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SEISMIC SAFETY EVALUATION OF AN EXISTING TANK SUPPORTED BY PILES IN LIQUEFIABLE GROUND

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SUMMARY

To evaluate pile foundation responses in liquefied soil accurately, non-linear soil-pile foundation interaction as well as excess pore water pressure generation must be taken into account. In this paper, a tank for 60,000kl LNG(Liquefied Natural Gas) supported by 561 steel pipe piles at coastal reclaimed land in Tokyo bay is modeled and the dynamic response analysis is performed. A practical numerical model proposed by Miyamoto et al.[2-4] is used. The pile foundation is modeled as lumped masses connected with bending-shear elements. And lateral and shear interaction soil springs are connected to the lumped masses of pile foundation. The rotational spring related to the axial stiffness of the piles is also incorporated at the pile head. Analysis for seismic response of soil-pile foundation system is conducted at the following two stages. Firstly, the soil responses are calculated by non-linear effective stress analysis using the computer code YUSAYUSA proposed by Ishihara and Towhata[1]. Then, the obtained displacement and excess pore water pressure time histories at each depth are applied through the corresponding non-linear lateral interaction soil springs. The analytical results show that the tank and piles are confirmed to be safe during strong earthquakes.

INTRODUCTION

The Hyogo-ken Nanbu Earthquake of January 17, 1995 caused severe damage to all kinds of structures supported by piles at many locations. During this earthquake, soil liquefaction occurred at coastal reclaimed land of Hanshin area and the large ground displacements induced by the soil liquefaction caused severe damage to structures supported by piles. Seismic responses of structures supported by piles in liquefied ground are significantly influenced by soil non-linearity and excess pore water pressure generation during strong earthquakes. Therefore, it has become very important to evaluate the seismic safety of structures supported by piles at coastal reclaimed land.

In this paper, dynamic response analysis of an existing tank is conducted, and the seismic safety of the structure is evaluated. A tank for 60,000kl LNG(Liquefied Natural Gas) supported by 561 steel pipe piles at coastal reclaimed land in Tokyo bay is modeled.

2. CONDITIONS FOR ANALYSIS

The pile-supported tank shown in Figure 2.1 is analyzed. The tank is supported by a total of 561 steel pipe piles (508 mm in diameter, 20.6 m long). Each steel pipe pile consists of three different pile segments. The soil properties used in this analysis are shown in Table 2.1. The physical properties are calculated from an empirical formula, using SPT blow counts (N-values). The layer Ds3 is assumed to be the input bedrock. The input ground motion (see Figure 2.2) is the earthquake record in the NS direction at the Japan Meteorological Agency(JMA)'s

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Kobe Observatory during the Hyogo-ken Nanbu Earthquake of 1995. The input maximum acceleration is normalized as 300 Gal.

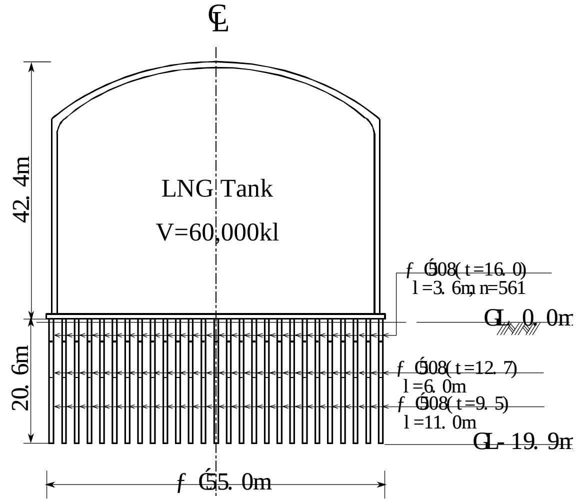

Figure 2.1 LNG TANK

Table 2.1 Physical Properties of Soil

Soil layer GL(m)	SPT blow count (N-value)	Unit weight γ (tf/m^3)	Initial shear wave velocity V_{s0} *1 (m/s)	Shear strength τ *2 (tf/m^2)	R-O model constants α	r
±0 - 3.5 Fs - 9.5	18	2.00	210	0.6 ~ 7.6	10.0	2.6
As, Ac - 13.2	7	1.98	150	6.4 ~ 7.7	10.0	2.6
Dc 1 - 14.7	14	1.60	240	13.4 ~ 13.8	5.0	2.4
Ds 1 - 22.2	42	1.85	280	15.1 ~ 19.9	10.0	2.6
Dc 2 - 30.4	19	1.67	270	18.1 ~ 21.8	5.0	2.4
Ds 2 - 38.4	29	1.83	250	21.5 ~ 25.7	10.0	2.6
Dc 4 - 43.5	19	1.79	270	26.5 ~ 28.6	5.0	2.4
Ds 3	62	1.94	320	–	–	–

