SEISMIC FRAGILITY ESTIMATION OF GEOSYNTHETIC-REINFORCED EMBANKMENT

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Abstract

Seismic risk assessment approach can be stochastically and seamlessly applied to various civil engineering structures subjected from small earthquakes to large earthquakes exceeding the prescribed seismic design force. In the seismic risk assessment of embankments, seismic hazard and fragility of the embankments should be determined rigorously, and the seismic risk of the embankments can be obtained by multiplying them. Meanwhile, the railway design standard in Japan has recommended using permanent seismic displacement as an index obtained by a displacement method based on the Newmark's sliding block analysis method to assess the restorability of the embankments subjected to an earthquake. According to the railway design standard in Japan, the seismic fragility of the embankment shall be calculated using the permanent seismic displacement by the Newmark's sliding block analysis implementing into Monte Carlo simulation with various soil strength and tensile strength of the reinforcement. The above method is exact and precise, but high calculation cost and inefficient for practice. Therefore, this paper proposes a practical estimation method of the seismic fragility of reinforced embankments subjected to an earthquake for practical seismic risk assessment. At first, strong seismic motion records from January 1997 to September 2018 observed in Japan were collected, and strong seismic motions database adjusted by Arias Intensity was newly created for the calculation of seismic fragility of the embankments. Analytical models are set as reinforced embankment models with different embankment heights according to the railway design standard in Japan. Sensitivity analysis for the seismic fragility estimation of the reinforced embankments was conducted with various embankment height, average values of friction angle of the backfill soil, and tensile strength of the primary reinforcement. Finally, a practical and straightforward fragility curve estimation equation using design parameters which are commonly used to check the embankment's stability is proposed for the practical use..

Keywords: embankment, reinforced soil, seismic risk assessment, fragility, Newmark method

1. Introduction

Japan is situated at a rare point in the world where four tectonic plates converge, and is thus located in a place where earthquakes are likely to occur with about one-tenth the number of earthquakes in the world. It is estimated that there are about 2000 active faults in Japan. Therefore, seismic design, construction, and maintenance of civil engineering structures are essential to realizing a sustainable society in Japan against high seismic risk. In seismic design approach for civil engineering structures, probabilistic risk assessment (PRA) is considered to be promising, taking into consideration the probability of occurrence of earthquake motion and the seismic fragility of civil engineering structures with various material strength. For example, a framework for developing probabilistic seismic hazard curves for sliding displacement was proposed [1]. They created displacement hazard curve, which provides the annual rate of exceedance for a range of displacement level. Wang and Rathje (2018) proposed probabilistic and logic tree frameworks and applied to a site in California [2]. The application of the approach describes the ground motion hazard, site characterization data, and development of the logic tree for analysis using the available data.

To construct seismic fragility curves is essential for seismic risk assessment. They relate the seismic intensity to the probability of reaching or exceeding a level of damage (the predefined limit state) for each element at risk. In this study, this seismic probability is called a seismic limit state exceedance probability. The seismic fragility curve is generally expressed as the seismic limit state exceedance probability according to the level of seismic intensity parameters, including peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD).

The fragility curve of embankments is often described by a cumulative lognormal distribution function. Maruyama et al. (2010) successfully constructed an empirical fragility curves of embankments of the express way in Japan which are developed based on the records of the damage resulting from Northen-Miyagi earthquake in 2003, Tokachi offshore earthquake in 2003, Mid Niigata prefecture earthquake in 2004, The Niigataken Chuetsu-oki earthquake in 2007 [3]. The empirical fragility curve could be constructed with a function of PGV as the seismic intensity. JRC (2013) proposed location and scale parameters of the cumulative lognormal distribution function to determine the fragility curve of the road and railway embankments with heights of 2 and 4 m in each damage state (minor, moderate and extensive/complete damage) [4]. These parameters were derived from the numerical results of finite element dynamic analyses due to an increasing level of the PGA. Among the previous researches, however, there is no simple formula for estimating the seismic fragility curve of road or railway embankments for practical use with a wide range of material properties. If the fragility curve of the embankments can be easily obtained with generally given design parameters only, the PRA of the embankments can easily apply to evaluate their seismic performance rigorously and quantitatively in practice.

