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MODELING OF DYNAMIC BEHAVIOR FOR PORT STRUCTURES USING THE PERFORMANCE-BASED SEISMIC DESIGN

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Key words: seismic design, dynamic analysis, performance-based design, cellular quay wall, sheet pile quay wall.

ABSTRACT

The paper presents a pioneer study on numerical modeling of dynamic behavior for port structures using the performance-based seismic design in Taiwan. Since Taiwan is located in earthquake prone area, there is significant interest in improving prediction of the behavior of port structures subjected to seismic loading. The investigation of port structures using three different analysis methods including the simplified analysis, the simplified dynamic analysis, and the dynamic analysis were adopted for evaluating the performance of the cellular quay wall and sheet pile quay wall. In addition, the effective stress analysis with the consideration of the pore pressure generation and soil/liquid coupled analysis was conducted for the dynamic analysis. A new procedure to evaluate the feasibility of the performance-based seismic design for cellular and sheet pile quay walls were also proposed. Results obtained demonstrate that the performance-based seismic design can properly be applied on port structures. In addition, the dynamic analysis can be used to evaluate the interaction behavior of foundation and soil as well as to model pore pressure excitation which may be very useful to evaluate the soil liquefaction.

I. INTRODUCTION

Because Taiwan is located in the circum-Pacific seismic belt, the safety design of port structures over their life time subjected to earthquakes needs to be paid more attention than those in other countries. Especially, the foundation of port structures in Taiwan are commonly found in seabed sediments. The soil is mainly sand and clay inter-layered and seabed sediments are generally very soft. The consideration of economic design of port structures over the lifespan mainly depends on the seabed soil stability for the interaction of foundation and soil subjected to earthquake (Ku et al., 2014).

Fig. 1. Large area of liquefaction of a wharf in the Great East Japan Earthquake.

From an engineering point of view, port structures are soil-structure systems that consist of various combinations of structural and foundation types. Typical port structures are gravity quay walls, sheet pile quay walls, pile-supported wharves, cellular quay walls, quay walls with cranes, and breakwaters (Ministry of Transportation and Communications, 2005). In the past, many retention facilities in harbor areas were damaged by earthquakes, completely disabling harbor function and hindering the estimation of total losses and restoration costs and time (Lai et al., 1999). In fact, one of the major issues is a discussion on how to evaluate the seismic performance of a port structure based on the specific structural and geotechnical conditions (Whitman et al., 1985; EN 1998-1, 2004).

During the 311 Great East Japan Earthquake in 2011, the normal line of steel sheet pile quay walls of Onahama Port's No. 3 wharf were displaced 160 cm toward the sea, the floor covering between land-side and sea-side tracks subsided 30 cm, the back of the land-side track exhibited a height difference of approximately 1 m, a large area at the container yard was liquefied, as shown in Fig. 1, and ground equipment tracks were bent and deformed (Takahashi et al., 2011). The failure mechanism of the steel sheet pile wharf was inferred that the wharf experienced an earthquake load that exceeded the design value. When com-

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Fig. 2. Large area of liquefaction at a wharf in the Chi-Chi Earthquake, Taiwan.

bined with soil liquefaction, this greatly increased earth pressure, causing the steel sheet pile to tilt toward the sea. In addition, a large area of liquefaction in port area of a wharf in the Chi-Chi Earthquake was also observed in 1999, as shown in Fig. 2 (Lai et al., 2014).

Performance-based engineering is a novel engineering concept that involves engineering structure designs, construction, maintenance, and monitoring to achieve the estimated structure performance objectives (Burcharth et al., 2001). In the past, conventional building code seismic design is based on providing capacity to resist a design seismic force. Since it does not consider the performance-based engineering, the methodology of seismic design before performance-based design can not provide information on the performance of a structure when the limit of the force-balance is exceeded. Before performance-based engineering was applied, one can use the conventional design methods, such as the simplified analysis or the simplified dynamic analysis. However, if one demands that limit equilibrium in the simplified analysis not be exceeded in the conventional design for the relatively high intensity ground motions associated with a very rare seismic event, the construction cost will most likely too high.

The purpose of performance-based engineering is to ensure optimal structure design and construction. Thus, in any load scale, structures can satisfy the safety, economic, cultural, and historical requirements of the owner and society, and possess a certain level of reliability regarding performance characteristics throughout the structure lifecycle. The performance-based earthquake engineering is derived from performance-based engineering that emphasizes the seismic performance of the overall system, structure and non-structure components, and accessory equipment (SEACO, 1995). To facilitate the performance-based seismic design of port structures, this paper presents a pioneer study on numerical modeling of dynamic behavior for port structures using the performance-based seismic design in Taiwan. The investigation of port structures using three different analysis methods including the simplified analysis, the simplified dynamic analysis, and the dynamic analysis were adopted for evaluating the performance of the cellular quay wall and sheet pile quay wall. In addition, the effective stress analysis with the consideration of the pore pressure generation and soil/liquid coupled analysis was conducted for the dynamic analysis. A new procedure to evaluate the feasibility of the performance-based seismic design for cellular and sheet pile quay walls were also proposed.

