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Misko Cubrinovski
Kiso-Jiban Consultants, Japan

Kenji Ishihara
Science University of Tokyo, Japan

Takahiro Kijima
Science University of Tokyo, Japan

EFFECTS OF LIQUEFACTION ON SEISMIC RESPONSE OF A STORAGE TANK ON PILE FOUNDATIONS

Misko Cubrinovski
Kiso-Jiban Consultants
Tokyo 102-8220, Japan

Kenji Ishihara, Takahiro Kijima
Science University of Tokyo,
Chiba 278-0022, Japan

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ABSTRACT

During the 1995 Kobe earthquake widespread liquefaction occurred in fill deposits of man-made islands in the port area of Kobe. The liquefaction of thick fill deposits resulted in extensive sand boils, large lateral movement and settlement of the ground. A number of piles has been found damaged or collapsed due to the excessive ground response as above. One important observation from the Kobe earthquake was that fewer signs of liquefaction and lesser ground deformation were found in reclaimed deposits that have been treated by ground improvement measures. In order to investigate the effects of liquefaction and ground improvement on the performance of pile foundations, a detailed study was conducted on a well-documented case history from the Kobe earthquake. This paper highlights the effects of liquefaction and ground improvement by sand compaction piles on the response of an oil-storage tank supported on pile foundations.

On the west part of the man-made Mikagehama Island, a 2450 kl oil-storage tank is located. The tank is supported on 69 high-strength concrete piles with a diameter of 45 cm. The surface layer at the tank site is a 13.6 m thick deposit of reclaimed gravelly soil. The loose fill deposit is fairly uniform and has rather low SPT resistance of 5 to 6 blow counts throughout the depth. Around the perimeter of the pile foundation, sand compaction piles have been installed down to a depth of 15 m, thus forming a 4 m thick belt zone of improved soil around the foundation. As a result of the ground improvement, the SPT resistance in the fill deposit significantly increased and reached values of 10-30 blow counts in the SCP zone and 20-40 blow counts in the foundation soil respectively.

To investigate the response of the soil-pile-tank system induced by the Kobe earthquake and examine the effects of liquefaction and soil improvement, a series of effective stress analyses was carried out. In the analyses, both soil and pile were modeled as elastoplastic materials. Material parameters of the fill deposits were determined based on results from field investigations and multiple series of laboratory tests on undisturbed samples recovered by means of the ground freezing technique. The influence of oil oscillation and effects of sloshing were accounted for by a simple mass-spring system representing the dynamic action of a fluid in a tank.

It was found that even though the excess pore pressures reached the effective overburden stress level in both unimproved and improved soils, the computed maximum shear strains in the ground were significantly reduced as a result of the ground improvement. The effects of ground improvement were clearly reflected on the pile response through a reduction in both displacements and bending moments of piles. Peak bending moments exceeding the yielding level were computed at the depth of the interface between the liquefied fill layer and non-liquefied silty soil layer where also the largest actual damage to the piles was observed in a close inspection of two piles of tank TA72 by bore-hole camera recordings.

KEYWORDS
Liquefaction, case history, 1995 Kobe earthquake, oil tank, pile foundations, ground improvement, effective stress analysis

INTRODUCTION

In the 1995 Kobe earthquake, a number of piles has been found damaged or collapsed as a result of liquefaction of thick fill deposits in reclaimed lands. The liquefaction-related damage to the piles was essentially caused by an excessive lateral movement of the liquefied soil layer. One important observation from the Kobe earthquake was that fewer signs of liquefaction and lesser ground deformation were found in reclaimed deposits that have been treated by some ground improvement measures. In order to evaluate the effects of liquefaction and ground improvement on the performance of pile foundations, a well-documented case history from the Kobe earthquake of an oil-storage tank supported on pile foundations was investigated in details.

OIL STORAGE TANK AT MIKAGEHAMA ISLAND

Mikagehama is a man-made island in the port area of Kobe, where a number of LPG and oil-storage tanks are located. A layout of the tanks and other facilities at Mikagehama is shown in Fig. 1. The subject of this investigation is the oil-storage tank TA72, which is located on the west side of the island, about 20 m from the revetment line. The tank, with a diameter of approximately 15 m and a similar height, has a storage capacity of 2450 kl.