*1 Sandy soil V_{s0}= 80N$^{1/3}$
Cohesive soil V_{s0}=100N$^{1/3}$
*2 Caluculated from Mohr-Coulomb's equation τ= C+σ'tan ϕ
where C=cohesion, σ'=effective stress, and ϕ=internal friction angle.

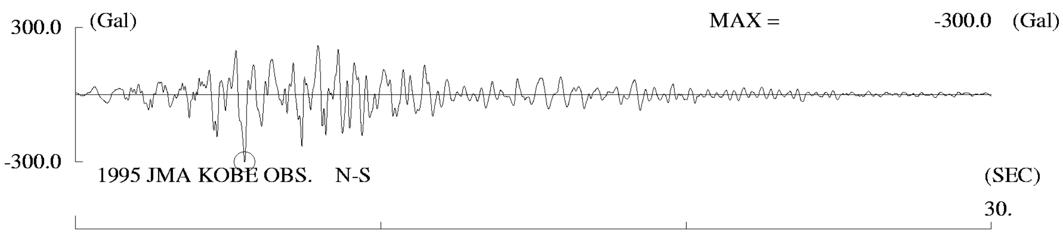

Figure 2.2 Input Ground Motion

3. ANALYSIS METHOD

The numerical model of the pile foundation consists of a single multi-mass with bending-shear element that models the pile group and an interaction soil spring at each mass point (see Figure 3.1) [3,4]. The super-structure is modeled as a four-mass system composed of (1) free liquid, (2) internal tank + confined liquid, (3) external tank, and (4) base slab. A rotational spring is defined at the base slab. Into the interaction soil springs are introduced the non-linearity due to relative displacement between the piles and ground and the effects of changes in effective stress due to the generation and dissipation of excess pore water pressures in free ground. Bending stiffness of structure is analyzed as being linear for the superstructure and non-linear for the piles.

Figure 3.2 shows a flowchart for the analysis. For calculation of interaction soil springs, it is assumed that non-linearity depending on shear strains in free ground due to seismic response also occurs in the soil around the piles. The initial interaction soil springs are calculated using equivalent stiffness of soil which is comparable to effective shear strain at each depth obtained from a total stress analysis of free ground. For multiple piles, it is necessary to take pile group effects into consideration. Interaction soil spring constants for multiple piles are evaluated by condensation of the interaction soil springs for the entire pile group that can be calculated by

