Earthquake Engineering and Engineering Seismology
Volume 2, Number 1, March 2000, pp. 43–54

Preliminary Analysis for Quaywall Movement in Taichung Harbour During the September 21, 1999, Chi-Chi Earthquake

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ABSTRACT

During the September 21, 1999, Chi-Chi earthquake, the Taichung Harbour suffered some damage, in which the reclaimed land of Piers #1 to #4 were liquefied and the quaywalls were displaced seaward about 1m. This paper is aiming at investigating the stability and sliding movement of those quaywalls. The stability analyses by considering the hydraulically filled backfills liquefied and not liquefied were performed by using the conventional pseudo-static method. For the sliding analysis, a simplified model was developed for back-analysis. The estimated affected area and associated excess pore water pressure are quite consistent to the extent of damage observed in the field.

Keywords: earthquake, quaywall, stability, sliding movement.

INTRODUCTION

The Chi-Chi earthquake occurred at 1:47am, local time on September 21, 1999. With a magnitude (M_w) of 7.6 and a fault rupture of length 105km, it was the largest in-land earthquake in Taiwan in this century. It caused significant damage in the nearby areas of Mid-Taiwan. The death toll is over 2300. About 10,000 buildings/houses collapsed, or heavily damaged. A lot of facilities including bridges, power and water supplies were damaged. Direct loss was estimated to be over 300 billion NT dollars.

The Taichung Harbour, an international port located near Taichung, was also partially damaged by this earthquake. Soil liquefactions occurred in some reclaimed areas and 5 piers made of caissons moved seaward. This paper is aiming at investigating the stability of those caissons in Taichung Harbour that experienced sliding movement during the Chi-Chi earthquake.

DAMAGE IN TAICHUNG HARBOUR

The Taichung Harbour, about 55km

northwest of the epicenter, is built of reclaimed land in four stages. It consists of a total of 45 docking piers. Piers #1 to #4A of length 1100m, located beside the North Docking Channel of the Harbour, were built in 1973 by method of caisson and hydraulic fill and completed in 1976 [1]. During the main shock of the Chi-Chi earthquake, the service areas of Piers #1 to #4A were liquefied and the waterfront quaywalls were displaced seaward about 1 meter as shown in Fig. 1 [2]. The typical profile of Piers #1 to #3 is shown in Fig. 2 [1]. Each pier consists of 10 caissons. Each reinforced concrete caisson has four cells, with a height of 19.6 meters and a width of 17.6 meters. It seats on layers of dumped cobbles and boulders. Inside the caisson, sands dredged from the sea were infilled to increase its weight. Behind the caisson wall, cobbles, boulders and a gravel layer of filter were deposited with a slope of 1:1.5 approximately to increase the stability of the caissons. Beyond that, the land was hydraulically filled with sands dredged from the sea, then paved with gravelly soils of thickness 30cm and a layer of asphalt pavement for ground facilities.

During the Chi-Chi earthquake, the loosely filled sands behind the caissons were liquefied due to strong shakings. Sand boils and cavities were extensively distributed at the service areas of Piers #1 to #4A. Sands erupted from underground can be found as far as 150 meters from the waterfront. The biggest cavity has a diameter over 30m and a depth of 4.2m. According to the field investigation reported by Lee and Chen [3], the damage in the area between Pier #1 to #3 were briefly sketched as shown in Fig. 3, and the profiles of settlement along three cross sections are shown in Fig. 4. Due to liquefaction, the caissons located at the waterfront moved seaward about 1 meter in average and the backfills behind settled about 70cm relative to the caissons. According to survey made by the Center of

