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Earthquake resistant design of gravity-type and soil structures with friction response spectrum

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ABSTRACT: Based on energy of earthquakes, soil and structures, a new framework is presented for the earthquake resistant design of soil and gravity-type structures. The mass-rough-boundary model is used to idealize the energy absorption of these structures and soil both macroscopically and microscopically. The friction response spectrum is introduced to determine the design seismic coefficient from the power spectrum of the ground motion.

1 INTRODUCTION

In 1965, Newmark pointed out that the kinematic energy of ground motion is the most significant factor in determining the response of a structure such as an earth or rock-fill dam by idealizing the resistance with the Coulomb friction. He derived relations to predict the maximum displacement of these structures with the critical resistance against sliding normalized by the peak ground acceleration and the peak ground velocity. His approach was reinforced both experimentally and numerically by many researchers (e.g. Ambraseys and Sarma 1967; Sarma 1979; Makdisi and Seed 1977). Based on the slip displacement vs. normalized critical acceleration chart computed by Franklin and Chang (1977), Richards and Elms (1979) have proposed a formula to calculate the seismic coefficient from the peak acceleration and the allowable displacement for retaining walls.

On the other hand, out of a number of sliding block analyses for recorded strong motions a conclusion has been drawn that numerically this approach was not to improve the conventional choice of the seismic coefficient but to justify it (Seed 1979; Hynes-Griffin and Franklin, 1984). As long as the effect of the earthquakes are expressed by the peak acceleration, Newmark's approach appears only to support the rule of thumb that the seismic coefficient of 50-80% of the expected peak acceleration is appropriate.

Theoretically, Newmark's approach is the counter-part of the Housner's response spectrum. The Coulomb damping can be compared to the viscous damping for its simplicity and fitness to the actual energy absorption in the soil and gravity-type structure. Considering the energy of earthquakes, structures and the soil, a general framework is presented where the effect of earthquakes can be rationally quantified.

2 FRICTION RESPONSE SPECTRUM

The permanent deformation of a soil and gravity-type structure has been idealized by the slip displacement S(t) of a mass m along a rough linear boundary which moves with the effective acceleration $\ddot{B}(t)$ in both horizontal and vertical directions in the gravity field G. The rigid-plastic internal resistance is represented by the Coulomb friction between the mass and the boundary with frictional angle ϕ. This system has a pair of principal axes X, inclined by the angle ϕ from the boundary pointing left and right from the mass, where the equation of motion of the mass can be written as

$$m\frac{d^2(S_x(t)+B_x(t))}{dt^2} = -mA_c \qquad (1)$$

in which

$$A_c = G\sin(\phi \pm \theta) \qquad (2)$$

is the critical value of the effective acceleration against sliding; θ is the inclination of the boundary from the horizontal. Usually, the up-hill critical acceleration, corresponding to the positive sign, is so large that the mass can be regarded to slide only in the down-hill direction. We will refer to this as the one-sided case. Eq (2) relates the magnitude of the critical acceleration with the principal direction. Practically, the error of regarding the horizontal as the principal direction is about A_c/G or 10-30%.

The friction response spectrum of a group of ground motions $B_X(t)$ with duration T is defined as the expected value of the maximum sliding displacement $S_X(t)$ of the mass for a given critical acceleration A_c:

$$S(A_c) = E\left[\max(S_x(t)) \text{ for } B(t), 0 \le t \le T\right] \qquad (3)$$

By integrating Eq (1) for the absolute displacement $d(S(t)+B(t))$ from the initiation of sliding at t_1 until it stops at t_2;

$$m\overline{A}_c\Delta S + \frac{1}{2}\Delta\, m\dot{B}^2\left(1 - \frac{2\overline{A}_c\Delta B}{\Delta\dot{B}^2}\right) = 0 \qquad (4)$$

where $\overline{A_c}$ and Δ indicates the average and increment between $t=t_1$ and t_2, respectively. Formally, for $Y=A_c$, $X=S$, B and for $Y=1$, $X=\dot{B}$:

$$\overline{Y}\Delta X \equiv \int_{X(t_1)}^{X(t_2)} Y dX \qquad (5)$$

The relative velocity starts from zero at t_1, takes a positive value, and becomes zero again at t_2. The relative displacement of the mass with respect to the boundary is monotonic for a single slippage. For one-sided case where only the down-hill sliding is considered, the slip displacement accumulates monotonically. The maximum value is the residual value at t=T. If $\theta=0$ or the boundary is horizontal, the mass moves left and right. We refer this as the two-sided case; the total slip displacement can be obtained by summing ΔS for both principal directions.

