IJE TRANSACTIONS C: Aspects Vol. 36 No. 06, (June 2023) 1075-1091

International Journal of Engineering

Journal Homepage: www.ije.ir

Evaluating Finn-Byrne Model in Liquefaction Analysis of Quay Wall and Cantilevered Retaining Wall Models

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P A P E R I N F O

Paper history:
Received 11 January 2023
Received in revised form 31 March 2023
Accepted 03 April 2023

Keywords:
Finn-Byrne Model
Liquefaction
Numerical Modeling
Quay Wall
Cantilevered Retaining Wall

A B S T R A C T

Two centrifuge tests on a quay wall and a cantilevered retaining wall with saturated granular backfills were simulated using Finn-Byrne model. Capabilities of Finn-Byrne model in liquefaction analysis of the quay wall and the cantilevered retaining wall were evaluated. The quay wall model subjected to a horizontal acceleration time history and the cantilevered retaining wall model subjected to a horizontal and a vertical time history. The constitutive model is a linear elastic – perfectly plastic model. Hooke's elasticity and Mohr-coulomb criterion for the yield surface were assumed for the backfill material behavior. The excess porewater pressure generation, acceleration, wall lateral displacement, lateral earth pressures, deformation pattern, and backfill settlements were monitored and compared with centrifuge tests' results. The results showed that the adopted model is suitable for stability and displacement analyses of the quay walls and cantilevered retaining walls. However, a care should be taken when assessing the backfill settlements and dynamic earth pressure behind the wall stem. The rest of the results showed a good agreement with the centrifuge tests' results.

doi: *10.5829/ije.2023.36.06c.06*

NOMENCLATURE

K	Bulk modulus (kPa)	K	Hydraulic conductivity (m/s)
G	Shear modulus	$\Delta\varepsilon_{vd}$	Incremental volume decrease
ϕ	Friction angle	ε_{vd}	Irrecoverable volume strain
C	Cohesion	γ	Cyclic shear strain
D_r	Relative density	C_1	Finn-Byrne model constant
γ	Unit weight (kN/m^3)	C_2	Finn-Byrne model constant
n	Porosity		

1. INTRODUCTION

Liquefaction is a catastrophic phenomenon in civil engineering that has become a major topic among engineers and reasearchers. Many researchers investigated the effect of liquefaction on geo-structures [1-3]. Some of them evaluated liquefaction potential [4-5] and showed how to mitigate that [6]. Predicting liquefaction via numerical models is a challenge and a complicated issue among geotechnical engineers. The Finn-Byrne model is a numerical model that introduces a formulation for porewater pressure generation in liquifiable soil media. This model was extensively employed for liquefaction analysis in different geo-structures such as earth dams, ground sites, element tests, tunnels, and sheet pile walls for stability and displacement analyses in research and practice. The use of the Finn-Byrne model in liquefaction analyses of cantilevered retaining walls and quay walls is rare in the literature. This paper shows the capability and limitations of the Finn-Byrne model in liquefaction analysis of cantilevered retaining walls and quay walls. For this purpose, two centrifuge tests on a cantilevered retaining wall and a quay wall with saturated granular backfills were numerically simulated via the Finn-Byrne model and the results compared with those of the centrifuge tests. Quay walls and cantilevered retaining walls are two types of the geo-structures that are common in practice

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Please cite this article as: A. Iraji, Evaluating Finn-Byrne Model in Liquefaction Analysis of Quay Wall and Cantilevered Retaining Wall Models, ***International Journal of Engineering, Transactions C: Aspects***, Vol. 36, No. 06, (2023), 1075-1091

and may subjected to seismic loads while the backfill is saturated. Because the backfill is usually granular and likely saturated, the liquefaction and excess porewater pressure in the backfill and under the foundation could affect the walls significantly and produce undesirable displacements. Quay walls are constructed in wharves. This type of structures should sustain wave loads in stormy weather and ice loads in winter. They should have durability against salty water and cold and hot temperatures. Cantilevered retaining walls are constructed to support soil mass in slopes and abutments of the bridges, soil mass in wharves, and backfills supporting railroads and highways. The backfill may be saturated when groundwater table rises because of heavy rains or tidal effects near the shorelines.

Byrne [7] introduced the Finn-Byrne model. This model is a modified and simple model to predict porewater pressure generation via coupling cyclic shear strain and volume strain. Byrne [7] examined the model via numerical modeling of cyclic triaxial tests with strain control. The tests had been reported by NRC [8]. The predicted porewater pressures were greater than the measured porewater pressures. The model was then evaluated by modeling cyclic load controlled undrained tests [9] and assessing liquefaction resistance that was satisfactory.

Many researchers used the Finn-Byrne model to perform liquefaction analysis of geo-structures. Vargas et al. [10] performed liquefaction analysis of an irregular site ground using the Finn-Byrne model. Sudevan et al. [11] performed an uplift numerical analysis of an underground structure in a saturated loose sand under a dynamic load using the Finn-Byrne model and compared results with results of a centrifuge test and those of a numerical simulation using Wang model conducted by Chian et al. [12]. The developed excess porewater pressure at around of the structure using the Finn-Byrne model was in good agreement with the centrifuge test and was more accurate than numerical results via the Wang model. They observed that the Finn-Byrne model overestimated the amount of the structure uplift by 25% and the Wang model underestimated that by 10 %. Masini and Rampello [13] assessed the behavior of large homogeneous earth dams under strong seismic loads using the Finn-Byrne model. They showed that neglecting excess porewater pressures during seismic loading can underestimate the dam settlements significantly. Chou et al. [14] used Finn and UBCSAND models and calibrated them for simplified liquefaction analysis. They emphasized that the simple analytical procedures with associated numerical simulations are mostly used for liquefaction mitigation solutions. They concluded that the Finn model cannot model stress-strain and stress paths observed in the laboratory tests. However, it provided reasonable excess porewater pressures. The relationship between Cyclic Resistance Ratio (CRR) predicted by the Finn model and the number of load cycles was reasonable. Banerjee et al. [15] performed numerical simulation of undrained cyclic triaxial tests using the Finn-Byrne and PM4SAND models. Both models predicted porewater pressure generation and cyclic stress paths well. However, the predicted stress-strain behavior was not good by both models. The Finn-Byrne model lacked in predicting the post-liquefaction behavior. Singh and Chatterjee [16] carried out liquefaction analysis of a cantilever sheet pile wall using the Finn-Byrne model. Various researchers conducted experimental methods to identify the failure modes and influential parameters on seismic response of quay walls [17-18] and some of them investigated only the seismic behavior [19-21]. Numerical seismic analysis and design of quay walls with saturated backfill is a challenge to engineers as it includes some of complexities such as proper prediction of excess porewater pressures and its effect on quay wall displacement and backfill deformation. Some researchers used advanced constitutive models to study seismic response of quay walls [22-24]. Madabhushi and Zeng [22] performed numerical analysis for centrifuge models on quay walls using advanced Pastor-Zienkiewicz Mark III model. They used fully coupled solid-fluid simulation. Yang et al. [23] performed numerical analysis of several centrifuge tests on quay wall under earthquake loads. They used a multiple-yield surface plasticity concept for the constitutive model. They conducted parametric study by changing permeability and soil relative density to assess amount of liquefied backfill and its effect on lateral spreading. They found that backfill permeability and dynamic properties are the parameters affecting the seismic behavior of quay walls significantly. Dakoulas and Gazetas [24] used advanced Pastor–Zienkiewicz elastoplastic constitutive model to simulate the seismic behavior of a quay wall from Rokko Island. They compared the numerical results with the results from Mononobe–Okabe theory. Abu Taiyab et al. [25] studied numerically and experimentally the effect of densification of loose sand around quay wall toe to mitigate damage to quay wall during earthquake loads. They used an elastoplastic multimechanism model called Hujeux model as the constitutive model. They found that the displacement of gravity quay wall is mainly because of shear strain occurred in the foundation. They also concluded that the densification of sand at the toe could prevent damage to quay wall considerably.

Wu [26] simulated the seismic behavior of San Fernando dam during the San Fernando Earthquake on 1971. A modified Martin-Finn-Seed model was used to predict the excess porewater pressure. Wu [26] predicted deformations and liquefied zones and compared them with the behavior observed in the Dam. The model underestimated vertical and horizontal displacements. However, the liquefactions zones were successfully assessed. Wang et al. [27] simulated an earth dam, a water front slope, and a rockfill dam under earthquake load to assess deformations and liquefaction using a bounding surface hypoplasticity model with nine model parameters. They predicted accelerations, wall deflections, and porewater pressure generations well. They underpredicted and overpredicted the vertical settlement and horizontal displacement of the crest, respectively. Dewoolkar et al. [28] employed program Diana-Swandyne II to simulate liquefaction behavior of the backfill in a cantilevered retaining wall model tested in a centrifuge apparatus. They used an advanced dynamic constitutive model, Pastor-Zienkiewicz Mark III model [29], for the backfill behavior. They underestimated the backfill settlement and predicted the excess porewater pressure well. Twelve parameters should be determined for the Pastor-Zienkiewicz Mark III model. The program Diana-Swandyne II was developed by Chan [30]. A bounding surface plasticity model with thirteen parameters was employed by Andrianopoulos et al. [31] to assess liquefaction in two projects. They evaluated liquefaction response in a Nevada sand layer with free-field condition in the first case. They assessed the performance of a rigid foundation on liquifiable Nevada sand in the second case. They investigated the excess porewater pressure in two cases. The settlement was assessed in the second case that was mostly underpredicted. Chakrabortty and Popescu [32] evaluated numerically the liquefaction potential in heterogeneous and homogenous soil layers. A simple frame of a structure was on the soil layers. The results showed that the excess porewater pressure in the heterogeneous soil layer was more than that of the homogenous layer. Kamai and Boulanger [33] used advanced bounding surface plasticity PM4SAND model to simulate liquefaction in a centrifuge test. The testing model consisted of two symmetrical slopes. There was an open channel between the symmetrical slopes. This model needs 11 parameters. They predicted lateral spreading, void redistribution, and dissipation patterns of the centrifuge test well.

