Intensity Measure Selection for Dynamic Analysis of Earth Dams

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ABSTRACT

A comprehensive study has been developed to determine the optimal Intensity Measure to be implemented in vulnerability and quantitative risk evaluation of earth dams. Newmark analyses and nonlinear time-history evaluations of a typical earth-core rockfill dam section were performed under a total of 23 near-field ground motion records. The comparison between the 19 IMs proposed was made on five properties: efficiency, practicality, proficiency, sufficiency and hazard computability.

Traditionally, peak ground acceleration has been used as the link between seismic hazard and structural analysis. However, results revealed that spectral parameters as velocity spectrum intensity and Housner intensity showed the best correlation with Newmark displacements, followed by the root mean square of acceleration. In general, velocity-related parameters and in particular, cumulative absolute velocity and sustained maximum velocity are the most appropriate IM regarding crest settlement.

KEYWORDS: seismic engineering, dynamics, earth dams

INTRODUCTION

One of the most critical issues at the present time is the quantitative risk assessment of structures. The susceptibility of a system to suffer serious consequences when subject to earthquake loads, i.e. the system seismic vulnerability, is frequently described using fragility curves. Fragility curves relate the probability of exceeding a specific damage state to seismic intensity. A detailed state of the art on the vulnerability assessment of buildings, lifelines, transportation infrastructures and critical facilities is presented in Pitilakis et al. (2014). Research progress is evident in studies concerning reinforced concrete and masonry buildings, bridges and railways but there are no references to dams and specifically to earth and embankment dams. A methodology for qualitative risk assessment of dams is proposed in Bureau (2003), in which risk factors and weighting points can be used to quantify the total risk factor (TRF) of a dam. The TRF depends on the dam characteristics as type, age, size, the downstream risk potential and the dam vulnerability, related to the seismic hazard of the site and obtained from curves based on expert judgment. This procedure was intended to quickly asses the potentially most vulnerable facilities in a large dam inventory and to assign priorities for seismic safety evaluation of the most critical dams. On the other hand, analytical procedures to derive fragility curves are based on the results of structural models under seismic loadings to derive damage distributions.

Over the past few decades, the Pacific Earthquake Engineering Research center PEER center has developed and improved a performance based methodology for seismic risk evaluation of buildings and bridges. The most interesting aspect of this probabilistic framework is the deconvolution of the process into independent steps linked by four generalized variables, Intensity Measures (IM), Engineering Demand Parameter (EDP), Damage Measures (DM) and Decision Variables (DV). A complete description of the methodology can be found in Deierlein et al. (2003). The variables are linked together using Equation 1, based on the total probability theorem:

$$\lambda(D) = \iiint G(DV|DM)\, dG(DM|EDP)\, dG(EDP|IM)\, d\lambda(IM) \qquad (1)$$

The focus of this paper is on the seismic demand model, the second step of the procedure, which provides a probabilistic relationship between a hazard IM and the structural response EDP (Wang, 2012).

An IM summarizes the earthquake attributes and defines the salient characteristics that affect the structural response. An IM is a property of an accelerogram that can be found simply and cheaply (Cornell, 2004). On this same issue, Padgett et al. (2008) proposed five properties of an IM that would qualify it as optimal. These are efficiency, practicability, proficiency, sufficiency and hazard computability.

In reference to the EDP, methods to analyze the seismic behavior of earth structures have shown major advances in the last two decades thanks to the increased computational power and evolution of engineering software. The methods can be grouped into three main classes: simplified analysis or pseudo-static approach, simplified dynamic analysis, such as Newmark method and its modifications, and fully dynamic analyses, which consider the soil as a continuum and deformable media. While displacements from simplified dynamic analyses provide an index to correlate with field performance (Jibson, 2011), research has shown that the last group is capable to adequately predict the structure performance and deformation behavior during and after an earthquake (Sica, 2002; Brigante, 2012). According to the International Commission on Large Dams (ICOLD, 2016) guidelines, safety concerns for embankment dams involve loss of strength of materials or excessive deformations (settlement, cracking of the impervious core). The permanent crest settlement C is a suitable parameter that may be adopted to characterize the dam response (Brigante, 2012). However, reliable time-history dynamic analyses often require an excessive number of records, having a significant impact on the variability observed in the EDP. Selection of real records based on strong motion parameters that are related to the structural response constitutes an efficient way to address this issue (Iervolino, 2008; Katsanos, 2010; Travasarou, 2003). Real accelerograms are more realistic than spectrum-compatible artificial records and easier to obtain than synthetic accelerograms generated from seismological source models (Bommer, 2004). For the most simplified procedures to estimate slope performance, Peak Ground Acceleration (PGA) has been chosen as primary IM, sometimes supplemented by additional parameters such as predominant period and significant duration (Travasarou, 2003). Previous studies with simplified dynamic analysis concluded that ground motion parameters such as Arias Intensity and Housner Intensity showed the best correlation with the permanent displacement of soils (Barani, 2010; Bray, 2007). However, comprehensive evaluation of the properties of an IM based on the results of fully dynamic analyses of earth dams remains undone and requires further research.

The aim of this paper is to provide guidance for appropriate IM selection for vulnerability and risk analyses of embankment dams. A 100 m high earth-core dam is presented as case study for comparing the relationship between 19 typical IMs and the seismic response of the structure. Newmark displacements and crest settlements from nonlinear dynamic analyses are used to evaluate the potential damage due to dynamic loading. The ground motion database compiled consists of 23 real ground motion records, characterized as near-fault records.

IM: DESCRIPTION AND PROPERTIES

Ground motion parameters describe at least one of the three main characteristics of earthquakes: amplitude, frequency content and duration (Kramer, 1996). The 19 different IMs considered in this work are listed in Table 1.

The parameters PGA, PGV, and PGD represent the maximum amplitudes of acceleration, velocity, and displacement of the records. The root mean square acceleration arms, is defined as:

$$a_{RMS} = \frac{1}{DS}\int_{t_1}^{t_2}[a(t)]^2\,dt \tag{2}$$

The DS is determined from the Husid plot (Husid, 1969), based on the interval during which the 5 to 95% of the total Arias Intensity IA is accumulated. The IA is computed as:

$$IA = \frac{\pi}{2g}\int_0^{t_{tot}}[a(t)]^2\,dt \tag{3}$$

The Characteristic Intensity IC is determined using the relation:

$$IC = a_{RMS}{}^{3/2}\sqrt{t_{tot}} \tag{4}$$

The Specific Energy Density SED is defined as:

$$SED = \int_0^{t_{tot}}[v(t)]^2\,dt \tag{5}$$

The Cumulative Absolute Velocity CAV is computed as:

$$CAV = \int_0^{t_{tot}} a(t)\,dt \tag{6}$$

The area under the acceleration response spectrum between periods of 0.1 and 0.5 s is defined as the Acceleration Spectrum Intensity:

$$ASI = \int_{0.1}^{0.5} Sa(\xi = 0.05, T)\,dT \tag{7}$$

Table 1: Summary of ground motion parameters

Ground Motion Parameters	Abbreviation	Units
Peak Ground Acceleration	PGA	g
Peak Ground Velocity	PGV	m/sec
Peak Ground Displacement	PGD	m
Peak Velocity and Acceleration Ratio	V_{MAX}/A_{MAX}	sec
Root Mean Square of acceleration	a_{RMS}	g
Root Mean Square of velocity	v_{RMS}	m/s
Root Mean Square of displacement	d_{RMS}	m
Arias Intensity	IA	m/sec
Characteristic Intensity	IC	$g^{3/2}sec^{1/2}$
Specific Energy Density	SED	m^2/sec
Cumulative Absolute Velocity	CAV	m/sec
Acceleration Spectrum Intensity	ASI	g . sec
Velocity Spectrum Intensity	VSI	m
Housner Intensity	HI	m
Sustained maximum acceleration	SMA	g
Sustained maximum velocity	SMV	m/sec
Effective Design Acceleration	EDA	g
Predominant Period	Tp	sec
Significant duration	DS	sec