The first term of Eq (4) is the work done to dislocate the mass on the rough boundary; the second is a product of the loss of the kinematic energy from the start to stop and a non-dimensional quantity of the critical acceleration normalized by the change of the velocity and displacement of the boundary. This can be summarized in the form:

$$A_cS=fK \qquad (6)$$

where A_c, S, and K stands for the representative value of the critical acceleration, slip displacement and the kinematic energy, respectively; f is a non-dimensional quantity to express what ratio of the kinematic energy is lost in dislocating the mass on the rough boundary. The critical acceleration of the mass-rough-boundary system expresses both the resistance and energy absorption of the mass for the slip displacement.

Eq (4) can be evaluated by assuming a correlational structure of the derivatives of the excitation. For a rectangular pulse with the peak acceleration A and peak velocity V, Newmark (1965) calculated

$$K=\frac{1}{2}V^2,\ f = 1 - \frac{A_c}{A} \qquad (7)$$

This is the simplest interpretation of Eq (4) for that the peak acceleration of the rectangular pulse A is equal to $V^2/2D$, peak velocity squared over the doubled peak displacement if the initial values are zero.

The energy content and the correlation structure of a process are expressed more generally by the Power Spectral Density Function which can be estimated by the Fourier amplitude spectrum:

$$S(\omega) = \frac{1}{2\pi s_0}\left|\int_0^{T_i} \ddot{B}(t)e^{-i\omega t}\, dt\right|^2 \qquad (8)$$

where s_0 is the denominator for the RMS values which may be determined so that the recorded peak value is expected to be observed once in the stationary Gaussian process with the same PSDF (Vanmarcke and Lai, 1980); T_i is the duration where the Fourier amplitude is calculated. Its shape and magnitude can be expressed by three series of parameters computed from the i'th central moment

$$\lambda_i = \int_{-\infty}^{+\infty} \omega^i S(\omega)\, d\omega \qquad (9)$$

as

$$\sigma_i = \sqrt{\lambda_{2i-4}},\ \omega_i = \frac{\sigma_{i+1}}{\sigma_i},\ \alpha_i = \frac{\sigma_{i+1}^2}{\sigma_i\sigma_{i+2}} \qquad (10)$$

which are RMS amplitudes, central frequencies and bandwidth indices of the i'th derivative of the displacement process, respectively. Assuming that the excitation is a segment of the stationary Gaussian process for duration s_0, RMS acceleration σ_2, central frequency ω_2 and the bandwidth parameter α_1, Igarashi (1986) obtained an analytical relation from the correlation theory of random process:

$$S(A_c) = \frac{s_0\sigma_2^2}{2\pi\omega_2 A_c}\exp\left(-\frac{1}{2}\left(\frac{A_c}{\sigma_2}\right)^2\right)\left(1+\frac{\pi}{2}\frac{\sqrt{1-\alpha_1^2}}{\alpha_1}\right) \qquad (11)$$

This can be transformed into Eq (6) by using relations in Eq (10). The kinematic energy is expressed by

$$K = \sigma_1^2 N_1 \qquad (12)$$

in which

$$N_1 = \frac{s_0\omega_1}{2\pi} \qquad (13)$$

is the effective number of cycles. And

$$f = \exp\left(-\frac{1}{2}\left(\frac{A_c}{\sigma_2}\right)^2\right)\left(\alpha_1+\frac{\pi}{2}\sqrt{1-\alpha_1^2}\right) \qquad (14)$$

As the critical acceleration increases from zero, the first exponential starts from unity and becomes practically zero for A_c greater than $3\sigma_2$. The second component is unity for the sinusoid ($\alpha_1=1.0$); $\pi/2=1.571$ for the white-noise ($\alpha_1=0.0$), reaching its maximum of 1.862 for $\alpha_1=0.537$. This is a general interpretation of the energy equation (4) of the mass-rough-boundary system, which shall be used in the proposed procedure.

3 CRITICAL ACCELERATION OF SOIL AND STRUCTURES

In most of the stability analyses and earth-pressure estimation for a soil and gravity-type structure, the rigid-plastic mechanism of the Coulomb friction is assumed for its potential sliding portion. The boundary of sliding is not always linear but curved and sometimes multiple. As the first order approximation, the sliding block model can be used to estimate the displacement of the sliding portion after the structure becomes unstable. In this case, the critical acceleration A_c of the model corresponds to

$$A_c = K_{hc}G \qquad (15)$$

where K_{hc} is the seismic coefficient to bring the structure just unstable in the analysis.