II. ANALYSIS METHOD

1. Performance-Based Methodology

The performance-based design is an emerging methodology, which was born from the lessons learned from earthquakes in the 1990s (Iai and Ichii, 1999). The objective of analysis in the performance-based design is to evaluate the seismic response of the port structure with respect to allowable limits.

In the performance-based design, appropriate levels of design earthquake motions must be defined and corresponding acceptable levels of structural damage must be clearly identified (Burcharth et al., 2001). Two levels of earthquake motions are typically used as design reference motions, defined as follows: Level 1 (L1): the level of earthquake motions that are likely to occur during the life-span of the structure; Level 2 (L2): the level of earthquake motions associated with infrequent rare events, that typically involve very strong ground shaking. If the lifespan of a port structure is 50 years, the return periods for L1 and L2 are 50 and 475 years, respectively. Because Taiwan is located in earthquake prone area, we need to consider very strong earthquake considerably. Therefore, Level 3 (L3): the return periods of 2500 years is defined and used in the performance-based design.

Once the design earthquake levels and acceptable damage levels have been properly defined, the required performance of a structure may be specified by the appropriate performance grade S, A, or B. The definition of grade S includes (1) critical structures with potential for extensive loss of human life and property upon seismic damage, (2) key structures that are required to be serviceable for recovery from earthquake disaster, (3) critical structures that handle hazardous materials, and (4) critical structures that, if disrupted, devastate economic and social activities in the earthquake damage area.

The definition of grade A is primary structures having less serious effects for (1) through (4) than Grade S structures, or (5) structures that, if damaged, are difficult to restore. The definition of grade B is ordinary structures other than those of Grades S and A. The principal steps taken in performance-based design are: (1) selecting a performance grade of S, A, or B; (2) defining damage criteria: Specify the level of acceptable damage in engineering; (3) evaluating seismic performance of a structure. The damage criteria for cellular pile quay walls and sheet pile quay walls are depicted in Tables 1 and 2, respectively.

2. Types of Analysis

A variety of analysis methods are available for evaluating the local site effects, liquefaction potential and the seismic response of port structures. These analysis methods are broadly categorized based on a level of sophistication and capability as (1) the simplified analysis, (2) the simplified dynamic analysis, (Nagao et al., 1995) and (3) the dynamic analysis (Finn et al., 1977; Iai et al., 1999). The simplified analysis is appropriate for evaluat-

Table 1. Damage criteria for sheet pile quay walls.

			Degree I	Degree II	Degree III
Residual displacement	Sheet pile wall	d/H	< 1.5% or d < 30 cm	N/A	N/A
		Note 1	< 3°	N/A	N/A
	Apron	Note 2	3 cm~10 cm	N/A	N/A
		Note 3	30 cm~70 cm	N/A	N/A
		Note 1	< 2°~3°	N/A	N/A
Peak response stresses/strain	Sheet pile wall above mudline		Elastic	Plastic	Plastic
	Sheet pile wall below mudline		Elastic	Elastic	Plastic
	Tie-rod		Elastic	Elastic	Plastic
	Anchor		Elastic	Elastic	Plastic

d: residual horizontal displacement at the top of the wall. H: height of deck from mudline.
Note 1: residual tilting towards the sea.
Note 2: differential settlement on apron.
Note 3: differential settlement between apron and non apron areas.

Table 2. Damage criteria for cellular pile quay walls.

			Degree I	Degree II	Degree III
Residual displacement	Cellular wall	d/H	< 1.5% or d < 30 cm	1.5%~5%	5%~10%
		Note 1	< 3°	3°~5°	5°~8°
	Apron	Note 2	3 cm~10 cm	N/A	N/A
		Note 3	30 cm~70 cm	N/A	N/A
		Note 1	< 2°~3°	N/A	N/A
Peak response stresses/strain	Cell		Elastic	Elastic	Plastic
	Cell joint		Elastic	Plastic	Plastic

d: residual horizontal displacement at the top of the wall.
H: height of deck from mudline.
Note 1: residual tilting towards the sea.
Note 2: differential settlement on apron.
Note 3: differential settlement between apron and non apron areas.

ing approximate threshold limit for displacements and/or elastic response limit and an order-of-magnitude estimate for permanent displacements due to seismic loading. The simplified dynamic analysis is to evaluate extent of displacement/stress/ductility/strain based on assumed failure modes. The dynamic analysis is to evaluate both failure modes and the extent of the displacement/stress/ductility/strain. The details of three different analysis methods are described as follows.

The simplified analysis was performed according to the PIANC port structure design standards (Burcharth et al., 2001), for which pseudo-static analysis based on a dynamic equilibrium was adopted. The basic principles of simplified analysis involve considering structures and bearing soil as rigid bodies and calculating the seismic factor of safety when the structure is resisting an actual earthquake. Therefore, only the factor of safety can be obtained. The sliding displacement and angle of inclination included in the performance requirements cannot be determined using the simplified analysis. Accordingly, the simplified dynamic analysis and the dynamic analysis may be needed.

