Reliability-based design for earth-fills against severe natural hazard events

Analyse de la fiabilité des barrages en terre face aux catastrophes naturelles

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ABSTRACT: A reliability analysis of the embankments is conducted considering the variability of the internal friction angle and the seismic hazard in this study. Also, the probability of overflow of earth-fills during heavy rains is evaluated. The rainfall intensity is dealt with as a probabilistic parameter, and the various rainfall patterns are tested by the Monte Carlo method. Finally, the total risk due to the earthquakes and the heavy rains is evaluated for an earth-fill site. As the result of this study, the possibility for the practical use of the proposed method for the plans of the maintenance of deteriorated earth-fill dams, has been verified. Especially, the results can be used effectively to evaluate the priority for the earth-fills to be improved.

RÉSUMÉ : Dans cette étude, une analyse de la fiabilité des barrages et levées en terre a été réalisée en prenant en compte la dispersion de l'angle de frottement interne et le risque sismique. La probabilité de débordement des levées en cas de fortes pluies a été évaluée. Dans cette méthode, l'intensité des précipitations est traitée en tant que variable aléatoire, et les différents schémas de précipitations sont testés par la méthode de Montecarlo. Le risque global lié aux séismes et aux fortes pluies sur un site de barrage ou levée en terre a été évalué. Les résultats obtenus montrent qu'il est possible d'utiliser la méthode proposée pour la définition des plans de maintenance des ouvrages en terre vieillissants. La méthode peut être utilisée pour évaluer le degré de priorité pour les barrages et levées en terre devant être renforcés.

KEYWORDS: risk evaluation, earth-fill, damage probability.

1 INTRODUCTION

There are many earth-fill dams for farm ponds in Japan. Some of them are getting old and decrepit, and therefore, have weakened. To mitigate disasters due to earthquakes and heavy rains, improvement works are conducted on the most decrepit earth-fill dams. This research deals with a strategy for the maintenance of geotechnical structures such as earth-fill dams. Since there is a recent demand for low-cost improvements, the objective of this research is the development of a design method for optimum improvement works at a low cost. A reliability analysis is introduced here in response to this demand.

Although the standard penetration test (SPT) N-values are frequently used to determine the soil parameters, the Swedish Weight Sounding (SWS) test is employed as a simpler method. Firstly, the statistical model of N-values is determined from the SWS results, and based on the model, the shear strength parameters are derived through the empirical relationships. Secondly, a reliability analysis of the embankments is conducted considering the variability of the internal friction angle and the seismic hazard. Thirdly, the probability of overflow of earth fills during heavy rains is evaluated. The rainfall intensity is dealt with as a probabilistic parameter, and the various rainfall patterns are tested by the Monte Carlo method. Finally, the total risk due to the earthquakes and the heavy rains is evaluated for an earth-fill site. In results, the possibility for the practical use of the proposed method for the plans of the maintenance of decrepit earth-fill dams, has been verified.

2 EARTHQUAKE HAZARD AND SAMPLE WAVE

The earthquake hazard curve is defined as the relationship between the maximum acceleration of an earthquake and the probability of exceedance during the specific period. In this study, the hazard data surrounding the Site H, which is the object of the seismic analysis, have been obtained from the Japan Seismic Hazard Information Station (J-SHIS) (National Research Institute for Earth Science and Disaster Prevention). The hazard curve corresponding to the maximum acceleration *a*, over the next 50 years, is depicted in Figure 1, which gives the probability distribution of the maximum acceleration. For the sequential value of the acceleration, the hazard function $H(a)$ is determined. As an input seismic wave, the representative wave of Nankai Trough Earthquake, is used in this study; the wave is shown in Figure 2.

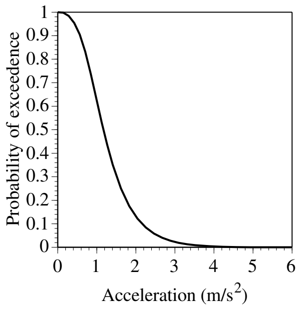


Figure 1. Earthquake hazard function at Site H over next 50 years.

