Estimation of Seismically-Induced Crest Settlement of Earth Core Rockfill Dams

Ki-Chun Park [1], Van-Quang Nguyen [1], Jung-Hyun Kim [1], Duhee Park [1,*] and Byoung-Han Choi [2]

[1] Department of Civil and Environmental Engineering, Hanyang University, 222 Wangsimni-ro, Sageun-dong, Seongdong-gu, Seoul 04763, Korea; parky229@naver.com (K.-C.P.); nguyenvanquang240484@gmail.com (V.-Q.N.); kjh3813@hanyang.ac.kr (J.-H.K.)

[2] Rural Research Institute, Korea Rural Community Co., Ansan-si, Geonggi-do 15364, Korea; han1020@ekr.or.kr

* Correspondence: dpark@hanyang.ac.kr; Tel.: +82-2-2220-0322

Abstract: The settlement at the dam crest induced by earthquake loading is an important design parameter widely used to evaluate the seismic performance of dams. Two-dimensional nonlinear dynamic analyses were performed to calculate the dam crest settlement of earth core rockfill dams (ECRDs). The numerical model was first validated against centrifuge test measurements. Comparisons showed that the numerical model provided reasonable estimates of the measured response. A series of dynamic numerical analyses were performed for a wide range of ground motions. The calculated settlements were revealed to compare well with field recordings of dam crest settlements. The settlements were also plotted against ground motion parameters, including free-field surface peak ground acceleration (PGA), peak ground velocity (PGV), Arias intensity (I_a), and moment magnitude (M). Predictive equations for the crest settlement were also presented. It wes demonstrated that PGA provided a good estimate of the settlement, but I_a provided the most favorable fit according to the calculations. Additionally, multiple parameters models were also presented to enhance the accuracy of the predictions.

Keywords: dynamic analysis; ECRD; crest settlement; ground motion parameter; predictive equation

1. Introduction

The settlement at the dam crest induced by seismic excitation is an important seismic design parameter for earth dams and is routinely used to evaluate the seismically-induced damage level of dams. Extensive data were collected and summarized to estimate the settlement characteristics of earth dams [1]. The dams considered were earth core rockfill dams (ECRDs), concrete-faced rockfill dams (CFRDs), hydraulic fill dams, and earthfill dams. An empirical formula was proposed that correlated the free-field outcrop peak ground acceleration (PGA) and moment magnitude (M) with the surface settlement. However, because a wide range of dams was used, a significant level of scatter was inevitable. Additionally, the strength of the correlation of various ground motion parameters other than PGA to potentially enhance the correlation was not investigated.

Recently, a series of numerical simulations was performed to evaluate the dynamic response of earth dams. Yang and Chi [2] performed a series of finite element limit analyses to evaluate the seismic stability of earth core rockfill dams (ECRDs). However, the crest settlement was not calculated. Pang et al. [3] performed numerical simulations to calculate the settlement of concrete-faced rockfill dams (CFRD) and to derive fragility curves. Zhang et al. [4] performed dynamic analyses to investigate the stress induced in the concrete slab of CFRDs. However, neither of the numerical models were

validated against recordings. The literature review demonstrated that a study using a validated or verified numerical model to calculate the dam settlement has not yet been performed.

The aim of this paper was to develop empirical equations to predict the crest settlement of earth core rockfill dams (ECRDs). The predictive equation could potentially be used for damage assessment of ECRDs. A two-dimensional (2D) nonlinear model validated against centrifuge tests was used to perform a series of dynamic analyses. Multivariate regression analyses were performed to correlate the calculated crest settlements with a number of ground motion parameters.