The simplified dynamic analysis is adopted to verify and inspect Degree II seismic performance. In this study, the Newmark sliding block analysis method based on permanent displacement analysis proposed by Newmark in 1965, (Newmark, 1965), shown in Fig. 3, was employed. The permanent displacement amount is defined as the amount of displacement that occurs when a sliding block (a wedge block formed by extending along the direction of the fracture surface) experiences an earthquake acceleration magnitude that exceeds the critical sliding acceleration ratio. The critical sliding acceleration is a crucial parameter of this method. However, the critical acceleration value exerts a substantial influence on the level of soil liquefaction.

The dynamic analysis is to examine the influence that earthquake acceleration, numerical analysis was adopted to simulate the nonlinear dynamic behavior of soil-sheet pile structural interactions. The seismic concern of the numerical modeling is the development of large displacement that could endanger the safety and serviceability of port structures. Such movements depend on the earthquake loading, the detailed design of the port structures, and the strength properties of the soil materials. In

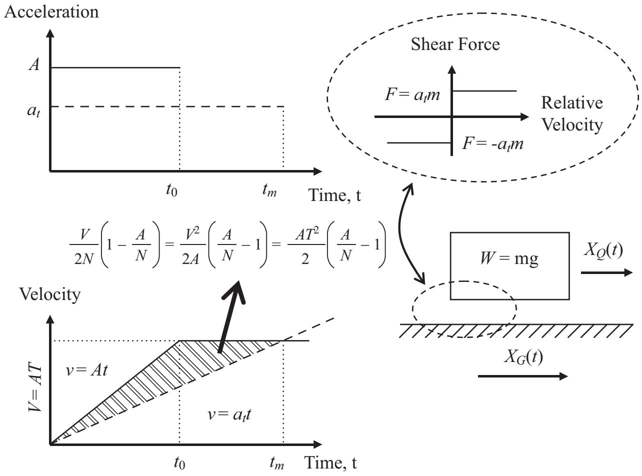

Fig. 3. the Newmark sliding block analysis method.

this study, the commercial software, Fast Lagrangian Analysis of Continua (FLAC) (Itasca, 2005), was adopted for the numerical analyses. The principle of the numerical modeling in this study does not intend to develop the numerical codes, but the conceptualization of the problem, such as the procedure to evaluate the seismic design of port structures in the sand and clay inter-layered soil, was emphasized.

The nonlinear dynamic analysis was performed to verify and inspect Degree III seismic performance. The FLAC program was adopted for the effective stress analysis. Because of the possibility of encountering stratum laminations in practice, this study adopted actual drilling data for stratum lamination. Therefore, multi-layered strata existed in the analytical case. Mechanical damping must be provided to consider energy losses during dynamic analysis. Rayleigh damping, which involves mass and stiffness dampers, was adopted for this case analysis. Critical damping ratios have been suggested for geotechnical engineering materials, generally 2%-5% (Itasca, 2005).

3. The Dynamic Pore Pressure Generation Model

To examine the influence that earthquake acceleration, numerical analysis was adopted to simulate the nonlinear dynamic behavior of soil-sheet pile structural interactions. Numerical simulations of the mechanical behaviors of soil materials were divided into two types. The first one is the total stress analysis. The total stress analysis assumes that the constitutive laws for soil materials are based on the relationship between total stress and strain. Therefore, if strain variation occurs in soil, the total stress is altered only. The fluctuations of the effective stress in the soil cannot be described. On the other hand, the second one is the effective stress analysis. The effective stress analysis indicates that under the effect of dynamic shear stress, the pore water pressure of soil increases with the dynamic shear stress of earthquakes. Thus, if a constitutive law based on effective stress is included in numerical stress analysis, the distributions of pore water pressure, effective stress, and deformation in soil can be determined by conducting dynamic effective stress analysis. This study employed the FLAC program embedded with the Finn model for effective stress analysis. The calculation is based on the explicit finite difference scheme to solve the full equations of motion, using lumped masses derived from the real density of surrounding zones.

The behaviors of the geomaterials are described by an elasto-plastic Mohr-Coulomb constitutive model. The assumption of the Mohr-Coulomb model constitutes an efficient tool for the investigation of the displacements under seismic loading. Coupled dynamic-groundwater flow calculations were also considered in the analysis. The assumption of an empirical equation proposed by Martin et al. (Martin et al., 1975), is adopted in the study.