Several researchers employed advanced dynamic elastoplastic constitutive models to assess liquefaction in different geo-structures. Validating these models and calibrating the large number of model parameters are time-consuming and boring works. Employing advanced constitutive models for numerical analysis of quay walls and cantilevered retaining walls under seismic loads may not be of interest to engineers as their use may be time-consuming and not cost-effective. For example, the Pastor-Zienkiewicz Mark III needs twelve parameters, the Hujeux model needs 22 parameters, and the PM4SAND needs 19 parameters. Determination of these number of parameters may be impractical for engineering problems. The present research uses and evaluates a simple constitutive model along with simple Finn-Byrne formulation to assess its capability for simulation of two centrifuge tests on a gravity quay wall and a cantilevered retaining wall during seismic loads. The constitutive model employed in this research and the Finn-Byrne formulation require 5 parameters, which all have physical interpretations: shear modulus, bulk modulus, cohesion, friction angle, and relative density. The numerical procedure deployed here in this study does not consider water hydrodynamic pressure for numerical modeling of the quay wall model for simplicity; however, this can be added in future works.

2. METHODOLOGY

Two centrifuge tests on a quay wall and a cantilevered retaining wall from the literature were selected to be simulated numerically via the Finn-Byrne model. The model XZ9 was selected for quay wall simulation and the model MMD12 was selected for cantilevered retaining wall simulation using the Finn-Byrne model. Nevada sand was used as the backfill for both of the quay wall and cantilevred retaining wall models. Some of the soil parameters were reported in the literature and the rest were calibrated via the reported element tests. FLAC was used to simulate both of the centrifuge tests numerically. A linear elastic – perfectly plastic constitutive model employed in the numerical models for both of the models. The elastic behavior and the yield surface obey Hook's elasticity and Mohr-Coulomb's law, respectively. The Finn-Byrne model was employed to predict the porewater pressure generation during seismic loads. Both of the centrifuge tests used absorbing materials at both sides of the models. Therefore, free field boundaries were used at both sides in the numerical models. Porewater pressures, accelerations, and the top lateral displacement of the quay wall model were assessed. Porewater pressures, accelerations, dynamic lateral earth pressures behind the wall, wall deflections, dynamic thrust and its point of action were evaluated and compared with those of the centrifuge test of the cantilevered retaining wall. These comparisons revealed abilities and shortcomings of the Finn-Byrne model in numerical simulation of the quay walls and cantilevered retaining walls at the similar conditions. The results of the numerical model of the cantilevered retaining wall model via the Finn-Byrne model compared with those of a numerical model conducted by an advanced and complicated constitutive model to show the effectiveness of the Finn-Byrne model compared to more complex models.

3. QUAY WALL

3. 1. Centrifuge Test of Quay Wall Zeng [17] reported three centrifuge tests of gravity quay walls in prototype scale performed at the Cambridge Geotechnical Centrifuge Center, i.e. XZ6, XZ7, and XZ9. The tests were conducted under acceleration of 80g. The relative densities of backfill for XZ6, XZ7, and XZ9 were 52.4 %, 25.8 % and 32.7 %, respectively. The backfill of XZ7 was dry and of the rest were saturated.

To investigate the effect of excess porewater pressure the model with lower relative density i.e. XZ9 was selected for simulation and assessment of the numerical model. Saturated loose granular material was used as the backfill. Figure 1 shows configuration of model XZ9 with the locations of porewater pressure transducers, accelerometers, and one LVDT. Figure 2 shows input motion history applied to the base of the model.

The soil used as the backfill was Nevada sand and its experimental tests were reported by Earth Technology Corporation [34]. Table 1 shows soil parameters used in the numerical model. All parameters except bulk and shear moduli were reported by Zeng [17] and Madabhushi and Zeng [22]. Bulk and shear moduli were calibrated using element tests on Nevada sand in VELACS project [34].

3. 2. Numerical Model The present work uses FLAC ITASCA to simulate the centrifuge model. FLAC uses finite difference method for numerical simulation and was used by many researchesrs in geotechnical numerical modeling [35, 36]. A linear elastic – perfectly plastic constitutive model was assumed for the soil medium. Hooke's elasticity was applied. The yield surface is Mohr-coulomb criterion and Finn-Byrne formulation was employed to predict pore pressure generation under dynamic loads. Finn-Byrne parameters are defined based on relative density.

Martin et al. [37] proposed a simple relationship between the volumetric strain and shear strain to simulate liquefaction. Finn and Byrne [38] developed this approach for drained condition. Finn et al. [39] developed this model by computing excess porewater pressure using volume constraints with an elastic rebound modulus. The tangent stiffness was dependent on shear strain and excess porewater pressure. Finn et al. [40] extensively developed this procedure. Then Byrne [7] introduced a simple method to calculate porewater pressure generation per Equation (1):

$$\frac{\Delta\varepsilon_{vd}}{\gamma} = C_1 \exp\left(-C_2\left(\frac{\varepsilon_{vd}}{\gamma}\right)\right) \tag{1}$$

$\Delta\varepsilon_{vd}$ is incremental volume decrease; ε_{vd} is irrecoverable volume strain; γ is cyclic shear strain; C_1 and C_2 are constants. This formulation shows an empirical relationship between the increment of volume decrease and the cyclic shear strain. When the volumetric strain, ε_{vd}, is accumulated the inceremental volumetric strain, $\Delta\varepsilon_{vd}$, decreases. The shear-induced volumetric strain increases with increasing number of cycles. When the primary effect i.e. irrecoverable volume strain occurs during a complete strain cycle, the secondary effect i.e. the porewater pressure increases.

Byrne [7] suggested the following equation to calculate C_1:

$$C_1 = 7600(D_r)^{-2.5} \tag{2}$$

where D_r is relative density and C_2 can be estimated as $\frac{0.4}{C_1}$ for many cases.

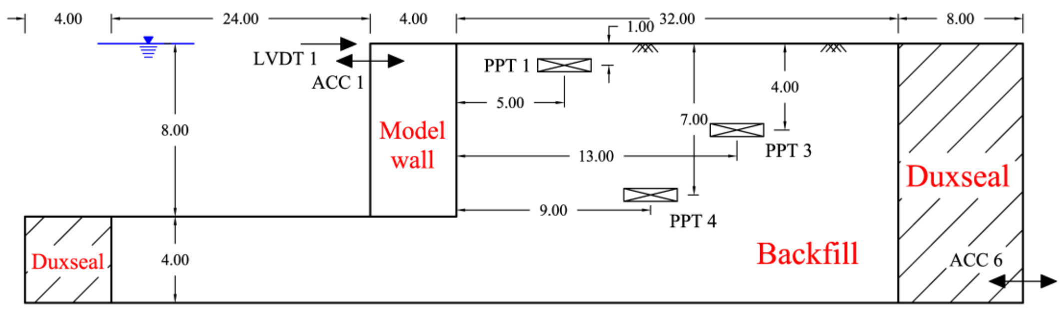


Figure 1. Model configuration of test XZ9 (dimensions in meter)

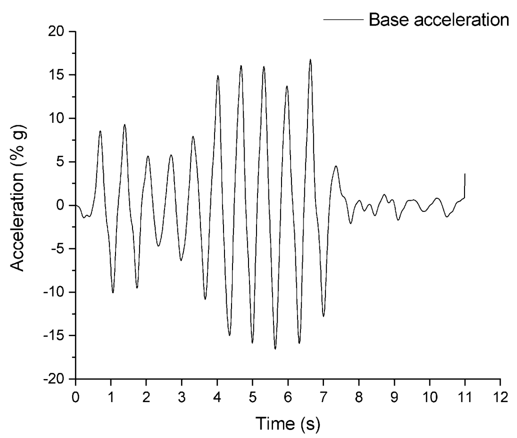


Figure 2. Acceleration history applied to model base

TABLE 1. Soil parameters

Bulk modulus (K) (kPa)	Shear modulus (G) (kPa)	Friction angle (ϕ) (°)	Cohesion (C) (kPa)	Relative density (D_r) (%)	Unit weight (γ) (kN/m^3)	Porosity (n)	Hydraulic conductivity (K) (m/s)
68993	7213	31.3	0.0	32.7	14.9	0.433	6.6×10^{-5}

The wall's material is an aluminum alloy and its behavior was assumed linear elastic. Aluminum Elasticity modulus was assumed 68.9 GPa and unit weight was assumed 27.0 kN/m^3. Because of the buoyancy effect submerged unit weight of the wall was applied in the numerical model, i.e. 17.0 kN/m^3 and bulk modulus of the water was assumed 2×10^6 kPa and 5% Rayleigh damping has been applied to the backfill.