2. Numerical Model

A series of 2D dynamic analyses was performed using the finite difference analysis program FLAC^{2D} (Fast Lagrangian Analysis of Continua in two dimensions), Version 7.0 [5] to calculate the settlement at the crest of an ECRD. The baseline numerical model of the ECRD is shown in Figure 1. The dam was 60 m and 260 m in height and width, respectively. The dimensions of the dam were assigned based on actual ECRDs constructed in Korea. The upstream and downstream slopes were set to 1:2.2 and 1:2.0, respectively. The slope of the core was set to 1:0.2. The thickness and width of the soil model below the dam were set to 40 m and 400 m, respectively. The shear wave velocity profiles of the rockfill and core sections were constructed from the empirical shear wave velocity equations of Sawada and Takahashi [6], as shown in Figure 2. The core profile lay between the lower and upper bounds of Sawada and Takahashi's [6] equations for the core. The rockfill profile also fell between the saturated and unsaturated rockfill profiles of Sawada and Takahashi [6]. The applicability of the empirical equations of Sawada and Takahashi [6] was confirmed using geophysical measurements [7] and inversion analysis results derived from artificial blast recordings [8]. Table 1 lists the properties used for the rockfill and the core. For the underlying foundation soil, the shear wave velocity was set at 1100 m/s.
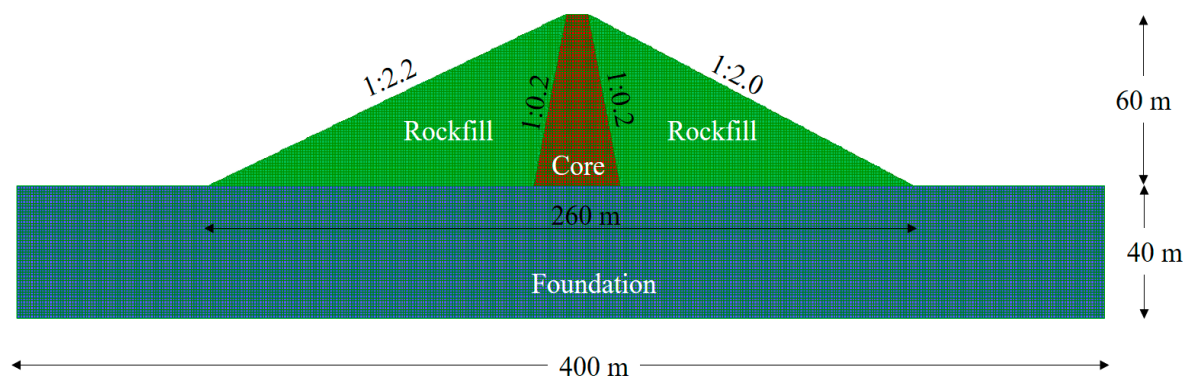

Figure 1. 2D finite element model of a baseline earth core rockfill dam (ECRD).

Table 1. Properties of the dam and foundation soil.

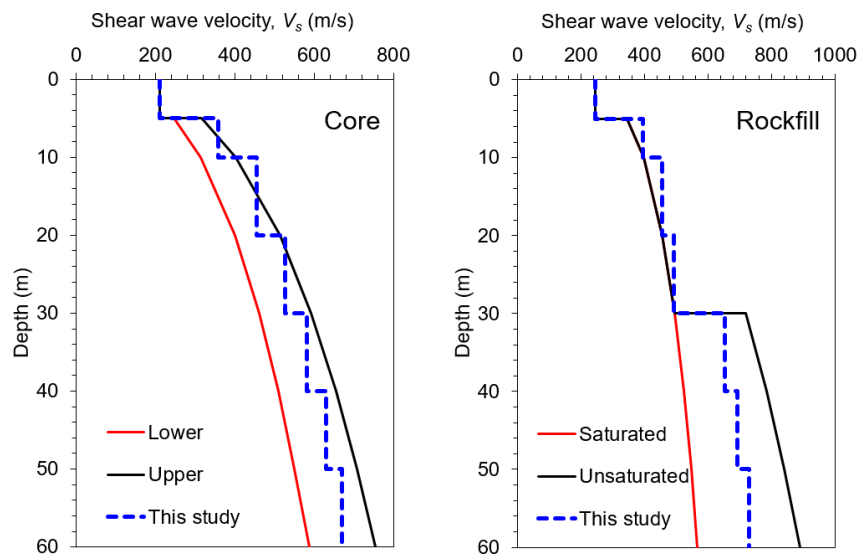| Layer	Unit Weight (kN/m^3)	Friction Angle (^{o})	Cohesion (kPa)	Poisson's Ratio
Rockfill	21	45	0	0.30
Core	20	27	32.5	0.35
Foundation soil	24	50	-	0.25

Figure 2. Shear wave velocity profiles of the rockfill and the core.