Microscopically, the sliding block model or mass-rough-boundary system idealizes the motion of a particle with Coulomb damping. The soil consisting the structure can be regarded as an assembly of many particles. During an earthquake, some particles may be locked by others and displace as a bulk. Other particles may dislocate from the others and may yield a permanent displacement or relative displacement to the bulk. For the dislocating particles, the other particles can be idealized by the rough boundary. In the differential space and time, the mass-rough-boundary analogy can be a reasonable physical model of the dislocation of particles with Coulomb damping. This consideration encourages us to apply the energy concept to the liquefaction of soil which is a major cause of the structural damage and has been left beyond the scope of the Newmark approach.

First of all, we shall make a physical assumption analogous to the Mohr-Coulomb static strength that the dislocation initiates when the shear stress of the bulk of soil reaches a critical value which is proportional to the initial effective confining stress σ_{m0}':

$$\tau_c = \sigma_{m0}' \tan \phi_c \qquad (16)$$

where ϕ_c is called the critical angle of dislocation. Critical acceleration of dislocation can be obtained from the stress distribution in the soil just before the dislocation initiates. For example, as for a soil at depth H of a free ground surface accelerated by uniform horizontal acceleration:

$$A_c = \frac{\sigma_{m0}'}{\rho H} \tan \phi_c \qquad (17)$$

From the start to just before the complete liquefaction, the pore-water can be regarded to be in the undrained condition, i.e. no flow occurs. The change in the internal energy is mostly attributed to that of the elastic potential E of pressure p:

$$E = \frac{1}{2} n C p^2 \qquad (18)$$

The increase of the pore-pressure from p_0 to p can be related with the external work done to the pore-water W through the elastic potential:

$$W = \Delta E = \frac{1}{2} n C (p^2 - p_0^2) \qquad (19)$$

where n and C is the volume ratio and the compressibility of the pore-water in the bulk of soil, respectively, which are assumed to be unchanged through the process. We assume that the work required to compress the pore-water is transmitted from the bulk to the water through the dislocating particles which displace along a total distance 2S relative to the bulk and pore-water with a steady acceleration Ac:

$$W = 2\eta\rho A_c S \qquad (20)$$

where η is a coefficient introduced to express the density of the dislocating particle effective to compress the pore-water; ρ is the bulk density.

4 ENERGY-BASED FACTOR OF SAFETY

The safety factors of gravity-type and soil structures have been defined by the ratio of the magnitudes of the resisting and driving forces to cause unstable states such as sliding, turning-over, up-lifting and liquefaction. Our approach takes the deformation of the structure after it looses the stability into account to determine the design value of the resistance. In other words, we will allow a structure to change its state within a limit. For this approach, we shall quantify the change of the state of the structure with that of the internal energy or the external work done to the structure. The safety margin of the structure against an earthquake for a limit state can be measured by a factor:

$$F_{le} = \frac{W_{l0}}{W_{e0}} \qquad (21)$$

in which subscript l, e and 0 denotes the limit, earthquake and the initial state, respectively. For example, W_{l0} is the work required to bring the structure to the limit state from the initial.

Provided the resistance of gravity-type and soil structures during the permanent deformation can be represented by the mass multiplied by the mean critical acceleration mAc, the sliding block analogy can be applyed to estimate the work done to deform the structure:

$$W_{v0} = \beta m A_c S_v, \quad v = l,e \qquad (22)$$

where coefficient β is introduced to relate the two sided-dislocation energy, i.e. the work done by the friction to the mass on the horizontal boundary, with one-sided case, for which our analytical expressions are derived. In Eq (20), we assumed β=2. Substituting external

works in Eq (22) for Eq (21) and cancel $\beta m A_c$, the energy-based factor of safety is expressed by the slip displacement of the limit-state S_l and of the earthquake-state S_e.