2. Objective

To quantitatively assess the residual risk of reinforced embankments by the railway design standard in Japan [5], this study proposes a practical and straightforward seismic fragility estimation equation of the embankments with the level of seismic energy calculated from the time history of ground accelerations. The proposed estimation equation of the seismic fragility of the reinforced embankment constitutes parameters of the prescribed probability distribution function which can be estimated from commonly used design parameters of unit weight, strength parameters of the backfill soil and tensile strength of the primary reinforcement only. The cross-section and reinforcement arrangement of the embankments conformed to the railway design standard in Japan [5]. Sensitivity analysis was carried out by changing the height of the embankment, friction angle of backfill soil in the embankment and tensile strength of the primary reinforcement. Seismic limit state exceedance probability of the embankments subjected to the earthquake was calculated with a predefined limit state of permanent seismic displacement and obtained by quasi-Monte Carlo simulation with the displacement method based on the Newmark's sliding block analysis method.

3. Method of seismic fragility assessment

Fig. 1 shows simple schematic figures of seismic PRA obtained from the results of hazard and fragility analyses. This study focused on the seismic fragility assessment of reinforced embankments subjected to an earthquake. The seismic fragility of the embankments can show the relationship between the seismic limit state exceedance probability and seismic intensity. In this study, the seismic limit state of the embankment was determined with permanent seismic displacement which can be calculated by the seismic displacement analysis method based on the Newmark's sliding block analysis method. The seismic limit state exceedance probability of the embankment can be calculated by the above permanent seismic displacement method implemented into quasi-Monte Carlo simulation with probabilistic distributed geomaterial unit weight and strength parameters and tensile strength of the primary reinforcement. In this study, Arias intensity [6] was used as an index indicating the seismic intensity. Strong seismic motion database was newly created to simulate a wide variety of seismic wave shapes of earthquakes that occurred in Japan, which will be explained as follows.

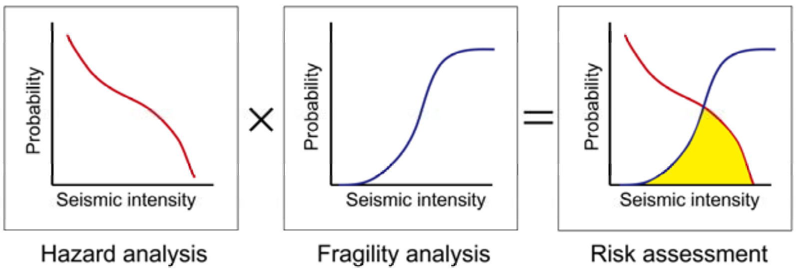

Fig. 1 – Seismic probabilistic risk assessment by hazard and fragility analyses

3.1 Strong seismic motion database

From the 124 earthquakes occurring from March 16, 1996, to September 6, 2018, and of Japan Meteorological Agency (JMA) seismic intensity scale "5 Higher" (JMA 2019) or higher, five seismography observatories in descending order of the peak ground acceleration were selected for each earthquake. For each earthquake in the specific seismography observatory, there are EW and NS components and opposite directions, which are four seismic motions. Therefore, the total number of collected seismic motions in the current database includes 2480 ($124 \times 4 \times 5 = 2480$) time histories of acceleration. These earthquakes caused damage to residential and industrial buildings and other types of infrastructure and triggered landslides, rockfall, and liquefaction, which were causing tremendous damage to life and property.

Fig. 2 shows a statistical analysis result of strong seismic motion database used in this study. The magnitude is likely to the normal distribution, and the average and coefficient of variation are 6.0 and 12%, respectively. Earthquake focal depth is likely to Poisson distribution and the average value is 27 km. Many earthquake focal depths are short, indicating inland earthquake

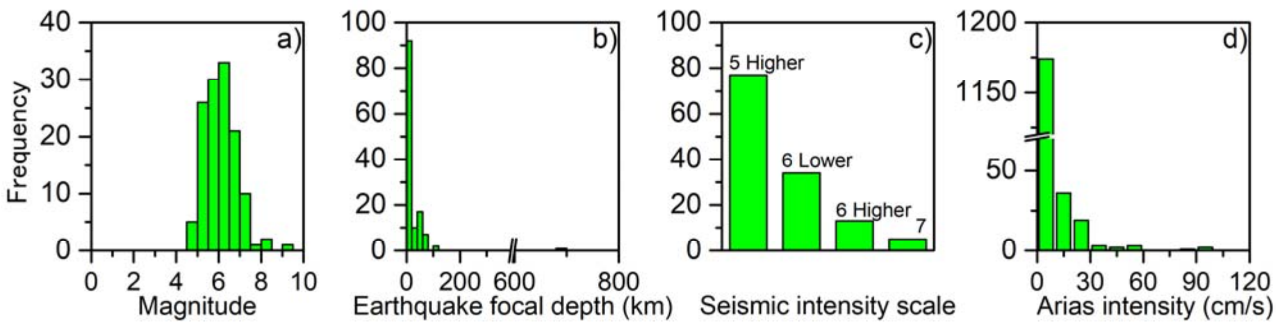

Fig. 2 – Characteristics of selected strong seismic motions: a) histogram of magnitude; b) histogram of earthquake focal depth; c) histogram of seismic intensity scale; and d) histogram of Aris intensity