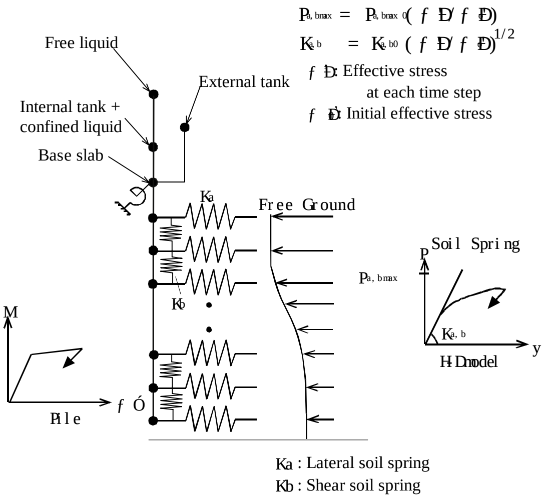

Figure 3.1 Analytical Model

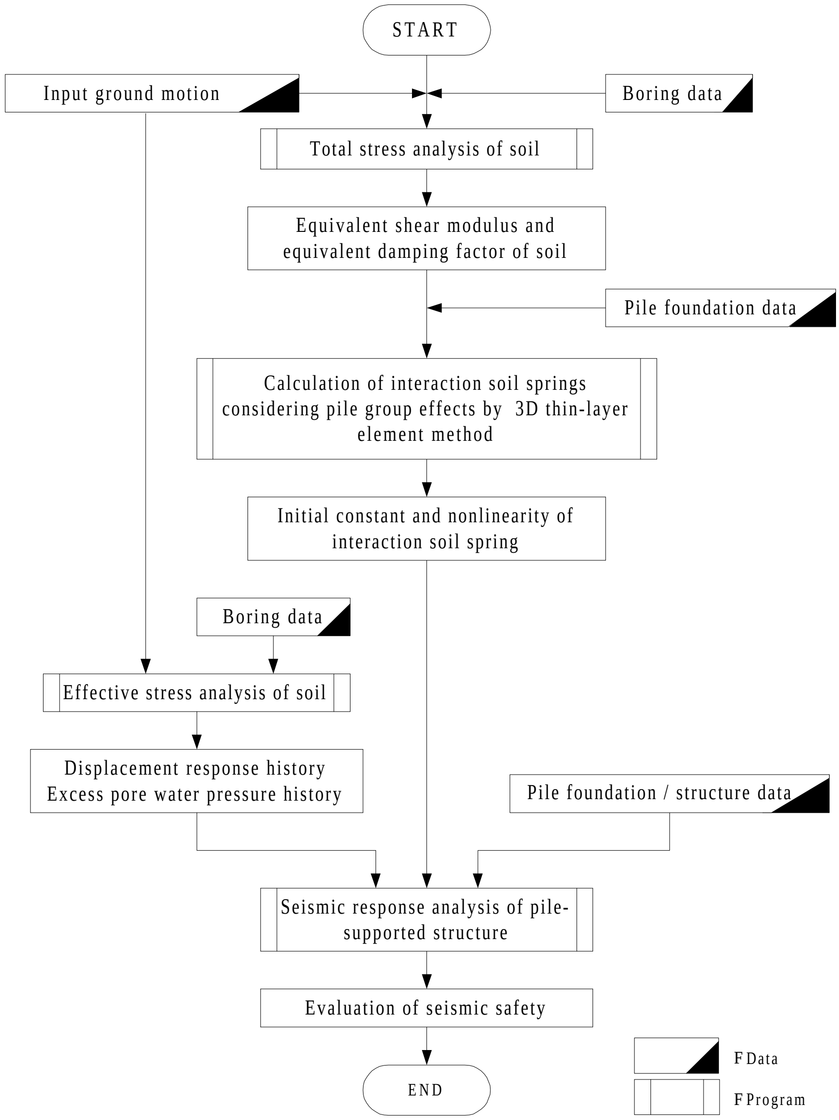

Figure 3.2 Flowchart for Dynamic Response Analysis of Pile-supported Structure in Non-linear Ground

a three-dimensional thin-layer element method. The condensed interaction soil spring constants are decomposed to the lateral soil springs and the shear soil springs. The seismic response of the lumped-mass system is analyzed by inputting the displacement and excess pore water pressure time histories at different depths obtained from a dynamic response analysis of free ground by using the computer code YUSAYUSA [1].

4. ANALYTICAL RESULTS

4.1 Free Ground

Figure 4.1 shows the maximum acceleration, maximum relative displacement, and excess pore water pressure (P.W.P) distributions in the soil. As can be seen from the figure, excess pore water pressures have generated in the sand strata at GL-3.5 to -7.5m and at GL-9.5 to -13.2 m and liquefaction in these strata is almost complete. Figure 4.2 shows time histories of excess pore water pressure. These figures indicate that there are different time lags at different depths before the soil liquefies completely. It can been seen that complete liquefaction occurs at the deeper levels about one second after the maximum acceleration of input motion shown in Figure 2.2, followed by complete liquefaction of the shallower soil layers.