The area under the pseudo-velocity response spectrum between periods of 0.1 and 2.5 s the Housner Intensity:

$$HI = \int_{0.1}^{2.5} PSV(\xi = 0.05, T)\, dT \tag{8}$$

Housner Intensity is similar to VSI, the Velocity Spectrum Intensity which is obtained from the absolute velocity spectrum, for the same period range.

The Sustained Maximum Acceleration SMA is the third highest absolute value of acceleration in the time-history and the Effective Design Acceleration EDA is defined as the peak acceleration value found after low pass filtering the input time history with a cut-off frequency of 9 Hz. Finally, the Predominant Period is the period at which the maximum spectral acceleration occurs in an acceleration response spectrum calculated at 5% damping.

The first stage in the PEER framework is the hazard analysis, which results in an IM that is a summary of the ground motion severity. Likewise, the output of the second stage is the EDP to characterize the structural response. To compare the adequacy of the evaluated IM, the properties proposed by Padgett et al.(2008), efficiency, practicality, proficiency, sufficiency and hazard computability, are evaluated. In a few words, efficiency refers to the accuracy in the prediction of the EDP given an IM and it is a measure of the scatter in the regression analysis, the smaller the dispersion the better. Practicality evaluates the correlation between these two variables and proficiency is a measure of the two properties described above.

The IM is sufficient if the EDP is independent of the earthquake intrinsic properties such as magnitude, fault type and source-to-site distance. The chosen parameter represents a larger level of information about the shaking, which overwhelms the dependency of the structural response on any ground motion characteristic (Iervolino, 2008). This is a fundamental property because it leads to a simpler evaluation and validates accelerogram scaling (Giovenale, 2004).Finally, the hazard computability is referred to the IM itself and the effort required to determine its hazard curve.

REAL RECORD SELECTION

In this study, the ground motion database compiled consists of 2 sets of real accelerograms extracted from the European Strong-Motion Database (http://isesd.hi.is/) and the PEER NGA Database (http://ngawest2.berkeley.edu/). The first set includes 11 records and the second 12 records, both with characteristics of near-field motions and increasing levels of PGA. All the time histories are recorded on soil classified as type A or B, according to the Geomatrix site classification (Abrahamson, 1996). The classification criterion is shown in table 2.

Near-fault motions are expected to produce significant damage to the structure (Bommer, 2001). Information on the ground motion selected data is presented in tables 3 and 4. The recorded signals were processed with Seismo Match V2.1.0 software [22] to determine the parameters from Table 1.

There is no unique definition for which a site may be classified as near or far-field. The first attempts to establish a limit were based on the source-to-site distance and magnitude of the event [23, 24, 25, 26]. Most recently, Martinez-Pereira and Bommer [27] defined the concept of near-source region considering the characteristics of the strong motion. Taking an intensity of VIII to be the threshold for motion that is potentially damaging to well-engineered structures, a lower bound was established for six strong-motion parameters: PGA, CAV, PGV, IA, I and aRMS. Records are only considered to be near-field if they simultaneously passed the proposed thresholds [21, 28]. The damage parameter I proposed in Fajfar et al. [29] is computed with relation (9):

$$I = PGV.DS^{0.25} \tag{9}$$

The lower-bound values of the parameters are given in table 5. The curves shown in figure 1 represent the upper limit of the magnitude-distance space defined by records passing all six thresholds. In the same figure, the pairs of magnitude and distance of the selected records are shown, confirming the adopted criteria.

Table 2: Geomatrix site classification

Site Class	Description	Definition
A	Rock	Vs > 600 m/s or < 5 m of soil over rock.
B	Shallow (stiff) soil	Soil profile up to 20 m thick overlying rock.
C	Deep, narrow soil	Soil profile at least 20 m thick overlying rock, in a narrow canyon or valley
D	Deep, broad soil	Soil profile at least 20 m thick overlying rock, in a broad valley.
E	Soft, deep soil	Soil profile with average Vs < 150 m/s.

Table 3: Set 1, near-fault records. Soil type A and B

Event	Record	Mw[1]	R_{RUP}[2] [km]	PGA [g]	Site Class
San Fernando, U.S, 1971	PUL-164	6.6	1.81	1.202	A
Duzce, Turkey, 1999	375-000	7.1	3.93	0.951	B
Tabas, Iran, 1978	TAB-LN	7.4	2.05	0.820	A
Kobe, Japan, 1995	KJM-000	6.9	0.96	0.805	B
Landers, U.S.,1992	LCN-260	7.3	2.19	0.713	A
Loma Prieta, U.S., 1989	WAH-090	6.9	17.47	0.659	B
Whittier Narrows, U.S., 1987	TAR-090	6.0	41.20	0.631	B
Kobe, Japan, 1995	KJM-090	6.9	0.96	0.587	B
Chi-Chi, Taiwan, 1999	TCU071-090	7.6	5.80	0.556	A
Chi-Chi, Taiwan, 1999	TCU068-090	7.6	0.32	0.555	A
Northridge, U.S., 1994	MUL-279	6.7	17.15	0.506	A

Mw=earthquake magnitude
R_{RUP}= closest distance to the fault rupture plane

Table 4: Set 2, near-fault records. Soil type A and B

Event	Record	Mw	R_{RUP} [km]	PGA [g]	Site Class
Cape Mendocino, U.S, 1992	CPM-000	7.1	6.96	1.497	A
Morgan Hill, U.S., 1984	CYC-285	6.2	0.53	1.298	A
San Fernando, U.S., 1971	PUL-254	6.6	1.81	1.160	A
Chi-Chi, Taiwan, 1999	TCU084-090	7.6	11.48	1.157	A
Chi-Chi, Taiwan, 1999	CHY080-090	7.6	2.69	0.968	A
Loma Prieta, U.S., 1989	LGP-000	6.9	3.88	0.966	A
Ardal	00158XA	6.0	4.00	0.891	A
Tabas, Iran, 1978	TAB-TR	7.4	2.05	0.852	A
Northridge, U.S., 1994	RRS-228	6.7	6.50	0.825	B
Chi-Chi, Taiwan, 1999	TCU095-000	7.6	45.20	0.698	B
Coalinga, U.S., 1983	PVY-045	6.4	8.41	0.592	A
Mammoth Lakes, U.S., 1980	LUL-090	5.9	16.88	0.408	A

Table 5: Lower-bound values for near field strong motion parameters.