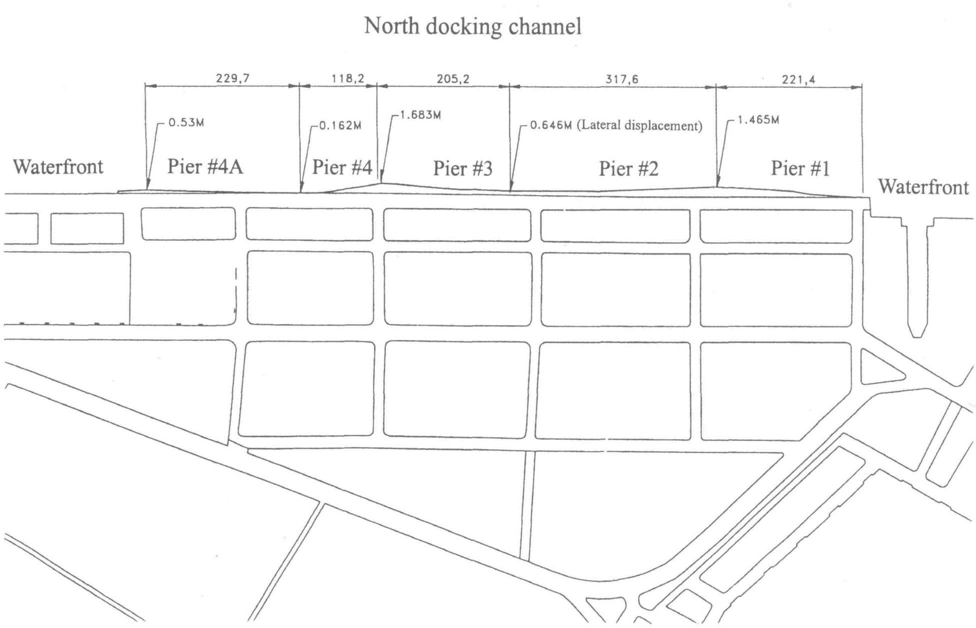

Fig. 1 Plan view and lateral displacements of Pier #1 to #4A [2]

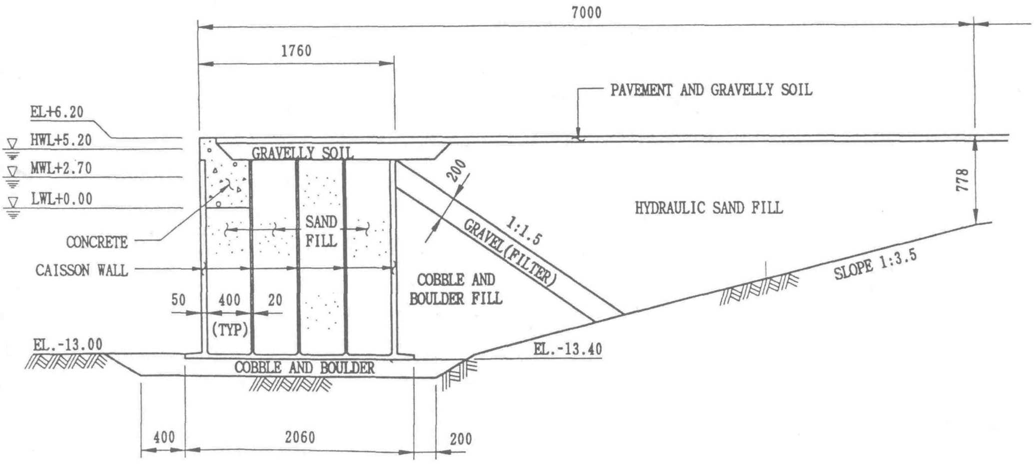

Fig. 2 Typical cross section of Pier #1 to #3 of Taichung Harbour [1]

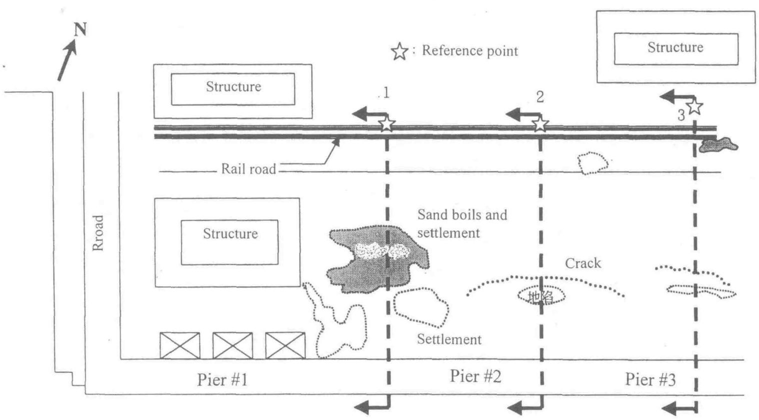

Fig. 3 Damages observed in the area of Pier #1 to #3 [3]

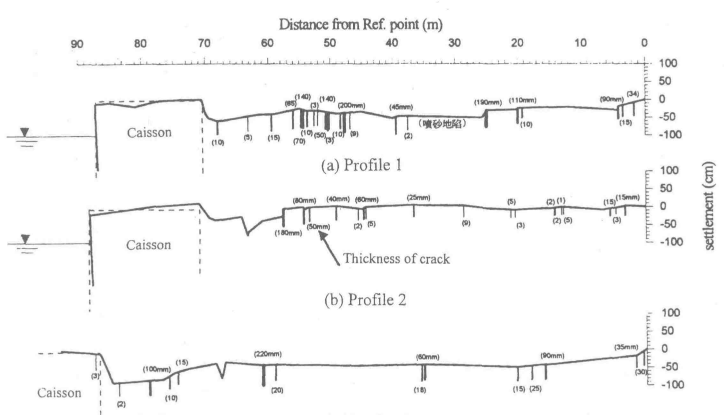

Fig. 4 Deformatiom profiles of Pier #1 to #3 [3]