Foundation and Soil Conditions

Cross sectional view of the tank and its foundation is shown in Fig. 2 whereas plan view of the foundation is shown in Fig. 3 respectively. The tank is supported on 69 high-strength concrete piles which are 23 m long and 45 cm in diameter. At the top, the piles are embedded in a 50 cm thick concrete slab.

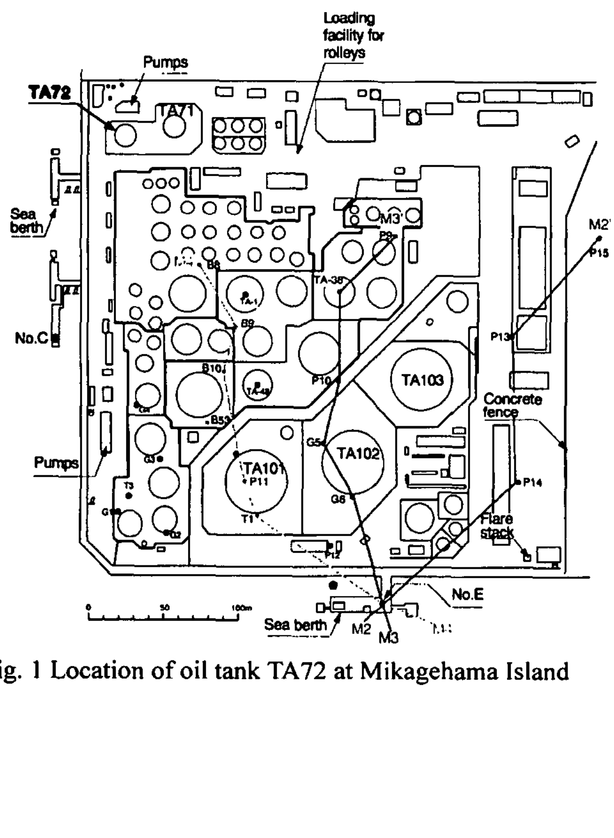

Fig. 1 Location of oil tank TA72 at Mikagehama Island

Ground conditions at Mikagehama Island are typical of the reclaimed land in the port area of Kobe, where a thick fill