IM	Units	Lower bound
PGA	g	0.20
CAV	g.s	0.30
PGV	cm/s	20.00
IA	m/s	0.40
I	$cm/s^{0.75}$	30.00
a_{RMS}	m/s^2	0.50

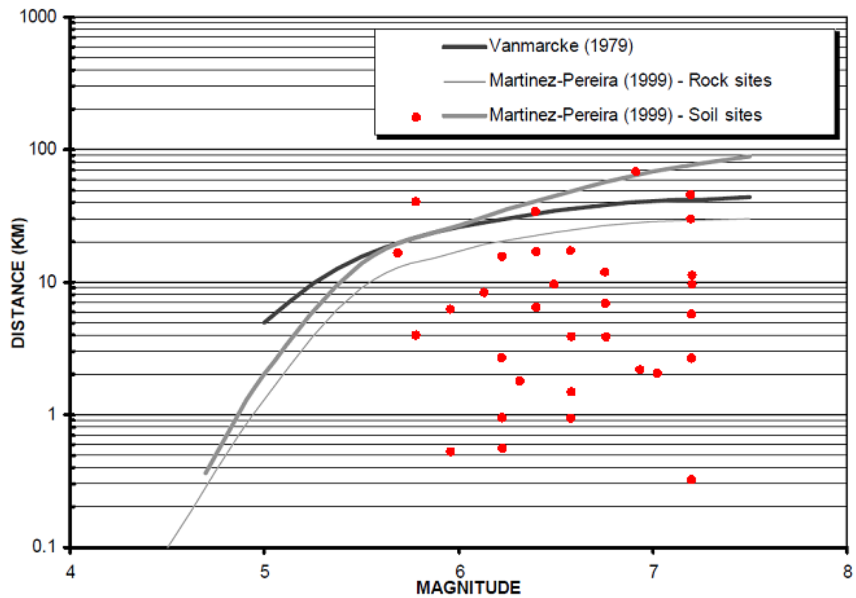

Figure 1: Location of selected records on the magnitude-distance space comprising the earthquake near-field, adapted from Martinez-Pereira and Bommer (1998).

CASE STUDY

A representative earth core dam with deterministic geometry and material characteristics is used as a case study to compare the typical IMs previously selected. The dam is introduced in Hunter and Fell [30]. In their work, the deformation behavior during construction, first filling and long term had been studied from a database of more than 130 real cases. The study was complemented by a finite difference model. The purpose was to define abnormal deformation and provide some guidance on the trends in behavior that are potentially indicative of slope instability. The typical dam section adopted is a 100 m height embankment with slope ratio 2:1 (H:V) with vertical core with similar shear strength properties to the rockfill, typical of well-compacted silty gravel materials with non plastic fines. Static analyses presented in [30] were chosen as the starting point for this research.

SIMPLIFIED DYNAMIC ANALYSES

The Newmark sliding block method (Newmark, 1965) has been widely discussed in the bibliography (Bray, 2007; Jibson, 1993). The method idealizes the soil as a rigid block and it is still the basis of several numerical techniques used to calculate earthquake-induced displacements in practice (Rathje, 2000). Once the acceleration from the earthquake record becomes equal to the yield acceleration, the frictional resistance is exceeded and sliding is initiated. The response is measure by the cumulative displacement D which is obtained by double integration of the acceleration time history. The yield or critical acceleration, ac, is a function of the static factor of safety of the potential sliding mass FS and its geometry and weight:

$$a_c = (FS - 1)g.\sin\alpha \quad (10)$$

In the first place, the yield acceleration was calculated. The static analysis was performed by a limit equilibrium procedure using the simplified Bishop method (Bishop, 1955). A critical surface search was performed, to find the circular slip surface with the lowest factor of safety, both in upstream and downstream shells of the dam. For a factor of 1.72 and a thrust angle of 26,11°, as depicted in figure 3, the yield acceleration of the mass is 0.317 g, which is a threshold for the beginning of sliding and it is constant throughout the analysis. Once the yield acceleration has been defined, Newmark displacement can be determined by double integration of those parts of the selected acceleration records above the critical value. The integration is performed twice, first for the actual time history and secondly for the same record reversed about the time axis up-slope, down-slope. In this case, the larger of the two values is used as the permanent displacement and the values are given in tables 6 and 7.

Due to the simplifications considered in the analysis, Newmark displacements must be considered indices of dynamic slope performance rather than precise predictions of actual slope displacement (Rathje, 2000; Jibson,2011). Wieczorek et al. (1985) proposed that the critical displacement would be 5 cm considering cracks that could be visualized when total slope failure occurred. Keefer and Wilson (1989) used 10 cm as the critical displacement for landslides. Finally, Jibson et al. (2000) concluded that the slope could be identified as failure when displacement became larger than 15 cm. Thus, the value of 15 cm is adopted as critical displacement.

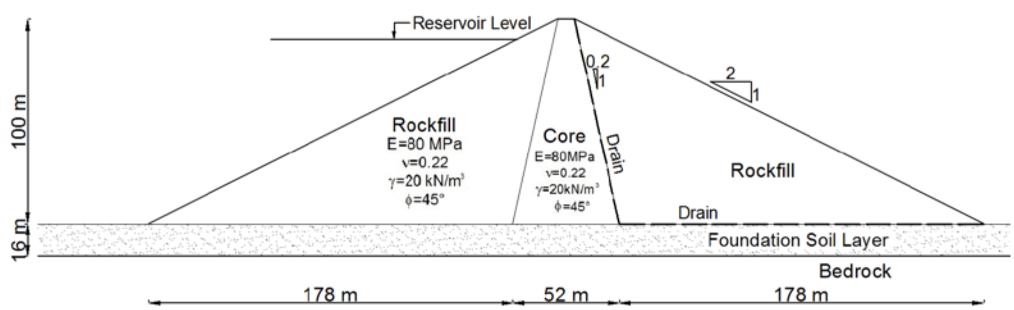

Figure 2: Earth dam section and material zones

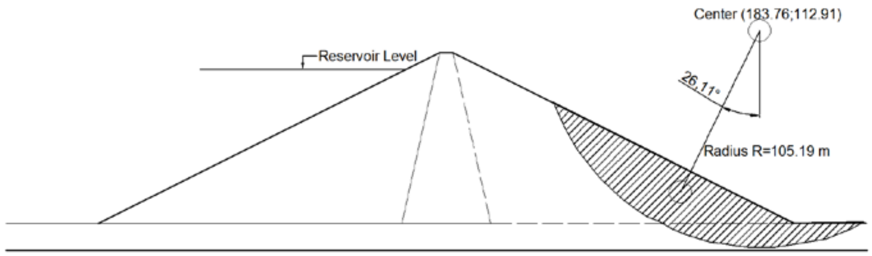

Figure 3: Geometry of the critical slip surface

Table 6: Set 1, near-fault records. Soil type A and B

Event	Record	D (cm)	C (m)
San Fernando, U.S, 1971	PUL-164	12.80	4.42
Duzce, Turkey, 1999	375-000	6.80	1.30
Tabas, Iran, 1978	TAB-LN	10.70	8.35
Kobe, Japan, 1995	KJM-000	26.30	5.29
Landers, U.S.,1992	LCN-260	1.80	2.39
Loma Prieta, U.S., 1989	WAH-090	1.60	2.23
Whittier Narrows, U.S., 1987	TAR-090	1.50	0.06
Kobe, Japan, 1995	KJM-090	13.90	4.74
Chi-Chi, Taiwan, 1999	TCU071-090	2.70	6.69
Chi-Chi, Taiwan, 1999	TCU068-090	3.10	7.59
Northridge, U.S., 1994	MUL-279	2.50	3.93