This formulation can be coupled to the structural element model, thus permitting analysis of soil-structure interaction brought about by ground shaking. Coupled dynamic-groundwater flow calculations can be performed in the analysis. This mechanism is well-described by Martin et al. (1975), who also note that the relation between irrecoverable volume-strain and cyclic shear-strain amplitude is independent of confining stress. They supply the following empirical equation, as shown in Eq. (1), that relates the increment of volume decrease to the cyclic shear-strain amplitude (γ) where γ is presumed to be the engineering shear stain.

$$\Delta\varepsilon_{vd} = C_1(\gamma\text{-}C_2\varepsilon_{vd}) + \frac{C_3\varepsilon_{vd}^2}{\gamma + C_4\varepsilon_{vd}} \tag{1}$$

$$\frac{\Delta\varepsilon_{vd}}{\gamma} = C_1 \exp\left(-C_2\frac{\varepsilon_{vd}}{\gamma}\right) \tag{2}$$

where C_1, C_2, C_3 and C_4 are constants. $\Delta\varepsilon_{vd}$ is the increment of volume strain and ε_{vd} is the accumulated irrecoverable volume strain. An alternative, and simpler, formula is proposed by Byrne (Byrne, 1991) as shown in Eq. (2). For the Byrne model, $C_1 = 8.7\left(N_1\right)_{60}^{-1.25}$ and $C_2 = 0.4/C_1$. This study adopted the Finn and Byrne model which revised from the model proposed by Martin et al. (1975). The Finn and Byrne model was selected because only two parameters need for the analysis. Due to the difficulties and limitations for conducting the geotechnical investigations in deep water, the two-parameter model such as the Finn and Byrne model is preferred in the planning stage. In addition, it is of importance that two parameters of the model can be directly obtained from the standard penetration tests. To clarify the difference of the Martin model and the Finn and Byrne model, we first conducted a numerical experiment. The parameters for this test were listed in Table 3. The width and the depth of the example are 50 m and 5 m, respectively. A sine wave with the maximum amplitude of 0.005 m and the frequency of 5 Hz was adopted for the input of the cyclic loading. The total computing time is 10 seconds. The computed pore water pressures at three observed points at different depth were recorded during the computation. As one can see the results ob-

Table 3. The parameters used in the numerical experiment.

Model	Martin	Finn and Byrne
Soil density (t/m^3)	2	2
Cohesion (Pa)	0	0
Friction angle (degree)	35	35
Porosity	0.5	0.5
Bulk modulus (MPa)	300	300
Shear modulus (MPa)	200	200
C_1	0.80	0.76
C_2	0.79	0.52
C_3	0.45	N/A
C_4	0.73	N/A

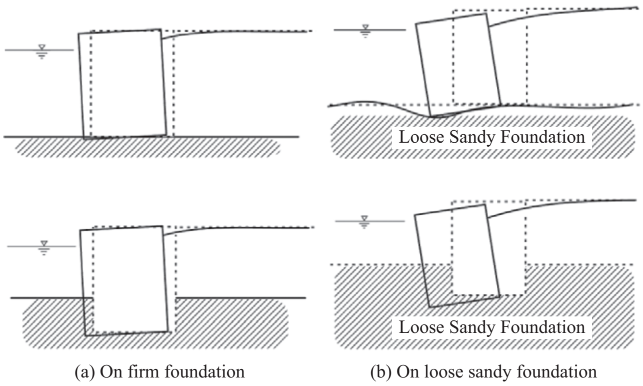

Fig. 5. Deformation/failure modes of cellular quay wall.

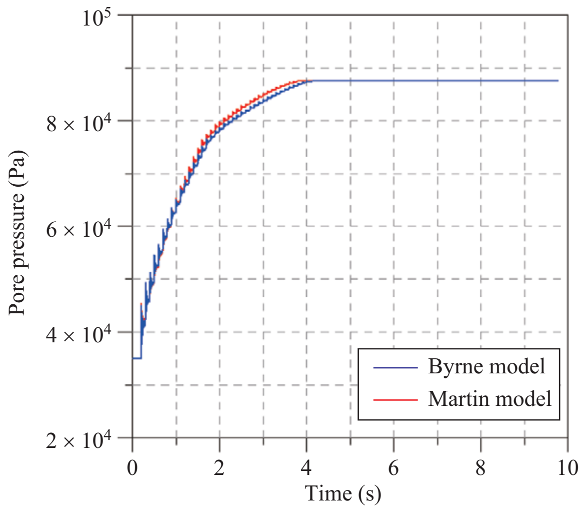

Fig. 4. The computed results of the Martin and Finn-Byrne models at the depth of 3 m.

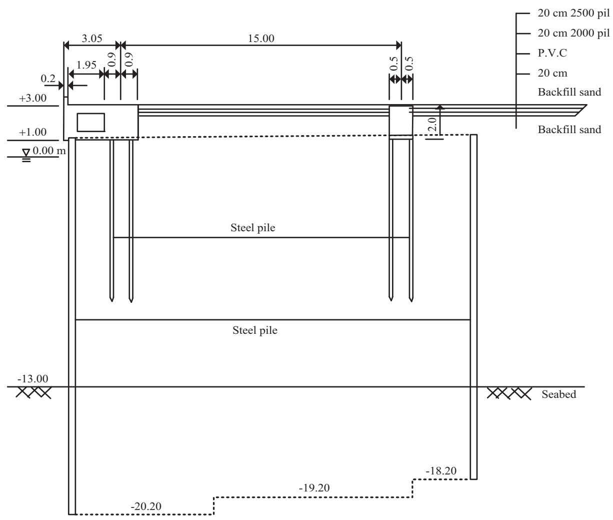

Fig. 6. A proposed cellular quay wall.

tained from the Martin model and the Finn and Byrne model are almost consistent with each other as shown in Fig. 4.

