Seismic Fragility of Weir Structures with Infinite Foundations in Korea

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Abstract

The key objective of this study was to conduct a probabilistic risk assessment and safety evaluation of weir structures. Seismic fragility analysis of this kind for natural hazards to weir structures requires rigorous evaluation for strong seismic ground motions. To that end, 2D simple linear elastic Finite Element (FE) and nonlinear FE models of weir structures — namely the Gangjoeng-Goryeong hydraulic systems — in consideration of Soil-Structure Interaction (SSI) were developed. Additionally, in order to evaluate energy radiation in the foundation, the infinite foundation model in ABAQUS was applied. Specifically, the probabilities of failure for the simple linear elastic FE model were compared with those for the model using infinite foundations under seismic wave propagation, after which a fragility analysis for the nonlinear FE model was conducted in comparison with the nonlinear weir structure model modeled with the infinite boundary. Prior to the evaluation of the seismic fragility analysis results, a simple time-history analysis was performed in ABAQUS in order to understand the seismic complex behavior of the weir structures. Interestingly, it was found that the seismic behavior of the FE model was significantly influenced by the infinite boundary condition, due particularly to the energy radiation of the infinite foundation as subject to strong seismic ground motion. Also, the limit states corresponding to the evaluated probabilities of weir-structural failure were defined, and a generic methodology for seismic fragility determination was characterized. The data derived from this study will prove relevant to the performance-based earthquake engineering and safety assessment of flood defense structures such as dams, reservoirs, and weir structures.

Keywords: Safety assessment, Fragility, Earthquake, Infinite Foundation, Weir

1. Introduction

In recent years, the increasing number of seismic ground motions in South Korea has prompted seismic safety assessment of civil engineering infrastructures such as nuclear power plants, dams/weirs and bridges for mitigation of the risk of seismic damage incurred during or after earthquakes. Notwithstanding the lack of damage thus far, civil engineering infrastructures' vulnerability to potential seismic risk due to earthquake has been recognized. Weir structures have been constructed on South Korea's four major rivers as flood defense works, the specific functions of which are to generate electrical power, control flooding and distribute water, among others. The failure of flood defense structures such as sea walls, dams/weirs and retaining walls will cause catastrophe in upstream and downstream areas. The destructiveness of the 2008 Wenchuan Earthquake in China, for example, led to a rigorous seismic safety assessment of dam structures in that area [1]. Also, the Fukushima Daiichi Nuclear Power Plant (NPP), which had been designed to withstand extreme hazards, was flooded by the tsunami triggered by the magnitude-9.0 Great East Japan Earthquake on March, 2011, due to the failure of the sea wall [2]. The resulting Fukushima NPP accident taught the lesson that infrastructural systems capable of providing suitable levels of protection against natural hazards such as strong earthquakes and flooding had to be developed or improved. Such infrastructures, moreover, must remain operational and continue functioning satisfactorily during and after incidents. In order to mitigate the seismic risk to flood defense structures therefore, it is essential to achieve design reliability in terms of dam- and wear-structural failure probabilities. For this purpose, seismic fragility analysis has been utilized both to evaluate the stability of civil engineering structures and to measure damage incurred by structural systems subjected to seismic ground motions.

Hwang *et al.* [3] derived a numerical-analytical method for simulation of the seismic behavior of highway bridges and the construction of fragility curves. In constructing their analytical fragility curves using nonlinear time-history analyses, they considered the uncertainties with respect to modeling, ground motions and site conditions and defined the damage characteristics of highway bridge structures. Tekie and Ellingwood evaluated the seismic fragility of concrete gravity dams based on a rational safety assessment of an existing flood defense structure, the Bluestone Dam located on the New River in West Virginia, USA [4]. In order to determine the seismic fragility of the concrete gravity dam structures, they identified four structural failure modes: concrete material failure, foundation material failure, sliding at dam, and deflection of top of dam relative to heel. Schweckendiek and Kanning's exploratory study included a Bayesian-based probabilistic risk assessment of foundation failure from seepage [5]. Ju and Jung formulated seismic fragility in terms of weir structures, using 60 ground motions to represent the ground-motion uncertainty for near and distant earthquakes [6]. Yao *et al.*, using three-dimensional finite element analysis, investigated the effect of various ground motions on the seismic fragilities of a high-arch dam, focusing on the assessment of the opening and slipping of a contraction joint and displacement at the dam crest [1].

