4th International Conference on
Earthquake Geotechnical Engineering
June 25-28, 2007
Paper No. 1734

SIMULATION OF SEISMIC BEHAVIOUR OF GRAVITY QUAY WALL USING A GENERALIZED PLASTICITY MODEL

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ABSTRACT

Well-documented case histories of damage during earthquake to port structures made of gravity retaining quay walls during the period (1964-2003) show that the damage is often associated with significant deformation of liquefiable soil deposits. Gravity quay walls failures such as these have stimulated much progress in the development of a deformation-based design method for waterfront structures, such as the effective-stress analysis method. This method has been applied in this paper based on an elasto-plasiticity constitutive model developed by Pastor et al. (1990) with some minor modifications which is incorporated into a new FEM code called (UWLC). The simulations of the proposed model are compared with the published monotonic and cyclic tests for different types of sand under different initial densities, confining pressures and different Cyclic Stress Ratios (CSR). Further, the FEM code is evaluated by re-analysing the typical Port Island PC-1 caisson type quay wall which was damaged by the 1995 Hyogoken-Nanbu earthquake. The results are compared with observed results which were obtained from the Ministry of Transport, Japan (1997) which includes seaward displacement, tilting and settlement which are known as typical failure modes of quay walls due to earthquake. Afterwards, several quay walls systems in different conditions have been analysed in order to identify the effect of each factor on the residual deformation of gravity quay walls.

Keywords: quay wall, liquefaction, earthquake, harbour, effective stress, constitutive model

INTRODUCTION

Gravity quay walls are the most common type of construction for docks because of their durability, ease of construction and capacity to reach deep seabed levels. The design of gravity quay walls requires sufficient capacity for three design criteria; sliding, overturning and allowable bearing stress under the base of the wall. Although the design of gravity quay walls is reasonably well understood for static loads, analysis under seismic loads is still in being developed.

During strong ground shaking, the pore water pressure of cohesionless saturated soils builds up. This increase in pressure not only causes the lateral forces on the wall to increase (which may make the wall fail), but also reduces the effective stress of the soil which may result in liquefaction.

The occurrence of the liquefaction in back fill was the main reason for the damage from earthquakes to gravity quay walls in 1964 at Nigata Port (Hayashi, *et al.*, 1966) , also in 1993 at Kushiro-oki and in 1994 at Hokkaido Toho-oki (Sasajima, *et al.*, 2003) . In addition, liquefaction caused major damage to port facilities in Kobe, Japan, in the 1995 Hyogo-ken Nanbu earthquake. Moreover, observations of 24 marine structures in the earthquake in 1999 at Kocaeli, Turkey showed the backfill of quay walls also

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liquefied and the quay walls were displaced seaward (Sumer, *et al.*, 2002) . The same observations were reported in 1999 during the Chi Chi earthquake in Taiwan, (Chen & Hwang, 1999) . The seismic coefficient method containing Mononobe-Okabe's formula is usually used in the structural design of gravity type quay walls to resist earthquake damage but this design method does not take account of the liquefaction of backfill ground and foundations (Sasajima, *et al.*, 2003) .

Gravity quay walls failures such as these have stimulated much progress in the development of a deformation-based design method for waterfront structures. Much significant experimental and theoretical research work has been done on the subject (Dakoulas & Gazetas, 2005; Iai, 1998; Iai, *et al.*, 1998; Iai & Sugano, 2000; Ichii, *et al.*, 2000; Inagaki, *et al.*, 1996; Inoue, *et al.*, 2003; Nozu, *et al.*, 2004; Sugano, *et al.*, 1996) Many advanced constitutive models are being developed and much deformation effective stress analysis has been carried out which has analyzed and studied the recorded cases history of the behaviour of gravity quay walls founded in liquefiable soils. In this study, an elasto-plastisity constitutive model developed by Pastor, *et al.* (1990) that can simulate the monotonic and cyclic behaviour of cohesionless saturated soils has been slightly modified and used to investigate liquefiable soils with a wide range of confining pressures and relative densities. The modified constitutive model has also been incorporated in the FEM code UWLC (Forum 8, Co., 2006). In order to assess the predictive capability of the proposed model, numerical results have been compared with published experimental results for monotonic and cyclic tests using different types of sand under different initial densities, confining pressures and different Cyclic Stress Ratios (CSR).

In order to identify the effect of each factor on the residual deformation of gravity quay walls, several quay wall systems in different conditions have been investigated. The research focused on the analysis of a typical Port Island PC1 caisson type quay wall which was damaged by the 1995 Hyogoken-Nanbu earthquake. The results are compared with observed results which were obtained from the Ministry of Transport, Japan (1997) which include seaward displacement, tilting and settlement which are known as typical failure modes of quay walls due to earthquake.

DESCRIPTION OF THE CONSTITUTIVE MODEL AND METHOD OF ANALYSIS

Martin, *et al.*, (1975) proposed a model for excess pore water pressure build up based on the incremental volumetric strain in drained condition between the shear work and the excess pore water pressure. Zienkiewicz, *et al.*, (1985) proposed a simple model for transient soil loading in earthquake analysis, referred to as Mark I, which gives a reasonable prediction of pore pressure changes occurring during cyclic loading. Pastor, *et al.*, (1985) extended the bounding surface and generalized plasticity of model Mark I to reproduce the behaviour of sands under both static and transient loading and they refer to it as Mark II, in 1986, Pastor and Zienkiwicz described the P-Z Mark III model and modelled the behaviour of sand in a hierarchical manner and were able to capture important features of soil behaviour under cyclic loading, such as the progressive decrease in the stiffness of soil with increasing pore pressure, accumulation of deformation, stress-dilatancy and hysteretic loops. Finally, Pastor, *et al.* (1990) outline the theory of generalized plasticity in which yield and plastic potential surface need not be explicitly defined, and show how a very effective general model describing the behaviour of sands under monotonic or transient loading can be developed. This model is currently one of the simplest and yet one of the most effective ones describing the full range of behaviours such as the cyclic behaviour of soil. In the present study the constitutive model for sand which was developed by Pastor et al. (1990) was adopted, after some minor modifications.