among collected earthquake. From the created strong seismic motion database, analytical seismic motion for the seismic fragility assessment of the embankments is arranged to adjust the specific Arias intensity [6]. The reason to select the Arias intensity as an index of the seismic fragility of the embankments is that the Arias intensity is a high correlation to the permanent seismic displacement calculated by the Newmark's sliding block analysis method [7, 8]. Arias intensity (I_A) can be calculated as follows:

$$I_A = \frac{\pi}{2g} \int_0^{t_{\max}} a_h(t)^2 \, dt \tag{1}$$

where t is time, $t_{\max}$ is maximum time after the effective seismic motion, g is gravity, and a_h is time histories of the horizontal acceleration. The I_A is the most comprehensive parameter expressing the energy content of an earthquake ground motion record. Fig. 2d shows that the I_A is likely to Poisson distribution and the maximum value becomes 99.5 cm/s. It is noted that the I_A strongly depends on the duration time. In this study, the main seismic motion was extracted from the recorded seismic motions to shorten the computational time of the seismic displacement analysis. Therefore, the calculated I_A in this study is smaller than that obtained from the recorded seismic motions.

3.2 Calculation of limit state exceedance probability

In this study, the permanent seismic displacement was calculated by the seismic displacement analysis method based on the Newmark's sliding block analysis method [9]. Hereafter, the seismic displacement analysis method used in this study is referred as the Newmark method. The adopted Newmark method is a simplified procedure employed in the design code of road and railway structures in Japan [5, 10], where the permanent seismic displacement of the reinforced embankments subjected to a strong earthquake can be calculated by integrating the equation of the rotational motion of a soil mass contained within the critical circular slip surface by assuming the failure mass as a rigid rotational block. The equation of rotational motion is solved for the rotation caused by the difference between the driving and the resisting moments. The critical slip surface is determined using the conventional modified Fellenius method [11] using a specific acceleration or seismic coefficient to yield a safety factor of 1.0. A requisite for such an analysis is the unit weight, friction angle, and cohesion of soil, and tensile strength of reinforcement. To calculate the permanent seismic displacement, it is not necessary to consider the input parameters in addition to the abovementioned ones. The feature of this analysis is that it is practically useful and less time-consuming regarding the calculation. In this study, the permanent seismic displacement is defined as a rotational displacement along the critical slip surface of the failure mass. The detailed of the calculation method can be referred to Shinoda et al. [12, 13]

In the present study, the quasi-Monte Carlo simulation was adopted to calculate the seismic limit state exceedance probability with the statistically distributed geomaterial unit weight and strength parameters and tensile strength of reinforcement. The critical slip surface of the embankment was determined to minimize the safety factor in each calculation of the quasi-Monte Carlo simulation.The seismic limit state of the embankments should be predefined before reliability analysis to calculate the seismic limit state exceedance probability. In this study, the seismic limit state is defined that the permanent seismic displacement of the embankments exceeds 50 cm according to the railway design standard in Japan [5]. This value is determined as a limit value for the restorability of ballast track in the railway design standard in Japan [5]. The quasi-Monte Carlo simulation can improve the calculation efficiency using a low-discrepancy sequence (LDS). The number of simulations was set as 10,000 to calculate the limit state exceedance probability. The LDS is one of the quasi-random numbers that has a uniform distribution (i.e., [14]). A feature of the LDS is that a set of quasi-random numbers in each simulation is unique about the number of simulations. Using the LDS, the uniformity of the random variable could be significantly improved. Based on the above, it is fairly reasonable to use the LDS for random numbers in the current Monte Carlo simulation.