Harbour and Marine Technology, Institute of Transportation, the normal line of the caisson waterfront has a relative horizontal displacement as shown in Fig. 1, which has a maximum horizontal displacement of 1.69 meter at the location between Piers #3 and #4. In addition, those caissons were also tilted a little bit toward the sea. The average tilting angle is about 1~3 degrees [2]. Due to the outward movement of the caissons, most of the interlockings between the caissons were found to have larger gaps after the earthquake, which permit the flow-in and -out of seawater during the tide variations. The big cavities observed on the ground surface were thought to be resulted from erosions of emerged seawaters.

GEOLOGICAL INVESTIGATIONS

For the construction of the Taichung Harbor, intense investigations for the site geological condition had been conducted. According to the Construction Report of Taichung Harbour [1], the original seabed at the location of Piers #1 to #4A has a profile as shown in Fig. 2. At the pier location, the slope is about 1:3.5 and then becomes more flat toward the shoreline. The original seabed consists of mainly sandy soils of loose to medium dense, interbedded with several thin layers of clayey silt or silty clay. Above the original seabed, sands dredged from the Navigation Channel and nearby areas are filled hydraulically to the present ground level. From the field reconnaissance after the earthquake, it can be concluded that the liquefactions observed are mainly occurred at the layer of hydraulically filled sands.

After the earthquake, several geological explorations had been conducted at Taichung Harbour, including SPT and CPT tests. Figure 5 shows one of the geological profiles obtained along the cross section of Pier #3. The results of explorations showed that the hydraulic sand fills are quite loose, with SPT-N values range from 5 to 14. From the results of cone penetration tests [4], it can be concluded that the tip resistance of the hydraulically filled sands are around 50kg/cm^2. The variations are quite large. At some locations, the tip resistances are much smaller.

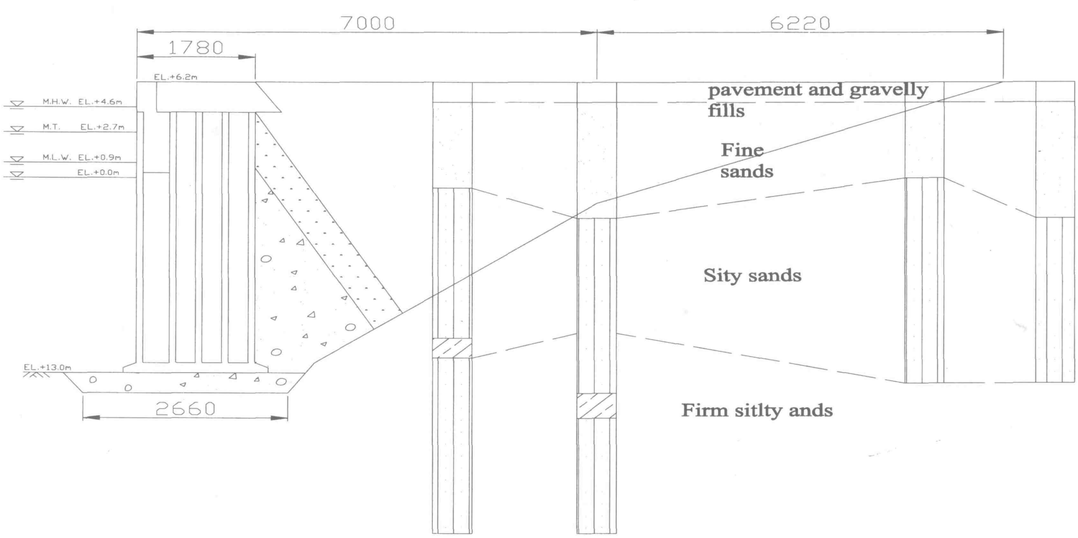

Fig. 5 Geological profile along the cross section of Pier #3

EARTHQUAKE GROUND MOTIONS

In Taiwan, an intense strong motion array called TSMIP has been installed island-wide. Near the Taichung Harbour, there are several seismograph stations in the near distance. Among them, the closest one is the station located at the Elementary School of Chingshui (Station No. TCU059), which is about 4.7km southeast of Taichung Harbour. For the main shock of the Chi-Chi earthquake, the recorded accelerograms in three directions are shown in Fig. 6. It has a peak ground acceleration (PGA) of 165 gals in the east-west (EW) direction and 152 gals in the north-south (NS) direction. The vertical component has smaller accelerations. Since there is no record directly recorded in the Taichung Harbour, the accelerograms recorded at the Chingshui Elementary School will be used as the input ground motions in subsequent analyses of this study.