CASE STUDY

1. Design Case of a Cellular Quay Wall

A cellular quay wall is made of a steel plate or steel sheet pile cell with sand or other fill. Resistance against inertia forces and earth pressures is provided by the fill friction at the bottom surface of the cell for a non-embedded cell (i.e., steel plate cofferdam type), or by the resistance of the foundation subsoil at the cell embedment. Typical failure modes during earthquakes depend on cell embedment and geotechnical conditions as shown in Fig. 5. Structural damage to a cellular quay wall is governed by displacements as well as stress states.

It is important to determine the preferred sequence of occurrence and degrees of ultimate states in the composite cellular quay wall system.

1) The Simplified Analysis

In this study, a cellular quay wall (Grade B) is proposed as shown in Fig. 6. The design parameters of the cellular quay wall is shown in Table 4. To verify and inspect Degree I seismic performance, the simplified analysis was employed. The computed factor of safety for the proposed cellular quay wall is about 1.92 which passed the design requirement.

2) The Simplified Dynamic Analysis

To verify and inspect Degree II seismic performance, the Newmark sliding block analysis method was employed. Fig. 7 demonstrates the computed displacement by the Newmark sliding block analysis method. Results show that the permanent displacement of 60.4 cm for the proposed cellular quay wall is occurred with the input of accelerograms from level II of earthquake motions shown in Fig. 8.

3) The Dynamic Analysis:

The functions of cellular quay wall involve withstanding additional pier loads and resisting lateral earth pressure, internal- and external water pressure, and the impact force and tension from vessels. To examine the influence that earthquake acceleration,

Table 4. Design parameters of the cellular quay wall.

Depth of the left steel pile	EL. -20.2 m	Unit weight of sea water (t/m^3)	1.03
Depth of the right steel pile	EL. -18.2 m	Height of the pile	21.2 m
Elevation of seabed	EL. -13 m	Groundwater level	EL. +0.98 m
The friction angle between pile and soil	15	Surcharge (t/m^2)	1.5

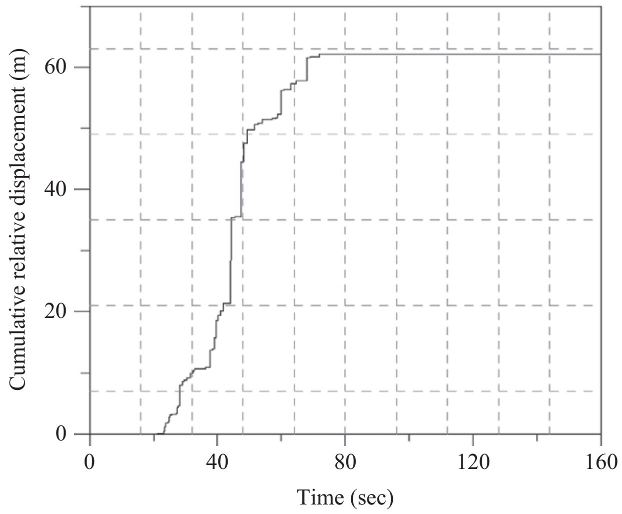

Fig. 7. Computed permanent displacement by the Newmark sliding block analysis method.

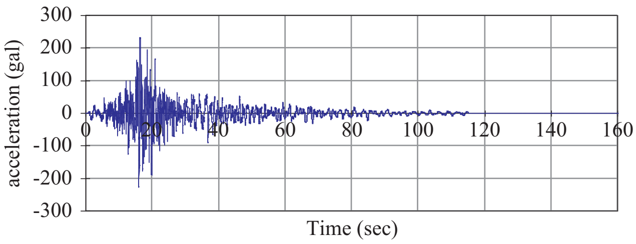

Fig. 8. Accelerations subjected to level II of design earthquake motions.

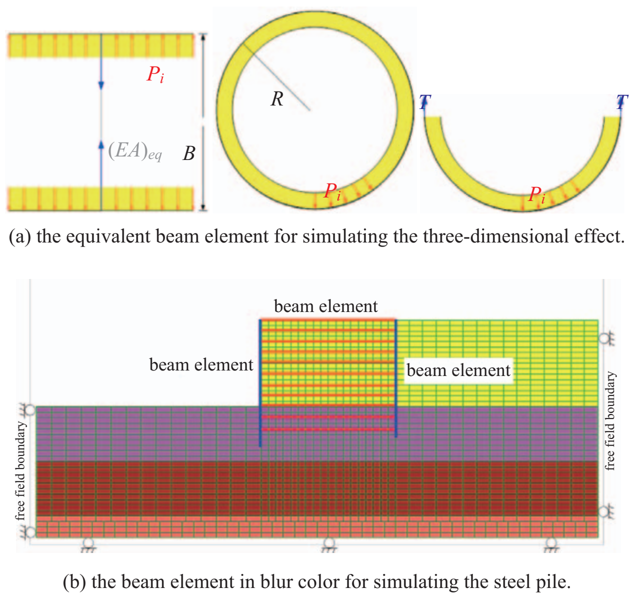

Fig. 9. The structural elements for simulating the cellular quay wall.

