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Seismic deamplifying effect of soft clay layers

Effet désamplificateur des couches d'argiles molles en cas de séisme

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ABSTRACT: The seismic deamplifying effect of soft clay layers is described. This deamplifying effect of soft clay layers on the propagated waves due to an earthquake is significantly influenced by the degree of consolidation of the soft clay underlying the site. In this paper, two buildings on Rokko Island are analysed. First, the static soil/water coupled analyses are carried out to assess the initial equilibrium conditions of the site just before the earthquake and, secondly, the dynamic response analyses are performed based on the data obtained from the static analyses. The emphasis is placed on evaluating the factor of safety against liquefaction at the various stages of consolidation and at the completion of consolidation. The results seem to be consistent with the overall phenomena observed in Port Island and Rokko Island.

RESUME: L'effect désamplificateur des couches d'argile molles en cas de séisme est décrit ici. Cet effet désamplificateur sur la propagation de l'onde sismique est significativement influencé par le degré de consolidation des couches d'argile molles au-dessous du site. Dans cette étude, deux immeubles construits sur Rokko Island sont analysés. Premièrement, les analyses statiques combinées du sol et de l'eau sont faits pour estimer les conditions de l'équilibre premier du site juste avant le tremblement de terre et, deuxièment, les analyses des réponses dynamiques sont exécutées, fondées sur les données obtenues par les analyses statiques. L'importance est mise sur l'évaluation du facteur de sécurité contre la liquéfaction aux différants stades de consolidation jusqu'au stade de consolidation commplète. Les résultats semblent compatibles avec tous les phénomènes observés à Port Island et à Rokko Island.

1 INTRODUCTION

On January 17, 1995, the Kobe area experienced a severe near-field earthquake. The magnitude of the event was 7.2 and the intensity at Kobe was determined to be more than 6. Port Island and Rokko Island, both of which are man-made islands placed on the seabed layer of soft clay, also suffered from the earthquake. The northern part of Port Island was approximately 15 years old at the time of the earthquake, while the southern part of Port Island was still under construction. The northern and southern parts of Rokko Island were 16 and 9 years old, respectively. As can be seen in Fig.1 by the area covered by boiled sand, the damage due to the liquefaction of sandy fill materials was more concentrated in the older parts of Port Island and Rokko Island. In this study, the authors draw attention to the different degrees of consolidation of the soft clay in the older and younger parts of these man-made islands. The seismic waves of the 1995 Kobe earthquake which were monitored at GL -83 m in Port Island (★ in the figure) by Kobe City office and used in the analyses in this paper, are summarized in the lower part of Fig.1.

2 DESCRIPTION OF SITE ANALYSED

The site analysed consisted of buildings located near the south-west corner of Rokko Island as indicated by ● in Fig.1. These buildings were 9 years old at the time of the earthquake and sustained practically no damage during the earthquake while the wharves near the buildings (within a distance of about 100 m) were severely damaged. The buildings, A and B, are sketched in the upper part of Fig.2. Building A is a single-story steel framed warehouse resting on concrete foundation beams (0.5 m wide and 3.0 m high) oriented in north-south direction. Building B, on the other hand, is a five story office building consisting of a steel framed reinforced concrete structure having a basement. Both buildings are supported by nodular piles, 400 to 500 mm in diameter and 12 m in length. This type of pile foundation was selected because of the beneficial effects of ground densification during pile installation. Several aspects of the dynamic soil-structure interaction depend on the stress state of the ground and/or structure immediately preceding the earthquake: (i) the dynamic material properties needed to analyse the seismic response of the ground depend on the initial conditions of both the ground and the structure at the time just before the earthquake, (ii) the liquefaction analysis is influenced by the stress state of the ground and, (iii) the bearing capacity of the structure with respect to the dynamic motion due to the earthquake is governed by the stress histories which the structure has experienced prior to the earthquake. Hence, in order to examine the influence of the earthquake motion on the structure, it is essential to carefully evaluate the initial stress states at the time just before the earthquake.