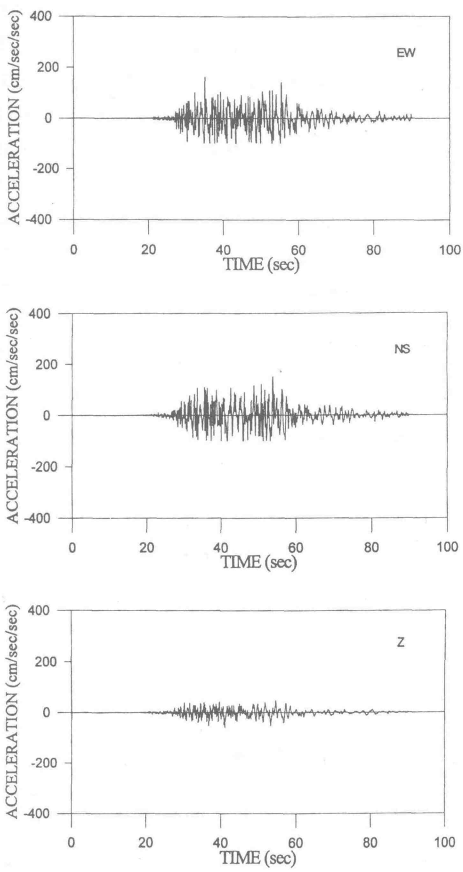

Fig. 6 Accelerograms recorded at Chingshui Elementary School

STABILITY ANALYSIS

In Harbour engineering, the stability of caisson-type quaywalls is usually checked based on pseudo-static analysis. Conventionally, the lateral earthpressure in earthquake is computed by using the Mononobe-Okabe Formula. For the case of horizontal ground surface, the coefficient of active lateral pressure can be calculated by

$$K_{AE} = \frac{\cos^2(\phi + \psi - \theta)}{\cos\theta \cos^2\psi \cos(\delta + \psi + \theta)\ \left[1 + \mathbf{A}\right]^2}$$

$$\text{where} \quad \mathbf{A} = \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \theta)}{\cos(\delta + \psi + \theta)\cos\psi}} \qquad (1)$$

and K_{AE} : coefficient of active lateral earthpressure

ϕ : angle of internal friction of sandy soils

γ : unit weight of soil

ψ : angle between wall surface and the vertical

δ : angle of friction between soil and wall

θ : angle given by the following equations; $\theta = \tan^{-1} k$ or $\theta = \tan^{-1} k'$, in which k and k' are the seismic coefficient and apparent seismic coefficient, respectively.

According to the Technical Standard of Port and Harbour Facilities, Ministry of Transportation, Japan [5], the dynamic pressure of water in backfills can be included in lateral earthpressure in earthquake when it is computed by employing the apparent seismic coefficient giving by

$$k' = \frac{\gamma}{\gamma - 1} k \qquad (2)$$

where γ is the unit weight of saturated soil, k is the seismic coefficient above groundwater table and k' is the apparent seismic coefficient. Besides, the dynamic pressure of water in front of the quaywall can be excluded because it can be compensated by the other factors in the whole course of design calculations.

To investigate the stability of the Taichung Harbour quaywalls, it is interesting to check against the conventional method commonly used in Japan for comparison purpose, i.e., to be checked according to the design procedure specified in the Technical Standard of Port and Harbour Facilities, Ministry of Transportation, Japan [5]. The soil parameters employed in current analysis are deduced from the Construction Report [1] and relevant Exporation Reports [1,2,4] for the Taichung Harbour. They are shown in Fig. 7, in which the water table in all area is adopted at EL.+1.9m according to the level of tides measured [4], the coefficient of shear resistance between the caisson and underneath cobbles/boulders is taken as 0.5 [1], and the unit weight γ of sand filled in the caisson were taken to be equal to $1.8 t/m^3$ ($17.64 kN/m^3$) and $2.0 t/m^3$ ($19.6 kN/m^3$), respectively, for two case studies.

According to the Technical Standards described above, the force diagram can be calculated and plotted as shown in Fig. 8, in which all the lateral forces are expressed in terms of the seismic coefficient k_h or implicitely in K_{AE}. Based on that, it can be used to evaluate the capability of caisson to resist the pseudo-static earthquake load.