4. Analytical model

The structures considered in this study are reinforced embankments, as shown in Fig. 3. The structural specification was by the railway design standard in Japan [5]. The heights of the embankments

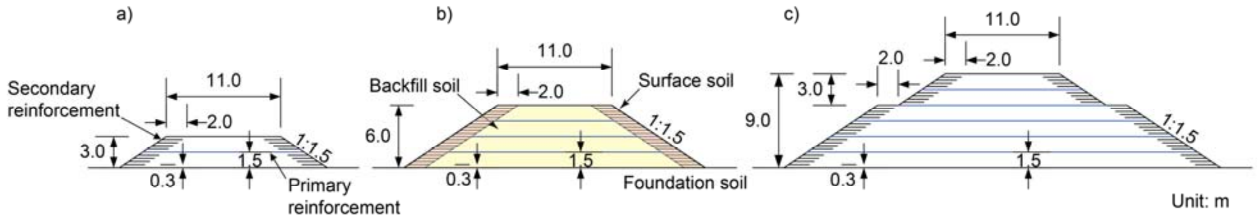

Fig. 3 – Analytical models of reinforced embankments with different embankment heights (H): a) H = 3.0 m; b) H = 6.0 m; c) H = 9.0 m

are 3.0 m, 6.0 m, and 9.0 m, respectively. The slope inclination is 1:1.5. The vertical spacings of the primary and secondary reinforcements were 1.5 m and 0.3 m, respectively. The length of the primary reinforcement was sufficiently long beyond the critical slip surface to resist the rotation of the soil mass, while the length of the secondary reinforcement was set constant at 2.0 m. A surcharge of 10 kPa was applied on the crest of the slope.

According to the railway design standard in Japan [5], the properties of the foundation soil, backfill soil, and surface soil require to be determined to evaluate the safety or reliability of a structure. In the current analysis, the foundation soil was assumed to have sufficiently high strength and stiffness. This means that the slip surface of the embankment subjected to an earthquake was assumed not to cross the foundation. Further, in practice, the surface soil along a slope is generally exceedingly difficult to compact, thus requiring a comparatively low friction angle. Moreover, the apparent cohesion of the unsaturated surface soil generally depends on the degree of saturation. The degree of saturation of the surface soil is usually comparatively high, owing to the effects of rainfall. This indicates that the cohesion of the surface soil may become lower than that of the backfill soil. Thus, the properties of the surface soil were modeled using a relatively lower friction angle and cohesion than the backfill soil. According to the railway design standard in Japan [5], the average friction angle of the surface soil is 5 in degree lower than that of the backfill soil.

To evaluate the statistical embankment soil properties, the Railway Technical Research Institute (RTRI) conducted statistical analysis with triaxial compression test results conducted by the RTRI and collected from a literature survey [16]. Tables 1 and 2 list the statistical soil properties of the backfill and

Table 1 – Statistical backfill soil property

Property	Average value	Location parameter of lognormal distribution	Scale parameter of lognormal distribution
Unit weight	18.0 kN/m^3	2.890	0.05
Friction angle	30°, 35°, 40°, 45°, 50°	3.401, 3.555, 3.689, 3.807, 3.912	0.10
Cohesion	6.0 kN/m^2	1.792	0.10

Table 2 – Statistical surface soil property

Property	Average value	Location parameter of lognormal distribution	Scale parameter of lognormal distribution
Unit weight	18.0 kN/m^3	2.890	0.05
Friction angle	25°, 30°, 35°, 40°, 45°	3.219, 3.401, 3.555, 3.689, 3.807	0.10
Cohesion	3.0 kN/m^2	1.099	0.10

surface soils adopted in this study referring to the round robin test and literature survey [15]. The COV of cohesion was assumed to be the same value of 10%. Each random variable was assumed to be statistically independent and lognormally distributed.

The technical committee of Japan chapter of International Geosynthetics Society carried out a round-robin extension test of high-density polyethylene (HDPE), polyester (PET) and vinylon geogrids to evaluate the statistical tensile strength of geogrids [16]. Table 3 lists the statistical properties of primary reinforcement adopted in the current analysis referring to the round robin test [16]. The COV of the tensile strength of geogrids was set at 5.0% in the current analysis. Based on the result of the trial Monte Carlo simulation, the effect of the secondary reinforcement on the seismic limit state exceedance probability calculation is minimal that the tensile strength of the secondary reinforcement is considered to be a deterministic value in this study. Each tensile strength of the primary reinforcements was assumed to be statistically independent and lognormally distributed.

