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FRAGILITY CURVES FOR ASSESSING THE SEISMIC VULNERABILITY OF GRAVITY QUAY WALLS

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Abstract. *Ports and marine structures have an important role in global transportation and world economy. Seismic vulnerability of marine structures can be evaluated through the probabilistic approaches in order to account for the uncertainties involved the structural capacity and seismic demand of such projects. The purpose of this study is to develop fragility curves for gravity quay walls. Numerical analyses are preliminarily carried out to capture the seismic displacement response of such structures to earthquake excitations. The recorded displacements of a previously conducted physical model test are employed to validate performance of the numerical modeling. A fully nonlinear soil model is used in the seismic analyses. Numerous seismic analyses are then performed using actual earthquake motions with various characteristics such as peak ground acceleration and predominant period. The results of numerical simulations are then analyzed through the probabilistic procedure to derive fragility curves for the gravity quay walls. Various performance criteria including serviceable, repairable, and near collapse damages are consequently determined by the fragility curves. Finally, results of this study are compared with the performance criteria recommended by the available performance-based guidelines of the port structures*

1 INTRODUCTION

Earthquakes have caused lots of damages to quay walls in the world. Thus, the seismic response of such structures is so important. The most important damages to gravity quay walls have been reported during the Hyogoken Nanbo 1995 earthquake in Japan. Seismic damages to gravity quay walls are typically evaluated in terms of horizontal displacement, settlement, and tilting. The main reason of these damages might be liquefaction occurrence in the soils located beneath and back of the quay walls or existence of infirm foundation under the wall. Zeng and Steedman [1] examined the seismic response of a wall restrained by sheet pile and concluded that increase in excess pore pressure significantly lowers the stiffness of soil and increases wall deformation. Zeng [2] studied behaviour of gravity quay wall through a centrifuge model. Zeng and Madabhushi [3] performed an effective stress numerical simulation using a finite element program known as SWANDYNE in order to examine the seismic response of quay walls. They compared results of numerical modeling with the measured outputs of centrifuge model and found good agreement. Cooke [4] simulated response of centrifuge tests of quay walls via the numerical models in FLAC software. Ichii [5] evaluated seismic vulnerability of Kobe port based on effective stress analysis and presented fragility curves for various conditions such as soil resistance parameters like N_{SPT} and also different $\frac{W}{H}$ ratio. Dakoulas and Gazetas [6] examined response of gravity quay wall in Rokko Island in kobe port using effective stress seismic analysis through Pastor model (1990). Dakoulas and Gazetas [7] used the elasto-plastic constitutive model of Pastor-Zienkiewicz and performed effective stress analysis. They examined the displacement, excess pore pressure, and plastic strain in a gravity quay wall of Kobe port. They found that by increasing the density of the weak soil under and back of quay wall, displacement and excess pore pressure will decrease. Chiou et al [8] developed fragility curves for a marine structure (pile supported wharf) in Taiwan. Yang et al [9] carried out numerical modeling of pile supported wharf via nonlinear time-history analysis under two ground motions suite in western United States. They draw the fragility curve for the wharf in two states, with or without crane, and also evaluated the damage probability for the structure at 2%, 10%, 50% return periods during 50 years.

In this paper, numerical analyses have been done to study the seismic response of a quay wall which was previously modeled in centrifuge. The centrifuge experiment was conducted in Cambridge University during the VELACS project in 1992. After the preliminary calibration of the numerical model and reasonable agreement between the numerical and experimental outputs, for the quay wall were subjected to 17 actual input motions. Fragility curves were obtained through the process of numerical outputs in order to study the seismic vulnerability of such structures within a probabilistic framework.

2 THE CENTRIFUGE MODEL

The case study is VELACS model number 11 test that is a gravity quay wall in centrifuge experiment conducted by Zeng [1] in Cambridge University. The physical model configuration and location of the instruments including excess pore pressure transducers, accelerometers, and LVDTs for horizontal displacement at the crest of wall are seen in Figure 1. The input seismic signal in the centrifuge test is shown in Figure 2 with the peak acceleration of 0.234g. The system soil is saturated Nevada sand with 60% relative. Liquefaction was occurred during the shaking and it produced horizontal displacement, settlement, and tilting in this structure.