In this light, the primary objective of the present study was to investigate the effect of the seismic response of weir structures using fragility methodologies and two-dimensional finite element models. The investigation's focus was the Gangjeong-Goryeong multi-functional weir structure recently erected on the NakDong River in South Korea. For parametric study of modeling uncertainty, the Finite Element (FE) models performed in ABAQUS were classified into two different models, namely 1) the linear/nonlinear FE model with soil foundations and 2) the linear/nonlinear FE model with infinite foundations, so as to avoid reflection of seismic wave propagation in the soil foundations. Twenty (20) realistic ground-motion records representative of near and distant earthquakes, all motions scaled to different intensity levels, were selected for ground-motion uncertainty. Additionally, the seismic responses including the stresses, displacement, and sliding of the weir structure as subject to strong seismic ground motions were assessed. Finally, the seismic fragilities of the weir structure with the simple FE model and infinite foundation models were identified as well as quantified according to a comparison of seismic demand and weir-structural capacity. The main purpose of this study was to suggest weir-structural designs capable of withstanding natural hazards in the forms principally of seismic ground motions and flooding. Probabilistic risk assessment of social and economical losses upstream and downstream of weir-structural failure sites was not part this study.

2. Description of Weir Structure

The power (3000 kW)-generating and drinking-water-supplying hydro structure known as the Gangjeong-Goryeong weir system was constructed in 2011 in southeastern South Korea. The non-overflow section (with rising sector gates) and overflow structure of the weir structure are 120 *m* and 833.5 *m*, respectively. The maximum height of the reservoir is 19.50 *m*, and the water level allowing for weir overflow is 9.47 *m*. The reservoir's maximum flood elevation is 24.02 *m*, and the weir structure's maximum storage volume is 92.3 million m^3. The structure's soil foundation consists of three layers: 1) sand; 2) gravel-sand mixture; 3) rock. The peak ground acceleration of the design response spectrum is 0.154*g* in the horizontal direction, and the elastic modulus of weir structure concrete is 24 *MPa*. Additionally, an 18 *MPa* elastic modulus is used for the mass concrete system. Further details on the weir structure's material properties are provided in Table 1. Its schematic design is shown in Fig. 1 [6].

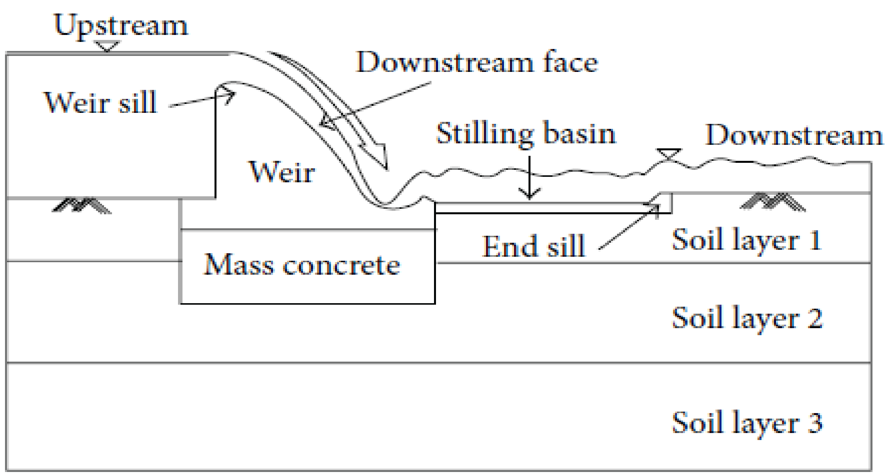


Fig. 1 – Schematic design of Gangjeong-Goryeong weir structure

Table 1 – Material properties of concrete gravity hydro complex structure

Structure	Elastic Modulus (MPa)	Poisson's ratio	Density (t/mm^3)
Weir body	24,000	0.167	2.4E-09
Mass concrete	18,000	0.167	2.4E-09
Steel	2000,000	0.25	7.85E-09
Soil foundation I	2	0.4	1.7E-09
Soil foundation II	25	0.4	1.9E-09
Soil foundation III	2,000	0.3	2.4E-09