Model evaluation

To test the validity of the proposed model, simulations are compared with the published monotonic and cyclic test of different types of sand under different confining pressure and initial relative densities. The experimental tests are those of Castro, (1969) and Toyota, *et al.*, (2004) .The model requires a total of 15 parameters which are shown in Table 1. Figure 1 shows a comparison between experimental and predicted results of the effective stress paths with different relative densities D_r =27%, D_r =44% and D_r =64% for undrained triaxial test of Banding sand (Castro, 1969) . Figure 2(a) compares the stress path during undrained cyclic triaxial test in Toyoura sand (Toyota*, et al.*, 2004) testing a sample with relative density D_r =38%, Figure 2(b) compares the Deviator stress verses Axial strain for the same sample. Figure 3(a) compares the stress path during undrained cyclic triaxial test in PI Masado sand (Moist placement) (Toyota, *et al.*, 2004) testing a sample with relative density e=0.58. Overall, the model seems capable of describing soil behaviours under monotonic and cyclic loading in undrained and drained conditions for a wide range of relative densities. From the figures it can be seen that the model gives excellent agreement with monotonic test results and while differences between the cyclic results do exist, the model captures the essential range of stress and strain behaviour. From these results, it is concluded that this soil model applied in the commercially available finite element code, UWLC, can be employed to assess liquefaction behaviour.

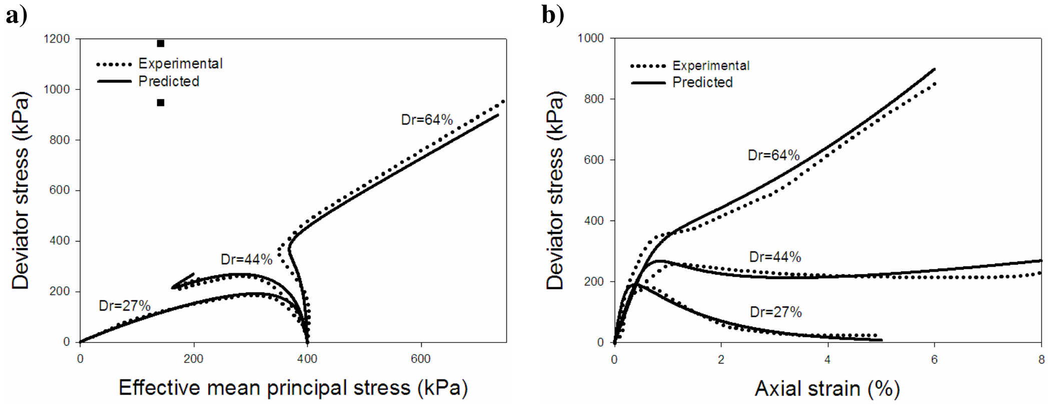

Figure 1. Simulation of experimental data from Castro, (1969) under monotonic undrained loading: a) stress path and b) stress strain relationship