The nonlinearity of the soil was reported to play an important role in the seismic response of slopes [9]. The Sig3 model, which is one of the most widely used nonlinear soil models among those included in the library of FLAC^{2D}, was used to simulate the nonlinear behaviors of the rockfill and the core. The Sig3 model was defined as follows:

$$M_s = \frac{a}{1 + \exp(-(L - x_0)/b)} \tag{1}$$

where M_s is the secant shear modulus, L is $\log(\gamma)$, and x_o, a, and b are curve-fitting parameters. Sig3 was used in a number of studies to represent the nonlinear soil response under seismic loading [9–13]. The parameters for the Sig3 model were selected to match the plastic index (PI) = 0 curves of Darendeli [14] in the middle of each soil layer. The Mohr–Coulomb model was used with the Sig3 model to simulate plastic deformation, which is induced at large shear strains. The small strain damping was modeled via the Rayleigh damping formulation, which was defined as follows [15]:

$$[C] = \alpha[M] + \beta[K] \tag{2}$$

where [C] is the damping matrix, [M] is the mass matrix, [K] is the stiffness matrix, and α and β are the Rayleigh coefficients that determine the frequency dependence of the damping formulation. These coefficients were defined as follows [15]:

$$\begin{aligned} \alpha &= \frac{4\pi\xi f_m f_n}{f_m + f_n} \\ \beta &= \frac{\xi}{\pi(f_m + f_n)} \end{aligned} \tag{3}$$

where ξ is the damping ratio. The formulation matched the target damping ratio only at the frequencies of f_m and f_n. In the analyses, the 1st and 5th modes were used, as recommended by Lee et al. [9].

FLAC^{2D} used the mixed discretization scheme to subdivide each quadrilateral element into two overlaid constant-strain triangle elements. It was reported that the mixed scheme accurately modeled the plastic collapse load [16]. Free-field boundary conditions were applied to the lateral boundaries to absorb reflected waves. The viscous damper proposed by Kuhlemeyer and Lysmer [17] was applied

at the lower boundary to absorb the downward propagating motion. The size of the elements was selected to propagate a minimum of 25 Hz components using the following equation:

$$H = \frac{V_s}{4f_{\max}}$$

(4)

where H is the height of the element, $f_{\max}$ is the maximum frequency of the input motion, and V_s is the shear wave velocity.

3. Validation of the Numerical Model

The numerical model was validated against the centrifuge test measurements presented by Kim et al. [18]. The centrifuge model and the corresponding numerical model are shown in Figure 3. The height and width of the ECRD were 5.2 m and 18.8 m, respectively. The model was significantly smaller than an earth core dam, but it was considered to be acceptable for validation purposes [18]. The following pressure-dependent equations reported by Kim et al. [18] were applied to assign the shear wave velocities of the rockfill and core.

$$V_s = 109(\sigma_c)^{0.24}$$

(5)

$$V_s = 41.8(\sigma_c)^{0.39}$$

(6)

where σ_c is the confining pressure (kPa) and V_s is the shear wave velocity (m/s). The equations were developed to match the resonant column test results of materials used in the centrifuge tests. Triaxial tests with specimens that were 50 mm diameter were also performed to measure the shear strengths. Table 2 lists the properties of the centrifuge model.

Table 2. Properties of the dam and foundation soil for the centrifuge model.