Table 7: Set 2, near-fault records. Soil type A and B

Event	Record	D (cm)	C (m)
Cape Mendocino, U.S, 1992	CPM-000	12.3	1.21
Morgan Hill, U.S., 1984	CYC-285	14.1	2.09
San Fernando, U.S., 1971	PUL-254	10.2	3.15
Chi-Chi, Taiwan, 1999	TCU084-090	94.6	10.02
Chi-Chi, Taiwan, 1999	CHY080-090	32	9.31
Loma Prieta, U.S., 1989	LGP-000	10.3	6.38
Ardal	00158XA	4.4	0.68
Tabas, Iran, 1978	TAB-TR	15.1	8.66
Northridge, U.S., 1994	RRS-228	40.4	5.3
Chi-Chi, Taiwan, 1999	TCU095-000	3.5	1.1
Coalinga, U.S., 1983	PVY-045	4.9	3.71
Mammoth Lakes, U.S., 1980	LUL-090	0	1.24

ADVANCED NUMERICAL ANALYSES

To complete the study of the impervious core dam, a plane strain finite element model was performed in PLAXIS 2D software (2012). A two-phase coupled effective stress model of the dam-foundation system was developed using 15-Node triangular elements. Staged construction of the dam was simulated by considering 10 material layers. The extension of foundation considered is 400 m from the toe in both directions, upstream and downstream and 16 m depth and the domain was discretized into 10007 elements and 81651 nodes. In dynamic analysis, the element size is chosen to ensure that a wave during a single step does not move a distance larger than the minimum dimension of an element (PLAXIS 2D Manual, 2012). Hardening Soil constitutive law was chosen for embankment material. The model involves friction hardening to model the plastic shear strain in deviatoric loading, and cap hardening to model the plastic volumetric strain in primary compression (Ti, 2009; Brinkgreve, 2012). The finite element mesh is depicted in figure 4 while table 8 summarizes the soil constitutive parameters assumed.

Table 8: Hardening Soil Model parameters

Parameter	Units	Core	Rockfill
γ_{unsat}	kN/m^3	20	20
γ_{sat}	kN/m^3	22.3	22.3
Poisson's ratio ν	-	0.2	0.2
E_{50}^{ref}	kN/m^2	33000	26000
E_{oed}^{ref}	kN/m^2	16500	26000
Power m	-	1	0.5
E_{ur}^{ref}	kN/m^2	100000	80000
Cohesion c'	kN/m^2	0	0
Friction ϕ	°	45	45

The amount of damping shown by a numerical system is determined by the choice of the constitutive model, also known as material damping, the integration scheme of the equations or numerical damping and the boundary conditions (Visone, 2010). For the HS model the damping from irreversible strains is not enough to model the real characteristics of soils (PLAXIS 2D, 2012; Visone, 2008; Elia, 2011). It is also not sufficient the numerical damping introduced by the implicit Hilber-Hughes-Taylor (HHT) scheme or α-method adopted for the calculations (γ=0.1). Hence, Rayleigh damping was adopted to account for the energy dissipation through the medium, with a target value of damping ratio ξ of 2.5% and absorbent boundaries were implemented along foundation limits using the Lysmer and Kuhlemeyer formulation (Lysmer, 1969). The analyses were performed under full reservoir condition. The vertical displacement of the center point of the dam crest was chosen as EDP.

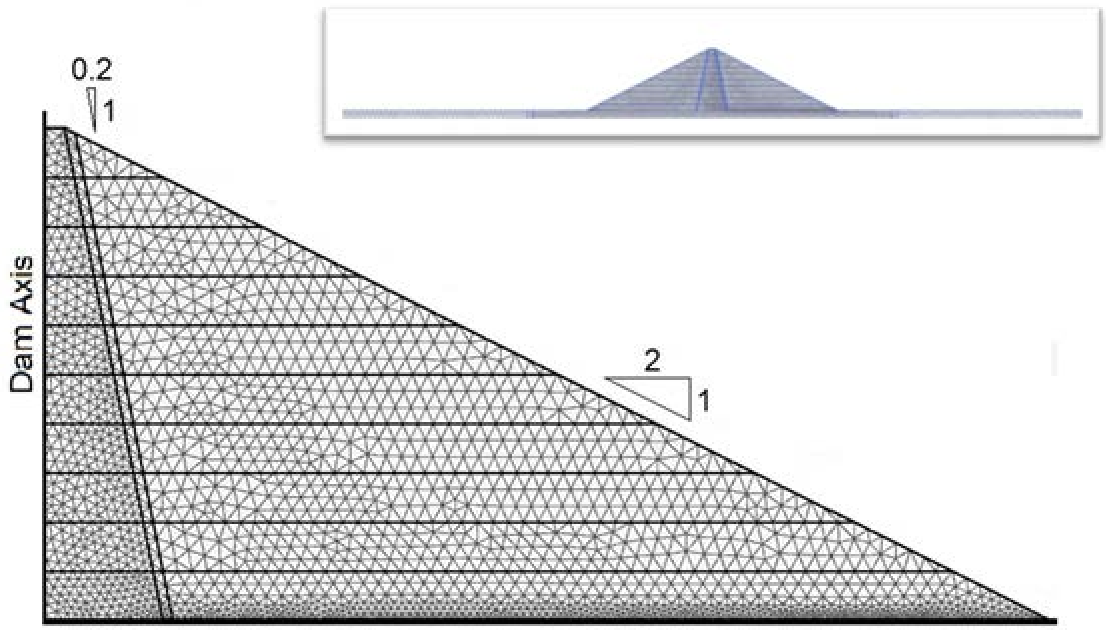

Figure 4: Finite element mesh

Despite the limited information pertaining to observed seismic-performance of earth dams during past earthquakes, Swaisgood (2003) had proposed an expression to estimate of the amount of crest settlement that would occur due to an assumed earthquake, characterized by its PGA and magnitude. Also, a relative degree of damage was associated to each percentage of crest settlement, reflecting that events with PGA greater than 0.5 g may induce serious damage, with settlements between 1 and 10 %. The results of the finite element analyses, presented in tables 6 and 7 agreed with these observations.