3. FE Modeling of Weir Structure

In this study, the commercial software ABAQUS platform was used to generate a numerical FE model of the Gangjeong-Goryeong weir structure [7]. This simple 2D FE model, illustrated in Fig. 2, was implemented using

4-node bilinear quadrilateral elements. The model with soil-foundation systems has dimensions of 83.5 *m* (x-direction) by 58.114 *m* (y-direction), and the soil-foundation models were assumed to incorporate Mohr-Coulomb materials. For consideration of the hydrodynamic pressure acting on the weir face, Westergaard's added mass methodology [8] was applied. Also, for the effect of soil-structure interaction assumed to be horizontal, the coulomb friction law was applied to the interaction among the weir body, mass concrete and soil foundations.

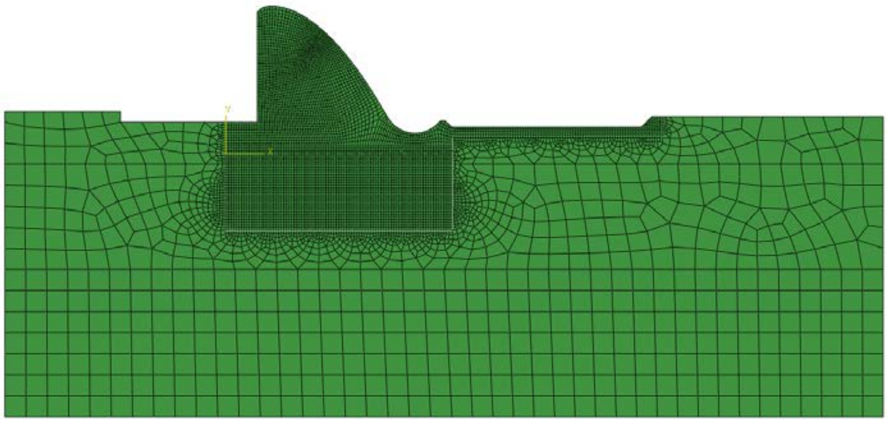


Fig. 2 – 2D FE model of weir structure

In order to avoid seismic wave reflection at the FE model mesh boundaries, it was necessary to consider the energy radiation in the infinite foundation. The absorbed energy or stress at those boundaries had to be characterized as well, because such time-domain-correspondent stress can be randomly generated. The nonlinear FE model with infinite foundations applied the same material properties as the above FE model, and CINPS4 in ABAQUS was conducted as the infinite foundations. This FE model with infinite foundations (IFE) is illustrated in Fig. 3, and the details with respect to elements and nodes are provided in Table 2. Further details in terms of the infinite foundation models in numerical analyses will be presented in a companion paper. However, the seismic behavior of the weir structure according to the FE models (with and without infinite foundations) will be discussed in the following sections.

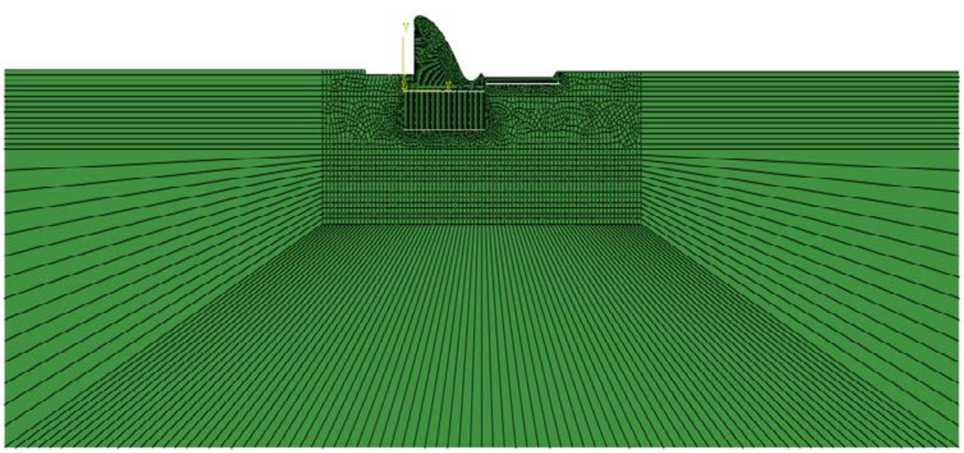


Fig. 3 – Infinite FE model of weir structure

Table 2 – Description of FE model of weir structure with infinite foundations

	Type	No. of Elements	No. of Nodes
Weir body	CPE4R	2929	3143
Mass concrete	CPE4R	2494	2610
Steel	T2D2	1529	1610
Soil layer I	CPE4R	384	477
Soil layer II	CPE4R	884	959
Soil layer III	CPE4R	294	310
Infinite element	CINPS4	142	143