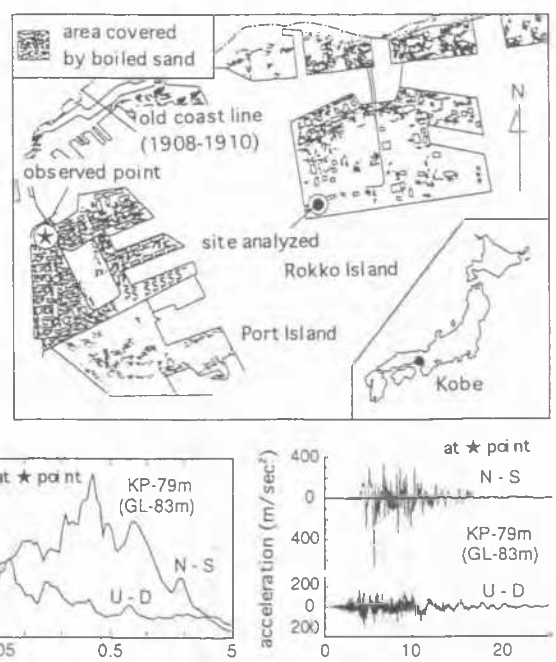
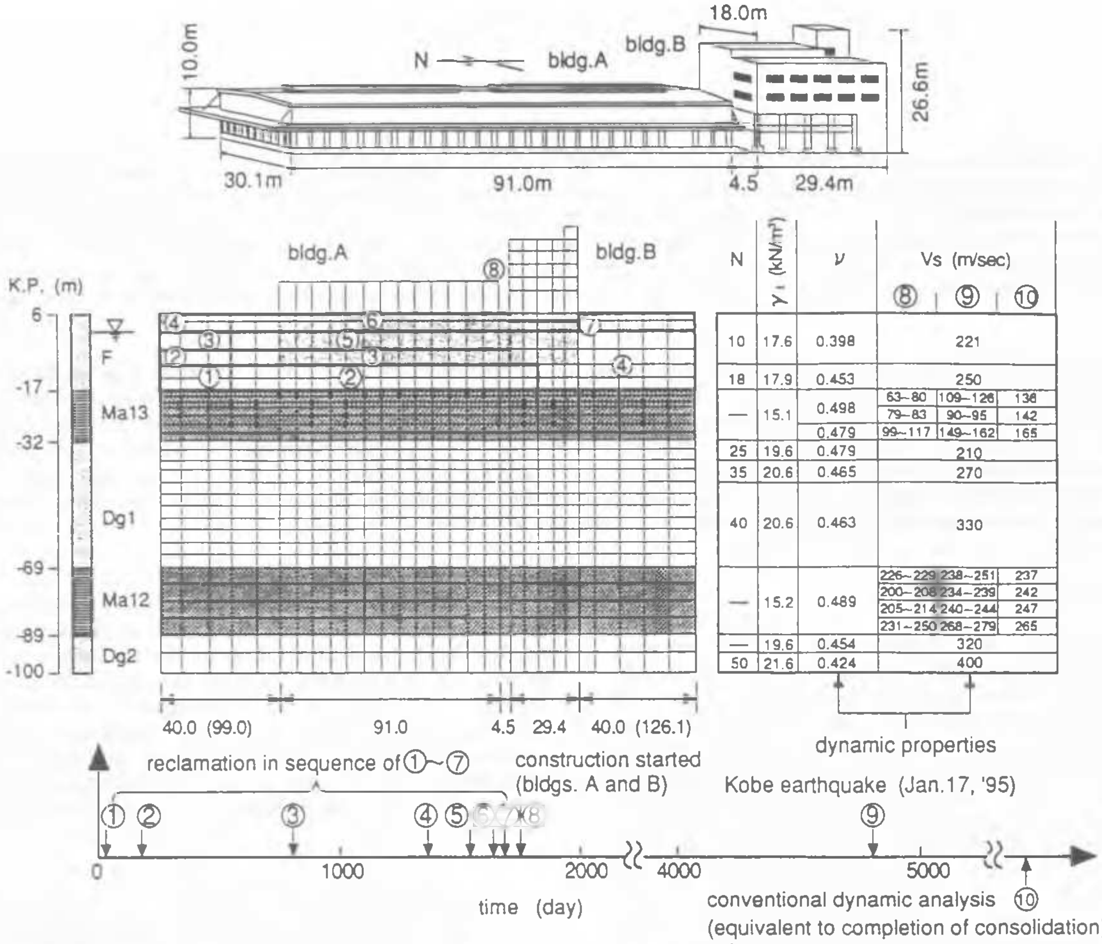