From the sliding analysis, the safety factor against the seismic coefficient is shown in Fig. 9 for the case with γ = $1.8 t/m^3$ (unit weight of sand in caisson). Based on that, it can be seen that the caisson of Taichung Harbour can resist a seismic coefficient of 0.12 (δ = 0°) to 0.15 (δ = 15°) without sliding. For the case with $\gamma = 2.0 t/m^3$, the caisson is heavier so that it can sustain a larger seismic load without any question. As for the safety factor of overturning, it is well recognized that the stability of overturning will not control the design and therefore the results of analysis will not be shown herein.

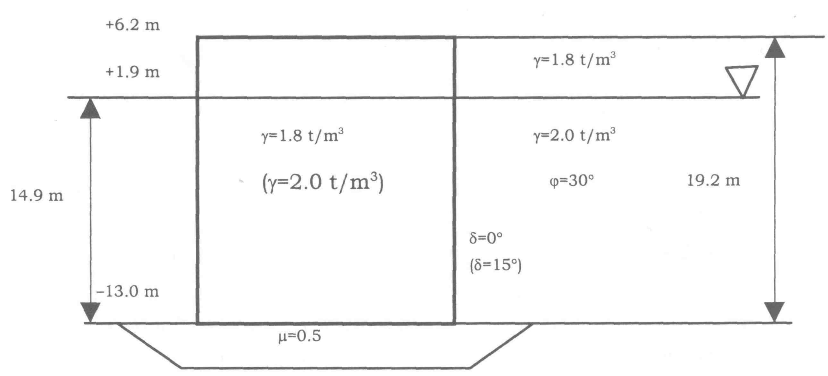

Fig. 7 Soil parameters used in analytical modeling

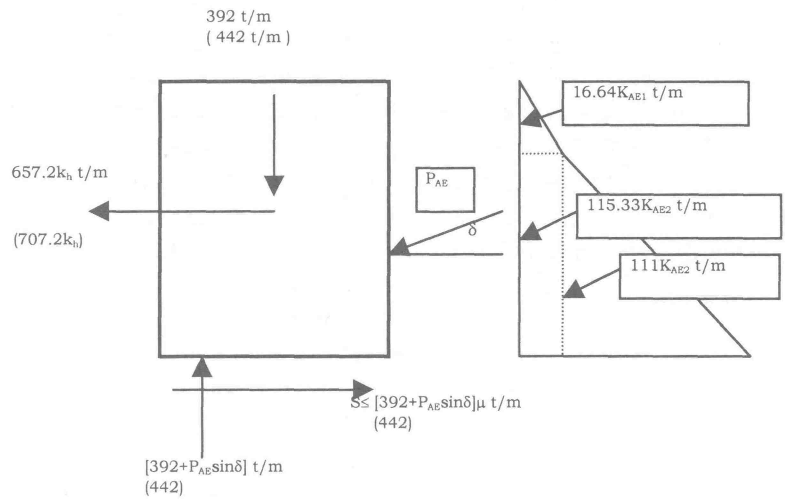

Fig. 8 Force diagram for stability analysis (backfill no liquefaction)

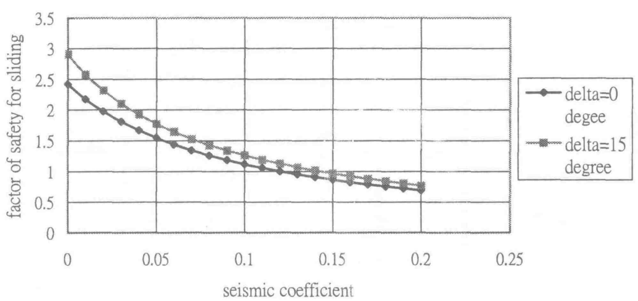

Fig. 9 Factor of safety vs. seismic coefficient from sliding analysis

The stability analysis made above is based on the assumption that the average shear strength of soils in the hydraulically filled region is equal to 30°, i.e., the soils in this region are assumed to be no liquefaction at all during the earthquake. With this no liquefaction assumption, the caisson so constructed can resist a lateral load with seismic coefficient of 0.12 to 0.15 based on a conservative estimation. For the main shock of the Chi-Chi earthquake, the peak ground acceleration recorded in the nearby seismometer (the Chingshui Elementary School) is about 0.16g. This magnitude of PGA is believed to be not able to trigger the sliding for the caisson under discussion, by recognizing that the equivalent pseudo-acceleration (or the seismic coefficient) is generally much smaller [6].