The centrifuge model benefits Duxseal material at both sides to absorb reflecting waves (See Figure 1). The numerical model uses free field boundaries at both side boundaries. In the free-field boundaries, the waves traveling from the main structure to the boundaries are absorbed properly. To simulate quiet boundaries, viscous dashpots are used along the lateral boundaries. Unbalanced forces of the free-field boundary are applied to the main boundary. The free-field method provides a boundary that behaves as an infinite boundary. Figure 3 shows boundary condition and mesh of the numerical model.

3. 3. Results

Figure 4 shows lateral displacement of quay wall top predicted by the numerical model at LVDT1 and compares it with the centrifuge test results. The numerical model predicted reasonably the residual displacement and displacement history of centrifuge test response after the second 5.

Figures 5(a), 6(a), and 7(a) illustrate excess porewater pressures generated at measurement points PPT1, PPT3, and PPT4 during the seismic excitation. The numerical model shows an increasing trend in porewater pressures at points PPT3 and PPT4 and matches reasonably well with observed porewater pressures in the centrifuge test. While residual porewater pressures from numerical model matches with centrifuge tests, in early stages of excitation, a negative excess porewater pressure is observed in numerical model at PPT1. The volumetric strain history by the numerical model at PPT1 is illustrated in Figure 5(b). A significant expansive volumetric strain occurred after second 1.5, which is resulted in negative excess porewater pressure. Then the contractive volumetric strain increases the porewater pressure. The expansive volumetric strain at PPT1 in early stages of the excitation can be from sudden displacement of quay wall due to lack of hydrodynamic pressure consideration in the numerical model. PPT1 is at the closer distance from the quay wall compared to PPT3 and PPT4. Therefore, a little expansive volumetric strain is observed at PPT1 while PPT3 and PPT4 does not show considerable expansive strain. The contractive volumetric strain is assumed positive and the expansive volumetric strain is assumed negative.

Figure 8(a) shows settlement and tilting of the quay wall after application of seismic loading in the centrifuge test reported by Zeng [17]. This figure shows 1.16 m lateral displacement of the wall while wall top lateral displacement measured during centrifuge tests (see Figure 4) shows 0.8 m lateral displacement at the end of seismic loading. Zeng [17] has not justified this difference. It seems lateral displacement of 1.16 m occurred several seconds after the end of loading and lateral spreading was not stopped after the end of excitation. Therefore, quantitative comparison of Figures 8(a) and 8(b) might not be possible. However, Figure 8(b) illustrates failure mode of settlement and tilting from numerical study conducted herein which is very similar to what observed in the centrifuge test.

Figure 9 shows the potential failure surface in the backfill for the quay wall model. The potential failure surface was not reported in the centrifuge test. The angle between the potential failure surface and the vertical line is about 46° in the numerical model. However, this angle was reported for two other centrifuge models with similar geometry and similar source for the backfill. These angles were 40.4° and 55.7° for models XZ7 and XZ6, respectively.

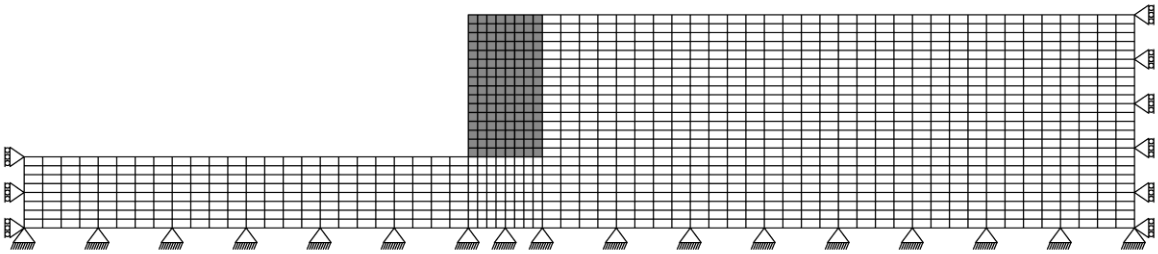


Figure 3. Boundary conditions and mesh for numerical model

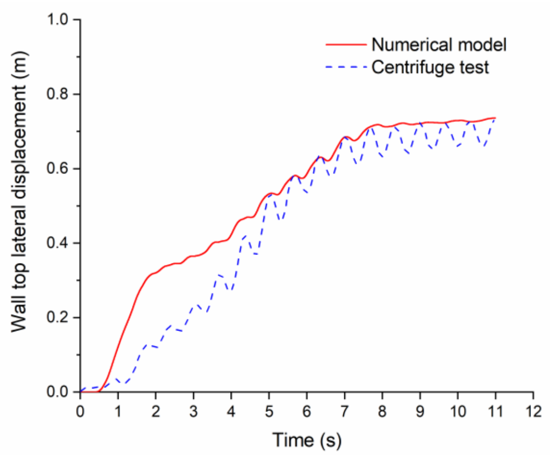


Figure 4. Top lateral displacement of quay wall at LVDT1, numerical versus centrifuge test results

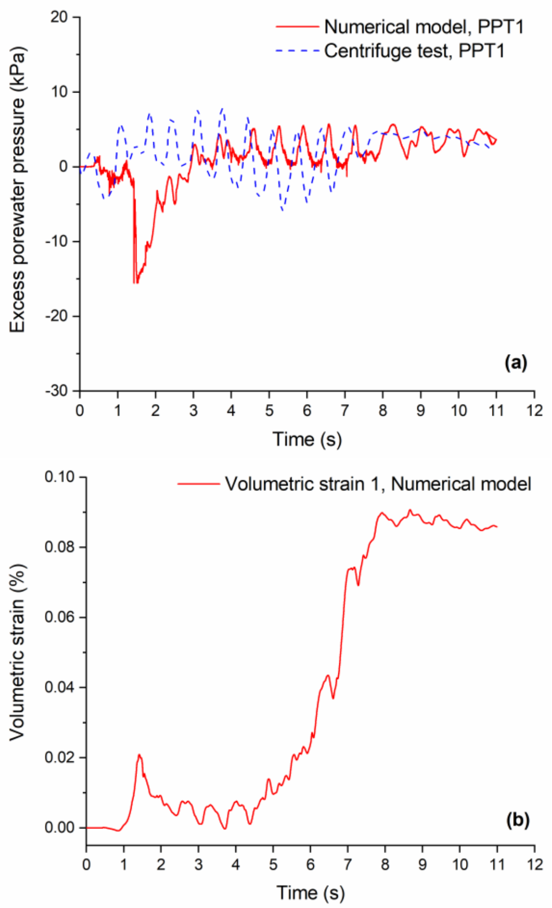


Figure 5. (a) Excess porewater pressure history at PPT1, numerical versus centrifuge test results, (b) Volumetric strain history from numerical model at PPT1

Figure 10 shows measured and predicted acceleration history via the Finn-Byrne model at ACC1. The predicted acceleration history is reasonable but is not as good as the excess porewater pressures and displacement history of the wall. It can be from this point that the hydrodynamic load of the water was not applied to the quay wall.

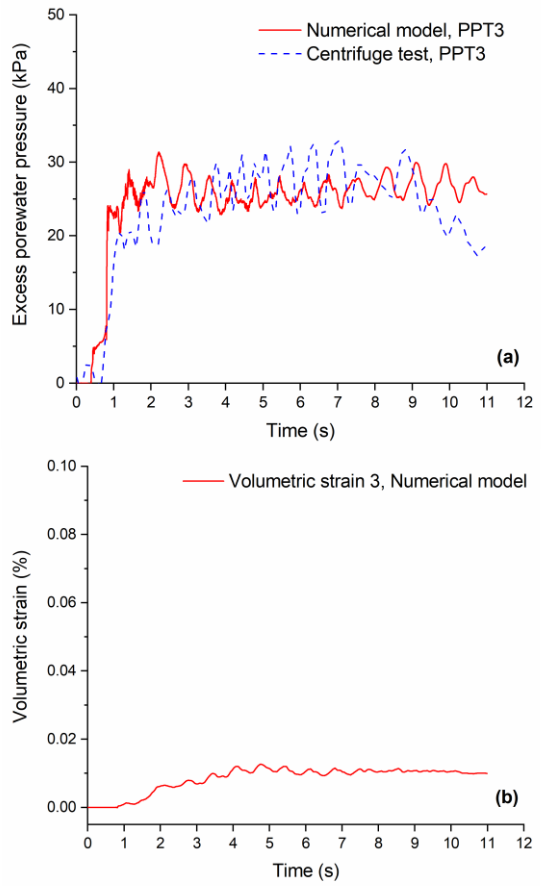
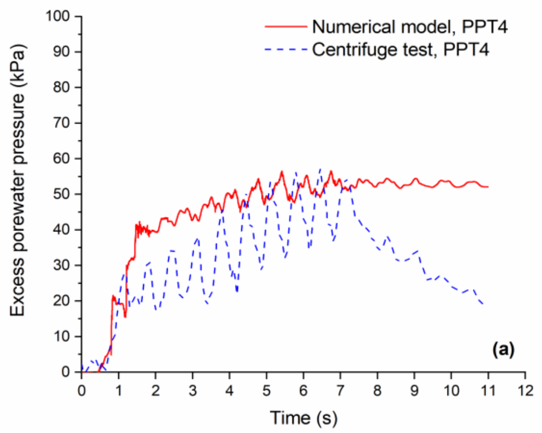