Table 3 – Statistical property of the primary reinforcement

Average value (kN/m)	Location parameter of lognormal distribution	Scale parameter of lognormal distribution
15.0, 30.0, 45.0	2.708, 3.401, 3.807	0.05

5. Sensitivity analysis

Figs. 4a to 4c shows the seismic fragility curves of reinforced embankments subjected to earthquakes obtained from the strong seismic motion database created in this study. In each figure in Fig. 4, embankment height and an average value of the tensile strength of the primary reinforcement were constant, but the various average value of the friction angle of the backfill soil. From Figs. 4a to 4c, lower the average value of friction angle in the backfill soil of the embankment, more significant the limit state exceedance probability.

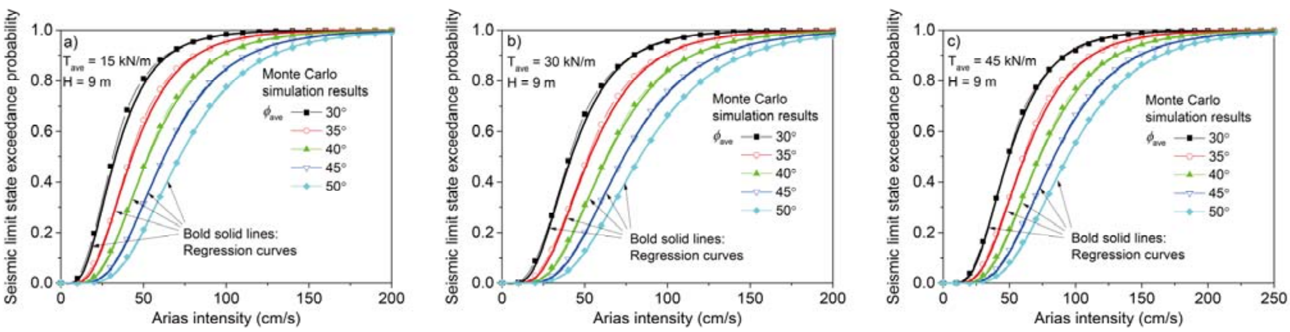

Fig. 4 – Fragility analysis results obtained from the quasi-Monte Carlo simulation with the Newmark method with a constant embankment height of 9 m, various friction angle of backfill soil in the embankment and tensile strength of the reinforcement: a) the tensile strength of the primary reinforcement of 15 kN/m; b) the tensile strength of the primary reinforcement of 30 kN/m; and c) the tensile strength of the primary reinforcement of 45 kN/m. Regression curves determined with each tensile strength of the primary reinforcement were also depicted

6. Seismic fragility estimation

The fragility curves shown in Figures 4a to 4c became very smooth, indicating a high possibility of numerical estimation using a regression equation. It is highly practical to estimate the fragility curve only with design parameters when usually checking the stability of the embankment. This study adopted the following lognormal cumulative distribution function with two parameters to fit the fragility curve obtained from the current Monte Carlo simulation:

$$P_e = \Phi\left(\frac{\ln I_A - \mu}{\sigma}\right) \tag{2}$$

where P_e is the seismic limit state exceedance probability, Φ is the cumulative distribution function of the standard normal distribution, μ is the location parameter of the cumulative lognormal distribution function, σ is the scale parameter of the cumulative lognormal distribution function.

Fig. 5a shows the location parameter μ identified from the result of Monte Carlo simulation with the average value of the tensile strength of the primary reinforcement of 30 kN/m. From Fig. 5a, it is found that the location parameter μ depends on $1/\tan\phi_{ave}$ and $\gamma_{ave}\ H/c_{ave}$. Therefore, the regression equation of the location parameter μ can be derived as follows:

$$\mu = \frac{A \cdot \dfrac{\gamma_{ave} \cdot H}{c_{ave}} + B}{\tan\phi_{ave}} + C \tag{3}$$

where A, B, and C are parameters. Fig. 5a also plotted the above regression equation using A = -0.0048, B = -0.6411, C = 5.072. The estimated location parameters using Equation (3) shows good agreement with that identified result by the Monte Carlo simulation.

Fig. 5b shows the scale parameter σ identified from the result of Monte Carlo simulation with the average value of the tensile strength of the primary reinforcement of 30 kN/m. From Fig. 5b, it is found that the scale parameter σ depends on $1/\tan\phi_{ave}$ and the sensitivity of the embankment height was not significant in the estimation of the scale parameter of the cumulative lognormal distribution. Therefore, the regression equation of the scale parameter σ can be derived as follows:

$$\sigma = \frac{D}{\tan\phi_{ave}} + E \tag{4}$$

where D and E are parameters. Fig. 5b also plotted the above regression equation using D = 0.0660, E = 0.3850. The estimated scale parameter σ using Equation (4) shows good agreement with that identified result by the Monte Carlo simulation.