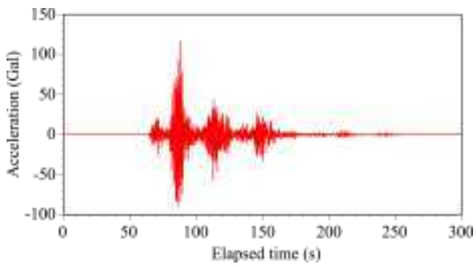


Figure 2. Sample wave due to Nankai Trough Earthquake at Site H.

3 RELIAIBILITY ANALYSIS OF A FILL-EMBANKMENT

3 .1 *Statistical model of an embankment*

A stability analysis is conducted and the risk is evaluated for an earth-fill dam at Site H. The statistical model of N-values derived from SWS is presented here. As the mean function of N-value, Equation (1) is derived, while the covariance function is defined as Equation (2), in which y (m) is the horizontal axis along the transverse section of the embankment , z (m) is the depth and h (m) is the elevation.

$$m = 1.89 + 0.157z \tag{1}$$

$$C_{ij} = 0.604 \cdot (1.24)^2 \exp\left(-\frac{|y_i - y_j|}{1.64} - \frac{h_i - h_j}{0.63} \right) \quad (i \neq j)$$
$$C_{ij} = (1.24)^2 \quad (i = j) \tag{2}$$

The analytical sections of the original embankment and the improved (restored) embankment are exhibited in Figures 3 (a) and (b), respectively. The embankment has been improved by constructing an inclined core, and by covering the original embankment with additional soil for reinforcement. The material properties are given in Table 1. The soil parameters are determined from the SPT N-values and laboratory soil tests. *Bs* means the embankment material; it is determined from the N-values based on the Swedish Wight Sounding (SWS) results instead of the SPT in order to consider the spatial distribution. The effective internal friction angle, $\phi' = \phi_d$, is obtained from the conversion, namely, Equations (3) and (4) (Hatanaka and Uchida 1996). In Equation (3), $3.0\varepsilon_f$ is the conversion error, in which ε_f is an $N(0,1)$ type normal random variable and the ratio of 3.0 is the standard deviation.

$$\phi' = (20N_1)^{0.5} + 20 + 3.0\varepsilon_f \tag{3}$$

$$N_1 = N_{SPT} / (\sigma_v'/98)^{0.5} \tag{4}$$

in which σ_v' is the effective vertical stress.

3 .2 *Reliability analysis*

The circular slip surface (CSS) method is employed as the stability analysis in this study. Random numbers are assigned for uncertain factors, and the stability of the embankments is evaluated as the probability of failure with the use of the Monte Carlo method. For the reliability analysis, Equation (5) is defined as a performance function, in which the internal friction angle is a probabilistic parameter.

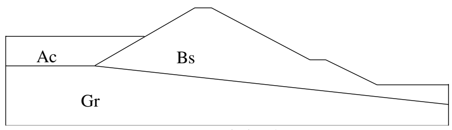
$$g = \sum_{i=1}^{n} (\tau_{fi} - \tau_{si}) l_i \tag{5}$$

where τ_f and τ_s are the shear strength and the shear stress, respectively, on the slip surface exhibited in Figure 4, which shows a slip surface across a finite element. In the figure, l_i is the length of the slip surface of element i, and n is the number of elements, which a slip circle crosses. The strength, τ_f is defined by the Mohr-Coulomb's law. Normal stress σ_n and shear force τ_s are defined in Figure 4, and calculated with the dynamic finite element method (LIQCA) (Uzuoka, *et al.* 2007) in this study. In the seismic response analysis, the linear-elastic model is employed for simplicity. The probability of failure is evaluated with Equation (6) through the use of the Monte Carlo method. The probability is also a function of the acceleration a.