RESULTS AND DISCUSSION

The comparison of results between the obtained Newmark displacements and crest settlements clearly reflects the observations made by Bray (2007), as provided in figure 5. Simplified procedures can lead to significant overestimation and some level of underestimation for cases where the ground motion is an intense near-fault motion. Strains and pore pressures increments are neglected and the potential critical surface with the lowest static factor of safety is not always the critical for dynamic analysis. Even though, there are still in practice because they are simple and inexpensive.

In a first attempt to evaluate the relation between structure response EDP and the earthquake IM, the Pearson correlation coefficient (Canavos, 1984) was calculated to measure the strength of the linear association between the variables. The results are given in figure 6. Newmark displacement D shows a strong correlation to VSI, HI, IA and IC while the lowest correlation is observed with respect to the V/A ratio. As for the crest settlement C, the strongest correlation is also given for HI and VSI and SMV and CAV and the lowest for EDA.

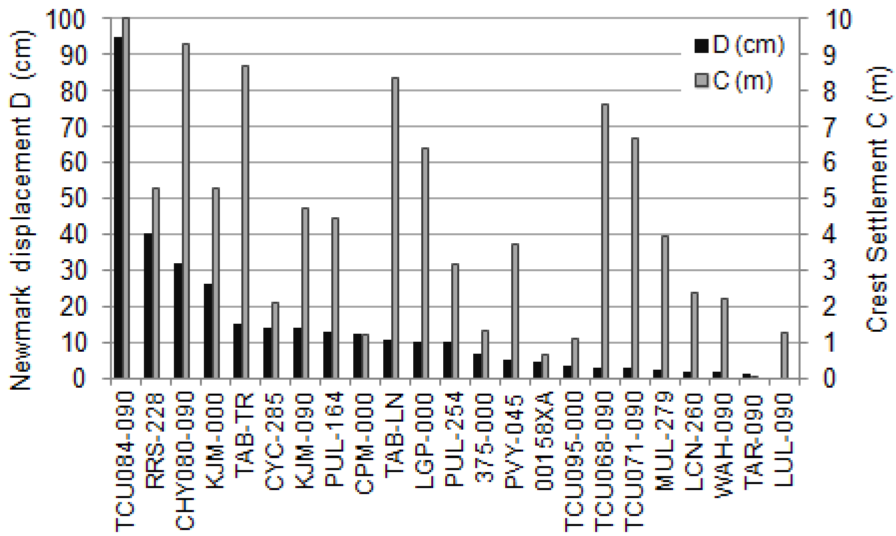

Figure 5: Newmark Displacement and Crest Settlement results for the selected ground motion records.

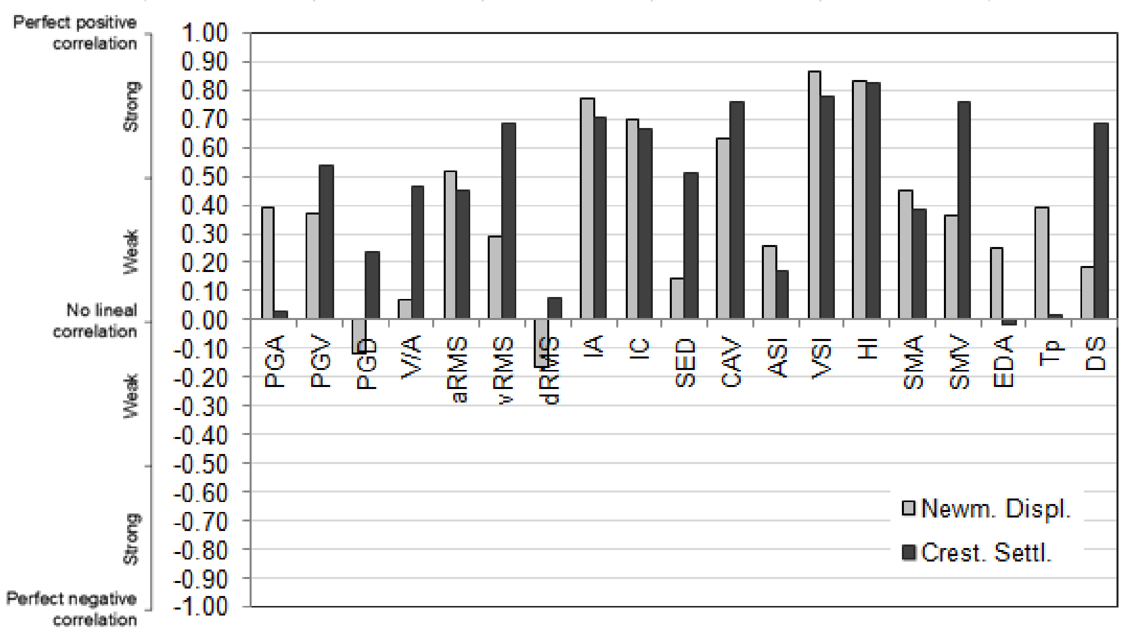

Figure 6: Pearson correlation coefficient between Newmark Displacement and Crest Settlement versus 19 candidates IM.

As proposed by Padgett (2008), the properties of the six IMs with strongest correlation with the EDP were evaluated to identify the optimal IM. Regression analyses of the logarithms of the IM and the response quantity in the following form were used:

$$\ln EDP = a + b \ln IM \tag{11}$$

where a and b are constants to be determined. The dispersion, $\beta_{EDP|IM}$, is a clear measure of efficiency. Considering N accelerograms,

$$\beta_{EDP|IM} \cong \sqrt{\frac{\sum\left[\ln(EDP) - \ln(a\, IM^{b})\right]^2}{N-2}} \tag{12}$$

The proficiency measure ξ is obtained as the ratio between $\beta_{EDP|IM}$ and the slope of the regression law b, the measure of practicality [6]:

$$\xi = \frac{\beta_{EDP|IM}}{b} \tag{13}$$

Tables 9 and 10 present the calculated coefficients for seismic response parameters respectively, Newmark displacement and crest settlement. Bold value indicates most practical, efficient, and proficient respectively. According to the selection criterion, the appropriate IMs would be VSI and HI in both cases.

Following, the IMs sufficiency condition was evaluated by performing a regression analysis on the standardized residuals from the previous analyses, relative to the earthquake magnitude and rupture distance in order to check if there was a correlation. Recall the standardized residual is the residual, i.e. the difference between the observed and predicted responses, divided by its standard deviation. An observation with a standardized residual that is larger than 2 (in absolute value) is generally deemed an outlier (Canavos, 1984). From the analyzed sets, the record from Whittier Narrows 1987 event TAR-090, was identified as an outlier and was thus not included in analysis.

A hypothesis test is useful to determine whether there is enough evidence in a sample of data to infer that a certain condition is true for the entire population.

The p-value is defined as the probability of rejecting the null hypothesis in an analysis of variance, where the null hypothesis states that the coefficient of regression is zero and it is a quantitative measure of sufficiency. An IM is assumed to be insufficient for p-values lower than 0.10 (Padgett, 2008). Tables 9 and 10 also present the p-values for the studied IMs. The six selected IMs are independent of the earthquake distance and magnitude for the simplified dynamic analysis. However, sufficiency condition is not met for the spectral parameters VSI and HI regarding crest settlement, but it is verified for SMV. Regression analyses for VSI and SMV are displayed in Figure 7.