Figure 6. (a) Excess porewater pressure history at PPT3, numerical versus centrifuge test results, (b) Volumetric strain history from numerical model at PPT3



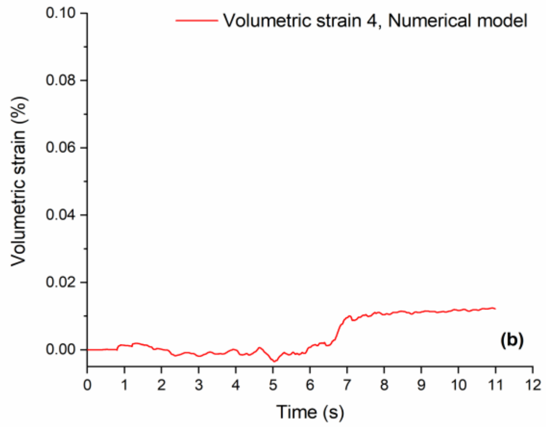


Figure 7. (a) Excess porewater pressure history at PPT4, numerical versus centrifuge test results, (b) Volumetric strain history from numerical model at PPT4

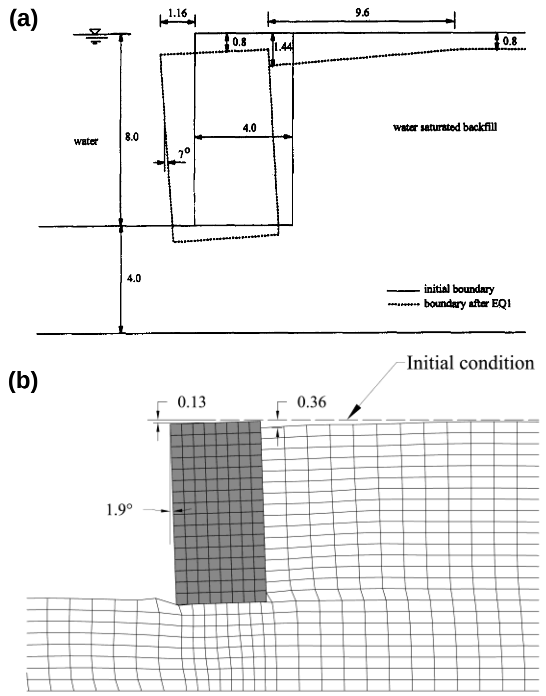


Figure 8. (a) Quay wall after application of seismic loading in centrifuge test (adapted from Zeng [17]) (b) Quay wall after application of seismic loading in numerical model

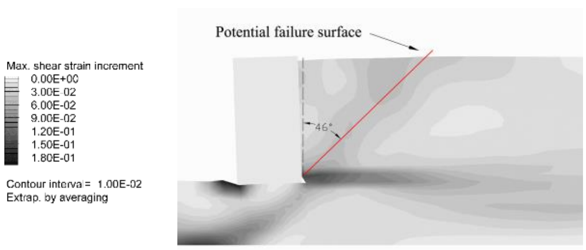


Figure 9. Potential failure surface in quay wall numerical model

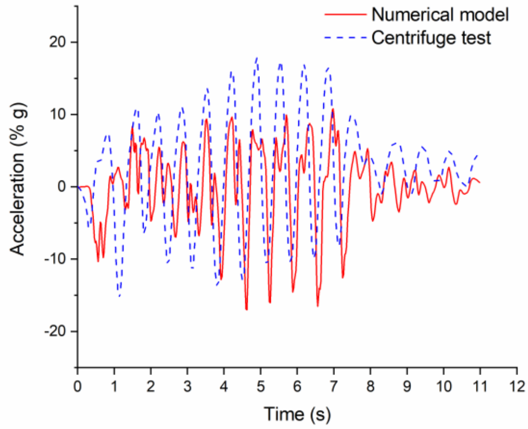


Figure 10. Measured and predicted acceleration history using Finn-Byrne model at ACC1

4. CANTILEVERED RETAINING WALL

4. 1. Centrifuge Test Dewoolkar et al. [41] carried out several centrifuge tests to evaluate seismic behavior of cantilevered retaining wall models with saturated and liquifiable backfill. A 400 g-ton centrifuge apparatus was used to perform the tests. The model MMD12 was used to simulate numerically via the Finn-Byrne model. The pore fluid used in the centrifuge tests was metolose water. Figure 11 shows configuration of the centrifuge test.

The backfill was Nevada sand No. 100. The relative density of the sand was 60%. Specific gravity was 2.67 and D_{50} was 0.1 mm. The minimum and maximum dry unit weights of the backfill were 13.87 and 17.33 kN/m^3, respectively. The wall was connected to the base rigidly. To make an absorbing boundary at the right side of the centrifuge model a duxseal panel was used at the behind of the backfill. T6061-T6 aluminum was used as the material for the wall. Poisson's ratio and young's modulus of the wall were 0.3 and 69×10^6 kPa, respectively. The density was 2787.7 kg/m^3. Scaling factor for the centrifuge model was 60. The model was subjected to an acceleration of 60 g in the centrifuge apparatus. 2% by weight metolose powder was mixed with deaired and distilled water and was used as the pore fluid in the soil medium. The metolose powder made the water 60 times more viscous than the water. Figure 12 shows horizontal and vertical input motions subjected to the testing model.

4. 2. Numerical Modeling FLAC was used for numerical simulation of the centrifuge test on the cantilevered retaining wall model. Yield surface was defined using the Mohr – Coulomb criterion. Soil behavior before the plastic deformation is linear elastic based on Hooke's elasticity. Excess porewater pressure in the numerical model was computed based on the Finn-

Byrne formulation. Table 2 shows soil parameters for the numerical model. The triaxial element tests conducted on Nevada sand were used to calibrate the soil parameters. The relative density of soil samples was 60 % [34]. Dewoolkar et al. [41] reported density of 2787.7 kg/m^3 and elasticity modulus of 69×10^6 kPa for the wall material which were used in the numerical model.

Elastic behavior was assumed for the wall stem. The scale factor of N was applied to reduce the soil hydraulic conductivity reported in Table 6. The reduced hydraulic conductivity was used in the numerical model to simulate the high viscosity of the fluid. Changing the fluid viscosity cannot be done in the numerical model. Density of the fluid used in the numerical model was 1000.0 kg/m^3. The bulk modulus of the fluid used in the numerical model was 2×10^6 kPa. Rayleigh damping of 5% was used in the numerical model. The friction angles between the backfill and the wall stem and between the backfill and the base were assumed 2/3Φ. (Φ = the backfill friction angle). Figure 13 illustrates mesh, geometry, and boundary conditions of the numerical model.

4. 3. Numerical Modeling By Dewoolkar et al. Dewoolkar et al. [28] simulated numerically the MMD 12 centrifuge test. They used an advanced dynamic elastic-plastic constitutive model, Pastor-Ziekniewicz Mark III model, for soil behavior. This model was developed using the generalized plasticity theory by

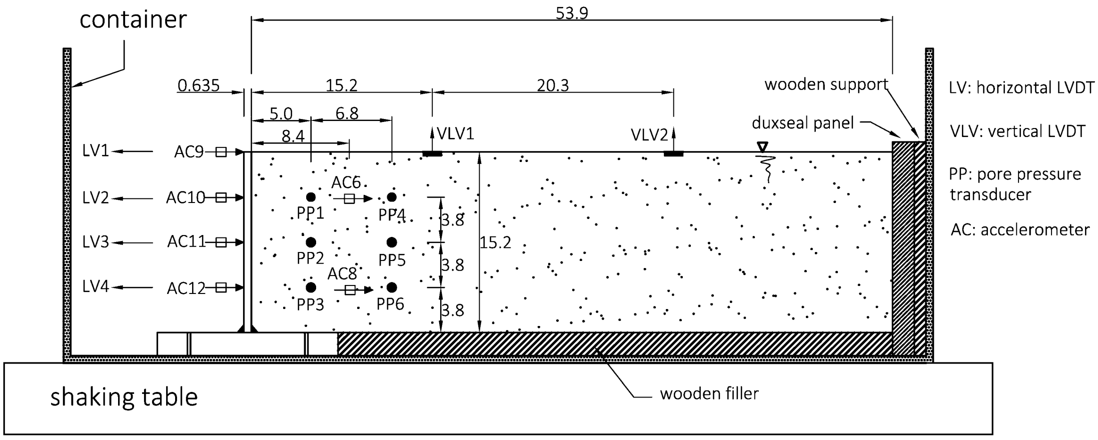
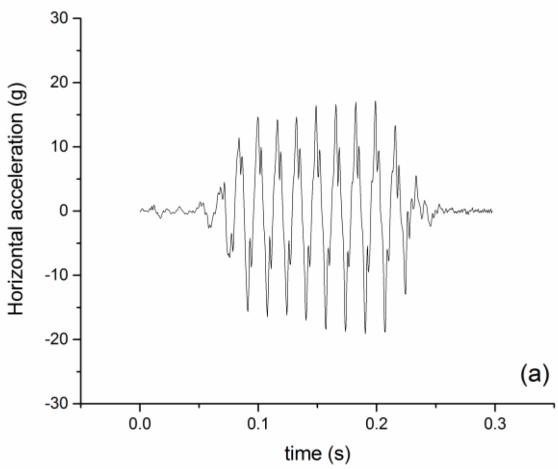