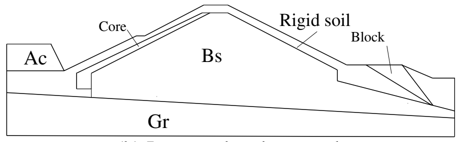
$$F_e(a) = \text{Probability } (g<0) \tag{6}$$

Table 1. Parameters of embankment materials.

Materials	Young's modulas (kN/m^2)	Cohesion (kN/m^2)	Internal friction angle (°)	Unit weight (kN/m^3)	Permeability (m/s)	Poisson's ratio
Bs (Sat)	12,350	0	*	20.9	8.55×10^{-6}	0.3
Bs (Unsat)	12,350	0	*	19.8	8.55×10^{-6}	0.3
Ac	1,000	0	37.4	15.0	2.71×10^{-8}	0.2
Core	16,800	0	37.4	20.9	9.95×10^{-8}	0.2
Rigid soil	16,800	0	37.4	20.9	8.55×10^{-6}	0.3
Block	16,800	200	50.0	23.0	8.55×10^{-6}	0.3
Gr	25,000,000	200	50.0	23.0	6.06×10^{-6}	0.3



(a) Original.



(b) Improved and restored.

Figure 3. Analytical sections of earth-fills.

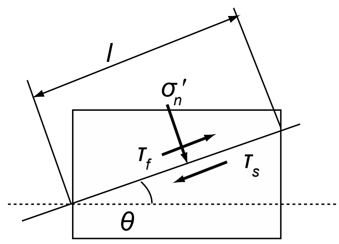


Figure 4. Slip surface across an element.

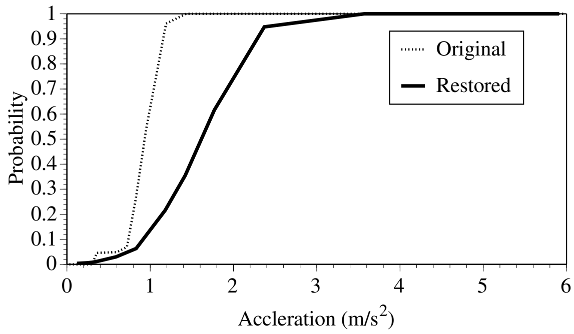


Figure 5. Fragilities of earth-fills.

For the internal friction angle ϕ' of the embankment material, B_s is dealt with as a random variable. Firstly, the random numbers considering the spatial distribution derived from Equations (1) and (2) are assigned to the NSWS. Secondly, the random variable N_{SPT} is evaluated by $N_{SPT} = N_{SWS}$ $(1+0.354\varepsilon_r)$ by considering the conversion error ε_r (Nishimura, *et al.* 2015), and then the ϕ' is obtained with Equation (3), including the conversion error term $3.0\varepsilon_f$. The Monte Carlo method is iterated 10,000 times.

In the calculation of the probability of failure $F_e(a)$, corresponding to maximum acceleration a, the level of the wave in Figure 2 is adjusted so that the maximum acceleration coincides with the target acceleration value. The fragility curves are obtained for the original and the restored embankments, respectively, as shown in Figure 5. For the original section, the probability $F_e(a)$ reaches to 100% at the acceleration of 1.5 (m/s^2), while, for the restored section, the probability approaches to 100% at surround of the acceleration of 3.0 (m/s^2). Considering the hazard function $H(a)$, the damage probability due to earthquakes over the next 50 years, P_{fe50} is defined as:

$$P_{fe50} = -\int_0^\infty \frac{dH(a)}{da} F_e(a) da \tag{7}$$

4 QUASI-RAINFALL MODEL

In this research, the rainfall events continuing for 72 hours are simulated based on the annual maximum rainfall intensities obtained from the rainfall data records in Okayama City, Japan for a span of 44 years. A dam break almost happens within 24 hours on an empirical basis. To cover all cases, the longer consecutive rainfalls for 72 hours are used. The cumulative distribution function $G_k(x)$ of rainfall intensity x (mm/h), for k (=1~72) hours after the rain starts, is determined with the mean rank method as follows:

$$G_k(x) = m_k(x)/(N+1) \tag{8}$$

where $m_k(x)$ is the number of rainfall intensities after k hours that do not exceed x, and N denotes the number of years. Then, rainfall intensity x (mm/h) is transformed into random variable y following the standard normal distribution as

$$y = \Phi^{-1}(G_k(x)) \tag{9}$$

where Φ is the standard normal distribution function. Then, the correlation coefficients ρ_{ij} (i,j = 1, 2, …, 72) between probabilistic variable y after i hours and j hours can be estimated by the following equation.