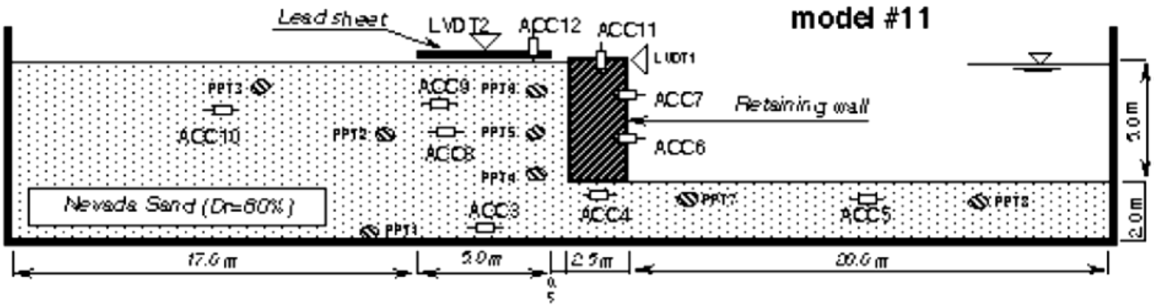


Figure 1. Configuration of the experimental model of the gravity quay wall.

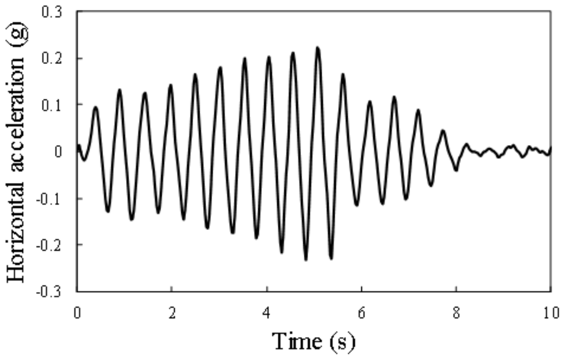


Figure 2. Time history of input acceleration for the centrifuge test at the base.

3 NUMERICAL ANALYSIS

In this study, the quay wall in VELACS model number 11 was modeled in FLAC 7 software which is a two dimensional finite difference program. The gravity block was made of aluminum alloy and so was considered as elastic material in modeling. A lead sheet with 0.1m thickness in prototype scale has been put on the soil surface as surcharge of the quay wall. The geometric features of the numerical model are shown in Figure 3. The properties of soil, caisson quay wall, and lead sheet are shown in Table 1. The friction angle between structure and soil is 17°. By the way, Finn model has been used to simulate liquefaction behavior while local damping was assumed 5%. Shear modules of the soil were estimated through the results of the resonant column tests conducted during the VELACS projects.

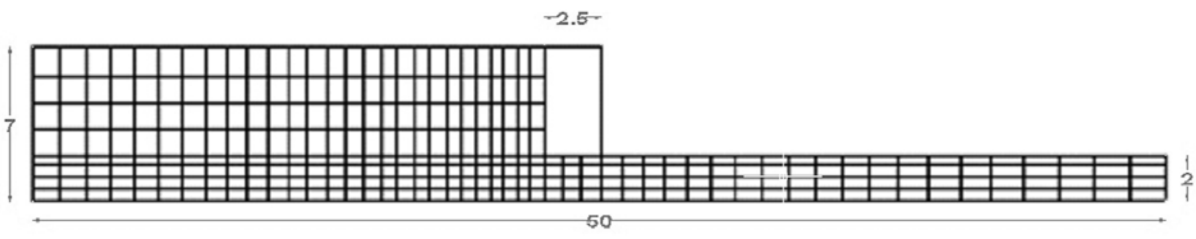


Figure 3. Numerical model mesh for the analysis

Material	Nevada Sand	Caisson Wall	Lead Sheet
Density (Kg/m^3)	1961	2700	11340
Shear modulus (KPa)	8.57E4	2.58E7	0.47E7
Poisson's ratio	0.4	0.334	0.431
Friction angle (°)	36	-	-