Layer	Unit Weight (kN/m^3)	Friction Angle ($^{\circ}$)	Cohesion (kPa)	Poisson's Ratio
Rockfill	21.09	40	2	0.30
Core	21	33	64	0.35

The numerical model was identical to the ECRD model described in the previous section, including the constitutive model, the damping formulation, and the lateral boundary conditions. The fixed condition was applied to the bottom boundary to simulate the rigid boundary of the centrifuge model, whereas the viscous dampers were used at the bottom boundary in the baseline ECRD model. Figure 4 compares the values measured by Kim et al. [18] and the computed *PGA* profiles from the model defined by this study. The numerical analysis was shown to provide reliable estimates of the dam response. It was therefore concluded that the numerical model could be used to predict the settlement of ECRDs.
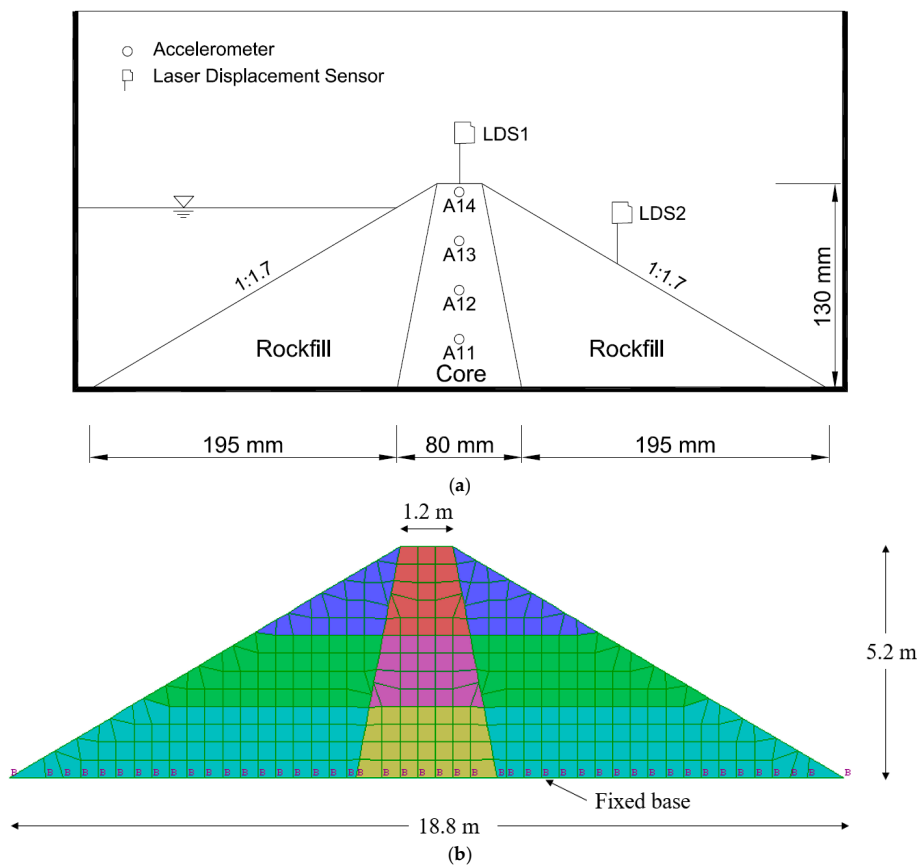
Figure 3. The models of ECRD used for validation. (**a**) Centrifuge model [18] and (**b**) numerical model of ECRD from this investigation.
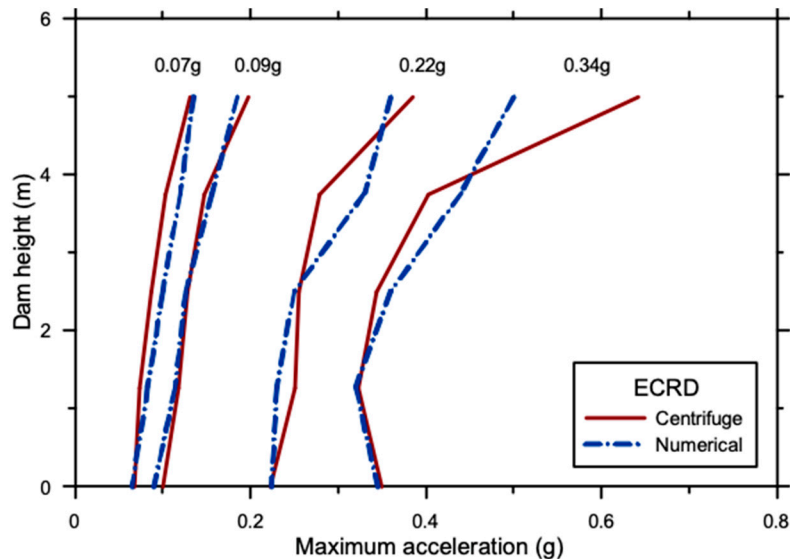