Figure 11. Configuration for centrifuge test of cantilevered retaining wall [41]

TABLE 2. Soil parameters of Nevada sand No. 100 used in numerical simulation

Bulk modulus (K) (kPa)	Shear modulus (G) (kPa)	Friction angle (ϕ) (°)	Cohesion (C) (kPa)	Relative density (D_r) (%)	Unit weight (γ) (kN/m^3)	Porosity (n)	Hydraulic conductivity (K) (m/s)
22027	5291	40.3	0.0	60	15.76	0.398	5.6×10^{-5}



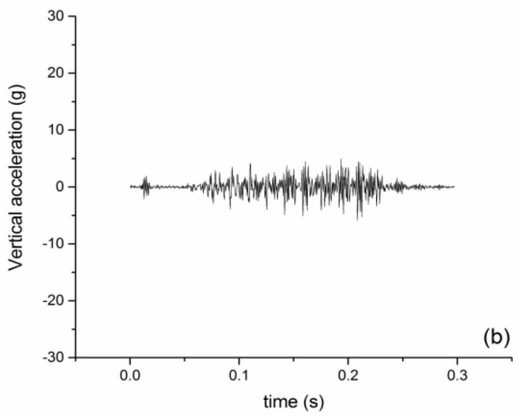


Figure 12. Acceleration histories subjected to numerical model a) horizontal acceleration b) vertical acceleration [41]

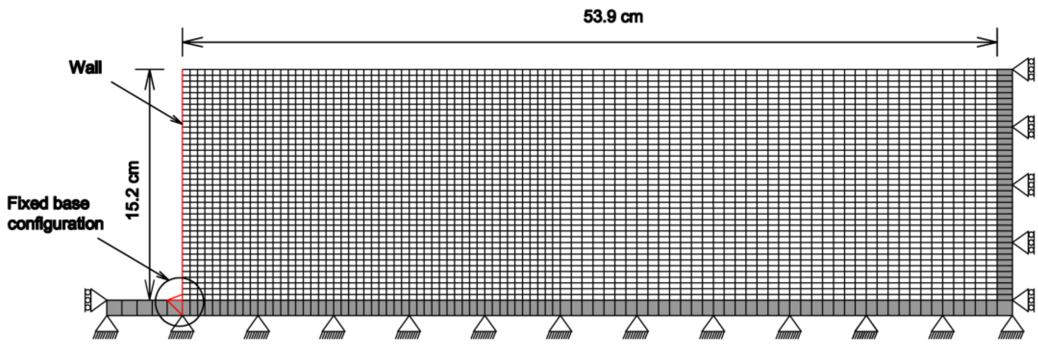


Figure 13. Mesh, geometry, and boundary conditions of the numerical model

pastor et al. [29]. The Pastor-Zienkiewicz mark III model is an advanced non-linear elastic - plastic soil model that was developed based on the generalized plasticity theory. This model can predict static and dynamic behavior of soil materials in drained and undrained conditions. This model is a well-established nonlinear model without explicit statement of yield and potential surfaces; instead, gradient vectors of the yield and potential surfaces are used. The elastic moduli in this model are dependent on mean effective stress. Several researchers used and developed this model for seismic analyses [42-44]. The adopted constitutive model in the present research uses linear elastic behavior while the Pastor-Zienkiewicz mark III model uses a non-linear elastic behavior. The size of the yield surface of the adopted model is constant while that changes in the Pastor-Zienkiewicz mark III model. The latter benefits from loading and unloading plastic moduli while the adopted model uses only a constant elastic modulus during loading and unloading. The loading and unloading tangent stiffness in the adopted model are the same while they are defined via different formulas in the Pastor-Zienkiewicz mark III model.

The Pastor-Zienkiewicz mark III model needs twelve parameters to be defined for fully dynamic analysis. The results of the numerical model using the Pastor-Ziekniewicz Mark III reported here and compared with the results of the Finn-Byrne model.

4. 4. Results Figures 14 and 15 show predicted and measured normalized porewater pressures at points PP1, PP2, PP3, PP4, PP5, and PP6. The porewater pressures were normalized to the atmospheric pressure of 101.325 kPa. Figure 14(b) compared the predicted porewater pressures by Pastor-Zienkiewicz Mark III model with those by Finn-Byrne model. As seen in all results from the Fin-Byrne model, the start of excess porewater pressures is at the beginning of the excitation whereas in the centrifuge test is at about 0.1 s. As mentioned, a constitutive model with linear elastic and perfectly plastic behavior was applied for the backfill.

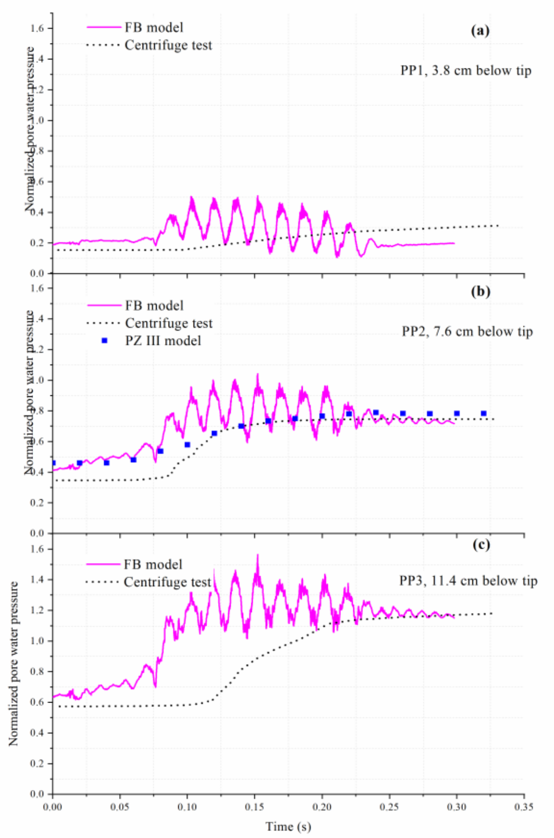


Figure 14. Comparison between measured and computed normalized porewater pressures using Finn-Byrne model and Pastor-Zienkiewicz Mark III model at PP1, PP2, and PP3

When the input motion is applied to the numerical model, the behavior of the backfill is elastic at low accelerations at the beginnings of the seismic load. No plastic deformation is observed at early stages and the waves are not damped and quickly conveyed to the above layers and produces excess porewater pressures at the beginnings of the seismic load. According to the general plastic behavior of the geo-materials, plastic deformation

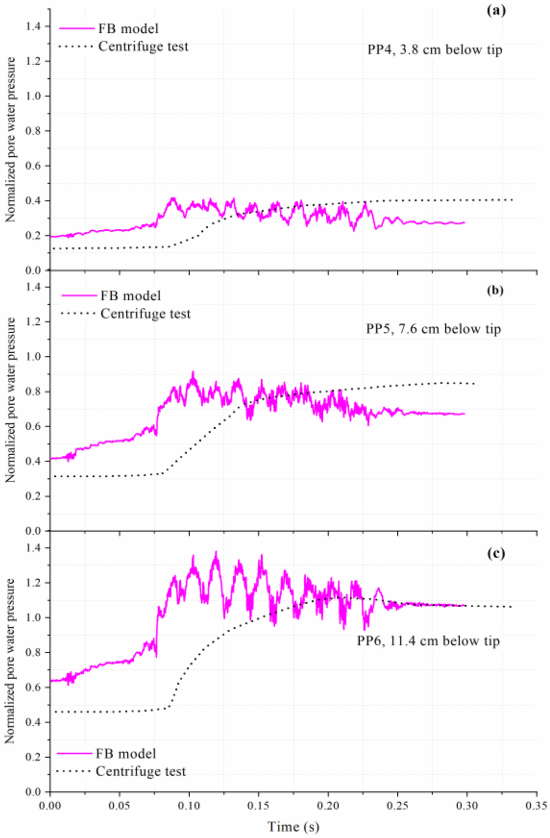


Figure 15. Comparison between measured and computed normalized porewater pressures using Finn-Byrne smodel at PP4, PP5, and PP6