As for the liquefaction, the state of the soil can be evaluated by the pore-pressure and its elastic potential. When it liquefies, the pore-pressure p increases from the original value p_0 to the initial effective confining stress σ_{m0}':

$$p_l = \sigma_{m0}' + p_0 \tag{23}$$

The external work required to bring the pore-pressure to p_l is computed by Eq (19). The work done by the ground motion is calculated by substituting the analytical expressions of the dislocation energy for $A_c S$ in Eq (20). For example, provided the critical acceleration A_c of the soil at depth H from a free ground surface can be expressed by Eq (17), the factor of safety against liquefaction is evaluated as the ratio of these works (Eq (21)):

$$F_{le} = \frac{nC_e\sigma_{m0}'(\sigma_{m0}'+2p_0)}{4\rho\sigma_1^2 N_1\left(\alpha_1+\frac{\pi}{2}\sqrt{1-\alpha_1^2}\right)} \exp\left(\frac{1}{2}\left(\frac{\sigma_{m0}'}{\rho H\sigma_2}\tan\phi_c\right)^2\right) \tag{24}$$

in which the coefficient η in Eq (20) is replaced by what shall be called the effective compressibility of the pore-water:

$$C_e = \frac{C}{\eta} \tag{25}$$

In Eq (24), the effect of a generic excitation is expressed by σ_1, σ_2, α_1, N_1. The strength of the soil is by n, σ_{m0}', p_0, ρ, C_e and ϕ_c. The last couple are introduced to quantify the resistance for liquefaction based on the mass-rough-boundary model. They can be determined in many ways from field and experimental data of liquefaction. For example, considering stress distribution in the specimen and putting F_{le} and α_1 as unity, an analytical form of the stress ratio vs. number of cycles for liquefaction similar to Eq (24) can be obtained. Comparing this with the undrained cyclic shearing test result, C_e and ϕ_c are evaluated.

5 GENERAL FRAMEWORK

The following is the general framework of the earthquake resistant design of gravity-type and soil structures with the friction response spectrum:

1. Determine the target earthquakes according to the seismicity of the site, for example by source parameters, distance, direction. Estimate the power spectral density function of the base-rock acceleration of the site.

2. Predict the power spectral density function of the effective acceleration of the structure in consideration to the amplification of the surface layers and the structure. Designate the effective ground motion by four of the energy-related parameters: strong motion duration, RMS amplitude, central frequency, and bandwidth index (s_0, σ_2, ω_2, α_1) or (N_1, σ_2, σ_1, α_1).

3. Specify the limit state considering the functional and safety requirements of the structure and the effect on the neighboring structures. If it concerns with the deformation of the structure, convert it in terms of the mean slip displacement S_l. Choose a factor of safety F_{le} that secures acceptable safety margin against the limit state and determine the design value of the mean slip displacement S_d;

$$S_l = aF_{le}S_d \tag{26}$$

in which 'a' is a factor required to convert the extreme value S_l into the mean.

4. In case that the soil is susceptible to liquefaction, compute the factor of safety against liquefaction F_{le} by Eq (24). If the safety margin is unsatisfactory, improve the soil. If the strategy against liquefaction is to support the excess pore-pressure and deteriorated soil by structural counter measures, estimate the pore-pressure from the F_{le} and use reduced material strength and increased load in the stability analyses.

5. Find the design value of the critical acceleration A_{cd} from the friction response spectrum $S(A_{cd})$ corresponding to S_d.

$$A_{cd} = S^{-1}(S_d) \tag{27}$$

6. Determine the appropriate value of the seismic coefficient K_d from the critical acceleration A_{cd};

$$K_d = b\frac{A_{cd}}{G} \tag{28}$$

where b is a factor allowing for the uncertainty in the stability analysis procedure. The seismic coefficient can be applied in the horizontal or in the principal direction which is inclined upwards by the angle

$$\alpha = \sin^{-1}\frac{A_{cd}}{G} \tag{29}$$

6 SAMPLE SPECTRA

To check the assumptions for the analytical expressions numerically, slip displacements are computed for the 140 sets of US accelerograms selected by McGire and Barnhard (1977) from the Caltech file and 52 sets of the Japanese in the NOAA (1981) file; two dimensional excitation is synthesized from both the vertical and horizontal components. For each set, 20 slip displacements are calculated for A_c/A increasing by 0.05 from -0.5 to 0.5. Three definitions of the durations T_i in Eq (8) are tested to calculate the Fourier amplitude spectrum, among which the strong motion parameters evaluated in Eqs (8) through (10) between the first and

last excursion of the ±0.5A of the horizontal component are found to explain the slip displacement best in the multivariable log-linear regression analyses; a few data which are less than 0.01cm are set to 0.01cm to keep the logarithm in a finite range. For the US records:

$$S = 0.214\,\frac{s_0^{1.122}\sigma_2^{2.405}}{A_c^{1.350}\omega_2^{1.118}}\exp\left(-0.886\left(\frac{A_c}{\sigma_2}\right)^2\right)F^{0.424} \quad (30)$$

where

$$F = 1+\frac{\pi}{2}\,\frac{\sqrt{1-\alpha_1^2}}{\alpha_1} \quad (31)$$

with the standard error of log-estimate 0.535 and the coefficient of determination 0.915. For the Japanese accelerograms:

$$S = 1.239\,\frac{s_0^{0.999}\sigma_2^{2.237}}{A_c^{1.356}\omega_2^{1.477}}\exp\left(-0.792\left(\frac{A_c}{\sigma_2}\right)^2\right)F^{0.535} \quad (32)$$

with standard error of log-estimate 0.664 and R-square 0.889. The Newmark type regression equation for the US records is

$$S = 0.262\frac{V^{1.85}}{A^{1.03}}\left(\frac{A_c}{A}\right)^{-1.90} \quad (33)$$

with the standard error 0.909 and R-square 0.770. For the Japanese

$$S = 0.315\frac{V^{1.60}}{A^{0.88}}\left(\frac{A_c}{A}\right)^{-1.97} \quad (34)$$

with standard error 0.969 and R-square 0.755.

The forms of the regression equations are determined according to the analytical solutions in Eqs (7) and (11); the exponents obtained from the log-linear regressions are close to the analytical ones for both US and Japanese records. The new regression Eqs (30) and (32) are found to explain the slip displacement with almost half of the standard error of the Newmark type. Fig. 1 compares the moving average of the slip displacements normalized by $\omega_2/\sigma_2 s_0$ with the analytical one computed for the Japanese data set. The analytical mean overshoots the data for smaller slip displacements, which may be attributed to the difference of the coefficient 0.792 from the analytical 0.5 in the exponential term. Judging from this plot and regressions, the analytical formula can be used as a reasonable estimation of the mean slip displacement.

Let us illustrate our procedure by the strong motion parameters listed in table 1 computed for the Kawagishi-chyo record of the 1964 Niigata earthquake and the mean value of the 52 sets of the Japanese records. Assume that by steps 1 and 2 of the general framework presented in chapter 5, the strong motion parameters are determined coinciding with the Japanese mean in Table 1; the limiting displacement is chosen as S_l=50 cm with a factor of safety 1.2 for a retaining wall in Japan. Then,

Table 1 Strong Motion parameters of Kawagishi-chyo Record(1964/06/16 13:01) and the Mean and Standard Deviation of the 52 sets of the Japanease Records

Component	Unit	NS	EW	Mean	S.D.
Magnitude	JMA	7.5		6.71	0.90
Focal Depth	km	40		35.1	18.3
Epicentral Distance	km	59		86.3	79.5
Record Duration	sec	34.02	33.92	56.58	35.72
Record Peak A	m/s^2	1.319	1.619	1.423	0.815
Corrected Peak A	m/s^2	1.289	1.709	1.765	1.014
Corrected Peak V	m/s	0.456	0.547	0.160	0.134
σ_2	m/s^2	0.467	0.810	0.823	0.716
s_0	sec	6.850	1.790	7.815	11.49
ω_2	rad/s	18.65	18.94	30.78	11.90
α_1		0.161	0.095	0.401	0.139
σ_1	m/s	0.156	0.448	0.092	0.093
ω_1	rad/s	3.00	1.81	12.59	7.49
N_1		3.27	0.52	17.58	44.18
Kinematic Energy	J/m^3	150.7	196.5	132.0	284.8