Figure 4. Comparison of measured peak ground acceleration (*PGA*) profiles from centrifuge tests and calculated profiles from 2D dynamic simulations.

4. Crest Settlement Calculations and Correlation with Ground Motion Parameters

A total of 20 ground motions were selected from the Next Generation Attenuation (NGA-West2) database (https://ngawest2.berkeley.edu/) to perform a suite of nonlinear numerical simulations,

from which empirical correlations between the ground motion parameters (GMs) and the crest settlement were developed. Detailed information regarding the respective ground motions, including *PGA*, peak ground velocity (*PGV*), and moment magnitude (*M*), were provided in the NGA-west2 database. These are summarized in Table 3. All 5% damped acceleration response spectra and the design spectrum of Korea [19] normalized to PGA are shown in Figure 5. The motions were imposed at the bottom of the computational domain, as shown in Figure 1.

Table 3. Details of input ground motion selected to perform nonlinear dynamic analyses.

| No.	Earthquake Name	Station Name	*PGA*(g)	*PGV*(cm/s)	Magnitude, *M*
1	Nahanni, Canada	Nahanni Station #3	0.148	6.1	6.8
2	Loma Gilroy, USA	Gilroy Array #2	0.17	14.2	6.9
3	Whittier Narrow, USA	Rancho Palos Verdes	0.186	4.6	6
4	San Fernando, USA	Pacoima Dam	0.209	18.9	6.6
5	Northridge, USA	Beverly Hills	0.217	9.8	6.7
6	Kocaeli, Turkey	Arcelik	0.218	17.7	7.4
7	Landers, USA	Yermo Fire	0.244	25.7	7.3
8	Hector, USA	North Palm Spr.	0.265	28.5	7.1
9	Loma Gilroy, USA	Gilroy Array #3	0.357	28.6	6.9
10	Parkfield, USA	Parkfield–Cholame	0.357	21.5	6.1
11	El Centro, USA	Imperial Valley Distr.	0.4	37.8	6.9
12	Mammoth Lakes, USA	Long Valley Dam	0.43	23.5	6.3
13	Duzce, Turkey	Bolu	0.727	56.4	7.1
14	Kobe, Japan	Nishi–Akashi	0.509	37.3	6.9
15	Tabas, Iran	Mhud–Bajestan	0.899	55	7.8
16	Chi-chi, Taiwan	Taiwan	0.057	2.3	6.2
17	Tottori, Japan	HYG007	0.035	1.2	6.61
18	Northridge, USA	Vasquez Rocks Park	0.15	11.1	6.69
19	Morgan Hill, USA	Gilroy Array	0.069	2.7	6.19
20	San Fernando, USA	Pasadena, Old Seismo Lab	0.095	5.8	6.61
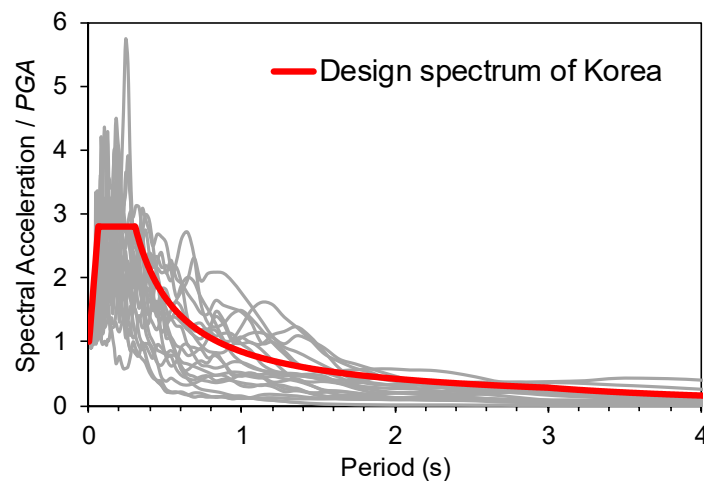
Figure 5. Spectrum of 5% damped response of input motions and the design response spectrum of Korea [19] normalized to the *PGA*.