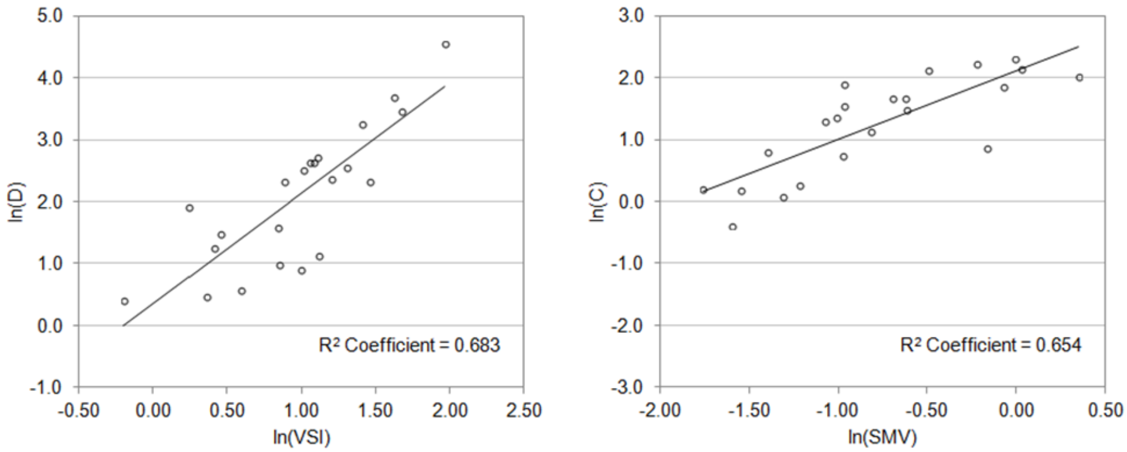

Figure 7: Newmark displacement and crest settlement regression analyses conditioned upon VSI and SMV respectively.

Table 9: Efficiency, practicality and proficiency comparisons for IMs with strong correlation with Newmark displacement

				p-value	
IM	$\beta_{EDP\|IM}$	b	ξ	M	R_{RUP}
a_{RMS}	0.96	**2.03**	**0.47**	0.49	0.48
IA	0.88	1.31	0.67	0.45	0.58
IC	0.88	1.77	0.50	0.78	0.66
CAV	1.05	1.03	1.01	0.59	0.15
VSI	**0.64**	**1.78**	**0.36**	0.38	0.89
HI	**0.79**	1.29	0.61	0.46	0.95

Table 10: Efficiency, practicality, proficiency and sufficiency comparisons for IMs with strong correlation with crest settlement

				p-value	
IM	$\beta_{EDP\|IM}$	b	ξ	M	R_{RUP}
v_{RMS}	0.57	0.98	0.59	0.33	0.58
IA	0.64	0.82	0.78	0.29	0.43
CAV	0.56	**1.26**	0.44	0.85	0.17
VSI	0.51	**1.34**	**0.38**	0.02	0.45
HI	**0.46**	1.22	**0.38**	0.04	0.59
SMV	**0.48**	1.10	0.44	0.68	0.67

Figure 8 shows a comparison of the sufficiency relative to earthquake magnitude and Newmark displacement and crest settlement given VSI and SMV, with p-values of 0.38 and 0.68 respectively.

Finally, hazard computability must be examined. As it refers to the effort to determine the hazard curves or maps, PGA is the most desirable IM because information is already available. Regarding VSI, it is also an IM with good predictability and there exists equations to estimate it directly (Bradley, 2009). Most recent equations allow HI prediction from the maximum absolute pseudo-velocity. Although SMV was found to be better than CAV representing the seismic response of the dam from advanced dynamic analyses, its estimation is rather uncertain, making it less desirable as ground motion intensity measure. On the other hand, exhaustive research has been done regarding CAV prediction (Campbell, 2012; Campbell, 2010)

The analyses have demonstrated that the most usual parameter, PGA is an inadequate IM, especially when nonlinear behavior is considered, as seen in figure 9.

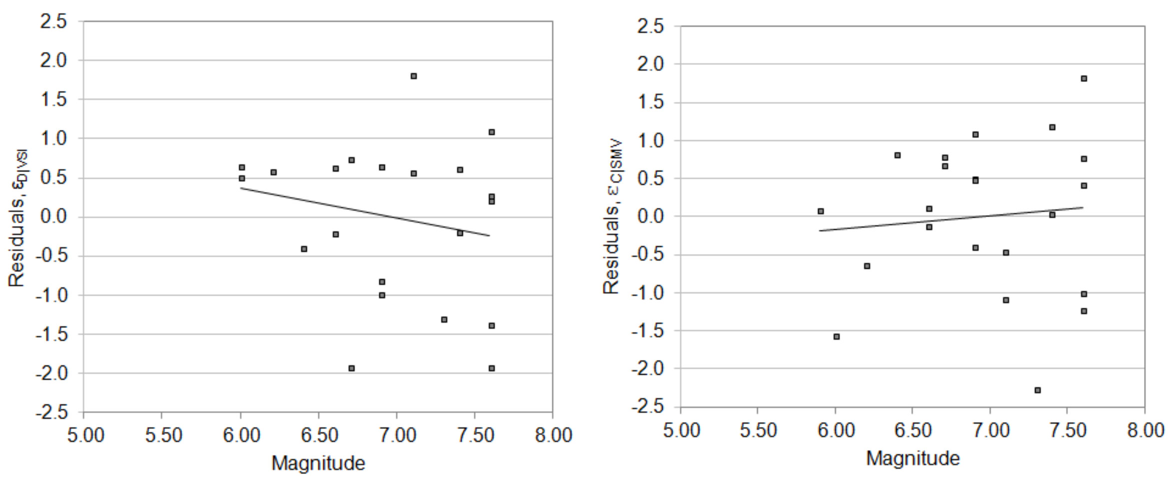

Figure 8: Sufficiency of two IMs (VSI, SMV) for Newmark displacement and crest settlement by evaluating magnitude independence