$$\rho_{ij} = \sum_{i=1}^{44} (y_i - \mu_i)(y_j - \mu_j) \Big/ \sigma_i \sigma_j \tag{10}$$

Here, the set of correlation coefficients is viewed as a matrix, namely, $\mathbf{R}=[\rho_{ij}]$. Since $\mathbf{R}$ is positive definite, the lower triangular matrix $\mathbf{L}$, satisfied with $\mathbf{LL}^T=\mathbf{R}$, is obtained by the Cholesky decomposition. A normal random number $\mathbf{Y}$ can be produced as follows:

$$\mathbf{Y} = \mathbf{Lz} \tag{11}$$

where $\mathbf{z}$ is standard normal random number generated by using Box-Muller method (Rubinstein 1981). Then, the normal random number $\mathbf{Y}$ is transformed into the random number X which has the distribution same as the actual rainfall using the following equation:

$$X_i = G_k^{-1}(\Phi(Y_i)) \tag{12}$$

If X is used directly as the quasi-rainfall, the pattern causing overflows, may be fixed, because the cases of the heavy rains are very limited. To prevent it, a method that the rainfall intensity is reduced or extended, keeping the shape of hyetograph is proposed. The Gumbel distribution is assumed for the distribution of the total rainfall T (mm/72hours) of the annual maximum 72 hours rainfalls, $f_G(T)$ is determined as follows;

$$T = \sum_{i=1}^{72} x_i \tag{13}$$

$$F(T) = \exp(-e^{-w}) \tag{14}$$

$$w = a(T - T_0) \tag{15}$$

where x_i is the intensity of the annual maximum 72 hours rainfalls after i hours, and a and T_0 are the parameters employed to adjust the observed data to the theoretical distribution function (Iwai and Ishiguro 1970). Then, the random variable T' is generated as total rainfall of the from Gumbel distribution $F(T)$ shown in Figure 6.

Lastly, the adjusted rainfall intensity, X' (mm/h) is determined for each hour as following equation.

$$X' = (T / \sum_{i=1}^{72} x_i) X \tag{16}$$

An example of a generated 72 hours rainfall series is exhibited in Figure 7.

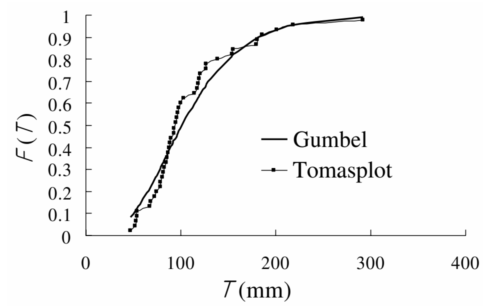


Figure 6. Cumulative distribution of maximum annual continuous precipitation.

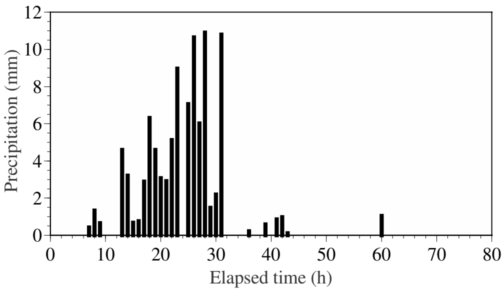


Figure 7. An example of quasi- precipitation.

Table 2. Outline of water reservoir and probability of overflow.