The crest settlements and settlement ratios were calculated from the dynamic analyses. The settlement ratio was defined as the vertical settlement normalized by the height of the dam. It should be noted that this definition was different from that of Swaisgood [1], where the settlement ratio was defined as the vertical settlement at the dam crest (including the settlement of the dam structure and the underlying soil) divided by the height of both the dam and the underlying soil. Swaisgood [1] used this ratio because the respective settlements of the dam structure and the underlying soil could not be separated from the measured crest settlements. Because the settlement of the underlying soil

was highly dependent on the site-specific stratigraphy and also because the foundation soil influenced both the numerator and the denominator, the difference in these definitions may either produce a higher or a lower ratio estimate. In this study, because stiff elastic soil was assumed to underlie the dam, only the calculated settlement in the dam structure was used to calculate the settlement ratio.

The calculated settlements were compared with Swaisgood's [1] data in Figure 6. The numerically calculated settlements fell within the range of measured values. The numerical settlements were lower than the measurements at low intensities (free-field *PGA* < 0.2 g). The differences in the dam dimensions, material properties, and underlying soil profiles may all have influenced the calculated settlement ratio. However, the results were very similar at *PGA* > 0.2 g. It was therefore further validated that the numerical model provided reasonable estimates of the crest settlement. It also highlighted that the difference in the definition of the settlement ratio did not have a pronounced influence on the calculated results for free-field *PGA* > 0.2 g. It also demonstrated that nonlinear dynamic analysis could be used as a seismic design tool to predict crest settlement and to evaluate the susceptibility of the dam damage.
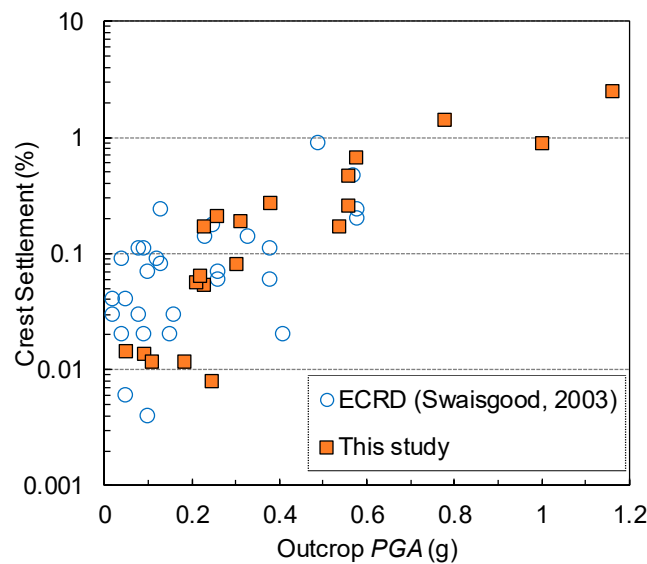

Figure 6. Comparison of crest settlement versus outcrop *PGA* data of ECRD presented in Swaisgood [1] and this study.