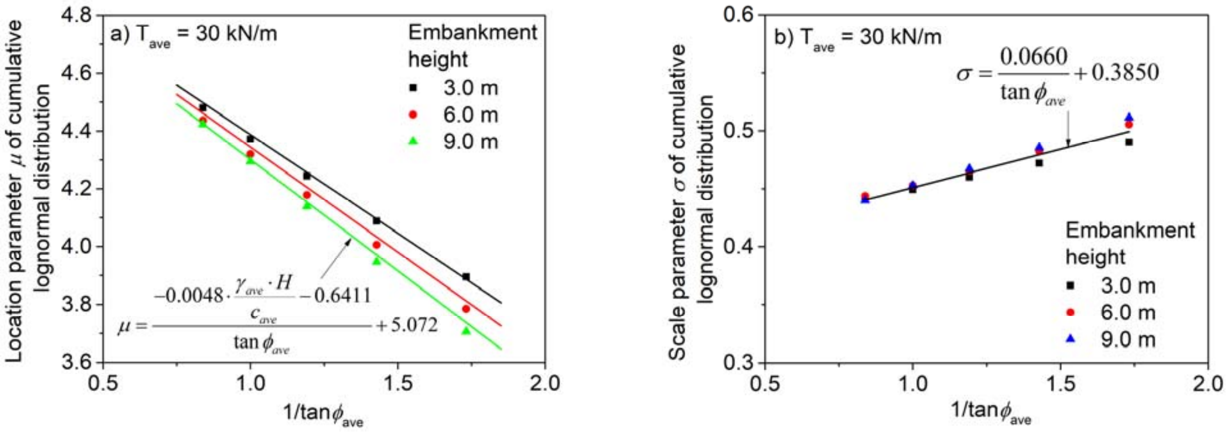

Fig. 5 – Parameters of cumulative lognormal distribution plotted to the inverse of $\tan\phi$ with embankment height of 3 m, 6 m, and 9 m and constant tensile strength of the primary reinforcement of 30 kN/m: a) location parameter and b) scale parameter.

Table 4 shows the above parameters A, B, C, D, and E with the average value of the tensile strength of the primary reinforcement. Fig. 4 shows the regression curves using the cumulative lognormal distribution using the above parameters A, B, C, D, and E. In Fig. 4, the approximated fragility is very good agreement with the results of the Monte Carlo simulation in the whole range of the tensile strength of the primary reinforcement.

Table 4 –Location and scale parameters of cumulative lognormal distribution with the averaged tensile strength of the primary reinforcement

Average value of the tensile strength of primary reinforcement (kN/m)	Location and scale parameters of cumulative lognormal distribution				
	A	B	C	D	E
15	-0.002	-0.820	5.01	0.091	0.370
30	-0.005	-0.641	5.07	0.066	0.385
45	-0.006	-0.660	5.15	0.055	0.390

7. Conclusion

This study proposed a practical fragility estimation equation of the standard models of reinforced railway embankments in Japan. The fragility curve could be estimated using a cumulative lognormal distribution function with location and scale parameters. To develop the simple fragility estimation equation, strong seismic motion database was created to be adjusted by Arias intensity. The collected seismic motion was recorded from the 124 earthquakes occurring from March 16, 1996 to September 6, 2018 and of JMA seismic intensity scale "5 Higher" or higher. Using this database, Newmark analysis implemented into quasi-Monte Carlo simulation using a low-discrepancy sequence was carried out to calculate the seismic limit state exceedance probability of the embankments with an index of the seismic permanent displacement. Sensitivity analysis for the fragility estimation of the reinforced embankments was conducted with various embankment height, average values of friction angle of the backfill soil, and tensile strength of the primary reinforcement. From the results of the sensitivity analysis, it is found that the location parameter μ depends on $1/\tan\phi_{ave}$ and $\gamma_{ave}\, h/c_{ave}$ and the scale parameter σ depends on $1/\tan\phi_{ave}$ with a prescribed average tensile strength of the reinforcement. To develop an unified fragility estimation equation of reinforced embankments, the location and scale parameters of the cumulative lognormal distribution could be successfully estimated using parameters of the embankment height, the average value of the friction angle of the backfill soil and the average value of the tensile strength, which are commonly used design parameters to check the embankment's stability. The proposed analytical approach to determine the fragility curve of the embankments is probably useful to apply the different standard structural configuration and statistical variability of the backfill soil and tensile strength of the primary reinforcement.

8. Acknowledgements

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