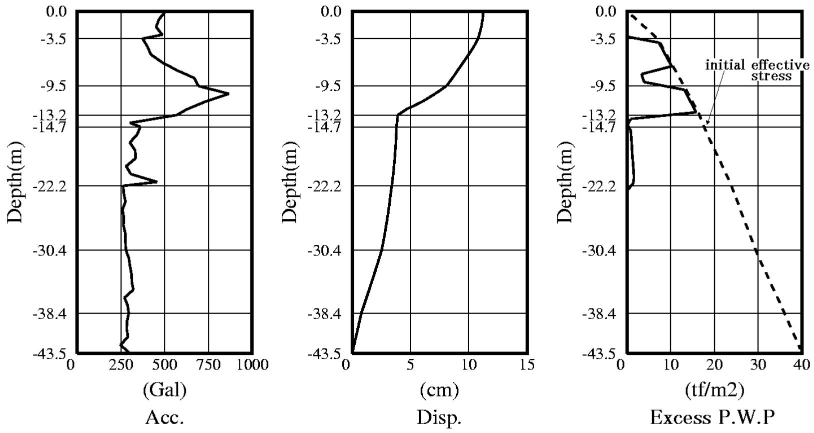

Figure 4.1 Maximum Response and Excess Pore Water Pressure Distributions in Soil

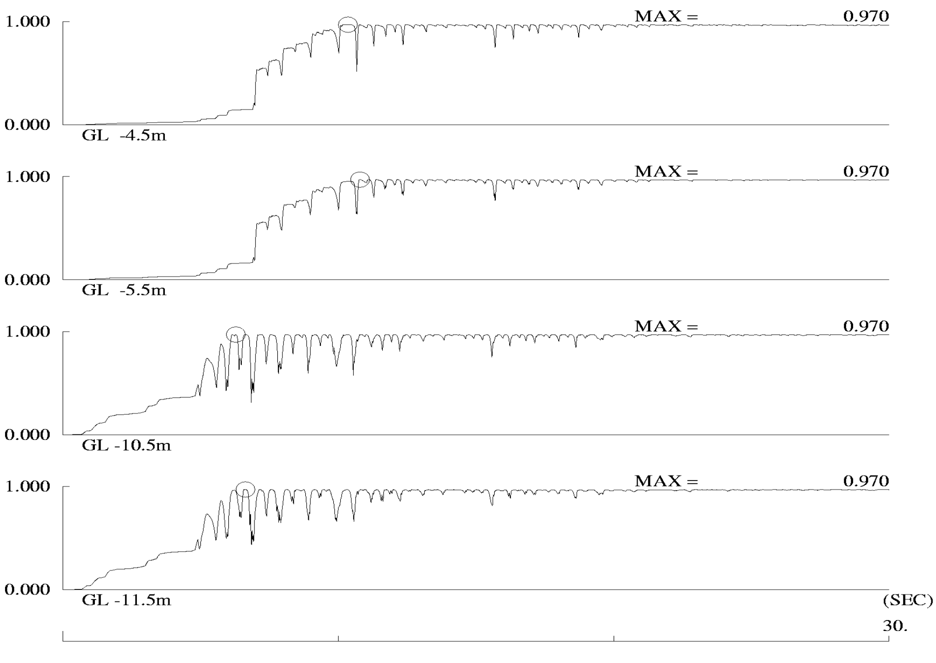

Figure 4.2 Time History of Excess Pore Water Pressure

4.2 Interaction Soil Springs

Figure 4.3 compares interaction soil spring constants (p561) that take pile group effects into account and those (p1) that do not, which have been calculated by the procedure described in the ANALYSIS METHOD section. A soil spring constant that does not take pile group effects into consideration is calculated by multiplying a soil spring constant for a single pile by the number of piles. As Figure 4.3 indicates, lateral soil springs are strongly affected by pile group effects, while shear soil springs are hardly affected. The pile group factor of the lateral soil springs for different soil layers, which is calculated by dividing a soil spring constant that takes pile group effects into account by a soil spring constant that does not, averages about 0.01. This value is smaller than the value given by the equation shown below, which is commonly used to calculate the pile group factors of soil spring constants in cases where lateral forces are applied to pile heads.