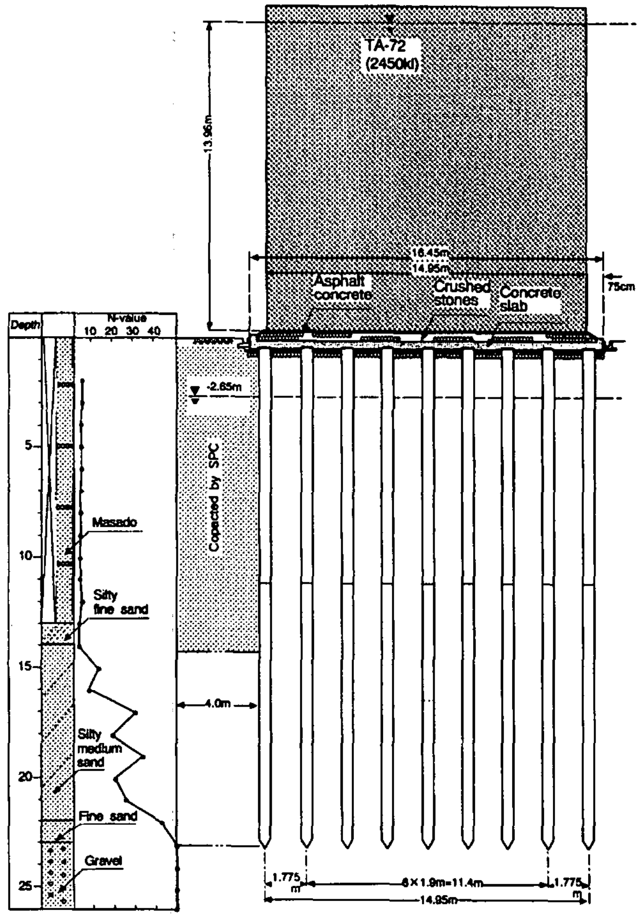

Fig. 2 Cross sectional view of tank TA72 and its foundation

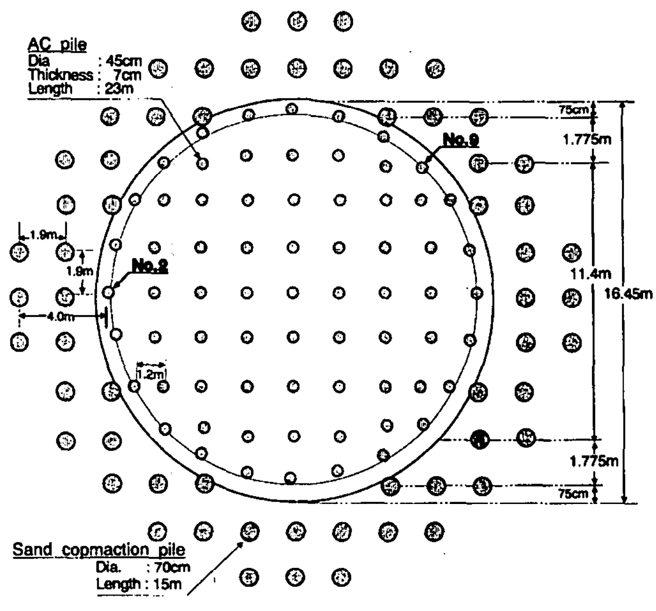

Fig. 3 Plan view of foundation

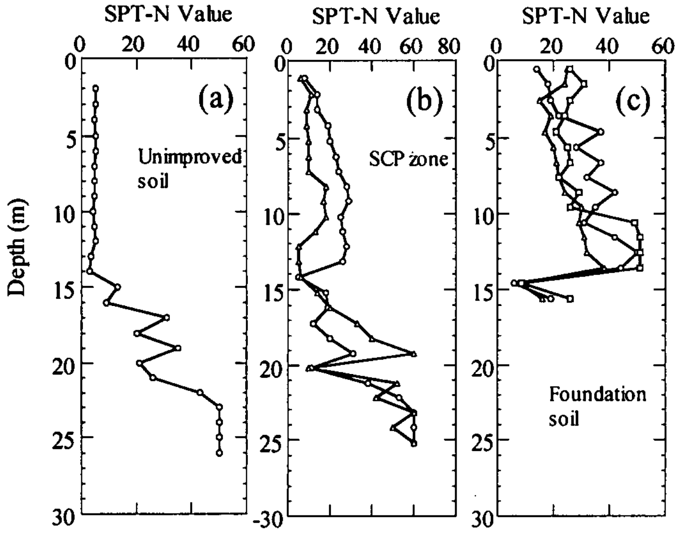

Fig. 4 SPT profiles: (a) unimproved soil; (b) SCP zone; (c) foundation soil

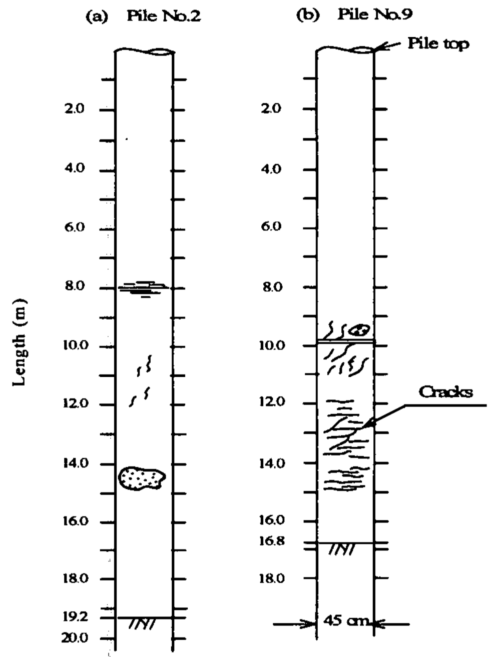

Fig. 5 Observed damage to piles of tank TA72: (a) Pile No. 2; (b) Pile No. 9

deposit of Masado overlies the original seabed layer and deeper gravel layers. At the TA72 tank site, the fill deposit of Masado and the underlying silty soil layer are 13.6 m and 10 m thick respectively. The ground water level is estimated at 2.5 to 3.0 m depth. Results of SPT measurements are shown in Fig. 4a where it may be seen that the original fill deposit of Masado has a uniform and rather low SPT blow count of 5 to 6 throughout the depth. The SPT resistance in the silty layer is in the range between 20 and 35 blow counts whereas the N value of the deeper gravel layer is 50.