Table 1. Material parameters in this study

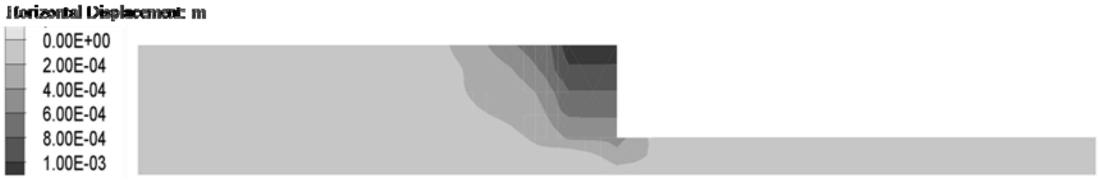
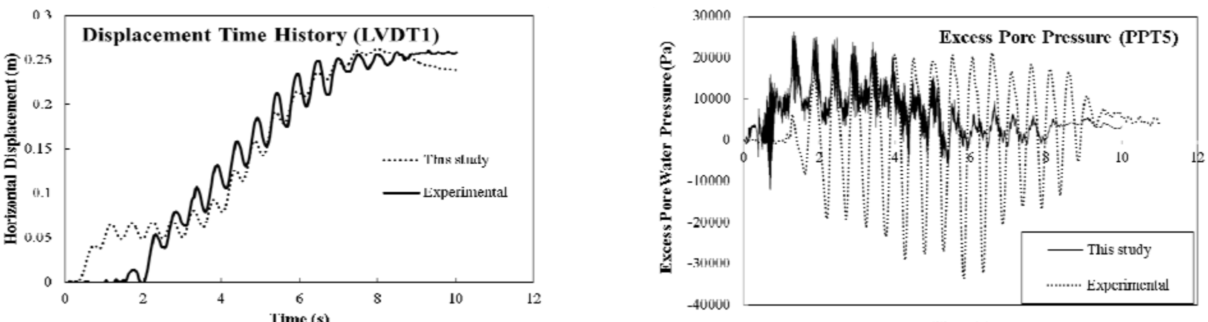


Figure 4. Contours of horizontal displacement of quay wall in static analysis

4 VERIFICATION OF PREDICTED AND MEASURED RESULT

Figure 4 shows contours of horizontal displacement of the system after the initial static equilibrium. This system was then subjected to the cyclic loading shown in Figure 2. The system response is attained via nonlinear time-history analysis and is compared with the centrifuge experiment outputs. Figures 5 (a and b) compares numerical and measured time histories of the horizontal displacement at the wall crest and the pore pressure buildup at PPT5 (see Figure 2). The maximum horizontal displacement at the top of the quay wall is predicted 0.25m via the numerical model, comparable with the measured value of 0.26m. Figure 6 also shows the time history of horizontal displacements on top of the wall which were predicted by the previous researchers. Figure 5 shows that the numerical model successfully predict the lateral deformation of quay wall while it fails to precisely predict the excess pore water pressure. For the current study, which deals with the displacement–based evaluation of seismic damage, the numerical model has sufficient preciseness for the subsequent numerical analyses.



(a) (b)

Figure 5. (a) Comparison of the numerical and experimental result in terms of the horizontal displacement on top of the wall (b) Comparison of numerical and experimental result in terms of the Excess pore pressure behind the wall

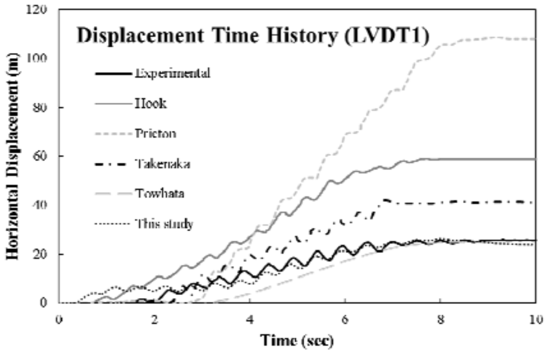


Figure 6. Comparison between the time history of horizontal displacement predicted by the previous researchers [12] with the prediction of the current study together with the experimental measurements

5 FRAGILITY CURVES

In the recent years many studies have been carried out for seismic behavior of a structural system. However, the performance improvement of a structural or geotechnical system is economically and scientifically critical

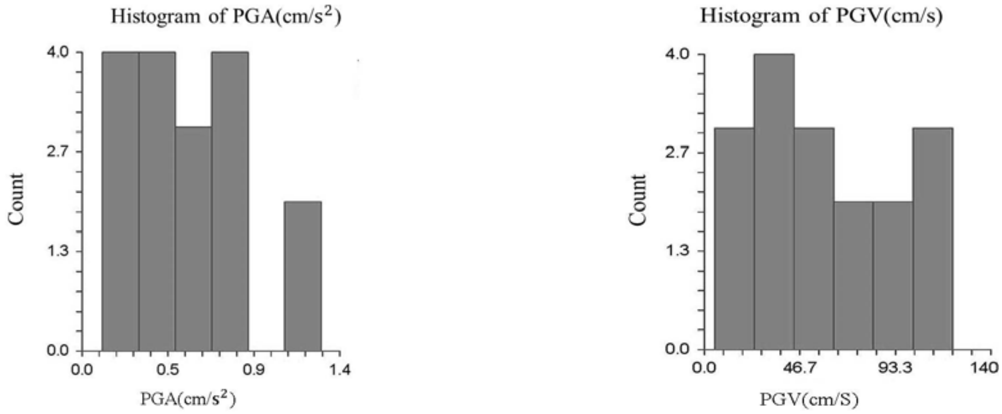
The seismic design of offshore structures might be better to quantify in probabilistic framework especially due to the existence of uncertainty in many cases. The probability of structural failure is a function of uncertainty in both structural capacity and earthquake demand. The capacity of a structure to withstand a load is a function of its geometry and material properties. Fragility curves can be used to evaluate the possibility of structural failures in large systems by considering the uncertainty in structural capacity and seismic demand