4. Seismic Ground Motions

This study focused on the effect of ground-motion uncertainty on the seismic behavior of a weir structure. Therefore, the ensemble of seismic ground motions for the seismic fragility analysis was generated according to the normalized same Peak Ground Acceleration (PGA) level as an intensity measure. The 20 seismic ground motions selected on the basis of the epicentral distance and magnitudes over (M_w) 6.0 are listed in Table 3. Taking the near-source effect of uncharacteristic pulse-like behavior with large acceleration magnitudes [4] into consideration, the Group I motions selected were within 10 *km* of epicentral distance, whereas the Group II motions selected were beyond 10 *km* of epicentral distance.

5. Definition of Seismic Fragility

In recent years, seismic fragility has been widely utilized in the safety assessment of infrastructures subject to seismic ground motions [9-11]. Seismic fragility represents simply the conditional probability of system failure due to both material and ground-motion uncertainty. In other words, the concept of seismic fragility is the likelihood of damage-level exceedance at a given Intensity Measure (IM) as a representation of randomness such as analytical models, structural materials, and ground motions on weir structures. Hence, in risk or safety assessment, severe damage-measure levels of flood defense structures are of interest. The following Eq. (1) defines seismic fragility with respect to the Engineering Demand Parameters (EDPs) related to Limit States (LS) and Damage Measure (DM).

Table 3 – Twenty (20) selected ground motions

No.	Group	Event	Year	Station	Mag.	Dist. (Km)	PGA(g)
1	I	Tabas Iran	Sep. 16th 1978	Tabas	7.35	1.8	0.8358
2		Imperial Valley	Oct. 15th 1979	Sahop Casa Flores	6.53	9.6	0.2874
3		Irpinia Italy	Nov. 23rd 1980	Bagnoli Irpinip	6.9	8.1	0.1394
4		Irpinia Italy	Nov. 23rd 1980	Sturno	6.9	6.8	0.2506
5		Morgan Hill	Apr. 24th 1984	Coyote Lake Dam	6.19	0.2	0.7109
6		N. Palm Springs	July 08th 1986	Morongo Valley	6.06	3.7	0.2182

7		Superstition Hills	Nov. 24th 1987	Parachute Test Site	6.54	0.9	0.455
8		Chi-Chi Taiwan	Sep. 20th 1999	CHY006	7.62	9.8	0.1301
9		Kobe Japan	Jan. 16th 1995	Nishi Akashi	6.9	7.1	0.5093
10		NorthRidge	Jan. 17th 1994	Newhall	6.69	3.2	0.583
11	II	Irpinia Italy-01	Nov. 23rd 1980	Rionero In Vulture	6.9	29.8	0.1059
12		Imperial Valley	Oct. 15th 1979	Calipatria Fire STA	6.53	23.2	0.1282
13		Imperial Valley	Oct. 15th 1979	Cerro Prieto	6.53	15.2	0.1691
14		Imperial Valley	Oct. 15th 1979	Compuertas	6.53	13.5	0.1862
15		Imperial Valley	Oct. 15th 1979	Delta	6.53	22	0.2378
16		Imperial Valley	Oct. 15th 1979	Parachute Test Site	6.53	12.7	0.1113
17		Imperial Valley	Oct. 15th 1979	Superstation MTN Camera	6.53	24.6	0.1092
18		Chi-Chi Taiwan	Sep. 20th 1999	CHY025	7.62	19.1	0.1592
19		Kocaeli Turkey	June 28th 1999	Arcelik	7.51	10.6	0.2188
20		New Zealand 02	Mar. 02nd 1987	Matahina Dam	6.6	16.1	0.2555

$$P(\lambda) = \left[EDPs \geq DM \,\middle|\, IM = \lambda \right] \tag{1}$$

As the equation reflects, seismic fragility analysis is integrative, being associated with structural dynamics, solid mechanics, hydrology, probability and statistics, not to mention FE analysis; accordingly, seismic fragility is defined by a lognormal Cumulative Distribution Function [12]. Fragility analysis of weir structures, as schematized in Fig. 4 below, proceeds as follows:

1) Develop linear and nonlinear FE models of weir structure with and without infinite foundations to characterize its structural-dynamic behaviors under seismic wave propagations;
2) Determine, by Probabilistic Seismic Hazard Assessment (PSHA) based on Uniform Hazard Spectra (UHS), the earthquake input needed for calculation of seismic fragility;
3) Scale the seismic ground motions to the equivalent Peak Ground Acceleration (PGA) values;
4) Conduct linear and nonlinear time-history analyses using scaled seismic demand parameters;
5) Compare the seismic responses of the structures with the capacity or limit state;
6) Formulate the seismic fragility in respect of the probability of failure at a given intensity level.

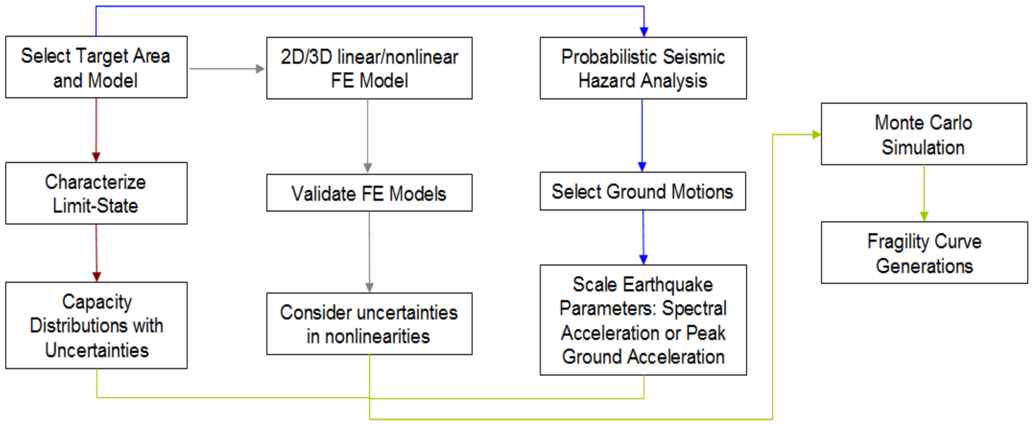


Fig. 4 – Flowchart of seismic fragility of flood defense structures

6. Seismic Fragility of Weir Structure

In the present study, prior to conducting the seismic fragility analysis of the weir structure, and in order to understand the complex behavior of that weir structure, the fundamental frequency (period=1.2388 sec) with a relatively high degree of effective mass participation (83%) was extracted using the simple linear elastic FE model, and the Rayleigh damping method with 5% damping ratio was used in the ABAQUS platform. Additionally, the relative displacements were assessed with both the nonlinear contact FE model (without infinite foundations) and the nonlinear contact IFE (see Fig. 5). Comparing the displacement histories, that of the weir structure with infinite foundations was more conservative than that of the simple FE model, due to the perfectly matched layers. Then, for the seismic fragility analysis, several limit states corresponding to failure modes were considered in this study: 1) tension- and compression-related material failure at the weir body and mass concrete (LS 1- compressive stress at weir body; LS 2-tensile stress at weir body; LS 3-compressive stress at mass concrete; LS 4-tensile stress at mass concrete); 2) sliding failure at weir structure/foundation interface (LS 5). More specifically, weir-structural sliding failure was classified into three damage measure states, namely 1) minor damage (3 *mm*), 2) moderate damage (13 *mm*) and 3) severe damage (153 *mm*), based on Tekie and Ellingwood [4].

