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ABSTRACT

The effect of the sand compaction pile method as a countermeasure for liquefaction mainly consists of three factors: increase in the density, increase in the horizontal effective stress and stabilization of microstructure. Proper evaluation of the effect of improvement is important for estimating the seismic behavior of the ground improved by the sand compaction pile method. How to incorporate the effect and its factors into an analytical model was investigated by simulating the seismic behavior of the ground at two sites during the 1995 Hyogoken-Nambu earthquake with the effective stress analysis method "FLIP." It was found that not only the increase in the density but also increase in the horizontal effective stress were important in explaining the effect of the sand compaction pile method. Moreover, a model taking account of both sand piles and the improved ground between them suggested a possibility of reproducing the behavior of improved ground under large ground motions more properly.

INTRODUCTION

An increase in the liquefaction strength of sand is achieved by three factors: an increase in the density, increase in the horizontal effective stress (increase in K_0), and stabilization of the microstructure (Yoshimi, 1990). Liquefaction strength is known to increase as density increases, which is normally evaluated in design practice. In addition, soil testing (Ishihara et al., 1976) and in-situ testing of improved ground (Harada et al., 1998) have revealed that an increase in the horizontal effective stress actually increases the liquefaction strength. The sand compaction pile method as a countermeasure for liquefaction is considered to increase the liquefaction strength of the ground by achieving all three factors, as it improves the ground by vibration. When evaluating the effect of the sand compaction pile method in detail, it is necessary to evaluate these factors separately. Such evaluation requires the liquefaction strength and shear modulus of sand obtained from subsurface exploration and soil testing before and after the improvement. However, SPT N-values are the only information available in many cases of actual design and are used for estimating such parameters as liquefaction strength. In such a case, an increase in the SPT N-value is usually regarded as a result of an increase in the density.

Most sites of ground that had been improved by the sand compaction pile method during the 1995 Hyogoken-Nambu earthquake withstood ground motions of PGA exceeding 200 cm/s^2, the designed improvement level. This suggests that another factor, such as an increase in the horizontal effective stress, contributed to the resistance in addition to the effect of increase in SPT N-value regarded as the increase in density, which is usually used in practical design.

Moreover, a model that estimates only the improved ground between sand piles, which is normally the case, can lead to underestimation of the effect of the sand compaction pile method. It is important to evaluate the effect of improvement in detail not only for the improved ground between sand piles but also for sand piles themselves during large ground motions. Changes in density from sand pile to sand pile are also important.

To evaluate the effect of improvement properly during such large ground motions as recorded at the Kobe area in the 1995 Hyogoken-Nambu earthquake, the following are considered to be important:

1) Evaluating the contribution of not only an increase in the density but also an increase in the horizontal effective stress (increase of K_0) to the liquefaction strength of soil.
2) Evaluating the strength of sand piles, which have a higher potential of liquefaction strength, as well as the changes in the effect of improvement of the areas from near the piles to the midpoint between piles, in addition to the conventional and conservative strength estimation based only on SPT N-values of midpoints between piles. Figures 1 and 2 show these effects.

SPT N-values, which are also used as an index to the effect of improvement in many cases, are considered to reflect both increases in the density and horizontal effective stress. It is important to examine the contributions of these effects included in an SPT N-value separately for evaluating the effect of improvement. Figure 3 shows the relationship between an SPT N-value and these effects.

In this study, in order to examine these problems analytically, the authors conducted an effective stress analysis of the behavior of two sites of improved ground in reclaimed land in the Kobe area during the 1995 Hyogoken-Nambu earthquake.

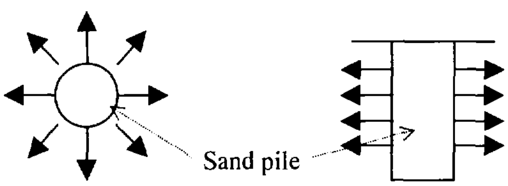


Fig. 1. Increase in the horizontal effective stress by sand compaction pile.

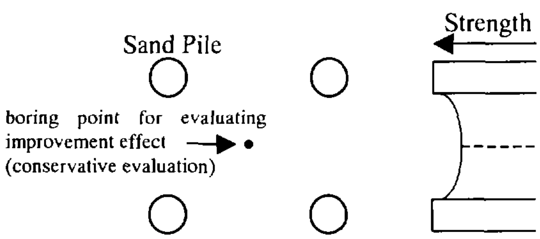


Fig. 2. Distribution of liquefaction strength of sand piles and those of improved ground between sand piles.