During the Chi-Chi earthquake, the hydraulic fills behind the caisson are known to having experienced liquefactions. The area of liquefactions can be assessed to be quite localized. However for the sake of evaluation, it is instructive to further check against the extreme case by assuming all the backfills being completely liquefied. Under that circumstance, the liquefied soils can be treated as a heavy liquid with a density of $2.0t/m^3$ [7,8]. Then the stability of the caisson can be checked by only considering the pressures of liquids in front of and behind the caisson, respectively. The pressure diagram calculated is shown in Fig. 10, in which the dynamic liquid pressure in front of and behind the caisson wall are calculated based on the Westergaard's formuls [9], giving by

$$\Delta P_{we} = \frac{7}{12} k_h \, \gamma_w \, b \, H^2 \tag{3}$$

$$H_g = \frac{2}{5} H \tag{4}$$

where ΔP_{we} : resultant force of dynamic water pressure

k_h : horizontal seismic coefficient

γ_w : unit weight of water

H : depth of water

b : width of retaining wall, usually take a value of 1m

H_g : height of application point of the resultant force

Based on the pressure diagram shown in Fig. 10, it is obvious that the caisson will not be able to resist the lateral pressure even for a very small seismic coefficient. It's not a real case, but it indicates that the soil behind the caisson have not experienced totally liquefaction during the Chi-Chi earthquake.

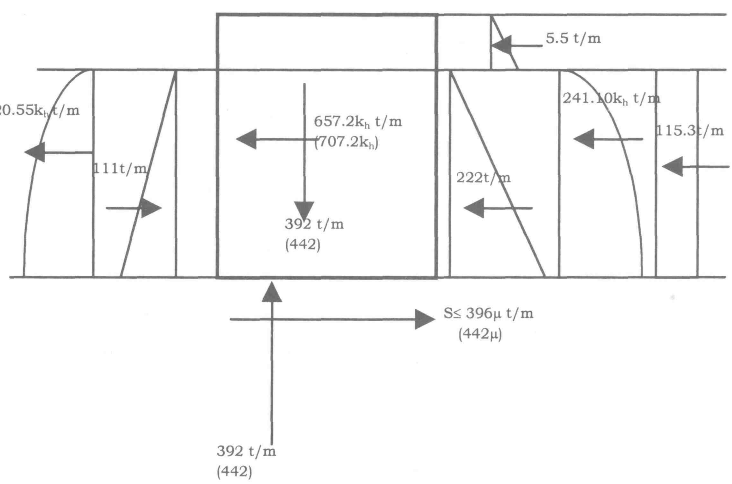

Fig. 10 Pressure diagram for stability analysis (backfill liquefied)

SLIDING ANALYSIS

To calculate the sliding displacement of a retaining structure during an earthquake, the sliding block model proposed by Newmark [8] has been commonly used in engineering practice [9]. That model can calculate the history of movement of a rigid block relative to the ground when the ground motion exceeds a critical acceleration *N*, i.e., when the resultant of horizontal driving forces on the block exceeds the shear resistance can be developed between the block and its foundation.

For the first study for the Taichung Harbour case, it is assumed that the backfills behind the caisson will not liquefied during the earthquake, and the shear resistance of the caisson foundation has a friction coefficient of 0.5 as previously specified. The accelerograms recorded at the Chingshui Station are taken as the input ground motions for current analysis. Based on that, the velocity and associated displacement of caisson relative to the ground are calculated as shown in Fig. 11. It can be seen that the calculated displacement of sliding for the caisson will only be 2.6mm for the EW component of earthquake excitations. For the case of NS component analysis, the sliding displacement obtained is even smaller.

Based on the analyses performed, it is concluded that the caisson will not slide in reality if the backfills had not liquefied at all. This can be used to explain that the lateral movement of the caisson during the main shock of Chi-Chi earthquake should be resulted from the effect of liquefactions occurred in the backfill area behind the caisson.

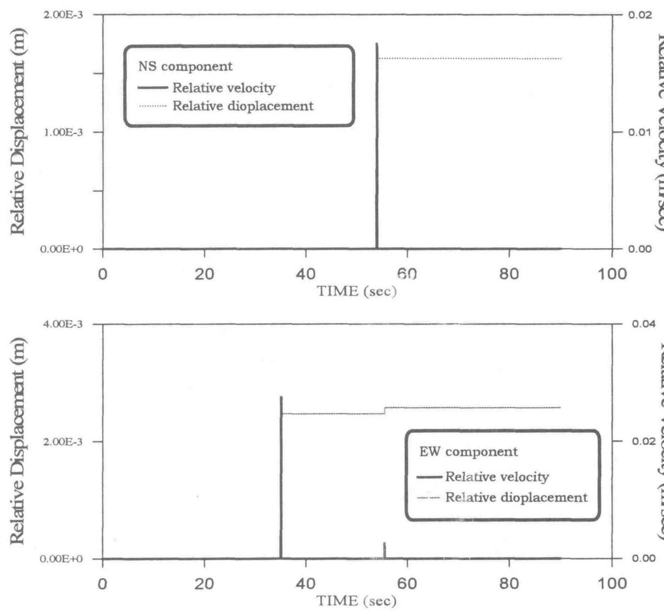

Fig. 11 Results of caisson sliding analysis (backfill no liquefaction)