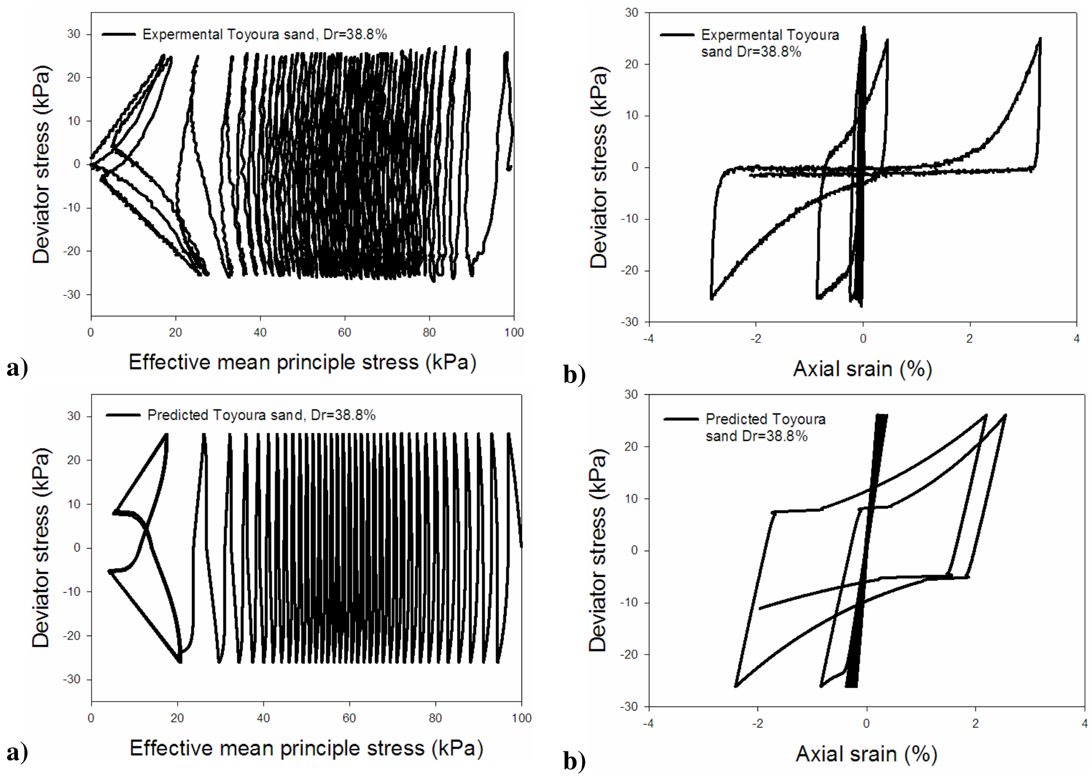

Figure 2. Simulations of experimental data from Toyota, et al., (2004) of Toyora sand (Moist placement), cyclic triaxial test: a) stress path and b) stress strain relationship

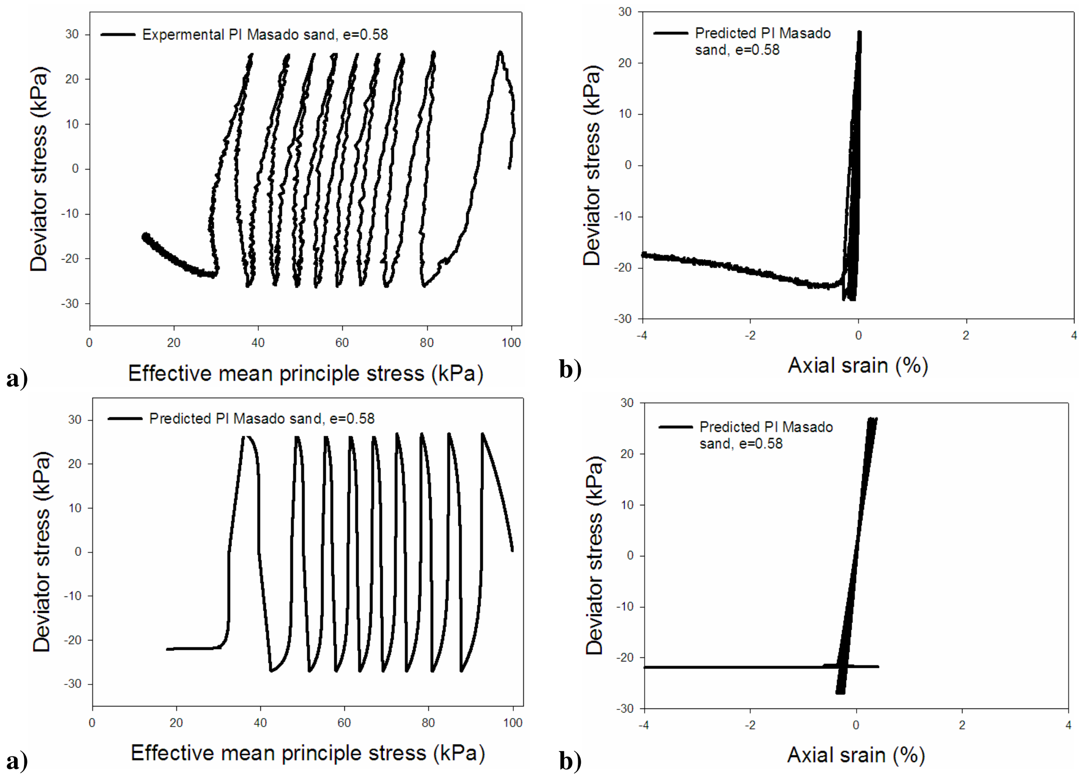

Figure 3. Simulation of experimental data from Toyota, et al., (2004) of PI Masado sand (Moist placement), cyclic triaxial test: a) stress path and b) stress strain relationship

Test	M_f	M_g	C	α_f	α_g	K_{ev0}	G_{es0}	m_v	m_s	β_0	β_1	H_0	H_{U0}	γ	γ_U	p'_0
Castro (27%)	**0.32**	**1.03**	**0.8**	**0.45**	**0.45**	**280**	**600**	**0.5**	**0.5**	**4.2**	**0.2**	**1000**	**0**	**0**	**0**	**400**
Castro (44%)	**0.545**	**1.32**	**0.9**	**0.45**	**0.45**	**195**	**300**	**0.5**	**0.5**	**4.2**	**0.2**	**560**	**0**	**0**	**0**	**400**
Castro (64%)	**0.495**	**1**	**0.9**	**0.45**	**0.45**	**160**	**250**	**0.5**	**0.5**	**4.2**	**0.2**	**1900**	**0**	**0**	**0**	**400**
Toyota (Toyora)	**0.77**	**1.42**	**0.85**	**0.45**	**0.45**	**220**	**140**	**0.5**	**0.5**	**6**	**0.3**	**520**	**19200**	**6**	**4.3**	**100**
Toyota (PI Masado)	**0.574**	**1.372**	**0.9**	**0.45**	**0.45**	**246**	**120**	**0.5**	**0.5**	**4.45**	**0.189**	**470**	**6950**	**6**	**4.3**	**100**