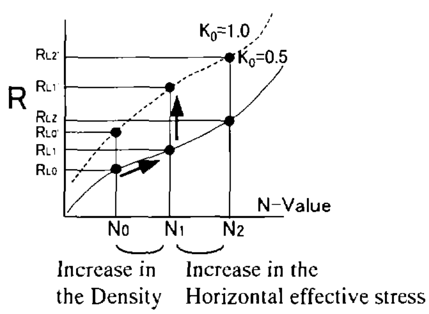


Fig. 3. Increase of SPT N-value by increase in the density and increase in the horizontal stress.

SOIL CONDITION AND MODELING OF THE SITES

Effect of improvement during Hyogoken-Nambu earthquake

The 1995 Hyogoken-Nambu earthquake caused severe liquefaction over wide areas of reclaimed land along Osaka Bay. However, there were some areas that scarcely liquefied among severely liquefied areas. Most of these areas had been improved by such methods as the sand compaction pile method, rod compaction method, and sand drain method. These methods proved to be effective in reducing the liquefaction potential and damage caused by liquefaction. In this study, the authors focused on the effect of the sand compaction pile method, and selected two sites that had been improved by this method and experienced the 1995 Hyogoken-Nambu earthquake. They are located in the western part of Nishinomiya-hama Island and in the southern part of Rokko Island, having different improvement ratios and degrees of damage. For these sites, effective stress analysis was conducted with various conditions including the horizontal effective stress and contribution of sand piles themselves. The authors investigated reasonable modeling of the ground improved by the sand compaction pile method during large ground motions, comparing the results and the actual behavior observed at the sites. Figure 4 shows the epicenter of the 1995 Hyogoken-Nambu earthquake and sites under analysis.

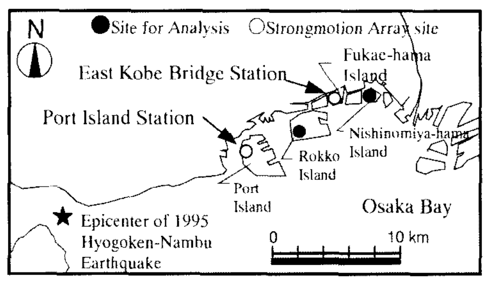


Fig. 4. Location of the sites.

Nishinomiya-hama Island site

One is on Nishinomiya-hama Island, which is one of the reclaimed lands between Osaka and Kobe. At the site, the sand compaction pile method was conducted at a relatively low improvement ratio of 5 % mainly for reducing consolidation settlement (Suwa et al., 1997). During the 1995 Hyogoken-Nambu earthquake, severe liquefaction occurred in unimproved areas of the island and another unimproved reclaimed land near the site. Sand boils also appeared even at improved areas of the site, which proved the occurrence of liquefaction at the site though the degree was not so severe (Inoue et al., 1996, Suwa et al., 1997). Since detailed soil conditions were not obtained, an analytical model was evaluated by referring to past tests and investigation from not only the site but also grounds nearby (Geo-Database Information Committee in Kansai, 1998). The surface layer

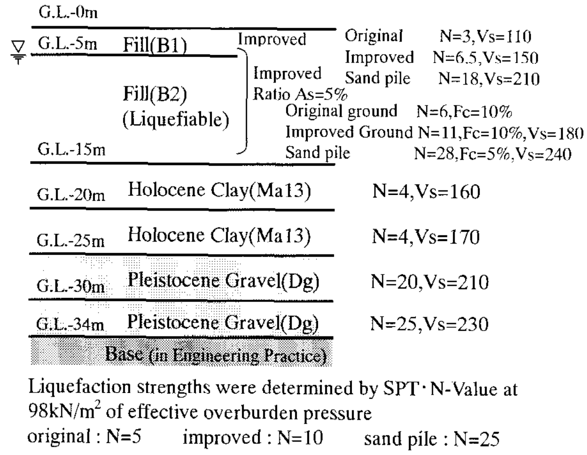
15 m in thickness was filled and improved, under which lies a Holocene clayey layer between G.L.-15 m and G.L.-25 m. A Pleistocene gravel layer appears under the clayey layer. G.L.-34 m was assumed to be the base in engineering practice where the shear wave velocity was more than 300 m/s according to the result of PS logging at a site in Fukae-hama Island (Isemoto et al., 1997) near the site. The ground water level was assumed at G.L.-3 m, because the ground water level is generally about 2 m above the sea level in reclaimed land around the site and the ground level is about 5 m above the sea level in Nishinomiya-hama Island. The liquefiable layer was therefore assumed to range between G.L.-3 m and G.L.-15 m.