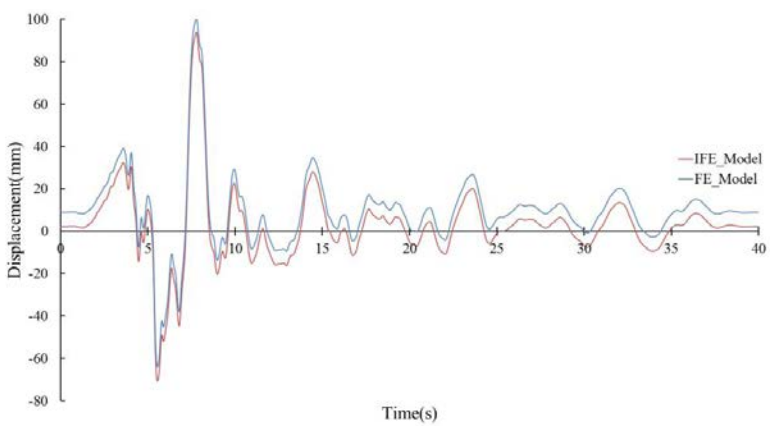
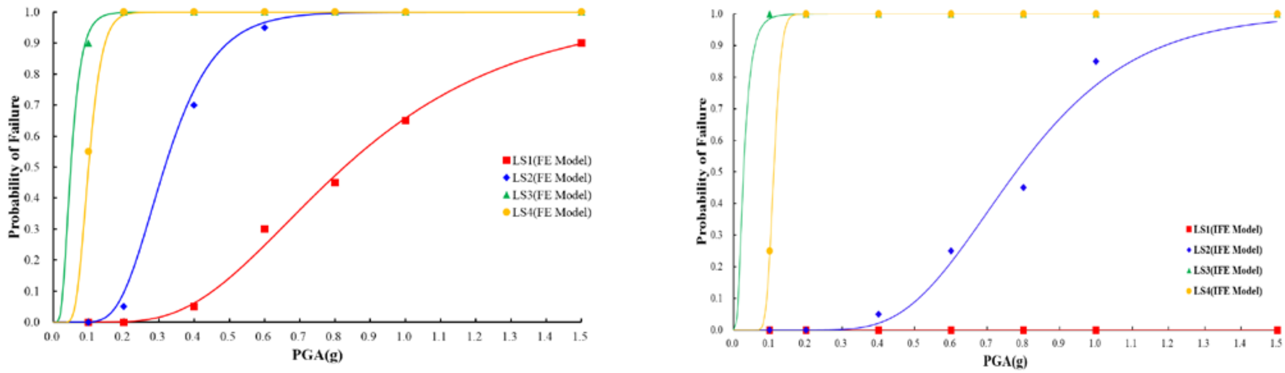


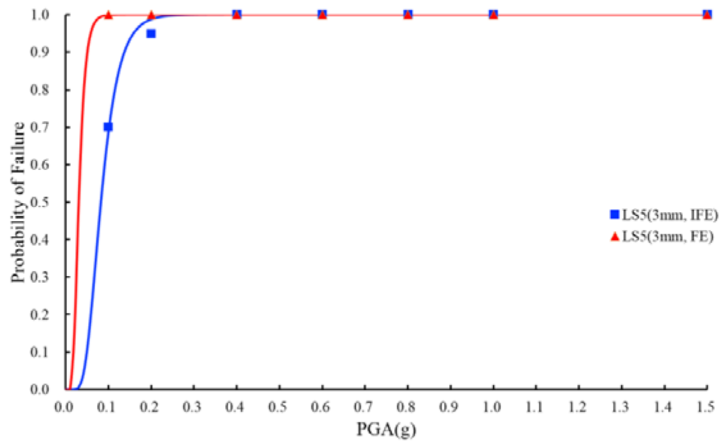
Fig. 5 – Displacement histories of weir structure

In the generation of seismic fragility curves for a weir structure subjected to strong ground motions, the present study applied a ground-motion ensemble of 20 selected earthquakes scaled to 0.1g, 0.2g, 0.4g, 0.6g, 0.8g, 1.0g, and 1.5g. The seismic fragility curves obtained by Monte Carlo Simulation (MCS) are plotted in terms of the nonlinear FE models with and without infinite foundations in Figs. 5 (a) and (b), respectively. They revealed, significantly, that the lognormal cumulative distribution function is suitable for fragility analysis.