	A (km^2)	$C\times B_s$	A_w(m^2)	h_d(m)	P_{fo}(%)
Original	0.126	1.72	4000	0.66	0.53
Restored	0.126	6.70	4000	0.60	0

5 EVALUATION METHOD FOR THE PROBABILITY OF OVERFLOW

At first, the quantity of inflow, discharge and storage are calculated. The inflow equation is defined as follows (JSIDRE 2002);

$$Q_{in} = f_p r A/3.6 \tag{17}$$

where Q_{in} is inflow to the reservoir (m^3/s), f_p is the peak runoff coefficient, r is the quasi- rainfall intensity (mm/h) and A is the area of the basin (km^2). Uniform random numbers are used for f_p in the 0.7 to 0.8 range (JSIDRE 2002). The discharge equation for a rectangular weir as used in this study is;

$$Q_{out} = CB_S h^{2/3} \tag{18}$$

where Q_{out} is discharge (m^3/s), C is discharge coefficient, B_S is width of spillway, and h is static or piezometric head on a weir referred to the weir crest. The storage of water in the water reservoir V_r over the full water level is estimated as follows;

$$V_r = A_w h \tag{19}$$

Table 3. Damage probabilities and risks over next 50 years.

	Original	Restored
Damage probability for earthquakes	0.6882	0.4294
Damage probability for overflow	0.2337	0.0000
Total damage probability	0.7611	0.4294
Total risk (1,000 JPY)	149,517	84,357

where A_w is area of water reservoir (km^2) and h is overflow head (m). The decreasing rate of the storage V with the runoff is:

$$dV_r / dt = Q_{in} - Q_{out} \tag{20}$$

The overflow head h is determined from Equation (20), and the maximum h within the 72 hours, is defined as the peak overflow head on the spillway. When h_p becomes greater than the design overflow head h_d, the overflow occurs. Then, the probability of overflow is defined by Equation (21) as the times of h_p < h_d in the iterations of the Monte Carlo simulation (Rubinstein 1981).

$$P_{of} = \mathrm{Prob}[h_d < h_p] \tag{21}$$

The result for a dam in Site H is described in Table 2. The probability of overflow in a year is evaluated as 0.53% for the original embankment and 0% for the improved one. From the value, the probability of overflow over the next 50 years can be estimated by Equation (22).

$$P_{fo50} = 1 - (1 - P_{fo})^{50} \tag{22}$$

The probability of overflow over the next 50 years at the Site H is 23.4% for the original embankment and 0% for the improved embankment.

6 TOTAL RISK EVALUATION

In Table 3, the damage probability over next 50 years for the earthquakes and heavy rains are presented. Based on the probabilities, the total damage probability P_{all} is evaluated by the Equation (23).

$$P_{all} = P_{fe50} + P_{fc50} \quad P_{fe50} \times P_{fc50} \tag{23}$$

The risk can be calculated by $P_{all} \times C_f$, in which C_f (=196 million JPY) is the damage cost by submergence due to the dam breach. The total risk is evaluated as 150 million JPY for the original embankment and 84 million JPY for the improved embankment, according to the table.

7 CONCLUSIONS

(1) Based on the statistical model for the internal friction angle, including the spatial distribution of the N-values, the two conversion errors, from the SWS N-value to the SPT N-value, and from the SPT N-value to the internal friction angle, and the seismic hazard for the Nankai Trough Earthquake, the reliability analysis has been conducted for an earth-fill embankment, and the damage probability over the next 50 years, has been evaluated.

(2) The probability of overflow caused by the heavy rains has been calculated for several earth-fill dams. We assumed that overflows will occur when the maximum overflow head on the spillway bed becomes greater than the design overflow head. The probability of overflow was then determined by the Monte Carlo simulation.

(3) The damage probability due to the earthquakes and heavy rain has been estimated 0.76 for the original embankment and, the probability of overflow is 0.43 for the restored embankment over the next 50 years. The difference between the risks for the two states of the embankment has been calculated as 66 million JPY, which is the effect of the improvement work.

8 ACKNOWLEDGEMENTS

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