In order to densify and improve the foundation soil, sand compaction piles were installed around the perimeter of the tank foundation, down to a depth of approximately 15 m. The arrangement of the compaction piles is schematically shown in Fig. 3. Standard penetration tests were conducted both in the sand compaction pile zone (soil in-between the compaction piles) and foundation soil (soil in-between the concrete piles) to quantify the effects of the ground improvement. The measured SPT resistance is shown in Figs. 4b and 4c for the SCP zone and foundation soil respectively. It is evident that as a result of the ground improvement, the SPT resistance of the Masado layer significantly increased and reached values on the order of 10 to 30 blow counts within the SCP zone and 20 to 40 blow counts in the foundation soil.

Damage Features

In the Kobe earthquake, extensive liquefaction occurred in the fill deposits of Masado soils at Mikagehama Island. Scattered sand boils, lateral movement, settlement and tilting of the ground were typical features of ground deformation (Ishihara, 1997). A number of cracks developed in the ground inland from the waterfront indicating that lateral spreading of the ground took place. The quay wall in the proximity to the tank TA72 moved towards the sea approximately 1 m resulting in a depression and large ground movement behind the quay wall. Results from ground surveying along a profile adjacent to the tank site indicate that the tank foundation was more or less affected by the lateral spreading.

To inspect the damage to the piles, a detailed field investigation was conducted on two piles of the tank TA72. The location of the investigated piles No. 2 and No. 9 is indicated in Fig. 3. A bore-hole camera was lowered into the inside of the hollow cylindrical piles to examine the damage to the piles. The outcome of the bore-hole camera recordings is summarized in Fig. 5, where distribution of cracks for the two piles is shown. It is important to note that the piles developed multiple cracks and suffered the largest damage at approximately 12 to 14 m depth, which is the depth at the interface between the fill deposit and silty soil layer.

LIQUEFACTION PROPERTIES OF FILL DEPOSITS

The fill material used for land reclamation (Masado) is a well-graded soil containing sand, some fines and a large portion of gravel. The gravel fraction is commonly in the range between 35 and 60 % while the fines content is from 5 to 15 %. Results of a series of cyclic undrained tests on undisturbed samples recovered by means of ground freezing technique from fill deposits in the port area of Kobe are shown in a summary form in Fig. 6. Here, in addition to the typical liquefaction strength plot showing a relationship between the cyclic stress amplitude and the number of cycles required to achieve a 5 % double amplitude strain in triaxial loading tests, the SPT

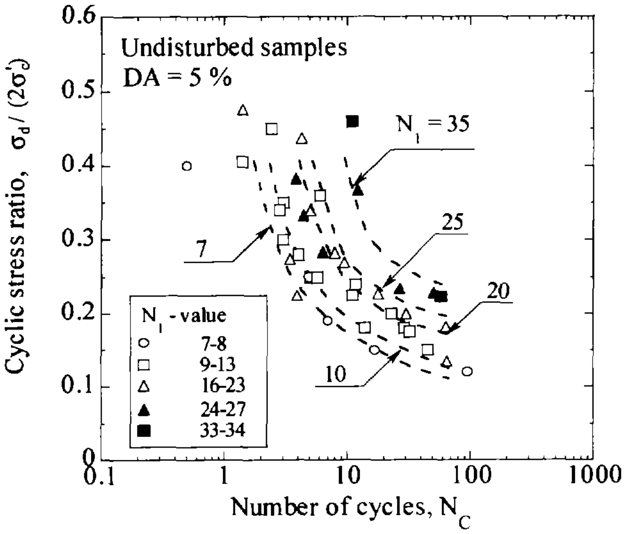

Fig. 6 Liquefaction strength of Masado soils

resistance corresponding to each sample or group of samples also is indicated. The large variation of the SPT resistance from 7 to 34 blow counts is due to the fact that the data shown in Fig. 6 contain results of tests on samples secured from both undensified and densified fill deposits. It is evident in Fig. 6 that the cyclic strength of Masado soils substantially increases with the SPT blow count.

NUMERICAL ANALYSIS

Since the overall response of the soil-pile-tank system is quite complex and is affected by a number of factors, a stepwise approach was adopted in the numerical study by separating it into two series of analyses. The first series examines the effects of liquefaction and ground improvement while the second one investigates the effects of lateral spreading. Here, the analyses concerning the effects of liquefaction and ground improvement are presented.