5.1 Ground motions

Fragility curves for seismic system is assessed based on numerical analysis in 17 input motions around the world as summarized in Table 2. These ground motions were recorded during 1971 to 1999 and some of which have occurred in the vicinity of offshore structures and caused lots of damages, such as Chi Chi earthquake in Taiwan port and Kobe earthquake in Kobe port. In the selection of these earthquakes it was tried to consider the important seismic parameters to be exist in a wide range. As can be seen in the histograms shown in Figure 7, seismic parameters such as PGA and PGV are reasonably distributed in the applicable range of these parameters. Similar strategy was considered for the other parameters such as T_p, T_m, I_a, and R_x. This type of selection for the input motions tries to reduce the possibility of considerable uncertainty in seismic demand. All of these input motions have been adopted from PEER (pacific earthquake engineering research center) strong motion database [12].

	Record	PGA(cm/s^2)	PGV(cm/s)	Magnitude
1	Anza 1980	0.131	5.1	4.9
2	Cape Mendocino 1992	0.229	7.1	7.1
3	Chi-Chi, Taiwan 1999	0.821	67	7.6
4	Chi-Chi, Taiwan 1999	0.311	34.2	7.6
5	Coyote Lake 1979	0.339	24.9	5.7
6	Duzce, Turkey 1999	0.348	60	7.1
7	Duzce, Turkey 1999	0.111	14.2	7.1
8	Erzincan, Turkey 1992	0.515	83.9	6.9
9	Imperial Valley 1979	0.775	45.9	6.5
10	Kobe 1995	0.616	120.7	6.9
11	Morgan Hill 1984	1.298	80.816	6.2
12	N. Palm Springs 1986	0.331	29.5	6
13	Northridge 1994	0.753	84.8	6.7
14	Northridge 1994	0.612	117.4	6.7
15	San Fernando 1971	0.268	25.9	6.6
16	San Fernando 1971	1.16	54.3	6.6
17	Tabas, Iran 1978	0.852	121.272	7.4

Table 2. Details of earthquake motions used in this study



(a) (b)

Figure 7. (a) Histogram of PGA for 17 record (b) Histogram of PGV for 17 record

5.2 Performance prediction

Performance-based design is a methodology born from the lessons learned from the previous earthquakes. In the conventional codes, seismic design is based on capacity to resist a design seismic force while structural performance is unknown when the capacity is exceeded. The International Navigation Association [10] proposed qualitative criteria for judging the degree of damage to a gravity quay wall based on the residual horizontal displacement at the top of the wall, as shown in Table 3. In this table, four damage states, I, II, III and IV, correspond to serviceable, repairable, near collapse, and collapse levels of a wharf structure, respectively. At the serviceable level, the structure should have minor or no structural damage and the structure should continue to function. At the repairable level, the structural damage is controllable and repairable. At the level of near collapse, the structural damage is extensive. At the level of collapse, the structural strength is completely lost. However, PIANC did not quantitatively provide the bound for each damage state. Damage states for every performance levels were defined based on PIANC guideline for fragility curves.

Level of damage	Degree I	Degree II	Degree III	Degree IV
Normalized residual horizontal displacement (d/H)	Less than 1.5%	1.5%-5%	5%-10%	Larger than 10%
Residual tilting towards the sea	Less than 3°	$3-5^\circ$	$5-8^\circ$	Larger than 8°

Table 3: Damage criteria for gravity quay wall by PIANC

5.3 System response

In this paper PGA is used as an intensity measure for analysis. For the fragility curves, all these input motions are scaled into 9 divisions from 0.1g to 0.9g based on PGA. In fact numerical analyses have been performed for more than numerous input motions. Responses obtained for each of the values can be assumed to be lognormally distributed with the probability density function (PDF) given by the following equation. λ and ζ are two parameters of the lognormal distribution of system responses which obtain from the mean (μ) and standard deviations (σ)values of the normal distribution of system response.