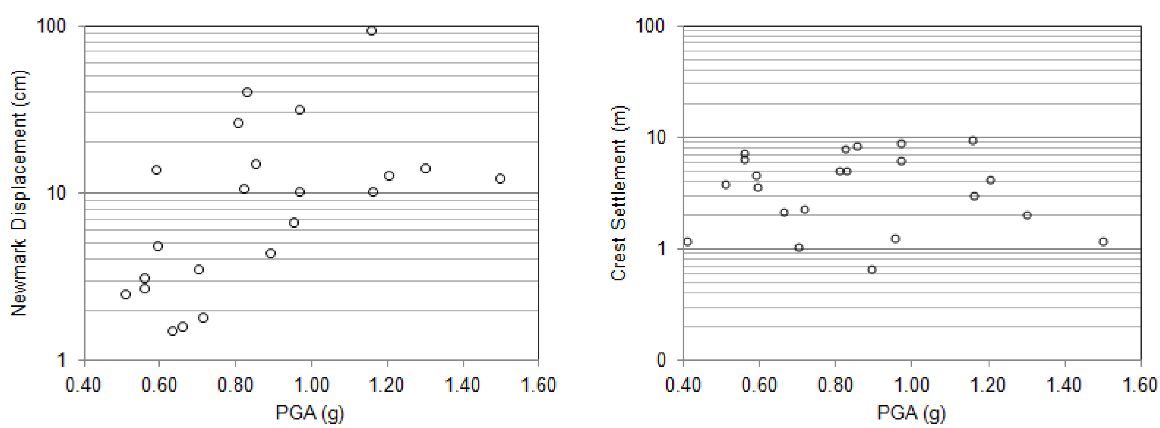

Figure 9: Scatter plots for the analyses results and PGA

For the analytical procedure the sliding mass is considered as a rigid plastic body and no permanent displacement occurs at accelerations below the yield value. Displacement is obtained directly by integration of the acceleration trace, and then, intensity parameters that are calculated from the ground motion record, such as aRMS, CAV, IA and IC, exhibit a strong correlation.

Velocity-related parameters, such as vRMS, CAV and SMV are effective indices predicting crest settlement. Even PGV is a better indicator of damage compared to PGA, as it is shown in figure 6. But, among these IMs, SMV is the most suitable because it captures the fact that repeated cycles of strong motion are required to induce permanent displacement. Geotechnical structures deformation is also associated with the duration of the event (Kramer, 1996), and correlation was found with a parameter representing this feature, DS, with a correlation coefficient of 0.69. The good correlation with IA could be attributed to the fact that this parameter is related to the energy content, including the effects of amplitude, frequency content and duration.

Spectral parameters, VSI and HI are competent indices at both levels of analysis, capturing effects of amplitude and frequency content in the range of the response of geotechnical structures. Although ASI is also a spectral parameter, the period range associated to this IM is 0.1-0.5 sec and so, the correlation with the earth dam response is weak. However, both parameters are insufficient with regard to the magnitude and crest settlement relation. Efficiency and sufficiency are the most desirable properties of and IM. If the IM is insufficient the results will be biased (Luco, 2007; Tothong, 2006).

According to these results, VSI and HI are optimal IMs for earth structures analyzed by Newmark method, whereas CAV and SMV are suitable when coupled dynamic analyses are performed.

CONCLUSIONS

The focus of this paper is on the selection of an appropriate IM for vulnerability and risk analyses of embankment dams. Simplified Newmark method and time-history nonlinear analysis were performed on a dam model using two sets of near-fault ground motion records. Although the comparison of methods is beyond the scope of this work, it can be concluded that simplified procedures can lead to significant underestimation of the potential damage of an earthquake. The comparison between the 19 IMs proposed was made on five properties: efficiency, practicality, proficiency, sufficiency and hazard computability, on the basis of the results of regression analyses.

The spectrum-based parameters with periods between 0.1 and 2.5 sec, namely VSI and HI are the most efficient IM, but sufficiency condition is not met for crest settlement with respect to the earthquake magnitude. Regarding Newmark displacement, intensity parameters obtained directly from the ground motion trace, such as aRMS, CAV, IA and IC has shown a good correlation and they are sufficient measures. Furthermore, aRMS is the most practical and proficient IM.

On the other hand, velocity-related indices correlate better with crest settlement than acceleration or displacement-related parameters. In fact, SMV is the most reliable IM but on the basis of the hazard computability, CAV is deemed superior.

The results are intended to provide guidance for the selection of optimal IM. As the study was limited to a specific case study of an earth-core rockfill dam, further investigation is needed to identify the influence of the variability of geometry and materials on the results. Moreover, a larger accelerogram database, including also far-field records, is needed to produce greater statistical validity.

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REFERENCES

1. Abrahamson NA (1996) Empirical models of site response effects. Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, pp. 23-28.
2. Barani S, Bazzurro P and Pelli F (2010) A probabilistic method for the prediction of earthquake-induced slope displacements. Proceedings of the fifth international conference on recent advances in geotechnical earthquake engineering and soil dynamics, San Diego, California, Paper No 4.31b.
3. Bishop AW (1955) The use of slip circle in the stability analysis of slopes. Geotechnique 5: pp.7–17.
4. Blume JA (1965) Earthquake ground motion and engineering procedures for important installations near active faults, Proceedings of the Third World Conference on Earthquake Engineering, Wellington, Vol. III: pp. 53-71.
5. Bommer JJ and Acevedo AB (2004) The use of real earthquake accelerograms as input to dynamic analysis. *Journal of Earthquake Engineering* 8: pp. 43-91.
6. Bommer JJ, Georgallides G and Tromans IJ (2001) Is there a near-field for small-to-moderate magnitude earthquakes?. *Journal of Earthquake Engineering*, 5(03): pp. 395-423.
7. Bradley BA, Cubrinovski M, Dhakal RP and MacRae GA (2009) Intensity measures for the seismic response of pile foundations. *Soil Dynamics and Earthquake Engineering* 29(6): pp.1046-1058.
8. Bray JD (2007) Simplified seismic slope displacement procedures. *Earthquake Geotechnical Engineering*, Springer Netherlands: pp. 327-353.
9. Brigante A and Sica S (2012) Seismic response of a zoned earth dam (case study). *Electronic Journal of Geotechnical Engineering*, 17(S): pp. 2495-2508.
10. Brinkgreve RBJ, Swolfs W and Engin E (2012) Plaxis 2D Materials Models Manual. Delft.
11. Bureau GJ (2003) Dams and Appurtenant Facilities in Earthquake Engineering Handbook. Chenh, WF and Scawthorn C,CRS press, Boca Raton.
12. Campbell KW (1981) Near-source attenuation of peak horizontal acceleration, Bulletin Seismic Society Am. 71: pp. 2039-2070.
13. Campbell KW and Bozorgnia Y (2010) Analysis of Cumulative Absolute Velocity (CAV) and JMA Instrumental Seismic Intensity (IJMA) using the PEER-NGA strong motion database. Berkeley: Pacific Earthquake Engineering Center.
14. Campbell KW and Bozorgnia Y (2012) Cumulative absolute velocity (CAV) and seismic intensity based on the PEER-NGA database. *Earthquake Spectra* 28(2): pp. 457-485.

15. Canavos GC (1984) Applied Probability and Statistical Methods. TBS The Book Service Ltd

16. Cornell CA (2004) Hazard, ground motions and probabilistic assessment for PBSD. Performance based seismic design concepts and implementation, PEER Report 5, pp. 39-52.

17. Deierlein GG, Krawinkler H and Cornell CA (2003) A framework for performance-based earthquake engineering. 2003 Pacific conference on earthquake engineering, Christchurch, New Zealand, pp. 1-8.

18. Elia G, Amorosi A, Chan AHC and Kavvadas M (2011) Fully coupled dynamic analysis of an earth dam. Geotechnique 61(7): pp. 549-563.