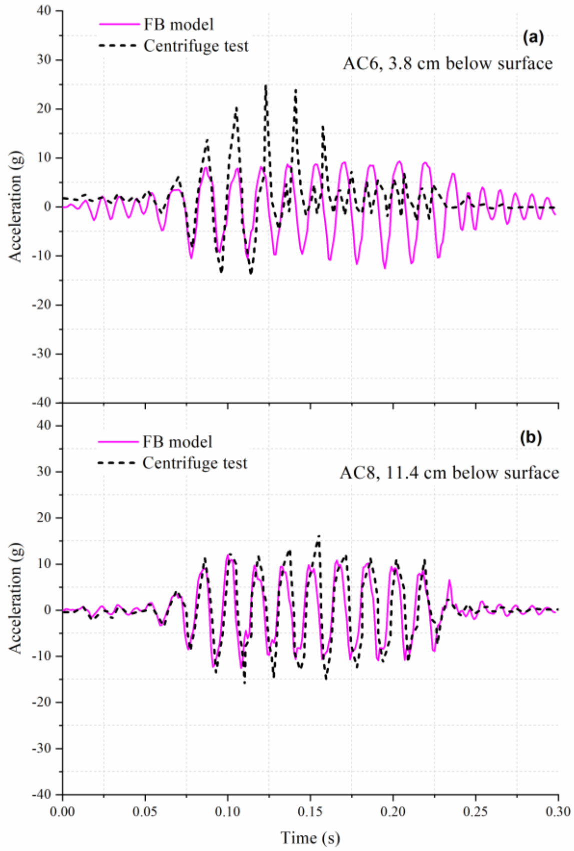


Figure 16. Comparison between measured and computed accelerations using Finn-Byrne model at AC6 and AC8 in the backfill

can be observed at the beginnings of the seismic loads even with low amplitudes of acceleration. Therefore, the accelerations at the early stages are damped in the bottom layers and the excess porewater pressure cannot be observed at the top layers.

A good agreement was observed between the residual porewater pressures of the numerical model and the centrifuge test at PP2, PP3, and PP6. The discrepancy between the residual normalized porewater pressures of the numerical model and the centrifuge test at PP1, PP4, and PP5 is about 0.1 to 0.15.

Figure 16 shows measured and predicted acceleration histories at AC6 and AC8 in the backfill. Figure 17 shows measured and predicted acceleration histories at AC9, AC10, AC11, and AC 12 on the wall stem. The agreement between the measured and computed accelerations is very good at AC8 in the backfill and at AC11 and AC12 on the wall stem. This is because that these points are at the lower levels in comparison to AC6, AC9, and AC10. The predicted accelerations at AC9 and AC10 are reasonable. The computed accelerations at AC6 are not in good agreement with the measured amplitudes. This can be attributed to the type of soil constitutive model adopted for the backfill.

Figures 18 and 19 show observed and predicted normalized lateral earth pressures behind the wall stem using the Finn-Byrne model. The lateral earth pressures at EP4, EP5, EP6, and EP10 were well predicted.

The lateral earth pressures at EP2 and EP3 were not predicted well. This can be attributed to mostly elastic response of the soil medium at locations close to the backfill surface behind the wall. Plastic deformation can occur from the start of the seismic load. The plastic deformation is one of the main reasons for damping in the soil medium. It seems that the elastic-perfectly plastic constitutive model and Finn-Byrne model cannot simulate damping in the soil medium and large oscillations are seen in the predicted values.

The predicted lateral earth pressures at EP8 and EP9 are a little more than those of the measured amounts, however, they are reasonable. This discrepancy can also be from the mostly elastic behavior of the medium at these locations.

Figure 20 show measured and predicted wall deflections normalized to the wall height at LV1, LV2, LV3, and LV4. The predicted wall deflections at the end of seismic load at all locations match reasonably with the observed deflections in the centrifuge test.

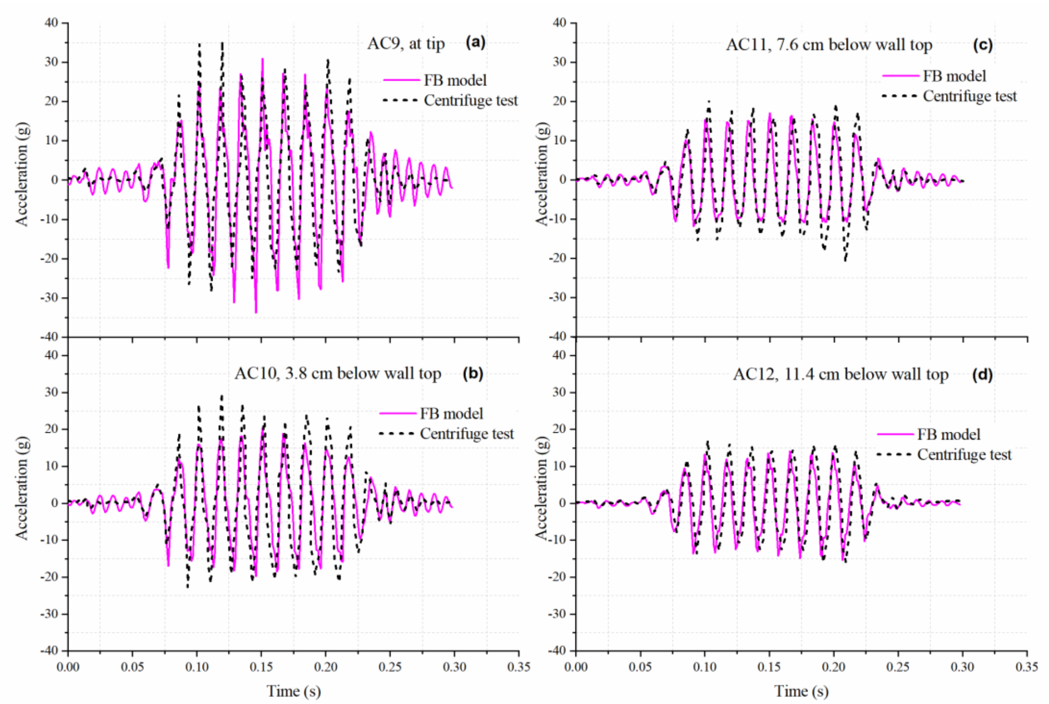


Figure 17. Comparison between measured and computed accelerations using Finn-Byrne model at AC9, AC10, AC11, and AC12 on the wall stem

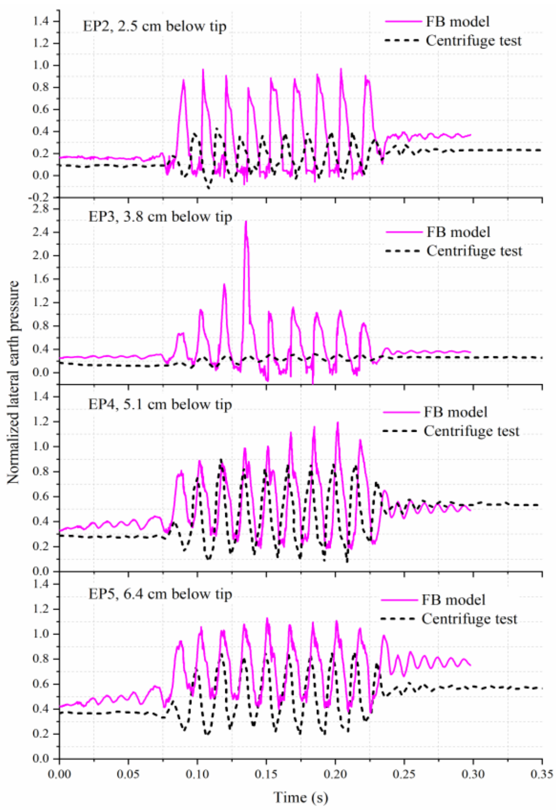


Figure 18. Measured and computed normalized lateral earth pressures using Finn-Byrne model at EP2, EP3, EP4, and EP5

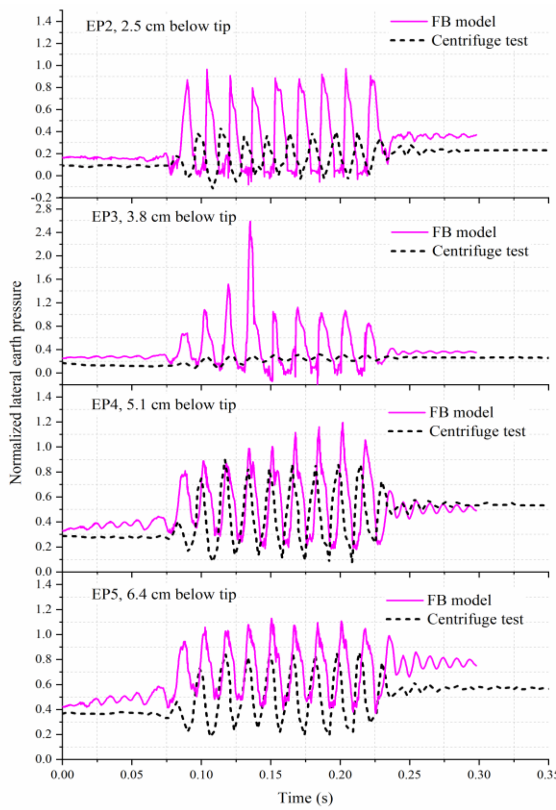


Figure 19. Measured and computed normalized lateral earth pressures using Finn-Byrne model at EP6, EP8, EP9, and EP10