A fully coupled effective stress method of analysis of saturated soil was used to analyze the soil-pile-tank system (FEM code Diana-J). As shown in Fig. 7, the employed 2-D numerical model consists of four node solid elements and beam elements representing the soil and the pile-tank system respectively. The piles were modeled by equivalent beams representing the properties of a group of piles in a given row. A spring-mass system was used to model the oscillation and sloshing of the oil. Two numerical models were analyzed: (a) U-model (unimproved ground model), in which effects of ground improvement were neglected and all of the fill soils were assumed to have the properties of unimproved fill deposits (Fig. 4a), and (b) I-model (improved ground model), in which effects of ground improvement were taken into account and three different soil conditions were distinguished, as indicated in Fig. 7. The acceleration record of the Kobe earthquake recovered at Kobe Port Island (GL-32m; E-W) was used as a base input motion in the analyses.

Soil Model. The large gravel content and relatively wide variation in the grain-size distribution of Masado soils cause difficulties in evaluating the relative density of these soils and complicate the conventional characterization of sandy soils based on D_r. For these reasons, Ishihara et al. (1998) suggested to make use of the steady state concept as an alternative way of characterizing Masado soils. Based on data of SPT and undisturbed samples, the following correlation between the void ratio and penetration resistance of fill deposits of Masado was established

$$e_c = e_0 - \frac{\sqrt{N_1} - 1.73}{22.1} \qquad (1)$$

where e_c is in-situ void ratio, e_0 is void ratio defined by the intersection of the steady state line with the void ratio axis in the e-p' plot, and N_1 is the normalized penetration resistance to an overburden pressure of 98 kPa. For a known values of e_0 and N_1, this expression can be used to evaluate the void ratio of the fill deposits.

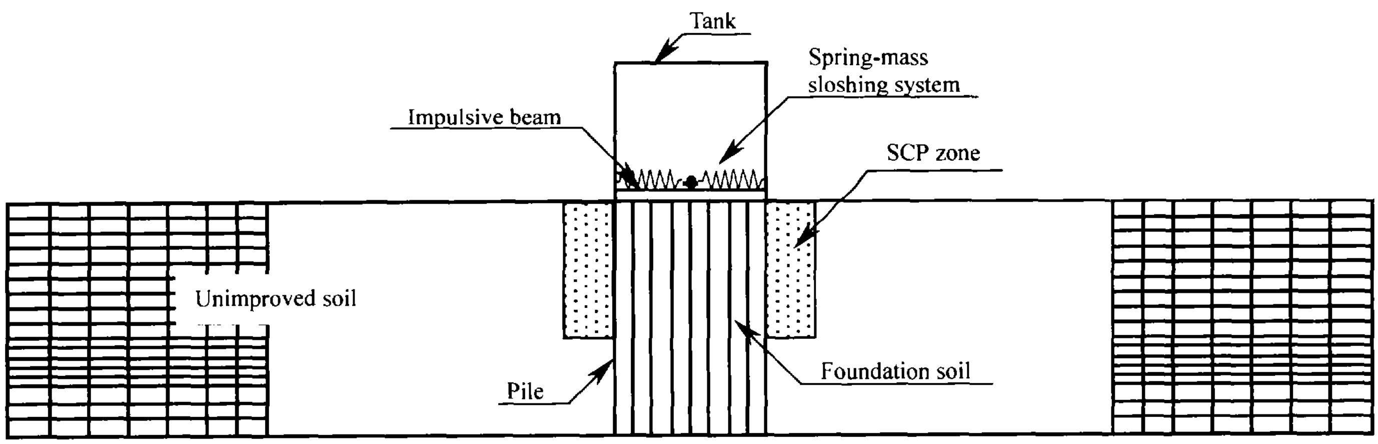

Fig. 7 Numerical model of the soil-pile-tank-fluid system

Table 1. Material parameters of Masado soils

Elastic	A = 199	n = 0.80	ν = 0.10
Stress-strain	$(\tau / p')_{max}$	$G_{N,max}$	$G_{N,min}$
	a_1 = 0.745	a_2 = 332	a_3 = 180
	b_1 = 0.10	b_2 = 60	b_3 = 10
Reference lines	UR-line	Steady state line	
	e_0 = 0.430	$e_s = 0.464 - 0.051 \log p'$	
Dilatancy	μ_0 = 0.18	M = 0.75	S_c = 0.012

An elastoplastic deformation law for sands which is based on a state concept interpretation of sand behaviour was employed in the analysis (Cubrinovski and Ishihara, 1998). A series of laboratory tests were conducted to determine the material parameters of Masado soils, as summarized in Table 1 (Cubrinovski et al., 2000). Using these parameters, the liquefaction strength of Masado soils was modeled for different densities or N_l values, as shown with the broken lines in Fig. 6. In these simulations, the void ratio was the only parameter that was varied according to the corresponding N_l value and correlation of Eq. (1).