Rokko Island site

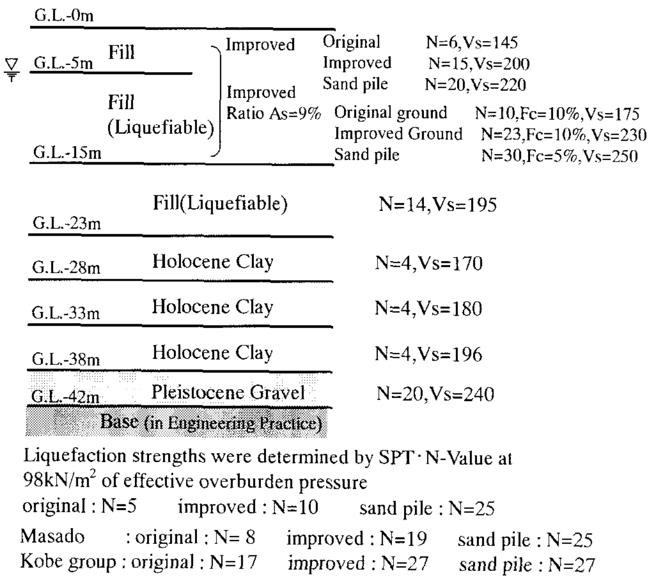
The other site is located in the southern part of Rokko Island in Kobe. At the site, the upper part of the reclaimed layer below G.L.-15 m was improved by the sand compaction pile method with an improvement ratio of about 9 % to increase the bearing capacity and prevent liquefaction. A large number of sand boils occurred, and the settlement was relatively large at about 30 cm during the 1995 Hyogoken-Nambu earthquake, suggesting that liquefaction was relatively severe in the northern part of Rokko Island. On the contrary, few sand boils occurred and the settlement was mostly less than 10 cm in the southern part of Rokko Island, suggesting that the degree of liquefaction was low even though it actually occurred. An analytical model was evaluated by referring to the results of past site investigations on and around the site. The surface layer of 23 m in thickness was filled, under which lies a Holocene clayey layer between G.L.-23 m and G.L.-33 m. The filled layer above G.L.-15 m was improved. A Pleistocene gravel layer appears under the clayey layer. The base in engineering practice was assumed to appear at G.L. -42 m from the result of many site investigations in Rokko Island (Geo-Database Information Committee in Kansai, 1998). Since the ground water level was assumed at G.L.-5 m, the liquefiable layer ranged between G.L.-5 m and G.L.-23 m. Figure 5 shows the soil profiles of the two sites.

Dynamic property of the soil

In this study, a method for proper modeling was investigated by comparing the analytical results with the actual behavior observed in reconnaissance after the earthquake in consideration of (1) unimproved ground (original ground), (2) increases in the density of soil by the sand compaction pile method, (3) increases in the horizontal effective stress, and (4) both effects of increase in the density and horizontal stress. Table 1 shows the cases examined in this study. For convenience, the increase in the SPT N-value was assumed to depend on the increase in the density of soil, and the effect of the increase in the horizontal effective stress was taken into consideration separately, because the contributions of these effects were unknown. As mentioned above, because the SPT N-value was also increased by the increase in the horizontal



(a) Nishinomiya-hama Island Site



(b) Rokko Island Site

Fig. 5. Soil profiles.

Table. 1. Case of numerical analysis.

		Modeling			
		Original Ground	Improved Ground		
			Only between sand piles	Considering sand piles	
				Averaged	Super-imposed
Effect of Increase in the Horizontal	Not Considering K_0=0.5	Case-1	Case-3	Case-5	Case-7
Effective Stress	Considering K_0=0.7(1.0)	Case-2	Case-4	Case-6	Case-8

stress, liquefaction strength may be overestimated to some extent in such a treatment (shown in Fig. 4). The method of evaluating these effects should be a subject for the future.