$$\lambda = \ln \mu - \frac{1}{2}\zeta^{2} \tag{1}$$

$$\zeta = \ln\left[1 + \delta^{2}\right] \tag{2}$$

$$\delta = \frac{\sigma}{\mu} \tag{3}$$

5.4 Fragility analysis

To estimate the probability of a system failure during a seismic risk, a fragility function $P\left[EDP > LS \middle| IM\right]$ is used. This probability function shows the response of the selected engineering demand parameters (EDP) exceed the structural state (LS) for a specific intensity of seismic excitation. Engineering demand parameters value can be achieved by using a set of input motions during nonlinear time history analysis of gravity quay wall. The amount of probability based on PGA for lognormal distribution of system response is calculated according to Eq 4.

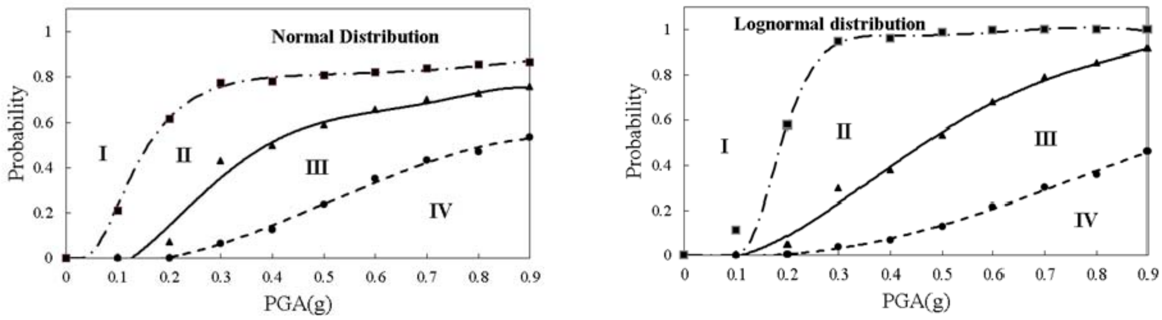
$$P\left[S > s \middle| PGA\right] = P\left[X > x_i \middle| PGA\right] = 1 - \phi\left[\frac{\ln(x_i) - \lambda}{\zeta}\right] \tag{4}$$

where $\phi(\circ)$ is the standard normal cumulative distribution function and x_i is the upper bound for x_i (i= I, II, III).

One can also obtain the probability for normal distribution of system responses based on PGA variation according to Eq 5.

$$P\left[S > s \middle| PGA\right] = P\left[X > x_i \middle| PGA\right] = 1 - \phi\left[\frac{x_i - \mu}{\sigma}\right] \tag{5}$$

Fragility curves of gravity quay wall in VELACS model number 11 is shown for two states of data by normal and lognormal distribution in Figure 8. Bounds of the damage states are selected based on the PIANC guideline. According to the statistical analyses of the results, it was found that the numerical results are more fit to the lognormal distribution. Therefore, fragility curves obtained from lognormal distribution is more appropriate. Figure 1(a) indicates that fragility curves achieved from normal distribution, probability of failure for PGA=0.5g in damage states I, II, III is 80%, 58.4%, 23.7% respectively. While Figure 1(b) indicates that fragility curves of system for lognormal distributions for PGA=0.5g in damage states I, II, III is 98.5%, 53.1%, 12.7% respectively.



(a) (b)

Figure 8: (a) fragility curve by normal distribution (b) fragility curve by lognormal distribution

6 CONCLUSION

In this paper Seismic risk assessment of a caisson gravity quay wall in centrifuge test is shown. Time-history analyses for this model were performed by nonlinear numerical methods. To evaluate the probability of the system, the seismic response of the system in terms of horizontal displacement at the top of the wall was evaluated. Fragility curves for the system was assessed based on the results of the numerical analyses by 17 input motions. Damage states for every performance levels defined based on PIANC guideline for fragility curves. According to the results and performance levels for three states including serviceable, repairable, and near collapse, the fragility curves for normal and lognormal data distributions have been presented. Since the distribution of results was more fit to lognormal distribution, the fragility curve of this distribution was found to be more reasonable. Damage probability for the performance levels of serviceable, repairable, and near collapse in PGA=0.5g have obtained 98.5%, 53.1%, 12.7%, respectively. When the peak ground acceleration increases to 0.9g, the probability of damages levels of serviceable, repairable, and near collapse increase to 99.9%, 91.2%, 46.4%, respectively.

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