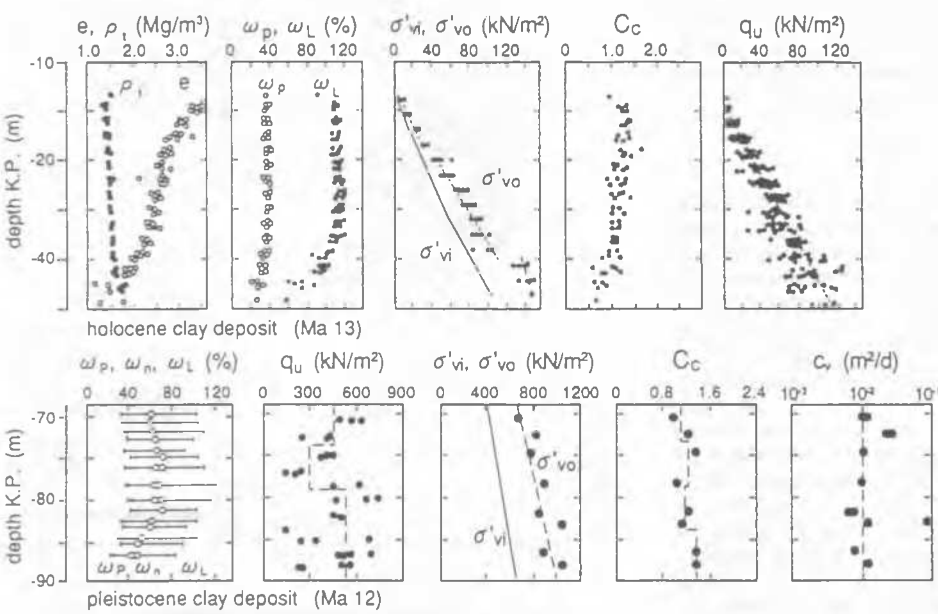
Figure 1. Possible liquefied zone (Hamada et al, 1995) and observed record

The event history diagram in the lower part of Fig. 2 indicates the construction sequence for the man-made island and the buildings, A and B. The reclamation work over the soft clay layer (Ma13) was completed in a little less than 2000 days. The events number ① to ⑦ in Fig. 2 indicate the order of the reclamation work, which was performed from the north to the south and from the bottom to the surface of sea. The time difference in each of the reclamation works brought a time lag in the progress of subsequent consolidation and, as a result, caused differential settlement of the ground. As can be seen from the event history diagram in Fig. 2, the construction of buildings A and B indicated by ⑧ in the figure began at the time just after the final stage of reclamation (⑦ in the figure). The 1995 Kobe earthquake occurred when the consolidation of the ground was in progress and the buildings were still settling unequally (⑨ in the figure). However, in the conventional seismic analyses, since complete consolidation (equivalent to the event ⑩ in Fig. 2) is assumed at the time of the earthquake, the initial conditions and the corresponding dynamic material properties may not represent actual conditions.



N	γ_t (kN/m³)	ν	Vs (m/sec) ⑧	⑨	⑩
10	17.6	0.398	221		
18	17.9	0.453	250		
—	15.1	0.498	63~80	109~126	136
			79~83	90~95	142
		0.479	99~117	149~162	165
25	19.6	0.479	210		
35	20.6	0.465	270		
40	20.6	0.463	330		
—	15.2	0.489	226~229	238~251	237
			200~208	234~239	242
			205~214	240~244	247
			231~250	268~279	265
—	19.6	0.454	320		
50	21.6	0.424	400		

Figure 2. Analytical modeling and construction sequence



N : SPT blow count
ν : Poisson's ratio
Vs : shear wave velocity
ρ_t : wet density
ω_p : plasticity limit
ω_L : liquid limit
ω_n : natural water content
σ'_{vi} : effective overburden pressure
σ'_{vo} : preconsolidation pressure
e : void ratio
C_c : compression index
q_u : unconfined comp. strength
c_v : coefficient of consolidation
PI : plasticity index

Figure 3. Soil properties before the Kobe earthquake (Kobe Port Wharf Co,1985, JSSMFE,1985)