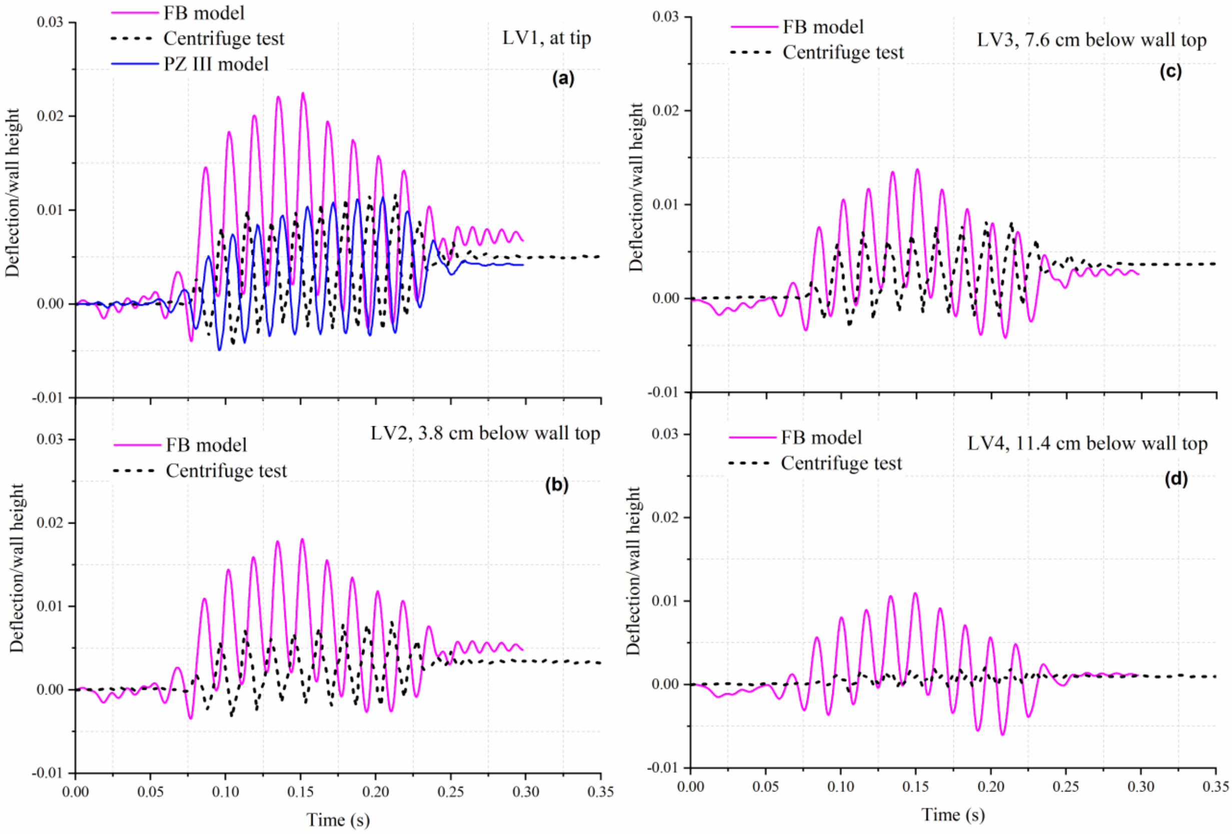


Figure 20. Measured and computed normalized wall deflections using Finn-Byrne model and Pastor-Zienkiewicz mark III model at LV1, LV2, LV3, and LV4

The predicted deflections during the seismic loading are greater than those of the centrifuge test. The deflection increases before 0.15 s and decreases after 0.15 s. This is because of the application of the baseline correction method. This method is used to correct raw input motions. In the seismic simulation of the structures, it may be observed a residual displacement or velocity at the end of seismic loads. This phenomenon is because of the nonzero output of the integral of the velocity time history. In this method, a low frequency velocity time history is added to the raw velocity time history. The integral of the corrected velocity time history, i.e., the final displacement should be zero. The residual predicted deflection matches reasonably with that of the centrifuge test. Figure 21 shows raw and corrected horizontal displacement time histories in the centrifuge test. The predicted deflections via Finn-Byrne model are consistent with the corrected horizontal time history in the centrifuge test (see Figures 20 and 21).

Figure 22 illustrates measured and predicted dynamic lateral thrust on the wall normalized to static thrust and point of action of the lateral total thrust. The point of action of the lateral total thrust was normalized to the wall height. The thrust increase due to the shaking at the end of excitation was 60 % in the centrifuge test while this increase in the numerical model was 75 %. The discrepancy is about 25% of the centrifuge test. The point of action of the total thrust was predicted well.

Figure 23 shows predicted and measured backfill settlement of the model. The numerical results were predicted via the Finn-Byrne model and Pastor-Zienkiewicz mark III model. As seen, the Finn-Byrne model cannot predict the settlement well. Dewoolkar et al. [28] predicted the backfill settlement via the Pastor-Zienkiewicz mark III model. As shown, the Finn-Byrne model is simple compared to the Pastor-Zienkiewicz mark III model and requires fewer parameters, however, the Finn-Byrne model predicted the settlement better than the Pastor-Zienkiewicz mark III model. The number of parameters for Pastor-Zienkiewicz mark III is twelve and for the Finn-Byrne model with elastic-perfectly plastic model is five. Note that Dewoolkar et al. [28] argued that the parameters of the Pastor-Zienkiewicz mark III model were calibrated so that the prediction of excess porewater pressures was better than the prediction of the settlement.

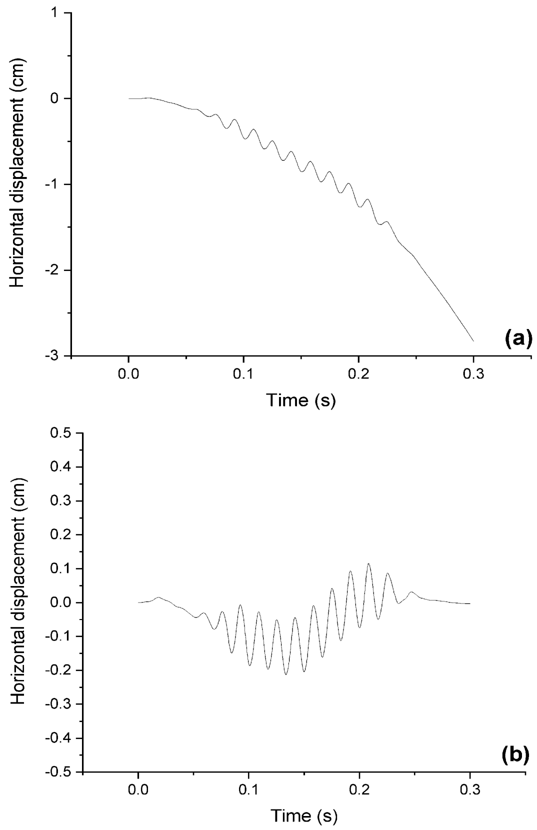


Figure 21. Time history of horizontal displacement in the centrifuge test (a) raw (b) corrected

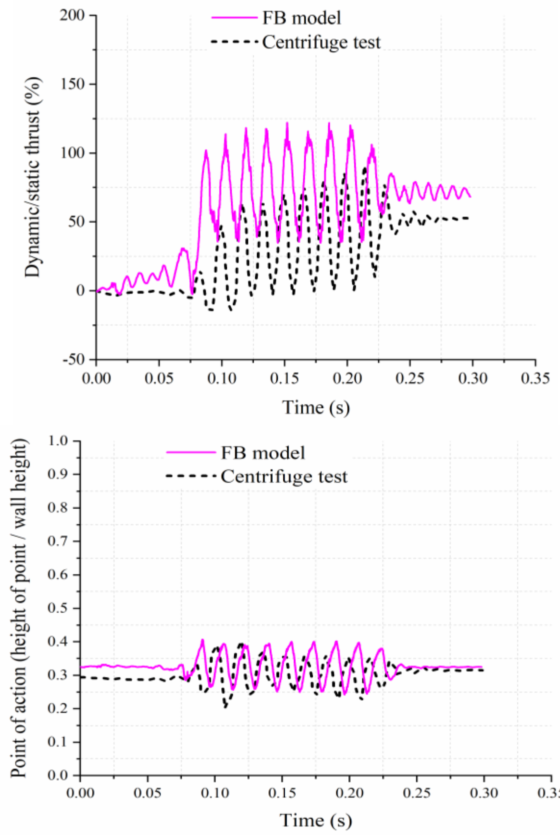


Figure 22. Measured and predicted dynamic thrust and its point of action (a) dynamic thrust normalized to static thrust (b) point of action for total thrust

5. DISCUSSION

Liquefaction in quay walls and cantilevered retaining walls may produce undesirable displacements and cause problems during the service period. Evaluating displacements and excess porewater pressures due to the liquefaction is yet a difficult job. The Finn-Byrne is a simple and relevant model to evaluate the liquefaction behavior. The literature review shows that the Finn-Byrne model was employed to assess the liquefaction behavior in different geo-structures. However, a few researches can be found to assess liquefaction behavior in quay walls and cantilevered retaining walls via the Finn-Byrne model. The present research was conducted to fill this research gap as the present topic is of interest to researchers and engineers.