Table 1. Material parameters used in simulation of figures 1 – 3

Case history

Kobe Port is located in an area 6km long and 12km wide, there are two man-made islands, Port Island and Rokko Island. The soils used for landfill were excavated from the Rokko Mountains to the north west of Kobe city, this soil is called PI Masado (Ikuo, *et al.*, 1996; Inagaki, *et al.*, 1996; Sugano, *et al.*, 1996) . Figure 4 shows Port Island which is divided into two phases, the first, referred to as phase 1, was constructed on the northern half of Port Island between 1966 and 1981, the rest is referred to as phase 2; landfilling in the southern half of Port Island was almost complete when the Hyogoken-Nanbu earthquake hit Kobe City in January 1995 (Toyota, *et al.*, 1996) . Kobe Port had significant ground subsidence as a result of liquefaction during the earthquake; the extent of liquefaction was intense on Port Island and over 250 caissons type quay walls were damaged there with a repair cost exceeding US$11 billion (Ishihara, 1997) . Those walls were constructed on loose saturated decomposed granite which had been used to replace the alluvial clay layer in order to attain the required bearing capacity for the foundations. The typical types of damage observed after the earthquake were: seaward displacement, approximately 5m maximum and approximately 3m average; the walls also settled approximately 1 to 2m and tilted approximately 4 degrees (Ichii, 2004) . Caisson type quay walls in Kobe Port including Port Island and Rokko Island were designed using the pseudo-static method, with limit equilibrium mechanics based on the Mononobe-Okabe method developed by Okabe, (1924) and Mononobe & Matsuo, (1929) using horizontal seismic coefficients ranging from 0.1 to 0.15 (Inagaki, *et al.*, 1996) .

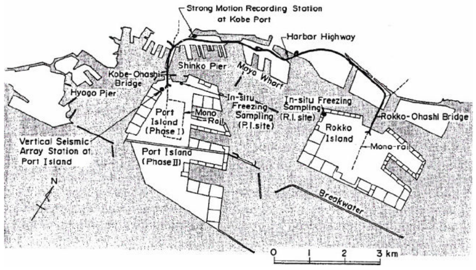

Figure 4. Plan of Kobe Port, showing the location of the recording station, quay walls and sites for geotechnical investigation after Inagaki, *et al, (*1996)

This study focuses on the analysis of typical Port Island caisson quay wall failure during the 1995 Hyogoken-Nanbu earthquake; the quay used as a case study is called PC1, constructed in phase one it is shown in Figure 5. The friction angle used in the design was 30 degrees except beneath the wall where a friction angle of 40 degrees was used, the factor safety was 1.2. The horizontal seismic coefficient was 0.1 and the vertical seismic coefficient was zero (Towhata, *et al.*, 1996) . This wall displaced toward sea approximately 2.75m, settled approximately 1.36m and tilted approximately 3 degrees on average as shown in Figure 6 as obtained by Ministry of Transport, Japan (1997)

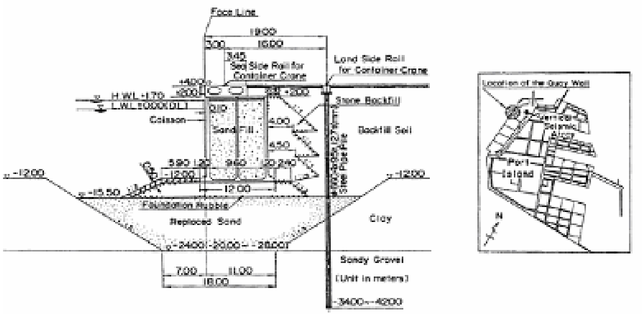

Figure 5. Cross section of a quay wall PC1 at Port Island after Inagaki*, et al.*, (1996)

The relative density of the backfill soil was obtained by the Standard Penetration Test (SPT) conducted by (Inagaki*, et al.*, 1996) and was equal to $D_r = 41.67$ %.

Case study and effective stress analysis method

The seismic performance of a typical Port Island caisson quay wall PC1 has been modelled using a dynamic nonlinear effective stress analysis method in order to evaluate the effectiveness of various soil improvement strategies. The computer program UWLC was used for this study. The UWLC code is a fully coupled numerical code, which can undertake initial stress analysis, one-dimensional and two dimensional dynamic finite element analyses based on total and effective stress.

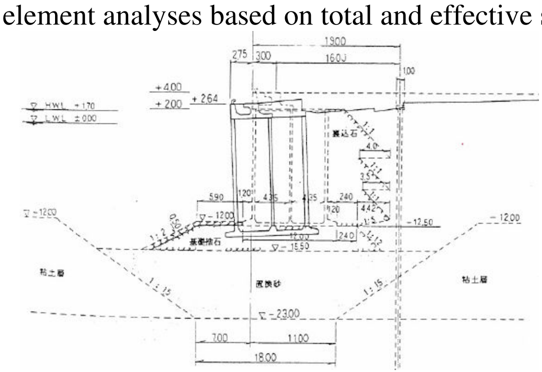

Figure 6. Cross section of the caisson quay wall PC1 in Port Island with observed residual deformation after Kobe 1995 earthquake, Ministry of Transport, Japan, (1997)

The finite element mesh shown in figure 7 was used for the analysis under plane strain conditions. A total of 1989 nodes and 644 elements were used. Five materials divided into nine zones were used in the analysis. The materials and their properties are listed in Table 2. The backfill and foundation layers are modelled by using the PZ-sand modified constitutive model described earlier; the parameters used in the calculation were obtained from the unique series of cyclic triaxial tests conducted prior to the earthquake by Nigase et al., (1995) and presented by Ishihara et al., (1996). In their work, undisturbed frozen samples of Massado soils excavated from the northern section of Port Island were tested with three relative densities. Since the SPT-N value tests presented by Injaki et al. (1996) showed the average density for reclaimed Port Island sand was 41%, as discussed above, the results of the sample with D_r =37% performed by Nigase and his group are used in this study. The results of the cycling loading tests in term of the cycles required to generate cyclic development of 5% double-amplitude axial strain are computed and compared with the triaxial tests on samples from Port Island conducted by Nigase and his group are shown in Figure 8 and the parameters are shown in Table 3.

