produces a ground motion intensity parameter, that exceeds a specified level, z, at the site is calculated. The inverse of ν corresponds to the definition of return period.
- The Operating Basis Earthquake (O.B.E.), widely referred to in dam engineering, defines the level of strong ground motion that the embankments or dams are likely to be exposed to during its operational lifespan.
- Considering that dams are typically designed for a lifespan of 100 years and conventional transport infrastructure embankments for 50 years, there is a consensus within the scope of O.B.E. evaluation that a level corresponding to a 50% hazard level over 100 or 50 years, equivalent to a strong ground motion event with a return period of 144 and 72 years, would be adequate.
- For hydraulic structures (dams) classified under high and very high-risk groups, there is inconsistency between defining the safety-oriented earthquake (S.E.E.) level identified through probabilistic studies with earthquake scenarios corresponding to a recurrence period of 2,475 years and the recommendation of 10,000-year recurrence periods by the ICOLD Bulletin 72, 2010 revision.
- For conventional transport infrastructure embankments and slopes, the selection of design earthquake recurrence periods is less complex compared to dams. In international literature, until 2006, recurrence periods of 475 years, corresponding to a 10% exceedance probability in 50 years, were used as the basis for design.
- After the AASHTO LRFD Bridge Design Specifications (2006), this recurrence period has been increased to the order of 1,000 years. This change has been accepted not only for bridges but also for key components of transportation systems, including highway embankments, fills, and retaining structures. In national specifications, however, earthquake scenarios corresponding to recurrence periods of 475 years are still used as the basis for highway embankments, fills, and retaining structures.
- In semi-static analysis methods, which are still widely used in the preliminary evaluation stage of embankments, the seismic forces acting on the dam body are determined by the product of the weight of the critical block, W, and the seismic "k" coefficient.
- The seismic coefficient is applied to the critical block using the least favorable combinations to model earthquake shaking in both horizontal and vertical directions. Typical seismic coefficient values used during that time ranged from 0.10 to 0.15.
- Ambraseys (1960) cautioned against relying on a predetermined constant seismic coefficient, advocating instead for the selection of the seismic coefficients based on seismic response analyses.
- Although semi-static analyses account for the embankment's inertia and the hydrodynamic effects of water loads on stability, they fall short in analyzing the combined interaction of the embankment body mass, foundation site conditions, water bodies, and earthquake shaking, and thus fail to reflect these interactions in the results.
- The equivalent seismic coefficient is shown to be selected based on allowable displacement, critical block geometry, earthquake moment magnitude (or duration), and other relevant factors.
- It was noted that the seismic coefficient commonly used in outdated literature, corresponding to 0.5 of the maximum ground acceleration, aims to target permanent displacements at approximately 1.5-2.5 cm levels. This conclusion was also supported by available sliding block permanent displacement predictive models.
- A wide range of permissible permanent displacement values is defined in the literature under the safety earthquake scenario for dams. Although related to factors such as dam height, zoning, filter thickness, and the strength and stiffness behavior of materials, it can be said that the stability of the dam body will not be significantly threatened when permanent displacements are below 1-1.5 meters (3-5 feet).

- Permanent displacements between 1.5 and 3.0 meters constitute a gray area, indicating the need for careful analysis and potentially reassessment. It should be recognized that these assessments are general and rough recommendations in the literature, and it is possible to encounter examples of dam performance that do not conform to these limits, whether stable or unstable.
- For transport infrastructure embankment or natural slopes, the decision is less complex. Depending on the importance and intended use of the retaining structure, permissible permanent displacement values for slopes typically range from 5 to 10 cm.

As the final remark, for dynamic assessment of embankments and natural slopes, seismic hazard assessment framework is recommended to be closely followed to evaluate multiple design scenarios with variable acceptable performance definitions. As part of preliminary design, pseudo-static analyses are widely performed to assess the performance of these geo-structures, during which seismic coefficient k, is recommended to be selected considering the geometry and the rigidity of the failure block, earthquake shaking characteristics and allowable permanent displacements. During these analyses, when the safety factor drops below 1.0, the embankment or slope should not be inaccurately classified as "close to failure." Despite considering the inertia of the embankment mass, pseudo-static analyses are insufficient for assessing the combined interaction of the embankment body mass, site conditions, earthquake, and reservoir (in case of dams), and may therefore not accurately represent their interaction. Thus, when these interactions are critical, pseudo-static assessments are recommended to be complemented by effective stress based dynamic response evaluations.

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