BACK ANALYSIS FOR QUAYWALL MOVEMENT

To further investigate the behavior of the quaywall movement during the earthquake, the quaywall of Pier #2 was chosen for study, because its backfill area is an open space without any structure. According to the survey made by the Center of Harbour and Marine Technology, Institute of Transportation [4], the movement of the caisson can be plotted as shown in Fig. 12. The top surface of the caisson experienced a horizontal displacement of 80cm and a tilt angle of 1°. It is estimated that the foundation of the caisson will have a horizontal displacement of 50cm. To back-calculate the earthquake force required to produce such a displacement in the quaywall, a simplified model is adopted as shown in Fig. 13. The horizontal profile is divided into three parts: the caisson, an equivalent rectangular area of cobbles/boulders/gravels backfills, and a soil wedge in the hydraulically filled sands.

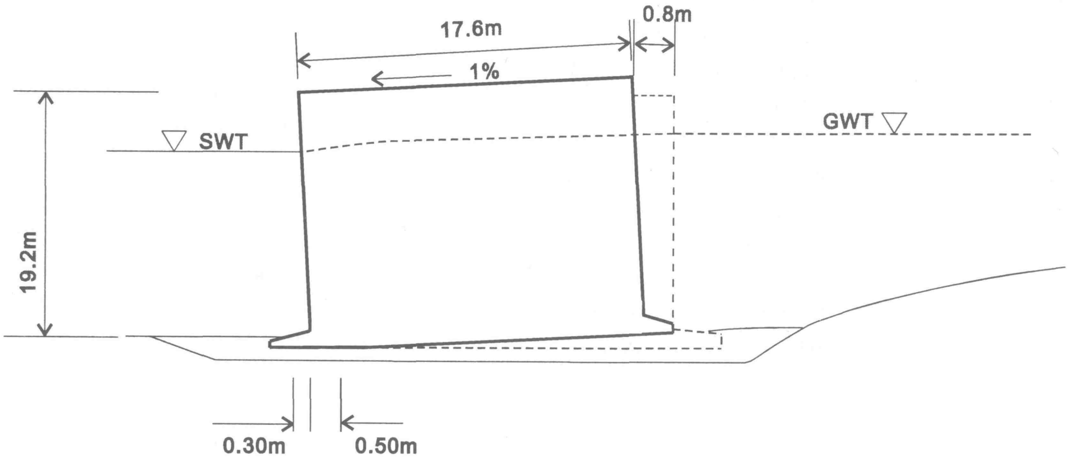

Fig. 12 Lateral displacement of caisson in Pier #2

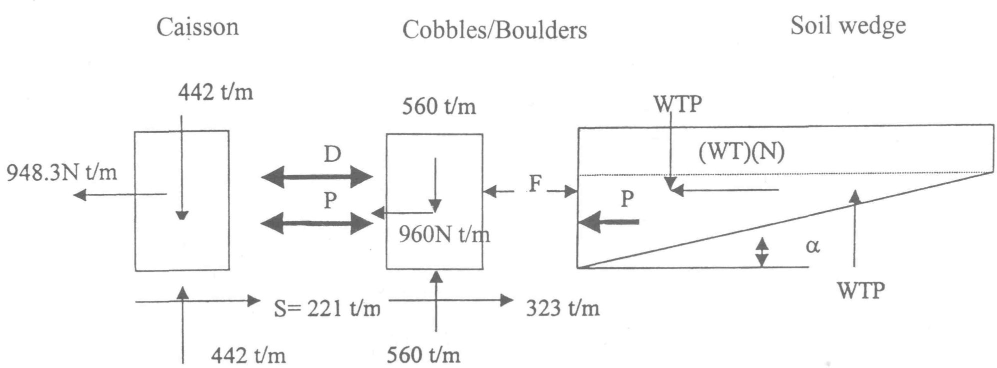

Fig. 13 Model for sliding analysis

This soil wedge behind is regarded as a soil body with a residual shear strength ϕ_r, resulted from the build-up of excess pore water pressure during the earthquake. Under this circumstance, the soil wedge can be assumed to have a inclined surface of α degree at the bottom. Therefore, it is further assumed that the soil wedge is bounded by a vertical surface (tension crack) at the location where the bottom α-surface intersecting the ground water level. It is thought that the finite sliding of the caisson was produced by the lateral pressure of the soil wedge in the hydraulically filled area. The lateral force includes the active earthpressure based on the residual shear angle ϕ_r, and an additional dynamic lateral force resulted from the earthquake effects. In the figure, the dynamic lateral force is represented by product of the weight of soil wedge (WT) and the critical ground acceleration (N) to produce the sliding of caisson. The definition of the critical acceleration (N) follows that proposed by Newmark [10] and Seed and Whitman [11].