Table 2. Material properties for model after Iai, *et al.*, (1998) and Dakoulas and Gazetas, (2005)

Materials	Density (ton/m^3)	Friction angle(^{o})	Permeability (m/s)
Land fill	1.8	37	$4*10^{-5}$
backfill and foundation	1.8	37	$4*10^{-5}$
Caisson wall	2.1		
Alluvial clay	1.7	30	$1*10^{-8}$
Rubble in backfill and foundation	2	40	$4*10^{-4}$

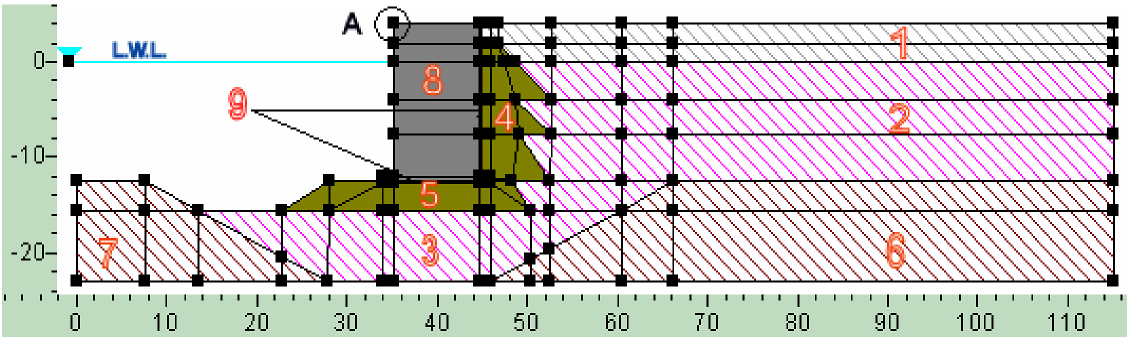

Figure 7. Geometry (in natural scale) and material zones of the Port Island PC1 quay wall; 1 is saturated backfill soil, 2&3 are submerged backfill and foundation soil, 4&5 are backfill and foundation rubble, 6&7are alluvial clay, 8 is the caisson wall and 9 is the interface

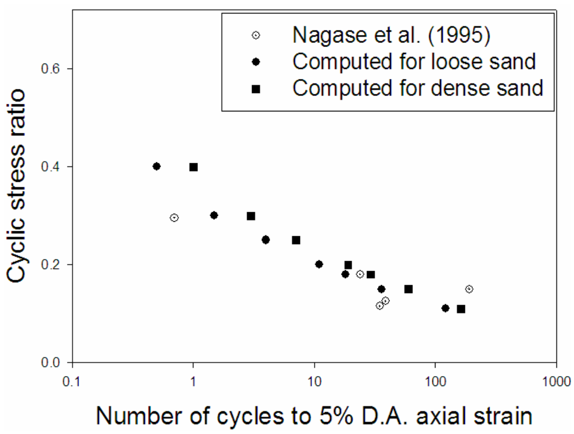

Figure 8. Cyclic stress ratio (CSR) from triaxial tests on samples from Port Island conducted by Nigase et al., (1995) (Ishehara et al., 1996): Comparison with predictions using modified PZ-sand model, D_r =37% and for dense sand $D_r \approx 75\%$

Table 3. Model parameters for relative density D_r =37% and for dense sand $D_r \approx 75\%$

Material parameter	M_f	M_g	C	α_f	α_g	K_{ev0}	G_{es0}	m_v	m_s	β_0	β_1	H_0	H_{U0}	γ	γ_U	p_0'
Loose	0.58	1.3	0.9	0.45	0.45	340	175	0.5	0.5	6	0.76	680	3000	8	7.1	100
Dense	0.78	1.55	0.9	0.45	0.45	310	175	0.5	0.5	6	0.7	600	2400	8	7.1	100

Practically, whether low water level or high water level is considered as a worst case for designing quay walls is not of significance as the active earth pressure on the wall is considered to be the failure force. In the low water level case the active pressure is less than in high water level the reason being the reduction in the density of backfill soil, on the other hand, the stability weight of the caisson wall will also be reduced for same reason, equally, the density of soil and the density of caisson will increase in the high water level case. Therefore, for practical engineering it is known that it is vital to design the caisson wall for both cases and then the worst case is taken. In this model the low water level is considered in the calculation as recommended by Elsharnobi et al., (2004), because it produces the worst case concerning stress at rock base level. As a result, the backfill layer is divided into layers, the top layer is saturated soil with unit weight $\gamma_{sat.}$ =1.8 ton/m^3 and 4m thickness, the submerged soil is measured under the water level with submerged unit weight $\gamma_{sub.}$ =0.8 ton/m^3.