Figure 7 shows the calculated settlement ratios plotted against *PGA*, *PGV*, arias intensity (I_a), and *M*. The multivariate regression analyses were performed to evaluate the goodness of fit. The functional form for the predictive equation of the crest settlement was as follows:

$$Ln(s) = \alpha_1 GM_1 + \alpha_2 GM_2 + \alpha_3 GM_3 + \alpha_4 GM_4 \tag{7}$$

where *S* is the settlement (%), $\alpha_1 - \alpha_4$ are the model coefficients, and $GM_1 - GM_4$ are the ground motion parameters. The coefficients and parameters are listed in Table 4. Also listed are the R^2 for each model. The model coefficients $\alpha_1 - \alpha_4$ and the coefficient of determination (R^2) were calculated using multivariate regression analyses.

Among the four GMs, I_a was revealed to have the strongest dependence on the settlement. For the single-parameter model, I_a was demonstrated to provide the best fit with the calculated settlements, resulting in $R^2 = 0.82$. *PGA* resulted in the second-best estimate of $R^2 = 0.77$. *M* was revealed to be positively correlated with the settlement, but produced the lowest R^2 of 0.33. The settlement prediction was further improved by using multiple parameters. When using *PGA* and I_a, the uncertainty was reduced by 10.0% and 5.3% compared with the use of the single parameters *PGA* and I_a, respectively.

The four-parameter model resulted in R^2 = 94.1%, which was 12.1% higher compared with the single-parameter model.
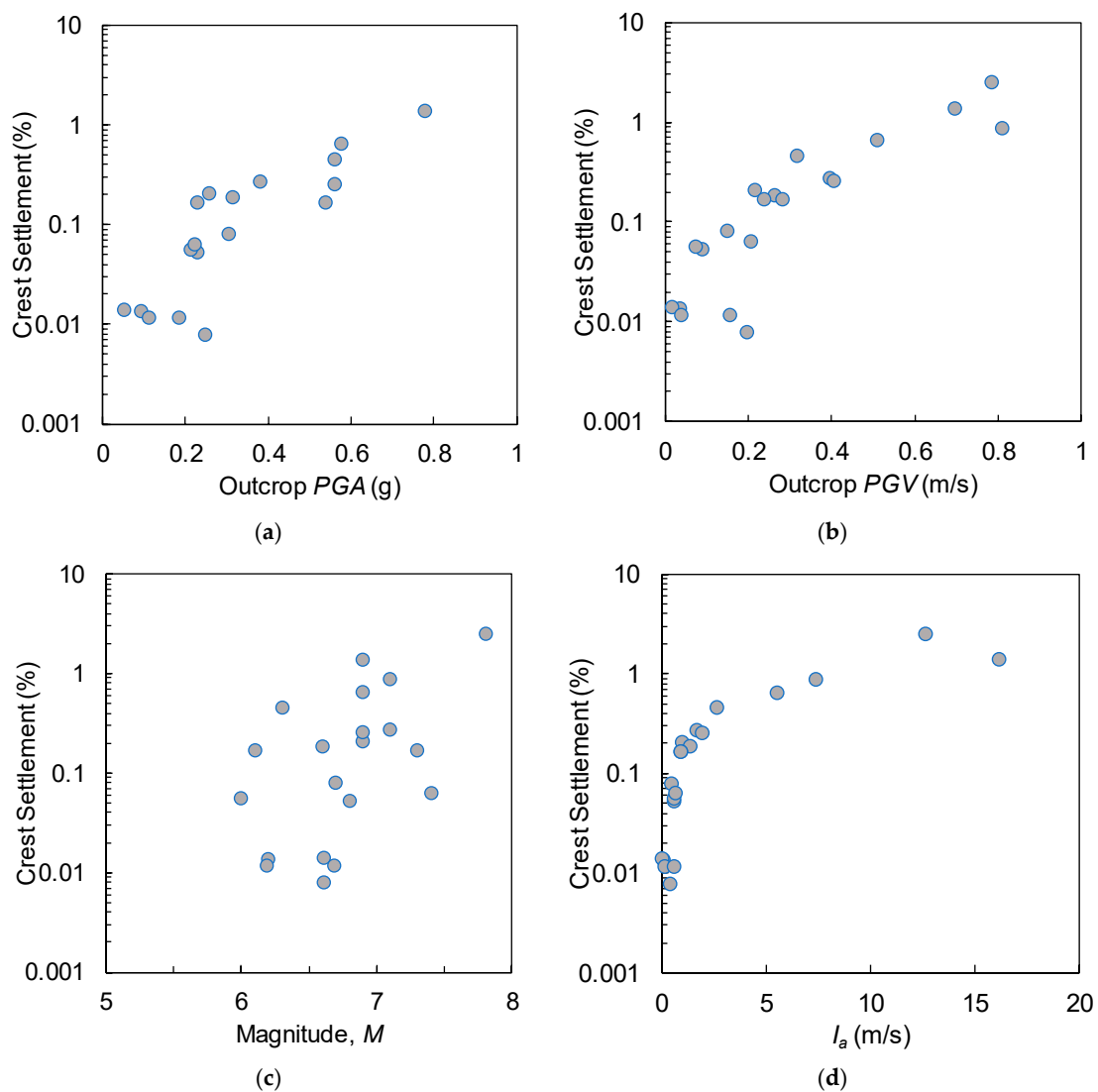