19. Fajfar P, Vidic T and Fischinger M (1990) A measure of earthquake motion capacity to damage medium-period structures. *Soil Dynamics Earthquake Engineering* 9(5): pp. 236–242.

20. Giovenale P, Cornell CA and Esteva L (2004) Comparing the adequacy of alternative ground motion intensity measures for the estimation of structural responses. *Earthquake engineering and structural dynamics* 33(8): pp. 951-979.

21. Hunter G and Fell R (2003) The deformation behaviour of embankment dams. Uniciv Report NºR-416. University of New South Wales, School of Civil and Environmental Engineering.

22. Husid LR (1969) Características de terremotos – análisis general. Revista del IDIEM 8. Santiago, Chile pp. 21-42.

23. ICOLD (2016) Selecting parameters for large dams—guidelines (revision of Bulletin 72). ICOLD Committee on Seismic Aspects of Large Dams, Bulletin, vol. 148.

24. Iervolino I and Manfredi G (2008) A review of ground motion record selection strategies for dynamic structural analysis. Modern Testing Techniques for Structural Systems. CISM Courses and Lectures Number 502, Springer Vienna; pp.131-163.

25. Jibson RW (1993) Predicting earthquake-induced landslide displacements using Newmark's sliding block analysis. Transportation research record.

26. Jibson RW (2011) Methods for assessing the stability of slopes during earthquakes—A retrospective. *Engineering Geology* 122: pp.43-50.

27. Jibson RW (2011). Methods for assessing the stability of slopes during earthquakes—A retrospective. *Engineering Geology* 122(1): pp. 43-50.

28. Jibson RW, Harp EL and Michael JA (2000) A method for producing digital probabilistic seismic landslide hazard maps. *Engineering Geology* 58: pp. 271–289.

29. Katsanos EI, Sextos AG and Manolis GD (2012) Selection of earthquake ground motion records: A state-of-the-art review from a structural engineering perspective. *Soil Dynamics and Earthquake Engineering* 30(4): pp. 157-169.

30. Keefer DK and Wilson RC (1989) Predicting earthquake-induced landslides, with emphasis on arid and semi-arid environments. In: Landslides in a semi-arid environment. *Inland Geological Survey Society* 2: pp. 118–149.

31. Kramer SL (1996) Geotechnical earthquake engineering. Pearson Education India.

32. Krinitzsky EL and Chang FK (1977) State-of-the-art for assessing earthquake hazards in the United States: specifying peak motions for design, Miscellaneous Paper S-73-1; Report 7, US Army Corps of Engineering Waterways Experiment Station, Vicksburg, Mississippi.

33. Luco N and Cornell CA (2007) Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions. *Earthquake Spectra* 23(2): pp.357-392.

34. Lysmer J and Kuhlemeyer RL (1969) Finite dynamic model for infinite media. Journal of Eng. Mech. 95(4): pp. 859–77.

35. Maniatakis CA, Taflampas IM and Spyrakos CC (2008) Identification of near-fault earthquake record characteristics. The 14th world conference on earthquake engineering Beijing, China; pp. 12-17.

36. Martinez-Pereira A and Bommer JJ (1998). What is near field? Seismic design practice into the next century, Balkema, Rotterdam, pp. 245-252.

37. Newmark NM (1965) Effects of earthquakes on dams and embankments, Fifth Rankine lecture. *Géotechnique* 15(2): pp. 139-160.

38. Padgett JE, Nielson BG and DesRoches R (2008) Selection of optimal intensity measures in probabilistic seismic demand models of highway bridge portfolios. *Earthquake Engineering and Structural Dynamics* 37(5): pp. 711-725.

39. Pitilakis K, Crowley H and Kaynia A (2014) SYNER-G: typology definition and fragility functions for physical elements at seismic risk. Geotechnical, Geological and Earthquake Engineering 27

40. PLAXIS 2D 2012 (2012) Computer software. Delft, Netherlands, Plaxis.

41. PLAXIS 2D 2012 (2012) Reference manual, Delft University of Technology and Plaxis bv The Netherlands.

42. Rathje EM and Bray JD (2000) Nonlinear coupled seismic sliding analysis of earth structures. *Journal of Geotechnical and Geoenvironmental Engineering* 126(11): pp. 1002-1014.

43. SeismoSoft. SeismoMatch (2012). A computer program for spectrum matching of earthquake records. SeismoSoft Ltd, Pavia, Italy.

44. Sica S, de Magistratis FS and Vinale F (2002) Seismic behaviour of geotechnical structures. *Annals of Geophysics* 45(6).

45. Swaisgood JR (2003). Predicting dam deformation caused by earthquakes - an update. ASDSO 2013 Dam Safety Conference, Rhode Island.

46. Ti KS, Huat BB, Noorzaei J, Jaafar MS and Sew GS (2009) A review of basic soil constitutive models for geotechnical application. *Electronic Journal of Geotechnical Engineering* 14: pp. 1-18.

47. Tothong P and Cornell CA (2006) Probabilistic seismic demand analysis using advanced ground motion intensity measures, attenuation relationships, and near-fault effects. PEER Report 11.

48. Travasarou T and Bray JD (2003) Optimal ground motion intensity measures for assessment of seismic slope displacements. 2003 Pacific Conference on Earthquake Engineering, Christchurch, New Zealand.

49. Visone C, Billota E and Santucci de Magistris F (2008) Remarks on site response analysis by using Plaxis dynamic module. Plaxis Bulletin, issue 23: pp.14-18.

50. Visone C, Santucci de Magistris F and Bilotta E (2010) Comparative study on frequency and time domain analyses for seismic site response. *Electronic Journal of Geotechnical Engineering* 15: pp. 1-20.

51. Wang Z, Dueñas-Osorio L and Padgett JE (2012) Optimal intensity measures for probabilistic seismic response analysis of bridges on liquefiable and non-liquefiable soils. Proceedings of 2012 Structures Congress, Reston, VA: ASCE, pp. 527-538.

52. Wieczorek GF, Wilson RC and Harp EL (1985) Map showing slope stability during earthquakes in San Mateo County, California. Miscellaneous Investigations Map I-1257-E. US Geological Survey.

53. Zhang W and Kezhong P (1997) Estimation on the duration of strong ground motion. Mitigating the Impact of Impending Earthquakes: Earthquake Prognostics Strategy Transferred into Practice, edited by Vogel, A. and Brandes, K., Balkema, pp. 149-156.

NOTATION

λ is the annual rate of exceedance of a specified threshold

G is a conditional probability.

Vs is the shear-wave velocity of the soil profile in the upper 30m.

g is the acceleration of gravity

α is the thrust angle

E_{50ref} is the plastic straining due to primary deviatoric loading

E_{oedref} is the plastic straining due to primary compression

E_{urref} is the Elastic unloading/reloading

m is the exponent for stress-level dependency of stiffness

DS is the duration of strong motion

t_1 is the lower limit of strong motion duration

t_2 is the upper limit of strong motion duration

t_{tot} is the complete duration of the acceleration record

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