Based on the model constructed above, all the forces are calculated and indicated as shown in Fig. 13. To do the analysis, the sliding displacement spectrum based on the earthquake record of

Chingshui Station is first generated as shown in Fig. 14. From this spectrum, it can be estimated that a critical acceleration ratio N/A of value 0.27 is required to produce a caisson movement of 50cm, where A is the peak acceleration of corresponding input ground motion. Based on that, the angle of α can be back-calculated, by iteration procedure, to have a value of 8 degrees. It is corresponding to the situation that the affected area will extend to a distance of 155m from the waterfront of the caisson, which is quite consistent to the farest point where ground failure occurred during the Chi-Chi earthquake [4]. Furthermore, by assuming that

$$\alpha = \tan^{-1} [(1 - r_u) \tan \phi] \qquad (5)$$

and

$$r_u = \frac{\Delta u}{\sigma_0'} \qquad (6)$$

in which, α is the inclined angle of the bottom surface of the soil wedge, ϕ is the angle of shear resistance of the backfill soils, r_u is the ratio of excess pore water pressure Δu relative to the effective overburden pressure σ_0'.

Based on Eq. (5) and the α value back-calculated, the ratio of excess pore water pressure (in the average sense) can be estimated to be equal to 0.76. This estimation is also quite consistent to the extent of liquefactions observed in the field.

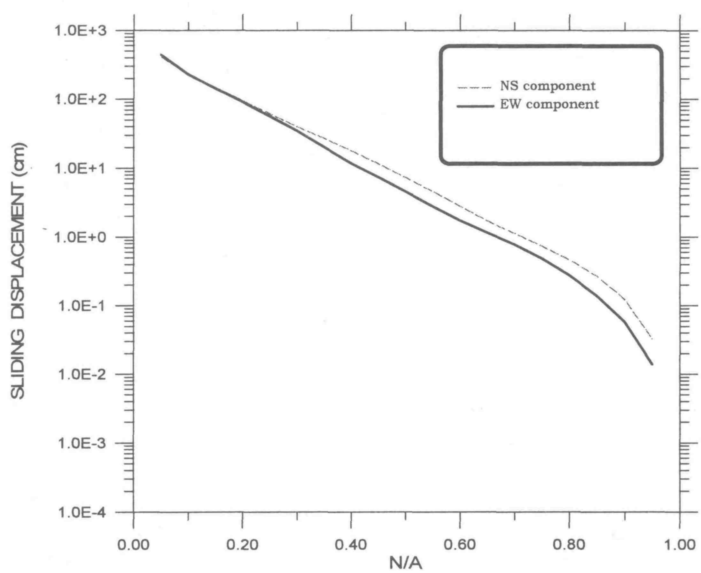

Fig. 14 Spectrum of sliding displacement for Chingshui accelerograms

CONCLUSIONS

Based on the preliminary studies presented herein, some general conclusions can be made as follows:

1. The Piers #1 to #4A of Taichung Harbour suffered some damage during the Chi-Chi earthquake. The reason of damage is mainly due to liquefactions of the loosely deposited sands in the reclaimed land. The potential of soil liquefaction in these areas has to be taken into account in restoration works for disaster mitiga-

tions in future earthquakes.

2. Based on the pseudo-static analysis, it can be found that the quaywall in Piers #1 to #4A can resist an earthquake load with a seismic coefficient of 0.12 to 0.15 if the backfills had no liquefaction at all.
3. The sliding of quaywalls during the Chi-Chi earthquake is due to the effects of soil liquefaction occurred. Based on the simplified model adopted, the extent of affected area and the average ratio of pore water pressure build-up can be reasonably estimated.

ACKNOWLEDGEMENT

The authors would like to thank for Professor J. C. Lee, H. T. Chen and J. H. Hwang of National Central University, and the faculties of Center of Harbour and Marine Technology, Institute of Transportation, for providing valuable reconnaissance data. Their contributions are deeply appreciated.

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