The wall is modelled as an elastic model having interface slip elements which are required between the quay wall model and soil to allow slippage and separation at the base and the back of the caisson with the friction angle at caisson bottom of 30 degrees and friction angle at the caisson back of 15 degrees. The clay zones are modelled using the Hardian-Drnevish model (HD). The seismic excitation input file in both horizontal (E-W) and vertical (U-D) directions respectively were recorded by a seismometer installed at a depth of 32m in the northern area of the Port Island site very close to PC1, adopted by the development Division, Kobe city, in October,1991 reported by Iwasaki & Tai (1996) . The peak acceleration value recorded reached 540gal (cm/sec^2) and 200gal (cm/sec^2) as shown in Figure 9. The digital data were collected from Ishii, (2006).

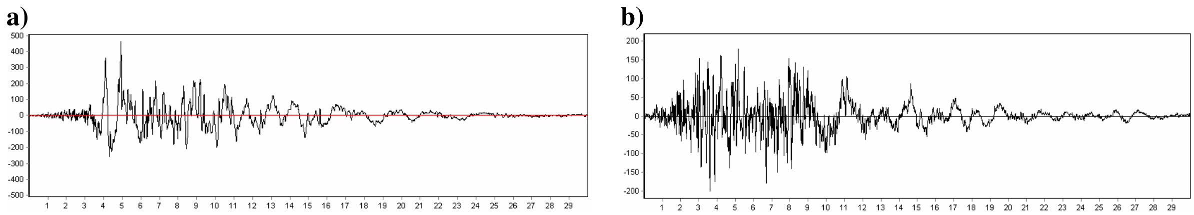

Figure 9. Recorded motions at Kobe Port during the 1995 Hyogoken-Nanbu earthquake a) Horizontal (E-W) component, b) Vertical (U-D) component

RESULTS AND DISCUSSIONS

Before the dynamic response analysis and to take the effect of gravity into account, a static analysis was performed to simulate the initial stress distributions. Figure 10 and Figure 11, below, shows the computed stress results for effective residual deformation including seaward displacement, settlement and tilting for Case 1 which corresponds to a typical quay wall section of PC1 in Port Island where the backfill, landfill and foundation are liquefiable soils. The figure shows the state 30 seconds after the end of the earthquake. The computed displacement at point A on the seaward side corner of the quay wall was 3.3m (2.75 m measured), the wall settled vertically by about 0.59m (1.36 m measured) and tilted into the foundation by 4.5 degrees (3 degrees measured),. The major factor which may have reduced the vertical displacement could be the absence of the effect of shaking parallel the face of quay wall. However the computed residual deformation results for Case 1 were consistent with field observations shown in Figure 6. Computed accelerations at upper sea side corner of the caisson for a quay wall are shown in Figure 12. Computed distributions of excess pore water pressure ratio for quay wall are shown in Figure 13. It can be seen from Figure 13 that maximum pore pressures were generated in the backfill rather than underneath the wall.

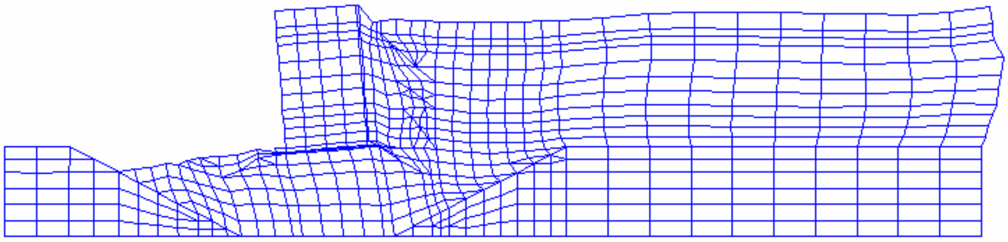

Figure 10. Case 1 computed deformation at the end of earthquake ($t = 30\,\text{sec}$) of Port Island caisson quay wall PC1

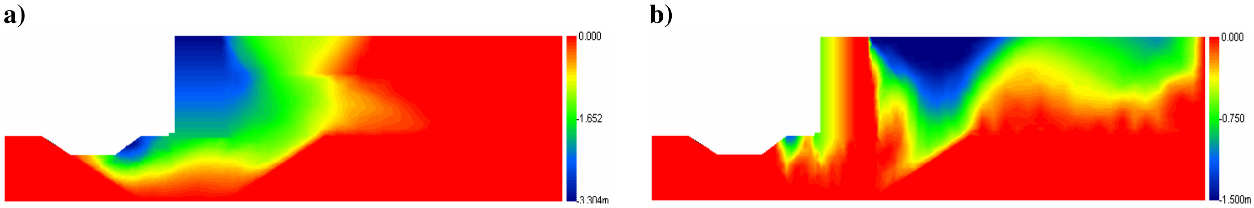

Figure 11. Case 1 computed displacements for quay wall after (30 sec); a) Horizontal and b) Vertical

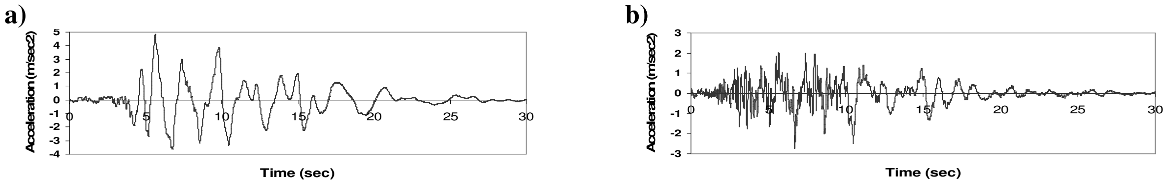

Figure 12. Computed accelerations at upper sea side corner of the caisson at (point A) for case 1: a) Horizontal and b) Vertical