Liquefaction strengths were basically determined by SPT N-values according to the method proposed by Tokimatsu and Yoshimi (Tokimatsu et al. 1983) as follows: The average SPT N-value at the site was assumed to be the value at mid-depth of the liquefiable layer. Based on this value, the SPT N-value under an effective overburden pressure of 98.1 kN/m^2 was obtained. This value was corrected into Na in regard to the fines content, and the liquefaction strength ratio defined by 15 cycles with a shear strain of 5% was then determined from the relationship between the cyclic stress ratio and the Na-value based on the Tokimatsu/Yoshimi method (Tokimatsu et al., 1983). The influence of the number of cycles was corrected by the relationship between the number of cycles and the scaling factor for the stress ratio (Seed et al., 1983) and the relationship between the magnitude of earthquake and the factor of liquefaction resistance (Yoshimi et al. 1978). Consequently, liquefaction strengths of 5, 10, 15, 26 cycles were obtained. The parameters of liquefaction strength for effective stress analysis were set by elemental simulation for fitting strength from the simulation to the determined liquefaction strengths mentioned above.

Shear wave velocities of filled layers were determined from the SPT N-values by Imai's formula (1) for Holocene sand (AIJ, 1993), because shear wave velocities were not investigated on both the original and improved ground.

$$Vs=80.6N^{0.331} \tag{1}$$

where Vs = shear wave velocity (m/s), N = SPT N-value

The values for the other layers were determined according to the results of PS logging tests around the site (Isemoto, 1997, Kobe city, 1995), because the shear wave velocities did not widely differ in these layers in each site.

In the Nishinomiya-hama Island site, because the SPT N-value after the improvement was unknown, the increase in the SPT N-value was determined from the relationship between the improvement ratio and the increase in the SPT N-value according to the investigation of the improved ground shown in Fig. 6 (Fudo Construction Co., Ltd, 1971). The improvement ratio was 5% at the site, and therefore the SPT N-value of the improved ground was supposed to increase to 10 from the value of the original ground of 5. In the Rokko Island site, the improvement ratio was about 9%, and the average SPT N-values of the original and improved ground were 8 and 19, respectively according to the site investigation. The material of the fill in the Rokko Island site was Kobe group formation instead of decomposed granite soil "Masado." In this study, two cases of filling with Masado and Kobe group formation were considered. Because the liquefaction strength of Kobe group formation was higher than that of "Masado" by between 0.02 and 0.07 (Ohta et al., 1997, Tanaka et al. 1999), the liquefaction strength of Kobe group formation was assumed to correspond to Na=17 (corrected SPT N-value) from the method mentioned above, that is the same as the strength in these researches. After the improvement, the SPT N-value was assumed to increase by 10 from that of the original ground to 27.

The increase in the horizontal effective stress was assumed to be expressed by the increase in the coefficient of earth pressure K_0, and its effect was examined. K_0 was set at 0.5 in the case of not considering the effect and 0.7 and 1.0 for Nishinomiya-hama Island and Rokko Island, respectively, in the case of considering the effect according to the relationship between the improvement ratio and the horizontal stress shown in Fig. 7 (Harada et al., 1998). Figure 8 shows the method of incorporating the effect of the increase in the horizontal effective stress (increase in K_0) into the analysis. At first, a static analysis by gravity was conducted, and, for incorporating the increase in K_0, an additional horizontal stress was then loaded as concentrated loads at nodes. Finally the dynamic analysis was performed.

The effect of improvement was normally estimated by the SPT N-Value at the midpoint between piles as a conservative estimation. However, a more accurate estimation is necessary for the design against strong ground motions. In modeling of improved ground, three cases were considered: (1) a conventional model in which only ground between sand piles was considered, (2) a model in which strength and rigidity of sand piles were considered and modeled into one element in weighted average of sand piles and ground in between

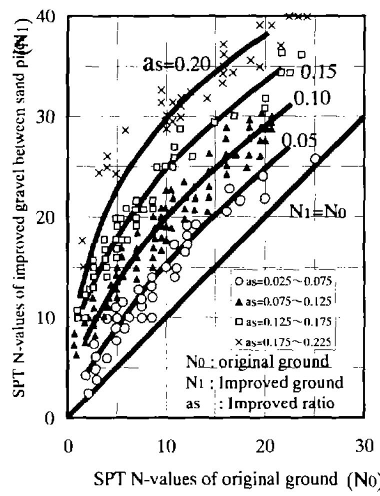


Fig. 6. Relationship between SPT N-values of original ground and those of improved ground between sand piles. (after Fudo Construction Co., Ltd.)

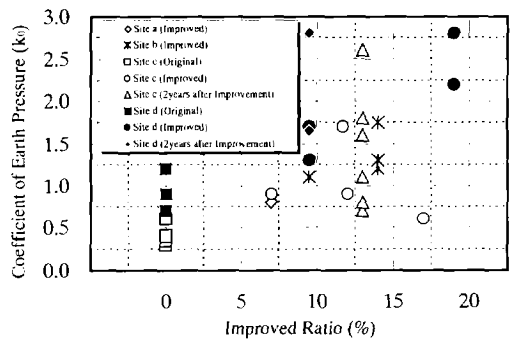


Fig. 7. Relationship between improved ratio and coefficient of earth pressure (K_0). (after Harada et al.)