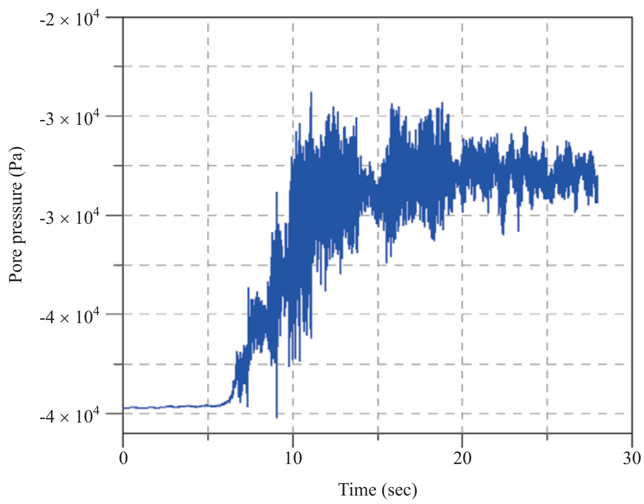

Fig. 10. The excitation of pore water pressure for the dynamic analysis.

numerical analysis was adopted. FLAC dynamic numerical simulation of the cellular quay wall is divided into ten steps: (a) establishing grid; (b) setting the material strength parameters; (c) applying boundary conditions; (d) adding structural elements and interface elements; (e) adding lateral sea water force; (f) specifying the groundwater table; (g) run to equilibrium; (h) using Finn model; (i) setting the damping parameters and applying dynamic boundary conditions; (j) applying seismic force and input accelerations subjected to level III of design earthquake motions. With the consideration of different soil stratums, the input soil parameters are listed in Table 5. Since the cellular quay wall is basically a three-dimensional structure, we adopted the equivalent beam elements to simulate the three-dimensional effects as shown in Figs. 9(a) and (b). Fig. 11 demonstrates the generation of the hydrostatic pressure.

Fig. 10 shows the computed excitation of pore water pressure of the dynamic analysis. Results obtained demonstrate that the maximum horizontal displacement of the cellular quay wall is 162 cm and the maximum bending moment of the cellular quay wall is 2.0 × 10^6 Nm which is still smaller than the allowable bending moment (3 × 10^6 Nm). The evaluation of the seismic performance of the cellular quay wall for level III of design earth-

Table 5. The soil parameters for different depths of soil.

Soil depth (m)	Material model	Density (t/m^3)	Friction angle	Bulk modulus (Mpa)	Shear modulus (MPa)	Permeability (cm/s)
13	M-C/ Finn	1.8	30	5.1	2.3	1×10^{-4}
23	M-C/ Finn	1.8	31	5.9	2.7	$1 \times 10\text{-}^{4}$
33	M-C	1.8	36	15	6.8	1×10^{-4}
37	M-C	1.8	40	18	8.4	1×10^{-4}

Table 6. Design parameters of the sheet pile quay wall.

Embedded depth (m)	5.9	Surcharge (tf/m^2)	1.5
Unit weight of sea water (tf/m^3)	1.03	Elevation of soil level in sea side (m)	-15
Elevation of sheet pile (m)	-20.9	The friction angle between sheet pile and soil (degree)	15

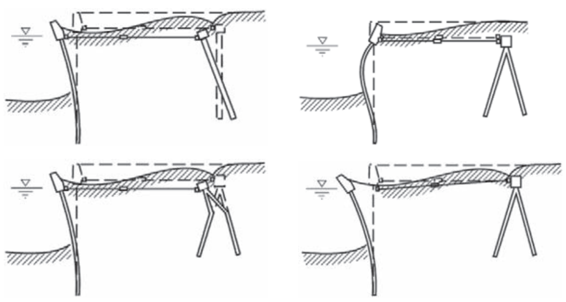
(a) On anchor (b) On sheet pile wall

Fig. 11. Deformation/failure modes of sheet pile quay wall.

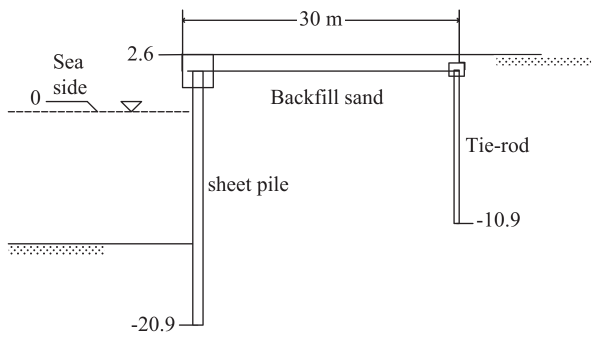

Fig. 12. A proposed sheet pile quay wall.

quake motions is still within the level of acceptable damage in engineering. Accordingly, it can be concluded that the proposed design of the cellular quay wall is acceptable.