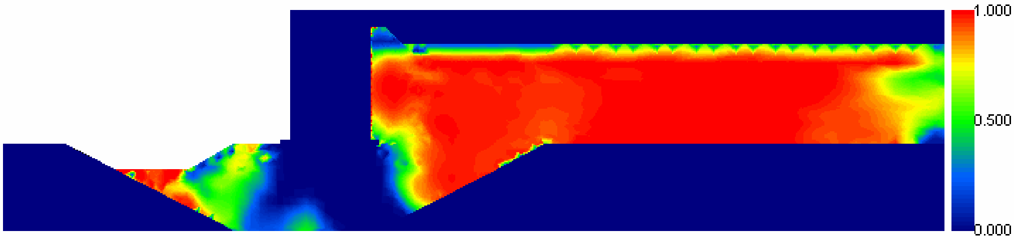

Figure 13. Case 1 computed distributions of excess pore water pressure ratio for quay wall after (30 sec)

In order to investigate the effect of ground improvement techniques on the liquefaction behaviour of the wall, Case 2 was analyzed using PZ parameters of dense sand, for both the backfill and foundation layers. The results are shown in Figure 14. Figure 14 demonstrates that the soil improvement leads to a reduction of the maximum displacement to 0.91m, from the 3.3m of case 1. The settlement at point A is reduced from 0.59 to 0.25m.

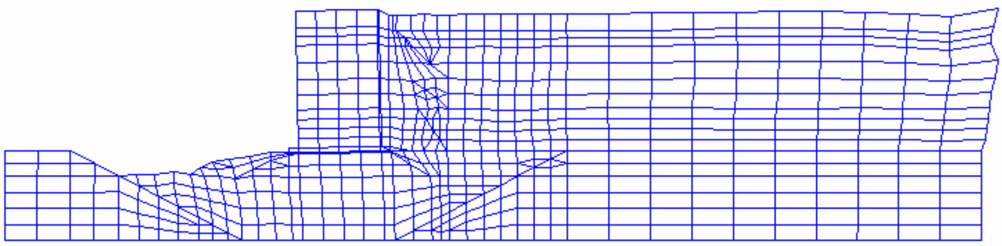

Figure 14. Case 2 computed deformation at the end of earthquake ($t = 30\,\text{sec}$) of Port Island caisson quay wall PC1 where the backfill and foundation are non-liquefiable

In Case 3 the effect of the size of the gravity quay wall is investigated. The equivalent regional horizontal seismic coefficient (K_h) for Kobe region of 0.15 according to the Earthquake Resistance Designed Codes in Japan reported by Ichii, (2003), and is presented in order to investigate the effectiveness of using the regional seismic coefficient in the design. It should be noted that PC1 was designed based on the simplified method which was developed by Okabe, (1924) and Mononobe & Matsuo, (1929) with K_h= 0.1, K_v= 0 and the safety factor = 1.2 (Inagaki, *et al.*, 1996). First, PC1 was designed with K_h = 0.1 by using the simplified method assuming the live load equal to 2 ton/m^2. The results show the required width for the quay wall to be stable in case of K_h= 0.1 was 10 m. Next the same procedure was used to evaluate the suitable quay wall width in case of K_h= 0.15. The new wall dimensions were width 11.5 m and height 16.5 m. The computed residual deformation results for Case 3 are shown in Figure 15. In this case the horizontal displacements have reduced by approximately 5% and settlement has slightly increased by approximately 10% when compared with Case 1 (this is probably due to the increased mass of the wall). Comparing the results of Case 2 and Case 3 to Case 1, it is clear that the improved backfill soil of caisson has a more pronounced effect on the performance of seismic resistance quay walls than increasing the wall width.

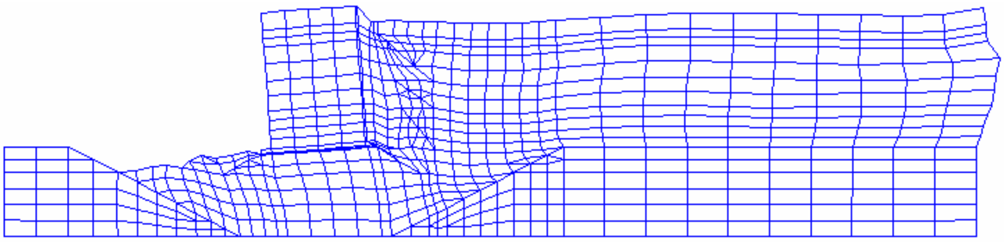

Figure 15. Case 3 computed deformation at the end of earthquake ($t = 30\sec$) of redesigned Port Island caisson quay wall PC1 with K_h= 0.15 where the backfill and foundation are liquefied

In Case 4 the design used in Case 3 with non-liquefiable soil was applied, in order to evaluate the performance of an idealized seismic design of gravity quay walls. The idealized design was achieved by using the regional seismic coefficient and with assumption that the backfill, landfill and foundation soils were improved enough to avoid the generation of pore water pressure during the earthquake. The results from Case 4 are shown in Figure 16. A summary of the major results of the analysis is given in table 4. The displacements in both directions the horizontal and vertical for each case are shown in Figure 17. Finally the results of Case 1, Case 2 and the observed deformations are summarized in Figure 18 by drawing the outline of the quay wall. This Figure clearly demonstrates that 1) the calculated and measured wall movement are similar and 2) that a soil improvement strategies of increasing the density of the soil, results in greatly reduced overall movement of the wall.