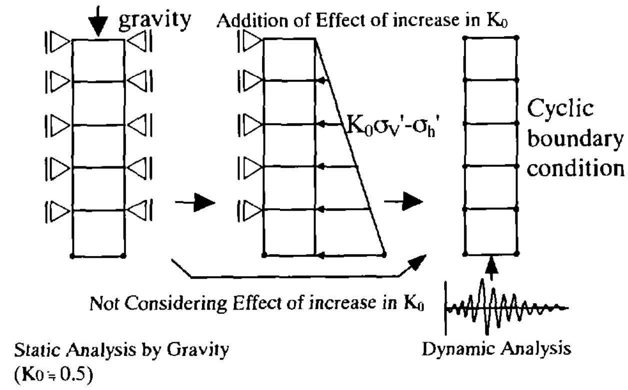


Fig. 8. Method for considering the effect of Increase in the horizontal effective stress in analysis.

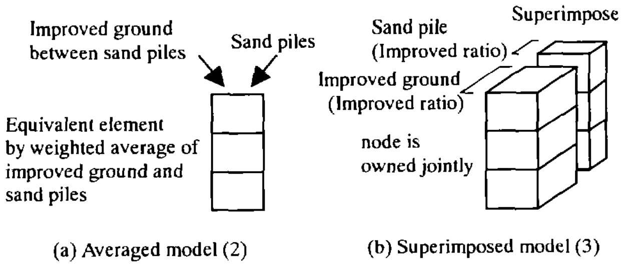


Fig. 9. Modeling of effect of sand piles themselves.

according to their contributions determined by the improvement ratio (averaged model), and (3) a model in which sand piles and ground in between were evaluated separately into two elements, which were superimposed according to their contributions (superimposed model). Figure 9 shows the methods of (2) and (3). The SPT N-value of the piles was set at 25. In the Kobe group formation case, the SPT N-value of the piles was set at 27, because the SPT N-value of improved ground was set at 27.

The authors conducted effective stress analyses (analysis code: FLIP) on the two sites using a model based on a multiple shear mechanism defined in a strain space (Iai et al. 1992). Table 2 shows the model parameters for the analysis. Records of vertical array observations during the 1995 Hyogoken-Nambu earthquake near the sites were used for input motions. An observed record at G.L.-34 m at the East Kobe Bridge station about 3 km to the west of the Nishinomiya-hama site was used for the site, whereas a record at G.L.-33 m at the Port Island station about 5 km to the west of the Rokko Island site was used for the site. The strong motion records were rotated to a predominant direction of N320E, because the shear stress would be largest in the predominant direction. Figure 10 shows the input motions.

RESULTS OF RESPONSE ANALYSIS

Nishinomiya-hama Island

Figure 11 shows the maximum response distribution of shear strain and excess pore water pressure comparing the effect of

Table. 2. Model parameters for analysis.

a) parameters

Symbol	Type of mechanism	Parameter designation
K_a	elastic volumetric	rebound modulus
G_{ma}	elastic shear	shear modulus
ϕ_f	plastic shear	shear resistance angle
ϕ_p	plastic dilatancy	phase transformation angle
H_m	plastic shear	upper bound for hystereetic damping factor
σ'_{m0}		effective confining pressure
p_1	plastic dilatancy	initial phase of cumulative dilatancy
p_2	plastic dilatancy	final phase of cumulative dilatancy
w_1	plastic dilatancy	overoll cumulative dilatamcy
S_1	plastic dilatancy	ultimate limit of dilatamcy
c_1	plastic dilatancy	threshold limit for dilatamcy