2. Design Case of a Sheet Pile Quay Wall

A sheet pile quay wall is composed of interlocking sheet piles, tie-rods, and anchors. Typical failure modes during earthquakes depend on structural and geotechnical conditions as shown in Fig. 11. Seismic performance of a sheet pile quay wall is based on serviceability. The damages of a sheet pile quay wall can be evaluated from the stress states and the displacement. It is important to determine the preferred sequence of occurrence and degrees of ultimate states in the sheet pile quay wall.

1) The Simplified Analysis

In this study, a sheet pile quay wall (Grade B) is proposed as shown in Fig. 12. The design parameters of the sheet pile quay wall is shown in Table 6. To verify and inspect Degree I seismic performance, the simplified analysis was employed. The computed factor of safety for the proposed sheet pile quay wall is about 1.7 which passed the design requirement. Sheet pile quay walls are comprised of reinforced concrete or steel sheet piles, levers, anchorage facilities, and back fillers. Aside from conducting the calculations suggested by PIANC according to safety factor analysis, this study recommends incorporating inspections of the maximum bending moment of steel sheet piles, anchor force, and maximum bending moment and penetration depth of anchor piles in Taiwanese port structure design baselines to provide more thorough analytical calculations.

2) The Simplified Dynamic Analysis

To verify and inspect Degree II seismic performance, the Newmark sliding block analysis method was employed. Fig. 13 demonstrates the computed displacement by the Newmark sliding block analysis method. Results show that the permanent displacement of 148 cm for the proposed sheet pile quay wall is occurred with the input of accelerograms from level II of earthquake motions shown in Fig. 14.

3) The Dynamic Analysis

The functions of sheet pile quay walls involve withstanding additional pier loads and resisting lateral earth pressure, internal and external water pressure, and the impact force and tension from vessels. To examine the influence that earthquake acceleration exerts on the RC, steel sheet piles, anchor piles, levers, and backfill soil of sheet pile quay walls more precisely, numerical analysis was adopted to simulate the nonlinear dynamic behavior of soil-sheet pile structural interactions (Seed et al., 1970; Towhata, 1987; Mylonakis, 2001). FLAC dynamic numerical simulation of sheet pile quay wall is the same as the cellular quay wall.

In this design case, the seismic force and input accelerations

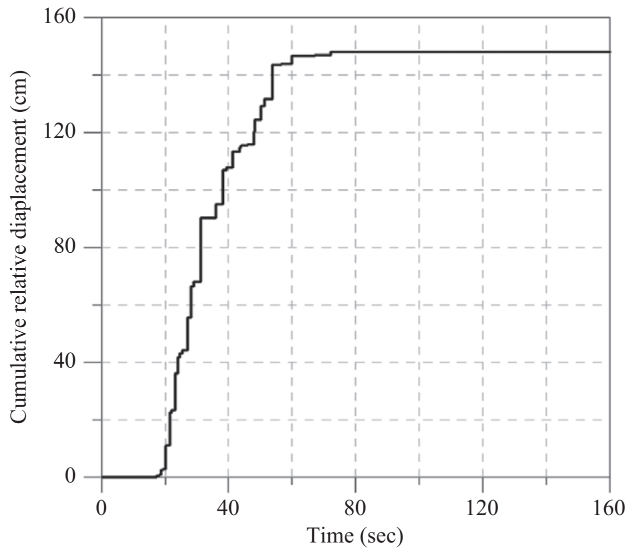

Fig. 13. Computed permanent displacement by the Newmark sliding block analysis method.

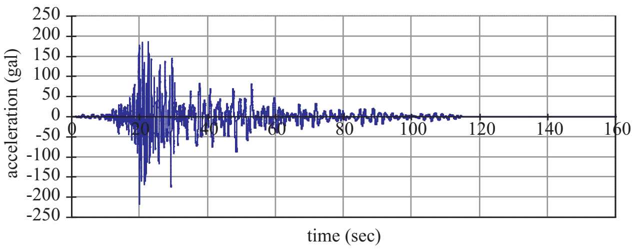

Fig. 14. Accelerations subjected to level II of design earthquake motions.

subjected to level III of design earthquake motions as shown in Fig. 15 was adopted. Fig. 16 demonstrates the numerical model for the sheet pile quay wall. Results obtained demonstrate that the maximum horizontal displacement of the sheet pile quay wall is 144 cm and the maximum bending moment of the sheet pile is 3,130,000 Nm which is greater than the allowable bending moment. The cable is also yield. The evaluation of the seismic performance of the sheet pile quay wall for level III of design earthquake motions is still within the level of acceptable damage in engineering. Accordingly, it can be concluded that the proposed design of the sheet pile quay wall may be acceptable.