$$\alpha = \frac{1}{\sqrt{n}} = 0.04 \qquad n: \text{number of piles (561)}$$

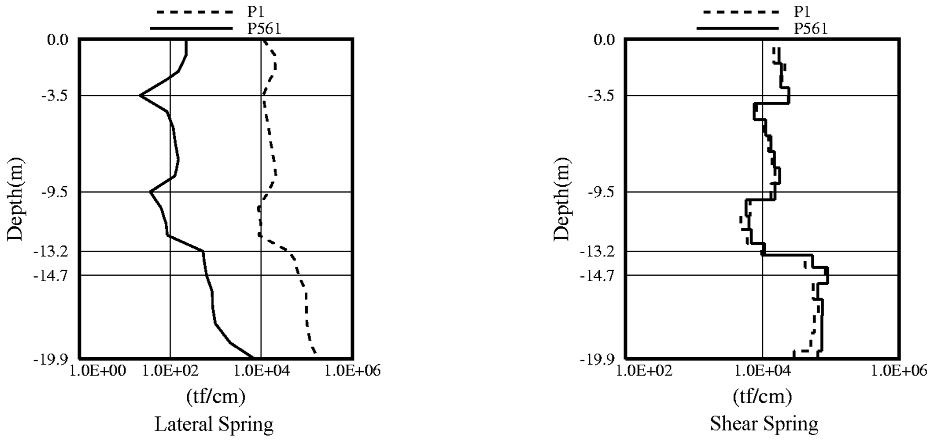

Figure 4.3 Comparison of Interaction Soil Springs

4.3 Results of Seismic Response Analysis of Tank-Pile Foundation-Soil System

4.3.1 Response Acceleration of Tank

Time histories of the tank and the base slab are shown in Figure 4.4. The maximum accelerations at the location of the base slab and of the internal tank are about 270 Gal and about 300 Gal, respectively. Since the static seismic coefficient used in design is 0.3, it can be concluded that the tank is safe.

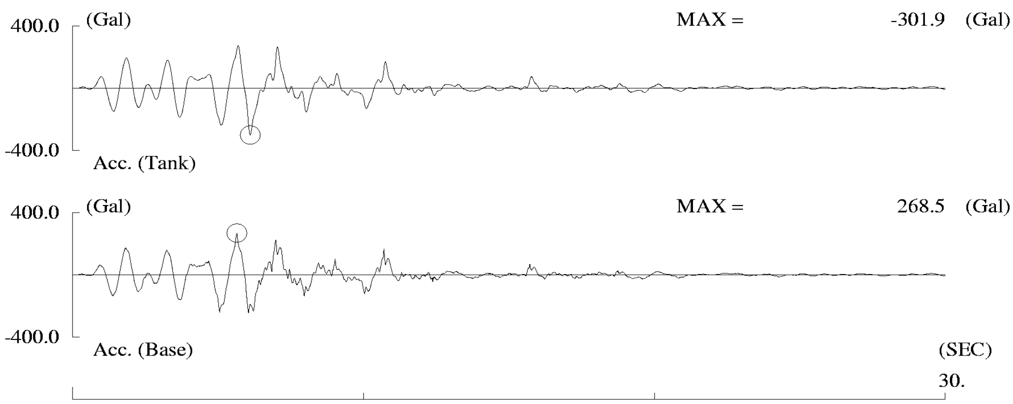

Figure 4.4 Time History of Above-ground Tank Acceleration

4.3.2 Evaluation of Seismic Safety of Piles

Pile stresses during an earthquake are composed of stresses that become large at and around pile heads because of inertial forces and stresses that become large in the ground because of ground displacements. From the standpoint of dynamic interaction, these pile stresses can be considered to correspond to inertial interaction due to inertial forces of structures and kinematic interaction due to ground motion inputs, respectively.

Pile stresses during earthquake (total pile stresses) are divided into inertial stresses and kinematic stresses. Kinematic pile stresses are calculated by assuming the weight of the superstructure to be zero. Figure 4.5 shows the distribution of total and kinematic pile stresses at different depths. Kinematic pile stresses at pile head are about 50%, indicating that stresses in the ground are mostly kinematic. The piles can be confirmed to be safe because the maximum stresses occurring in the steel pipe pile is smaller than yield stresses, regardless of the part of the pile (top, middle, or bottom pile segment (see Table 4.1).