b) Model parameters for Nishinomiya-hama Island

		$\sigma m_0'$ (kN/m²)	Gm_0 (kN/m²)	Km_0 (kN/m²)	ρ (t/m³)	Hmax	φf (deg.)	φp' (deg.)	Parameters for dilatancy s1	w1	p1	p2	c1
Fill 1	Original	23.2	26300	70100	2.1	0.20	40						
	Improved	23.2	47300	125700	2.1	0.20	42						
	Sand Pile	23.2	91700	224100	2.1	0.20	42						
	Average	23.2	49800	132500	2.1	0.20	42						
Fill 2	Original	94.9	44800	119100	2.1	0.20	40	30	0.005	4.7	0.48	1.0	1.7
	Improved	94.9	68800	183000	2.1	0.20	40	30	0.005	5.5	0.80	1.0	2.0
	Sand Pile	94.9	124000	329900	2.1	0.20	42	30	0.005	35.0	1.00	1.0	2.7
	Average	94.9	71900	191200	2.1	0.20	42	30	0.005	6.4	0.80	1.0	2.1
Holocene Clay		156.3	43500	115800	1.7	0.22	30						
		182.1	49100	130700	1.7	0.22	30						
Pleistocene Gravel		211.5	83800	222900	1.9	0.22	35						
		241.3	100500	267400	1.9	0.22	35						

c-1) Model parameters for Rokko Island ("Masado")

		$\sigma m_0'$ (kN/m²)	Gm_0 (kN/m²)	Km_0 (kN/m²)	ρ (t/m³)	Hmax	φf (deg.)	φp' (deg.)	Parameters for dilatancy s1	w1	p1	p2	c1
Fill 1	Original	36.788	41472	110344	2.0	0.20	42						
	Improved	36.788	77618	206517	2.0	0.20	42						
	Sand Pile	36.788	94178	250577	2.0	0.20	42						
	Average	36.788	79209	210731	2.0	0.20	42						
Fill 2	Original	110.363	60552	161109	1.0	0.20	42	30	0.005	5.0	0.7	1.0	1.8
	Improved	110.363	103968	276625	1.0	0.20	42	30	0.005	45.0	0.5	1.0	1.7
	Sand Pile	110.363	124002	329930	1.0	0.20	42	30	0.005	42.0	0.5	1.0	1.4
	Average	110.363	105800	281500	1.0	0.20	42	30	0.005	45.0	0.5	1.0	1.6
Fill 3		176.580	75272	200275	1.0	0.20	40	30	0.005	6.0	0.7	1.0	1.9
Holocene Clay 1		217.046	46240	123030	0.6	0.22	30						
Holocene Clay 2		239.119	51840	137930	0.6	0.22	30						
Holocene Clay 3		261.191	57760	153681	0.6	0.22	30						
Pleistocene Gravel		284.000	103680	275859	0.6	0.22	35						

c-2) Model parameters for Rokko Island ("Kobe group")

		Parameters for dilatancy s1	w1	p1	p2	c1
Fill 2	Original	0.005	10.0	0.5	1.10	2.45
	Improved ground	0.005	54.0	0.5	1.20	5.00
	Average	0.005	60.0	0.5	0.95	3.40
	Sand Pile	0.005	50.0	0.5	0.78	1.00
Fill 3		0.005	10.0	0.5	1.10	2.45

Improved : Improved ground between sand piles
Average : weighted average of improved ground and sand piles

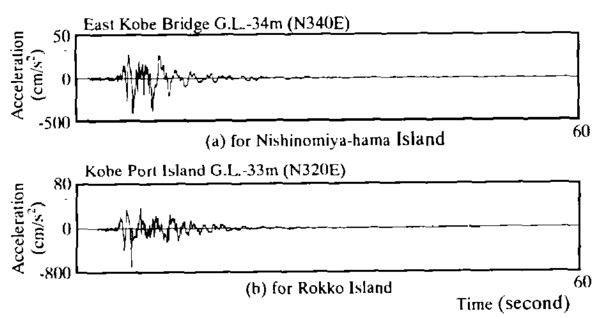


Fig. 10. Input ground motion for analysis.

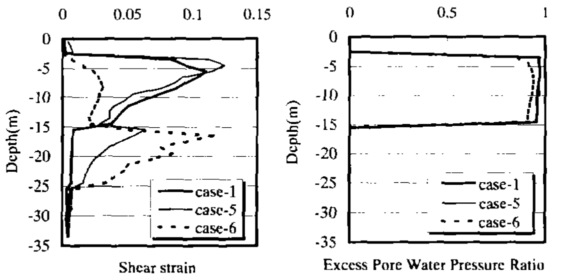


Fig. 11. Effect of factors for improvement.

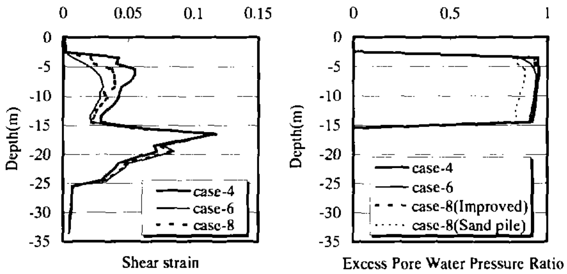


Fig. 12. Modeling of improved ground.