The present work uses a constitutive model with linear elastic and perfectly plastic behavior with Mohr-Coulomb criterion for yield surface and the Finn-Byrne model for porewater pressure generation. Therefore, the behavior of the soil medium may be purely elastic at the beginnings of the seismic loads. The pure elastic behavior decreases material damping in the soil medium.

Based on the soil media behavior observations, the plastic behavior may be seen even under the light loads.

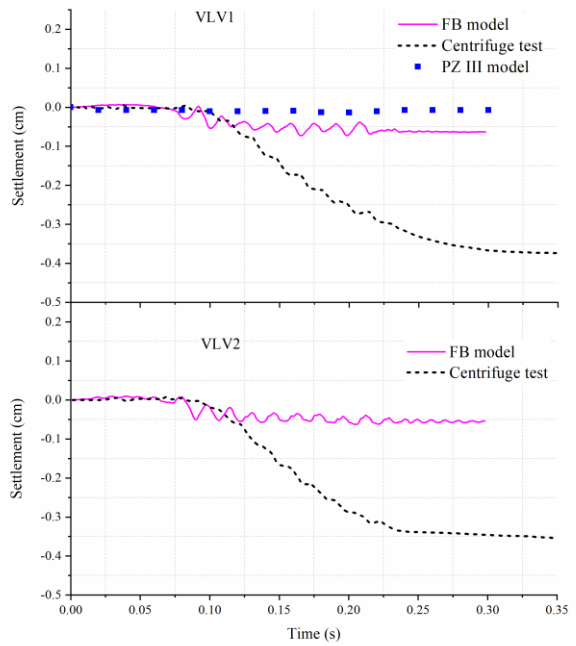


Figure 23. Measured versus predicted backfill settlement via Finn-Byrne model and Pastor-Zienkiewicz mark III model at VLV1 and VLV2

Therefore, the employed model cannot show material damping at the beginnings of the seismic excitations. The start of excess porewater pressures from the onset of the seismic excitation at the numerical model of cantilevered retaining wall is attributed to the shortcoming of the adopted constitutive model with the Finn-Byrne model in material damping simulation. When the plastic deformation starts in the numerical model, the maximum excess porewater pressures can be simulated well by the Finn-Byrne model for both of the quay wall and cantilevered retaining wall models. The residual porewater pressures were predicted well at some locations and not well for some other locations. The shortcoming of the Finn-Byrne model at the post-liquefaction stages were reported in the literature.

The predicted accelerations for both models are generally reasonable and the Finn-Byrne model may be recommended for acceleration amplification analysis in quay walls and cantilevered retaining walls with similar conditions.

The Finn-Byrne model computed the lateral displacement of the quay wall model satisfactorily. An abrupt displacement computed by the numerical model in the first couple of seconds is due to lack of water hydrodynamic pressures from passive side of the wall. The wall moves easier in the absence of water hydrodynamic pressure. The Finn-Byrne model computed the residual deflection of the cantilevered retaining wall stem reasonably. An increase in then a decrease in deflection was observed during the seismic load that was because of the influence of the baseline correction to the raw input motion. Large oscillations observed in the predicted deflection time history. This is due to the elastic-perfectly plastic behavior of the material and the weakness of the model in simulating the material damping.

The computed settlements via the Finn-Byrne model and Pastor-Zienkiewicz mark III were not satisfactory. The prediction of the Finn-Byrne model was better than that of the Pastor-Zienkiewicz mark III model. Note that the latter is more complicated than the Finn-Byrne model and needs more model parameters.

According to the results of the numerical model for the quay wall and cantilevered retaining wall models, the capability of the Finn-Byrne model in predicting excess porewater pressures in the backfill, accelerations, and lateral wall stem displacement is good. The results showed that the backfill settlements and dynamic earth pressures predicted via the numerical model did not match as well as the lateral displacements, excess pore water pressures, and accelerations with the results of the centrifuge test.

The poor prediction of the backfill settlement via the Finn-Byrne model shows that the adopted constitutive model cannot simulate volumetric strains properly during seismic loads. This is also true for the advanced Pastor-Zienkiewicz mark III model. The sudden drop in the predicted excess porewater pressure at PPT1 in the quay wall model shows the weakness of the adopted constitutive model in predicting volumetric strain. The Finn-Byrne model predicted the lateral displacement of the quay wall well and the residual lateral deflection of the cantilevered retaining wall reasonable.

When the values of the lateral displacements of the walls are desired, the Finn-Byrne model may be recommended for the displacement analysis. However, when the backfill settlement is desired, a care should be taken in using this model.

Since the Mohr-Coulomb criterion gives appropriate solutions in practical works and the prediction of the excess porewater pressures were satisfactory, therefore the adopted model is suitable for stability analysis of the quay walls and cantilevered retaining walls.

6. CONCLUSION

Several constitutive models were recently developed by researchers that can simulate liquefaction behavior. They include UBCSAND model, PM4SAND model, Pastor-Zienkiewicz Mark III model, a multiple-yield surface plasticity model, the elastoplastic multimechanism Hujeux model, the modified Martin-Finn-Seed model, and a bounding surface plasticity model. In the present research, the capability of the Finn-Byrne model in liquefaction analysis of a quay wall model and a cantilevered retaining wall model conducted by centrifuge apparatus was evaluated.

In the quay wall model, the predicted horizontal displacement of the wall, the excess porewater pressure in the backfill, and the backfill and wall deformation pattern via the Finn-Byrne model were well. The predicted acceleration history of the quay wall was reasonable.

In the cantilevered retaining wall model, the residual deflection of the cantilevered retaining wall was reasonable. However, large oscillations observed in the deflection history during the seismic load. This was attributed to the linear elastic – perfectly plastic behavior and weakness of this model in simulating appropriate material damping. The results showed the reasonable prediction of the excess porewater pressures, acceleration histories, and lateral displacement using the Finn-Byrne model. The computed dynamic earth pressures behind the wall stem were not as reasonable as the formers. In addition, the predicted backfill settlement was not satisfactory.

The results showed that the use of the Finn-Byrne model may be recommended to do stability and displacement analyses of the quay walls and cantilevered retaining walls with similar conditions.

Since geotechnical engineers often prefer simple numerical models to simulate and design the geo-structures, therefore, the simplicity of the Finn-Byrne model and its reasonable results reveals its effectiveness in the practice. When there is a need to check the excess porewater pressures, accelerations, and lateral displacements of the walls, the application of the Finn-Byrne model will be useful in research works. However, a care should be taken in assessing the backfill settlements and dynamic earth pressures behind the wall stem. This weakness indicates that the adopted constitutive model with the Finn-Byrne formulation cannot predict volumetric strain and dynamic active pressures during and after seismic loads. The future developments can be focused on the modification of the used constitutive model to address these problems while maintaining its simplicity.

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Persian Abstract

چکیده

دو آزمایش سانتریفیوژ بر روی یک دیوار وزنی حائل و یک دیوار طره‌ای حائل با خاک دانه‌ای و اشباع توسط مدل فین – بیرنه شبیه‌سازی شده است. توانایی مدل مذکور در آنالیز روانگرایی دیوارهای حائل وزنی و حائل طره‌ای ارزیابی شده است. دیوار حائل وزنی تحت یک رکورد زلزله افقی و دیوار حائل طره‌ای تحت همزمان دو رکورد زلزله افقی و قائم قرار گرفته است. مدل رفتاری استفاده شده برای خاک یک مدل با رفتار ارتجاعی خطی – پلاستیک کامل است. برای رفتار ارتجاعی از قانون هوک و برای سطح تسلیم از معیار موهر – کولمب استفاده شده است. پارامترهایی نظیر اضافه فشار آب حفره‌ای، شتاب دیوار و خاک، تغییرمکان افقی دیوارها، فشار جانبی لرزه‌ای ناشی از خاک در پشت دیوار، الگوی تغییرشکل خاک پشت دیوار و نشست در خاک پشت دیوار در مدلهای عددی بررسی شده و با نتایج آزمایشهای سانتریفیوژ مقایسه شدند. نتایج نشان دادند که مدل رفتاری به کار گرفته شده برای تحلیل پایداری و تغییرشکل لرزه‌ای دیوارهای حائل وزنی و حائل طره‌ای ابزاری مناسب است. با این حال به علت ضعف نسبی این مدل در ارزیابی نشست خاک پشت دیوار و فشار جانبی دینامیکی در پشت دیوار، بایستی احتیاطهای لازم در خصوص استفاده از این دو پارامتر به عمل آید. نتایج نشان دادند که مدل عددی مذکور در خصوص پیش‌بینی سایر پارامترهای دیوار عملکرد رضایت‌بخشی دارد.