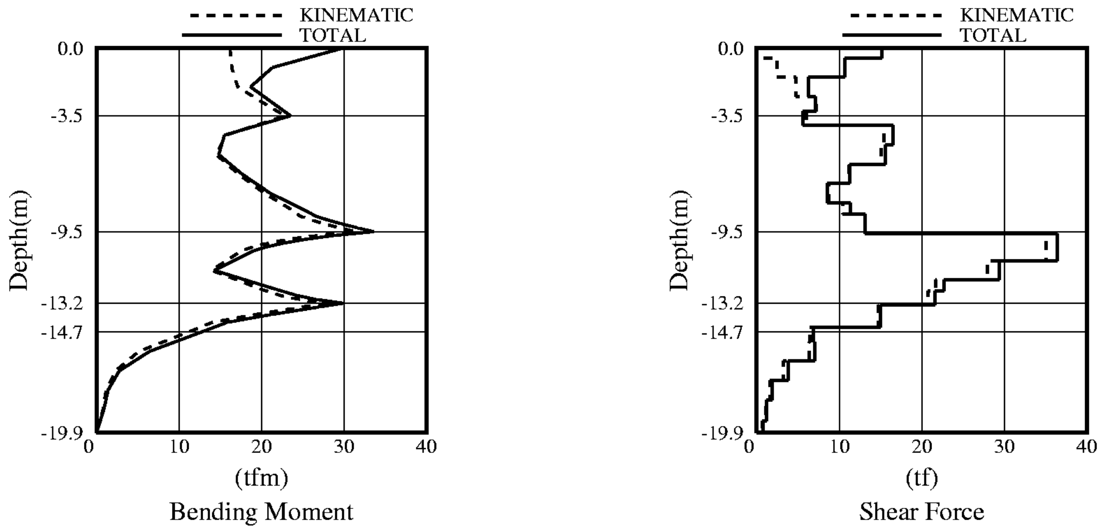

Figure 4.5 Pile Stress Distribution

Table 4.1 Maximum Bending Moment and Maximum Shear Force

	Bending moment (tf m)		Shear force (tf)	
	Maximum bending moment M_{max}	Fully plastic moment M_p *)	Maximum shear force Q_{max}	Yield shear force Q_y *)
Top Pile	29.8	80.7	15.2	296.6
Middle Pile	28.1	61.7	16.5	227.1
Bottom Pile	33.6	42.6	36.4	158.4

* Fully plastic moment Mp and yield shear force Q_y are calculated as follows:
$M_p = M_{p0} \cdot \cos(\pi/2 \cdot \alpha)$, $M_{p0} = Z_p \cdot \sigma_y$, $\alpha = N/(\sigma_y \cdot A)$

$$Qy = \tau_{xy} \cdot A, \qquad \tau_{xy} = \sqrt{\frac{\sigma_y{}^2 - \sigma_x{}^2}{3}}$$

where

M_{p0} : fully plastic moment in the case where there is no axial force, α : ratio between yield axial force and actual axial force N in the case where there is no moment, Z_p : plastic section modules, σ_y : yield stress for steel, N : axial force acting on each steel pile pile, A : cross-sectional area of steel pipe pile, τ_{xy} : shear stress σ_x : axial stress

5. PARAMETER STUDY

This section describes the results of a parameter study in which the maximum acceleration of input ground motion used in the seismic response analysis was varied.

5.1 Response of Free Ground

The maximum acceleration, maximum displacement, and excess pore pressure water (P.W.P.) distributions for input maximum accelerations of 200, 300, and 400 Gal are shown in Figure 5.1. As shown, the liquefaction zone becomes larger as acceleration increases. Relative displacement of soil increases sharply at GL-9.5 to -13.2 m where the soil liquefies almost completely in all cases.