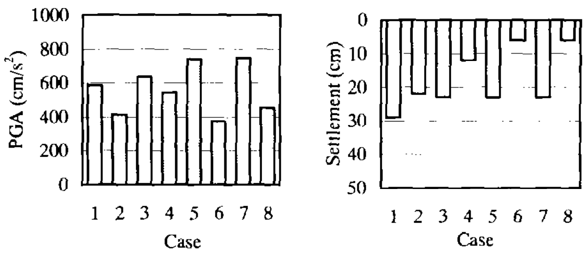


Fig. 13. PGA and Settlement of ground.

factors of improvement. In the original ground, the excess pore water pressure of the filled layer built up to as high as the initial effective overburden pressure, which basically corresponded to the level of occurrence of the observed liquefaction. In the case of not considering the increase in the horizontal effective stress (K_0=0.5), despite the relative high liquefaction strength, the excess pore water pressure of the filled layer built up to as high as that of the unimproved case and a large shear strain also appeared at the same level as the unimproved case. Therefore this case did not express different behavior of improved ground. On the other hand, in the case of considering the increase in the horizontal stress (K_0=0.7), the shear strain level was reduced to about 1/3 of the original case, though excess pore water pressure build up to a high level. This corresponded with the reduction in the damage of liquefaction by the improvement observed at the site.

Figure 12 shows the excess pore water pressure and shear strain distribution comparing different methods of modeling of the improved ground. Though the pore water pressures were almost the same, the shear strain of the case considering sand piles (case 6, 8) decreased to 2/3 or 1/2 from the case not considering sand piles (case 3). This suggests a possibility of properly estimating the effect of the sand compaction pile method under strong ground motions by taking account of the effect of sand piles themselves as well. The settlements were evaluated using the relationship between shear strain and volumetric strain of liquefied sandy layers according to Ishihara & Yoshimine (Ishihara & Yoshimine, 1992). The relative density of the reclaimed layer was set at 65% for the original ground and 75% for the improved ground referring to the results of soil tests (Tanaka, 1999). Figure 13 shows the evaluated settlements. The original ground settled approximately 29 cm. Though the settlement was similar to the original case at 23 cm in the case of not considering the increase in the horizontal stress, the settlement decreased to between 6 and 8 cm when considering the increase in the horizontal stress. Though the observed settlement in Nishinomiya-hama Island was not clear, the case considering both factors of the increase in the density and the increase in the horizontal stress was found capable of evaluating the settlement adequately, judging from the data of the settlement in unimproved areas and the decrease in the settlement in improved areas of reclaimed land near Nishinomiya-hama Island (Kobe City, 1995, Yasuda et al., 1996). It is considered that the effect of the sand compaction pile method and seismic behavior of improved ground were evaluated properly by considering both factors of the increase in the density and the increase in the horizontal effective stress.

Rokko Island

In the Rokko Island case, the surface layer above the water level was as thick as 5 m, and an unimproved layer remained under the improved layer even in the improved case. Figure 14 shows the maximum response distribution of shear strain and excess pore water pressure comparing the effect of factors of improvement. In the unimproved layer, though excess pore water pressure built up to about 90% of the initial effective overburden pressure, the shear strain was relatively smaller than that of the Nishinomiya-hama Island case. This indicates that liquefaction was not severe or occurred partially and slightly, which corresponded with the observed phenomena of small or no damage around the site. The improved layer did not liquefy in both cases of "masado" or Kobe group formation, suggesting that the improvement was effective in preventing liquefaction at this site. These results corresponded with the fact that settlement was especially small at the improved site in the southern part of Rokko Island where settlement was generally small. The effect of the horizontal effective stress was not so large, however, the excess pore water pressure was reduced to about 1/2 in the case considering it.

Figure 15 shows the excess pore water pressure and shear strain distributions comparing the modeling of the improved ground. The pore water pressure and shear strain were almost the same in all cases. The difference in the effect of sand piles did not appear clearly in these cases, presumably because the liquefaction strength of the ground between sand piles was sufficient to prevent liquefaction.