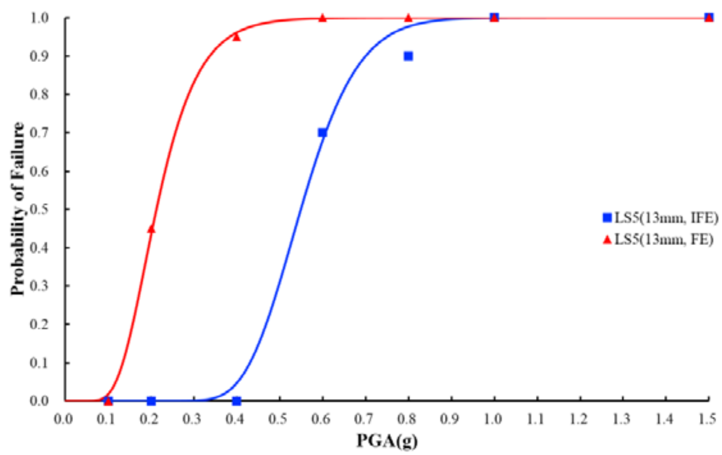


(a) FE model without infinite foundations (b) FE model with infinite foundations

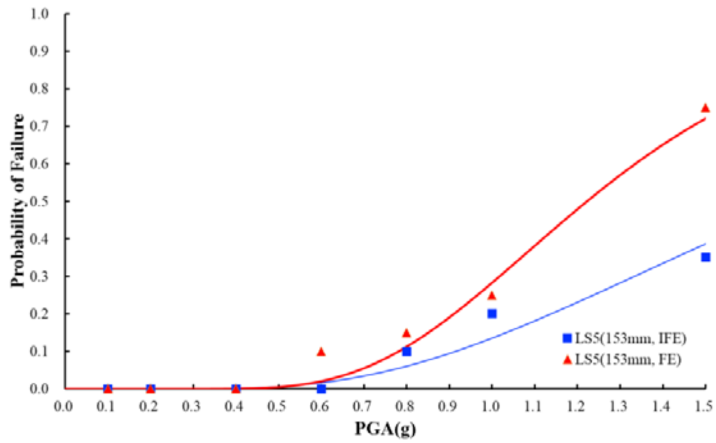
Fig. 6 – Seismic fragility curves of weir structure



(a) Minor damage level with respect to sliding effect of weir structure



(b) Moderate damage level with respect to sliding effect of weir structure



(c) Severe damage level with respect to sliding effect of weir structure

Fig. 7 – Seismic fragilities in terms of sliding failure of weir structure

As shown in Fig. 6, the probability of failure of the weir structure without infinite foundation models was much higher (about 30%) relative to that of the weir structure with infinite foundations at LS-1, the latter suffering no failure up to 1.5g. Also, the weir structure modeled without infinite foundations was more vulnerable than that modeled using the IFE at LS-2. However, the probability of failure of mass concrete systems was not significantly different between the simple FE model and IFE. On those bases, it was determined that the infinite foundation models were sensitive to the weir body structure, due to the absorbed energy or seismic wave reflection at the mesh boundaries. Furthermore, the seismic fragilities for the sliding effect at the weir structure/foundation interfaces depicted in Fig. 7 indicated that the probability of failure was shifted to the right side with increasing damage-measure level. As shown in Fig. 7(c), the seismic behavior of the weir structure at the severe damage level (sliding LS-153 *mm*) can be significantly affected by the nonlinearity at the interfaces, though the probability of system failure was less than 5% at PGA 0.6g. Also of note was the fact that the seismic fragility for sliding effects was more sensitive to the limit-state characterization.

7. Conclusion

The purpose of this research was to understand the seismic behavior of weir structures subjected to strong ground motions and to evaluate their seismic fragility using two different FE models — 1) the weir FE model without infinite foundations and 2) the weir FE model with infinite foundations — to avoid seismic wave reflection at the mesh boundaries. To that end, inelastic time-history analyses for 20 selected seismic ground motions were performed while representing the weir structures' various characteristics of randomness and uncertainty. Finally, five (5) different limit-state characteristics were defined to generate the fragilities. The results showed that the weir structure based on simple foundation models was more vulnerable than that based on infinite foundation models; for the fragility of sliding effects moreover, the two model types' trends with respect to the probability of failure were similar.

8. Acknowledgements

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