In the case of the numerical model with improved ground (I-model), three soil zones with different densities were distinguished: (1) unimproved soil, (2) soil in the SCP zone, and (3) foundation soil among the concrete piles. Based on the SPT data shown in Fig. 4, average N_l values of 5, 20 and 30 were approximated throughout the depth of the unimproved soil, soil in the SCP zone and foundation soil respectively. Subsequently, corresponding void ratios of 0.407, 0.306 and 0.26 were calculated by using Eq. (1). Thus, the three different soil conditions indicated in Fig. 7 were modeled by using the material parameters listed in Table 1 and the respective value of the void ratio as determined above.

Pile-Tank-Fluid Model. Flexural properties of the piles were modeled by a nonlinear moment curvature relationship with the cracking, yielding and ultimate bending moments defined as: M_C = 68 kN-m, M_Y = 204 kN-m and M_U = 238 kN-m. The tank, on the other hand, was modeled with linear rigid-beams.

Effects of the oil oscillation were modeled by the simplified dynamic system for a fluid in a tank proposed by Housner (1963). According to this model, the fluid action is represented by a two-mass system: a mass M_i that is attached rigidly to the tank, and a mass M_s that can oscillate horizontally against a restraining spring. The first system represents the effects of the impulsive pressure or fluid that moves with the tank walls while the mass-spring system represents the effects of sloshing fluid. At the time of the earthquake, the oil level was at 3.41 m resulting in the following masses and corresponding elevations: M_i = 144 t, M_s = 268 t, h_i =1.27 m and h_s = 1.79 m (Fig. 7). It was found that the period of the sloshing oscillation was approximately 5 seconds.

RESULTS AND DISCUSSION

Ground Response

Computed excess pore pressures and maximum shear strair throughout the depth of the fill layer are shown in Figs. 8a an 8b respectively. Here, the response of the unimproved soil that computed for the free field condition; this response wa found to be practically identical for the unimproved an improved ground models. It is evident in Fig. 8a, that the fi layer completely liquefied, as illustrated by the excess poi pressures reaching or exceeding the effective overburde stress σ'_v. In the SCP zone and foundation soil, the poi pressure exceeded the effective overburden stress as a result c additional lateral stresses induced by the interaction betwee the piles and surrounding densified soils. The fill soil withi and around the foundation exhibited cyclic response typical c dense sands with a large fluctuation in the pore pressures an pronounced cyclic mobility.

Even though the maximum excess pore pressures developed t a similar level in both unimproved and improved soils, th computed maximum shear strains shown in Fig. 8b clearl demonstrate the effects of the ground improvement. It may b seen in this figure that the maximum shear strains reache about 3.5 % in the unimproved soils while the correspondin shear strains were less than 2 % and about 1.5 % in the soil c the SCP zone and foundation soil respectively. Thus, th influence of the ground improvement by sand compactio piles was mostly reflected in a significant reduction of th shear strains or ground deformation in the foundation soil.

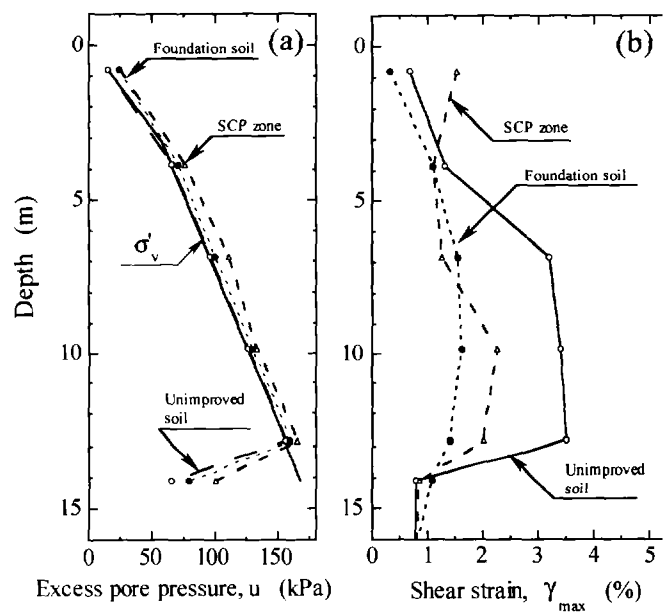

Fig. 8 Computed ground response: (a) maximum excess pore water pressures; (b) maximum shear strains