The settlements were evaluated using the same method as the Nishinomiya-hama Island cases. The relative density of the reclaimed layer was set at 75-80% for the original ground and

90% for the improved ground referring to the results of soil tests (Tanaka, 1999). Figure 16 shows the settlement. The original ground settled about 15 cm. Being estimated as a maximum value, it was slightly larger, but roughly corresponded with the observed settlement ranging from 0 to 20cm with an average of 10cm at unimproved areas in the southern part of Rokko Island (Kobe city, 1995, Editorial committee for the report on the Hansin-Awaji earthquake disaster, 1998). In the improved cases, though liquefaction did not occur in the improved layer, an unimproved layer under the improved layer liquefied and caused settlement. The settlement was about 8 to 9 cm in the "masado" case, and 6 to 7 cm in the Kobe group case. The settlement was reduced by about 1 cm by the effect of the horizontal effective stress in each case. The settlement observed at the site was much smaller at 0 to 5cm. In the estimation, the settlement can be evaluated more adequately by considering the improvement, particularly both factors of the increase in the density and the increase in the horizontal stress, because the effect of improvement is expressed as a reduction in the settlement to 1/2, and because the settlement tends to be overestimated as mentioned above.

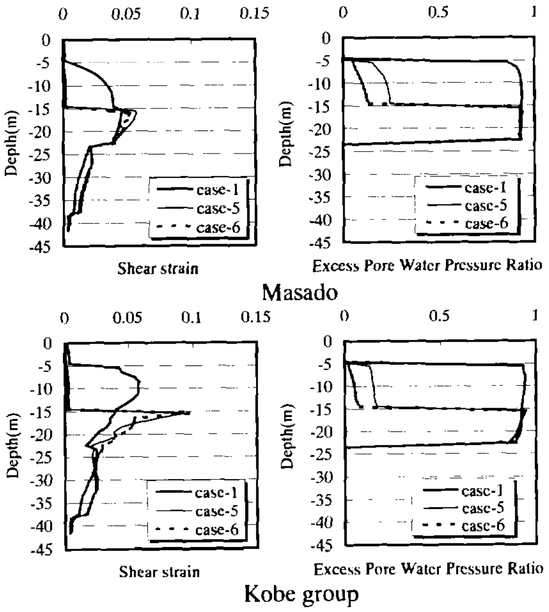


Fig. 14. Comparison factors for improvement.

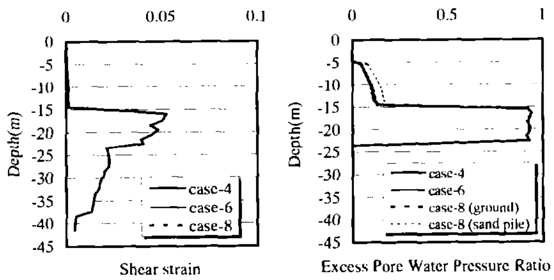


Fig. 15. Comparison of modeling of improved ground.

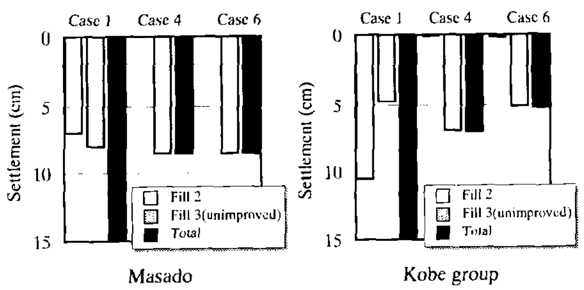


Fig. 16. Settlement of ground.

As a result of these analyses at two sites, it is considered that the effect of the sand compaction pile method and the seismic behavior of improved ground were evaluated properly considering both factors of the increase in the density and the increase in the horizontal effective stress.

CONCLUSION

The following conclusions were obtained:

1) It was found that there was a possibility of explaining well the actual behavior of the ground during strong ground motions, by considering the increase in the horizontal effective stress and the strength and rigidity of sand piles themselves regarding the effect of the sand compaction pile method.

2) By considering the addition of the effect by the increase in the horizontal effective stress, the liquefaction strength of the improved ground was estimated to be larger, leading to behavior of the improved ground widely different from the case without the addition. Whereas the conventional method tended to overestimate the residual settlement, the method incorporating the increase in the horizontal stress well explained the behavior and damage of the ground. However, there is still room for discussion as to how the effect of the increase in the horizontal effective stress should be estimated.

3) Shear strains were reduced in the model considering both the improved ground and sand piles when compared with those in the model considering only the improved ground between sand piles. A model including sand piles themselves as well as the ground between the piles can lead to a more proper estimation. Besides, a more detailed evaluation of changes in the liquefaction strength and dynamic characteristics between improved ground and sand piles are considerable for reasonable design under strong ground motions, such as records of the 1995 Hyogoken-Nambu earthquake.

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