IV. DISCUSSIONS

1. Performance Regulation

The International Navigation Association has recommended acceptable standards for cellular quay wall and sheet pile quay wall. However, only displacement inspections have clear quantitative standards in Degree I. In addition, seismic performance is difficult to evaluate in comparatively weaker soil strata. Thus, because sheet pile and gravity-type piers follow the same analytical theorem, this study included displacement inspections of the acceptable standards for gravity-type piers as the basis for

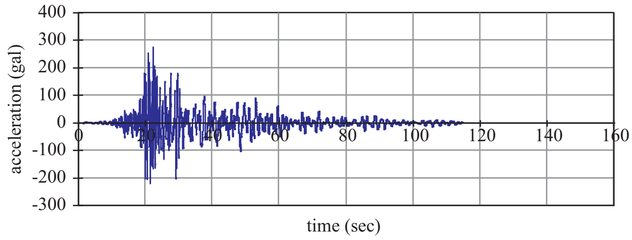

Fig. 15. Accelerations subjected to level III of design earthquake motions.

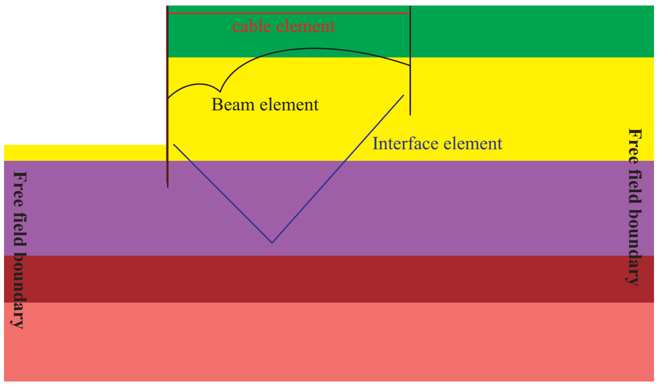

Fig. 16. The numerical model for the FLAC analysis.

assessing performance. PIANC suggested that sheet pile-style piers are not rigid structures. Although displacement can be included in inspections, structural component inspections provide a more crucial reference when assessing reparability. For relevant acceptable performance standards and displacement inspections, besides the stress states of structure components, the performance parameters of sheet pile quay walls include subsidence of parallel displacements, aprons, and anchor facilities. Although acceptable standards of gravity-type pier performance are currently used as a reference, its applicability and accuracy are not guaranteed. Accurate and appropriate standards should be established as a reference for subsequent designs.

2. Feasibility of Preliminary Designs

Regarding preliminary designs, the suggestions of this study were based on the seismic performance requirements of structures. Design engineers should determine the size and detail of each structure section according to the level on the seismic force and conduct an elastic analysis. This suggested method enabled design engineers to produce preliminary designs, regardless of the traditional design methods they typically employed. Therefore, few limitations may exist in preliminary design. Moreover, comparisons of these analyses were used as a reference for evaluating soil liquefaction when conducting the simplified dynamic analysis of the design cases in this study. When seismic force of the same level as different design earthquakes was encountered, the failure displacement differed. This indicates that despite possessing the same peak ground acceleration, the time-history waveform characteristics of an earthquake involve differing energy levels, which generate differing results.

V. CONCLUSION

In this study, we conducted a complete design case study of cellular and sheet pile quay walls by performing simplified analyses, simplified dynamic analyses, and nonlinear dynamic analyses. The results confirmed the feasibility of the seismic performance design proposed in this study.

1. Simplified Analysis

The simplified analysis was employed in the preliminary design stage for confirmation and inspection analysis. The simplified analysis was performed according to the PIANC pier structure design standards, for which pseudo-static analysis based on a dynamic equilibrium was adopted. The basic principles of simplified analysis involve considering structures and bearing soil as rigid bodies and calculating the seismic factor of safety when the structure is resisting an actual earthquake. Therefore, only the factor of safety can be obtained. The results from the simplified analysis may reduce the failure rates of the preliminary design parameters and sizes, enhanced user familiarity with the analytical method, and reduced the complexity of employing numerous analytical methods for analysis and inspection.

2. Simplified Dynamic Analysis

To verify and inspect Degree II seismic performance, the Newmark sliding block analysis method was employed. The permanent displacement amount is defined as the amount of displacement that occurs when a sliding block experiences an earthquake acceleration magnitude that exceeds the critical sliding acceleration ratio. However, the critical acceleration value exerts a substantial influence on the level of soil liquefaction. For the cellular quay wall, the inner backfill soil is an extremely crucial material. High material liquefaction indicates a high degree of damage to the cellular quay wall.

3. Nonlinear Dynamic Analysis

The nonlinear dynamic analysis was performed to verify and inspect Degree III seismic performance. The FLAC program was adopted for modeling soil stratum laminations in practice. Mechanical damping must be provided to consider energy losses during the dynamic analysis. Rayleigh damping, which involves mass damping and stiffness damping, was adopted for this study. Critical damping ratios have been suggested for geotechnical engineering materials (generally 2%-5%).

Finally, in this design case, the acceptable standards for the performance of cellular and sheet pile quay walls of Degree II or above established by PIANC did not possess quantitative standards for displacement inspections. Future studies are suggested to establish the appropriate standards.

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