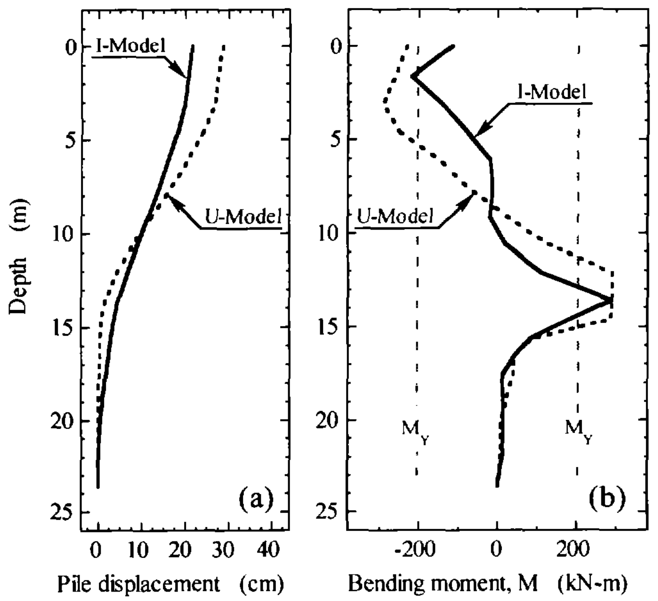

Fig. 9 Computed response of piles: (a) displacements; (b) bending moments

Response of Piles

Computed displacements and bending moments of end-piles (the left-most row of piles in Fig. 7) are shown in Fig. 9. Here, results of the unimproved ground model (U-model) and improved ground model (I-model, shown in Fig. 7) are comparatively shown. Each data set presented in this figure is for respective time of occurrence of a peak displacement at the pile top or a peak bending moment at the interface between the fill layer and underlying silty soil layer at 13.6 m depth.

It is evident in Fig. 9 that the peak displacement at the pile top was reduced from 29 cm to about 20 cm as a result of the ground improvement. In the case of the model with ground improvement, the pile displacement throughout the depth of the fill layer was reduced to a half of that of the unimproved model; on the other hand, the displacement within the silty soil layer increased for 1 to 3 cm.

Generally, the distribution of bending moments along the pile length was similar in the analyses with unimproved and improved ground models except for the zones of peak bending moments near the pile top and at the interface between the liquefied fill layer and non-liquefied silty layer. The largest contribution of the ground improvement is seen in the reduction of the bending moments in the uppermost 5 m of the pile where the bending moment was brought back from the ultimate moment to a level below the yielding moment. Similarly, the pile flexural response was reduced in the zone of the interface between the Masado layer and silty soil layer. However, even in the analysis with the improved ground model, the bending moment at the interface exceeded the yielding level. This response is in accordance with the observed damage to the piles shown in Fig. 5.

CONCLUDING REMARKS

Effects of liquefaction and ground improvement by sand compaction piles on the response of an oil-storage tank on pile foundations was investigated. It was found that ground deformation within the foundation soil was significantly reduced as a result of the ground improvement resulting in a consequent reduction in both displacements and bending moments of the piles. The present study clearly exemplifies the importance of the ground deformation and displacement pattern on the response of piles when surrounding soils liquefy. Thus, even a similar level of excess pore pressures induced by a strong earthquake may result in a substantially different ground deformation and response of soils of different densities.

It is to be mentioned that the actual response of tank TA72 was more or less affected by lateral spreading of the ground. In addition, the geometry of the foundation and configuration of the ground improvement indicate that 3-D effects may have been pronounced in the response of the soil-pile-tank system. The presented numerical results in this study, therefore, should not be understood as an attempt to simulate the actual response of tank TA72, but rather as an effort to quantify the effects of sand compaction piles on the performance of the pile foundation. Influence of lateral spreading and 3-D effects will be subject of a detailed investigation in a subsequent study.

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