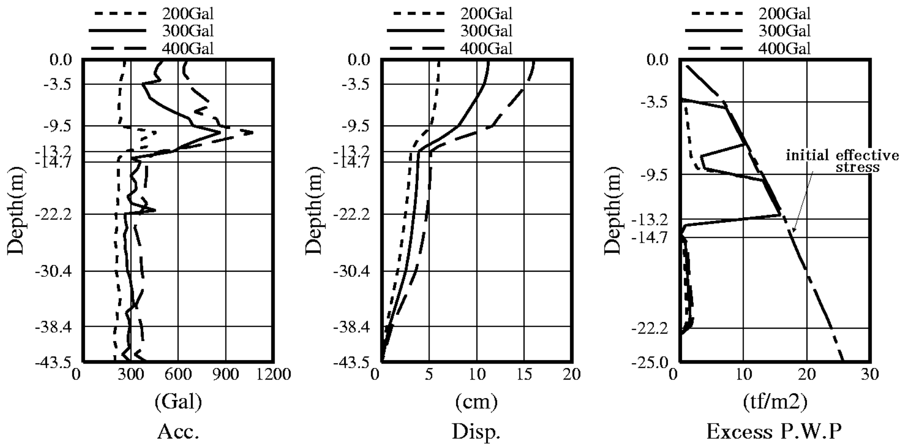

Figure 5.1 Maximum Response and Excess Pore Water Pressure Distributions

5.2 Pile Stresses

Figures 5.2 and 5.3 show the distribution of kinematic and inertial stresses at the pile head and at GL-9.5 m which corresponds to the boundary of soil strata. Pile stresses at the pile head and at GL-9.5 m do not reach the fully plastic moment level as the maximum acceleration increases from 200 to 400 Gal. The numbers in parentheses are the percentages of kinematic pile stresses (total pile stress=100%). It can be seen that as acceleration increases, the percentage of kinematic pile stresses at the pile head increases considerably. At GL-9.5 m, the percentage of kinematic pile stresses account for more than 90% at all acceleration levels.

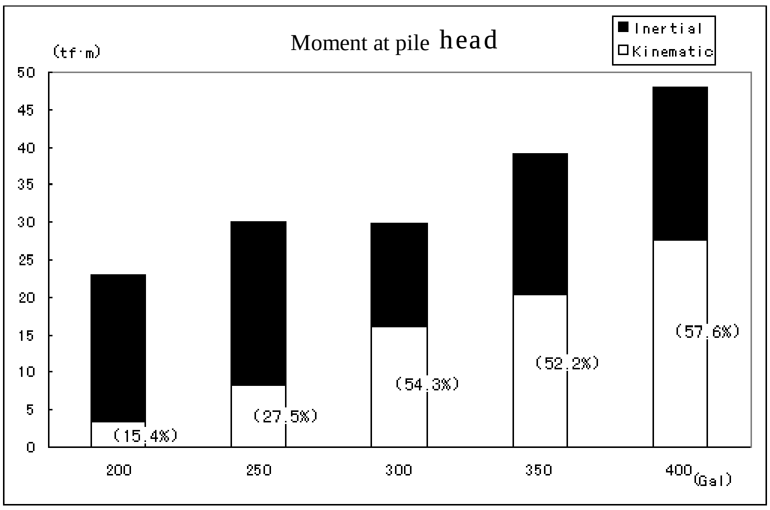

Figure 5.2 Comparison Between Kinematic and Inertial Pile Stresses at Pile Head

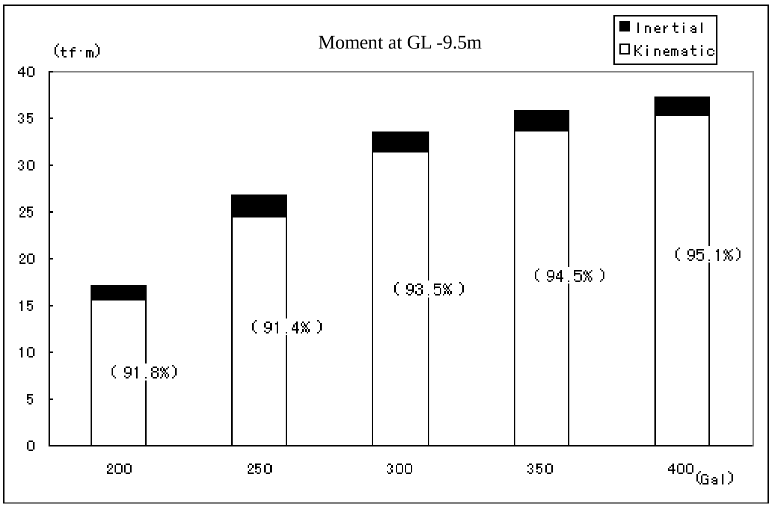

Figure 5.3 Comparison between Kinematic and Inertial Pile Stresses GL-9.5m

6. CONCLUSIONS

This study resulted in the following findings:

1. A dynamic response analysis of an existing tank supported by pile group were conducted by taking into account non-linear pile-soil interaction as well as excess pore water pressure generation. The analytical results show that the tank and piles are confirmed to be safe during strong earthquakes.

2. Bending moment at the pile head occurs not only because of inertial forces of superstructure, but also under the influence of ground displacements. As the input acceleration increases, bending moment due to ground displacement becomes larger and the contribution to the pile stresses increases by soil liquefaction.

3. Bending moment in the ground is mainly caused by ground displacement. As the input acceleration increases, the bending moment at the boundary of soil strata becomes larger because of amplification of ground displacement in the liquefied soil layer.

7. REFERENCES

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