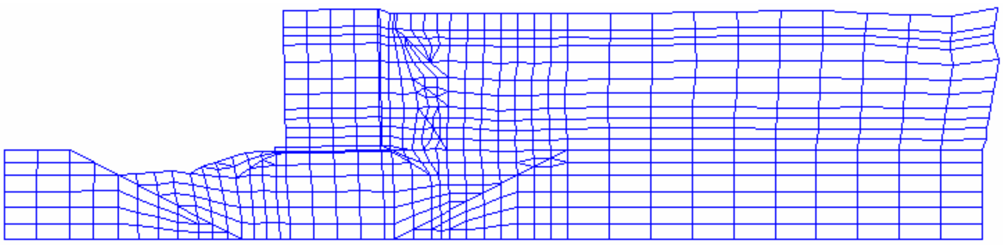

Figure 16. Case 4 computed deformation at the end of earthquake ($t = 30\sec$) of redesigned Port Island caisson quay wall PC1 with Kh=0.15 where the backfill and foundation are non- liquefiable

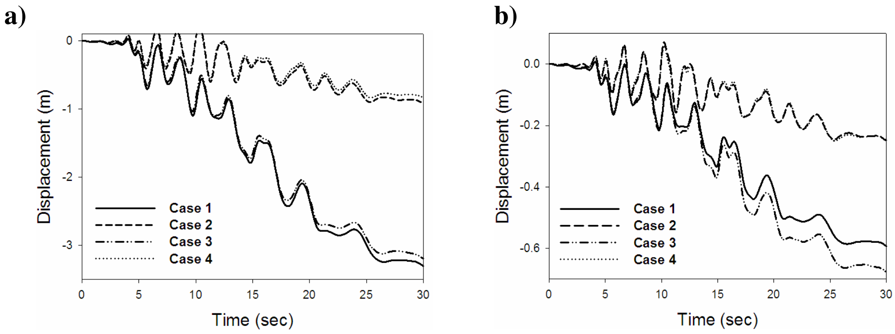

Figure 17. Computed displacement time history at the upper seaward corner of caisson (point A) for all cases: a) horizontal direction and b) vertical direction

Table 4. Summary of computed results of parameter study for quay wall PC1

Case	Wall Diminutions (m)		Horizontal seismic coefficient (Kh)	Displacements (m)		Rotation (degree)	Backfill and foundation conditions
	Width	Height		Horizontal	Vertical		
Case 1	10	16.5	0.1	3.3	0.59	4.5	Loose
Case 2	10	16.5	0.1	0.91	0.25	1.5	Dense
Case 3	11.5	16.5	0.15	3.19	0.68	4.25	Loose
Case 4	11.5	16.5	0.15	0.83	0.25	1.4	Dense

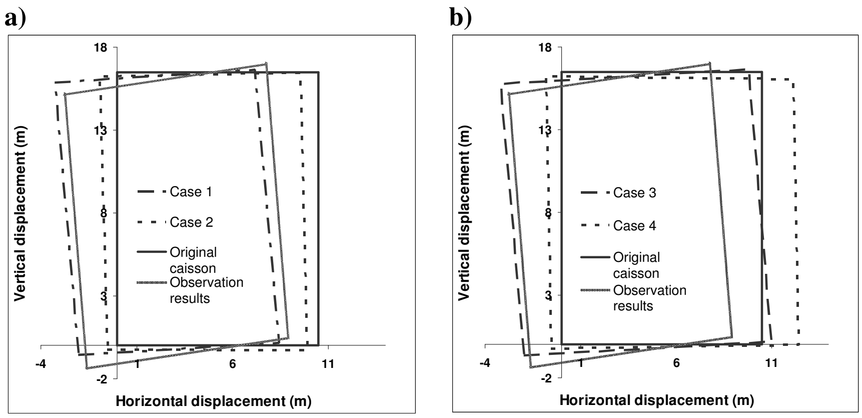

Figure 18. Comparison between computed deformation with original caisson and observation deformation: a) case 1 and 2 and b) case 3 and 4

CONCLUSIONS

A Two-dimensional effective stress method of analysis based on an elasto-plastic constitutive model of Pastor (1990) with slightly modifications has been used for the analysis Port Island quay-wall PC1. The model was first validated by simulating published monotonic and cyclic test results. The results of the monotonic testing showed excellent agreement between the physical and numerical experiments. For the cyclic tests, the simulations were able to capture the general behaviour exhibited in the physical experiments (i.e. stress and strain histories of the numerical simulations showed comparable shapes to the physical experiments).

A model of Port Island quay walls was then developed using a finite element package UWLC and the influence of the density of the soil and wall dimensions on the liquefaction behaviour of the soil was investigated. Both vertical and horizontal accelerations were applied to the model and the results compared to the observed field measurements.

The performance of the quay wall is summarized as follows:

1) Computed overall displacement and rotations of the wall were similar to those observed in the field.
2) Improving the backfill soil of caisson reduces the vertical settlement at the toe of the wall by over 200% while the horizontal displacement is reduced by over 350%.
3) Increasing the width of the wall also reduced the horizontal displacement of the wall (although not as much as increasing the wall width), while the vertical settlement increased slightly.

Finally, Effective stress analysis is powerful tool can describe the seismic response of port structures including liquefaction failure modes.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the technical support of FORUM 8 for the software used in this work. The first author wishes to thank Mr. Hirokazu NAKAMURA for the fruitful discussions throughout the research.

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