Figure 7. Crest settlement plotted against ground motion parameters. (**a**) *PGA*, (**b**) peak ground velocity (*PGV*), (**c**) moment magnitude (*M*), and (**d**) Arias Intensity (*I$_a$*).

Table 4. Coefficients for the crest settlement prediction model.

	1-Parameter Model				2-Parameter Model			3-Parameter Model			4-Parameter Model
GM_1	PGA	PGV	I_a	M	PGA	PGA	PGA	PGA	PGA	PGA	PGA
GM_2					PGV	I_a	M	PGV	M	PGV	PGV
GM_3								M	I_a	I_a	I_a
GM_4											M
α_1	1.803	2.16	0.127	0.774	1.735	1.606	0.845	2.197	0.748	1.616	2.15
α_2					0.089	0.264	0.079	−0.847	0.203	−1.25	−2.422
α_3								0.339	0.075	0.094	0.1
α_4											0.396
R^2	0.773	0.71	0.334	0.82	0.774	0.803	0.873	0.811	0.89	0.891	0.941

5. Conclusions

A series of nonlinear finite difference analyses was performed to calculate settlements at the crest of earth core rockfill dams (ECRDs) subjected to earthquake loading. Before performing the analyses,

the numerical model was validated against centrifuge test recordings. The conclusions drawn from this study were as follows.

1. The nonlinear model of the dam was demonstrated to provide reliable estimate of the seismic response of ECRDs when compared with the centrifuge test measurements. It was therefore revealed that the nonlinear model was capable of predicting the settlement and could be used for the evaluation of the seismic performance of ECRDs.
2. The calculated settlements using twenty ground motions were compared with field recordings. The results compared well with the recordings for peak ground acceleration (*PGA*) exceeding 0.2 g. It was therefore further validated that the numerical model could be used in the seismic design.
3. The settlements were also plotted against ground motion parameters, including the free-field surface *PGA*, peak ground velocity (*PGV*), Arias intensity (I_a), and moment magnitude (*M*). Predictive equations for the crest settlement were also presented. It was demonstrated that *PGA* provided a good estimate of the settlement. However, I_a provided the most favorable fit with the calculations. Therefore, using the primary parameter to predict the crest was recommended.

Author Contributions: D.P. and K.-C.P. conceived the idea, outlined the work, and reviewed the paper; V.-Q.N. performed numerical analyses and co-wrote the paper; J.-H.K. analyzed the calculated responses and performed multivariate regression analyses; B.-H.C. outlined the scope of the work and reviewed the manuscript.

Conflicts of Interest: The authors declare no conflict of interest.

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