Note: ρ=1900kg/m^3 is assumed for computing kinematic energy

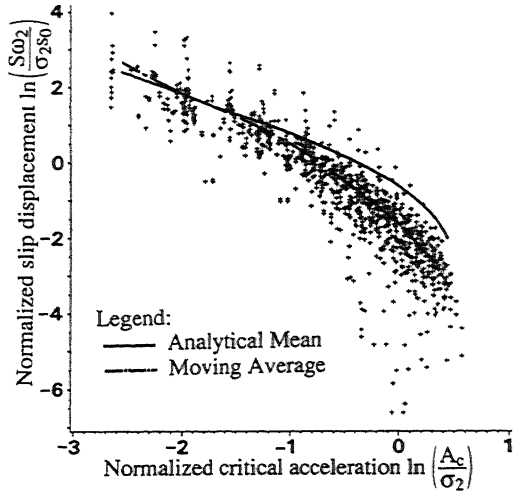

Fig. 1 Analytical mean vs. moving average of normalized slip displacement

the factor a in Eq (26) can be chosen as exp(3x0.664)=7.3 for the 3-σ limit and the design mean slip displacement is calculated as 5.7 cm. From the friction response spectrum obtained by substituting the strong motion parameters in Eq (11), the design value of the critical acceleration can be found to be A_{cd}=100 cm/sec^2 (Fig 2). If we choose the factor b to allow for the uncertainty of the succeeding structural calculations as 1.0, the design seismic coefficient is determined to be K_d=0.102. Similarly if the limiting displacement is selected as 5cm for an important gravity-type structure, K_d is calculated as 0.18. Incidentally, these values are

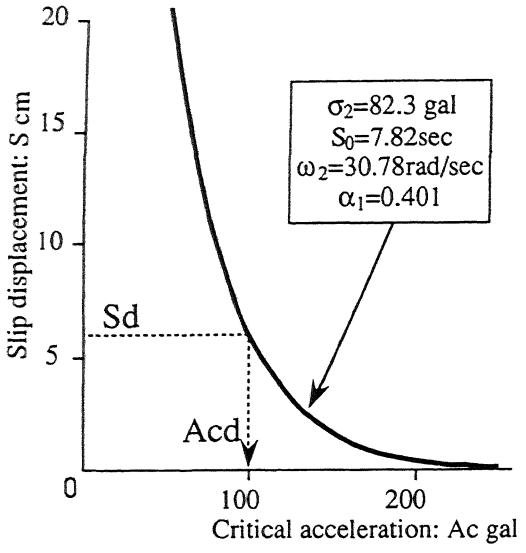

Fig. 2 Friction Response Spectrum

Table 2. Soil conditions and F_{le} of Kawagishi-chyo

Depth	-8	m
Water Table	-2	m
Porosity n	0.469	
Effective Confining Pressure σ_{m0}'	40.4	KPa
Initial Absolute Pore-Pressure p_0	160.1	KPa
Earth Pressure Coefficient K_0	0.5	
Bulk Density ρ	1900	kg/m^3
Effective Bulk Density ρ'	1000	kg/m^3
Relative Density Dr	45	%
Stress Ratio for Liquefaction	0.15	Rl(5)
Stress Ratio for Liquefaction	0.11	Rl(20)
Dislocation Angle ϕ_c	11.9	deg
Effective Compressibility C_e	2.69E-8	Pa^{-1}
Critical Acceleration Ac	0.560	m/s^2
F_{le} of Liquefaction (NS)	0.59	
F_{le} of Liquefaction (EW)	0.21	

similar to those in the current Japanese codes. proposed method is illustrated to produce reasonable seismic coefficients for a typical strong motion observed in Japan.

JSSMFE (1989) reports soil conditions surrounding the Kawagishi-chyo building in the basement of which the accelerograms in Table 1 were recorded while the sever liquefaction in the 1964-Niigata earthquake. The pertinent parameters to calculate the factor of safety against liquefaction by Eq (24) are borrowed from the report and tabulated in Table 2. The effective compressibility and the critical angle of dislocation are calculated from the stress ratios in the procedure explained in Chapter 4. Critical Acceleration is computed by Eq (17). The safety factors are computed about 0.4 with Eq (24), which indicate that the ground motion in Table 1 can supply more than twice as much energy to liquefy the soil in Table 2. This corresponds to the field observation and illustrates that our procedure can quantify the strength of soil and the effect of earthquakes in the physical scale.

7 CONCLUSIONS AND ACKNOWLEDGEMENTS

1. The earthquake resistant design of soil and gravity-type structures can be rationalized by using the friction response spectrum to determine the seismic coefficient from design earthquake and the design value of the limiting displacement with a factor of safety.
2. Conditions of the design earthquakes can be stated more precisely with RMS values rather than their peak values.
3. Consistent values of design seismic coefficients can be assigned for different types of structures in a specific region, using (elastic) response spectrum and friction response spectrum if the design earthquake is given by its power spectrum.
4. The safety margin against liquefaction can be evaluated by the friction response spectrum of earthquakes and the effective compressibility and critical angle of dislocation of the soil.

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