Table 1. Input parameters used in static analyses

depth (m)	D	Λ	M	ν'	c_v (m²/d)	Ko	Ki	α	$\dot{v}_0$ (l/d)
-18~-23	0.082	0.48	0.84	0.42	5.8×10^{-3}	0.73	0.99	7.2×10^{-3}	2.2×10^{-7}
-23~-28	0.081	0.54	0.95	0.42	5.5×10^{-3}	0.73	1.00	7.0×10^{-3}	2.0×10^{-7}
-28~-33	0.081	0.44	0.77	0.41	5.2×10^{-3}	0.68	0.89	7.2×10^{-3}	2.0×10^{-7}
holocene clay deposit (Ma 13)									
-69~-74	0.098	0.89	1.55	0.42	0.01	0.73	0.83	8.6×10^{-3}	2.5×10^{-7}
-74~-79	0.098	0.61	1.07	0.42	0.01	0.73	0.83	8.6×10^{-3}	2.5×10^{-7}
-79~-84	0.098	0.69	1.21	0.42	0.01	0.73	0.84	9.0×10^{-3}	2.7×10^{-7}
-84~-89	0.116	0.50	0.87	0.41	0.01	0.73	0.81	1.0×10^{-2}	3.0×10^{-7}
pleistocene clay deposit (Ma 12)									

M : critical state parameter
D : coefficient of dilatancy
Λ : irreversibility ratio (=1-Cs/Cc)
Ko : coef. of earth pres. at rest
K_i : coef. of in-situ earth pres. at rest
α : secondary compression index
$\dot{v}_0$: initial volumetric strain rate
ν' : Poisson's ratio in terms of effective stress

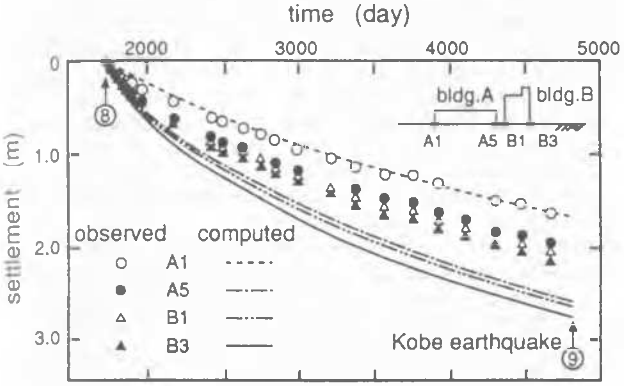


Figure 4. Comparison between computed and monitored settlements

3 STATIC ANALYSIS (ELASTO-VISCOPLASTIC SOIL/ WATER COUPLED ANALYSIS)

In order to assess the initial conditions at the time just before the earthquake, the reclamation work and subsequent construction of the buildings are numerically simulated, carefully following the construction sequences, by using the elasto-viscoplastic soil/water coupled finite element technique (DACSAR, Iizuka and Ohta, 1987). Table 1 provides the input parameters needed for clay layers in the static analysis, which are determined mainly from the unconfined compression strengths by the semi-theoretical procedure proposed by Ohta et al. (1988) based on soil property data summarized in Fig.3. The soil properties of the soft clay layers in Fig.3 are the data at the time before reclamation of man-made island. Sandy deposits (F, Dg1 and Dg2 in Fig.2) are assumed to be linearly elastic and their elastic moduli are estimated from an empirical equation E_0=2.75N (MPa) relating the coefficient of deformation, E_0, and the SPT blow count, N. For the area where nodular piles were driven, the compaction due to pile driving increased the blow count three fold resulting in elastic moduli three times as large as the above values.

The settlement curves obtained from the static finite element analysis are compared with those monitored at locations, A1, A5, B1 and B3 from the start of construction of the buildings until the time of the earthquake as shown in Fig.4. It can be seen that the computation has successfully simulated the monitored settlements within the range of error of 20 %. The maximum bending moment on the concrete foundation beams of building A is calculated to be 1.6 MNm from the monitored settlement and 1.5 MNm from the finite element computation. These are in good agreement. It should be noted that no attempt has been made to obtain good agreement by adjusting the values of the input parameters.

4 DYNAMIC RESPONSE ANALYSIS (DYNAMIC F.E. ANALYSIS BASED ON COMPLEX RESPONSE METHOD)

The dynamic soil-structure interactions are analysed using the complex response method by Lysmer et al. (1975) in which the non-uniform initial stress distribution during consolidation is taken into account. The initial shear modulus, G_0, of clays, needed in the dynamic analyses, is estimated from the empirical relationship by Zen et al. (1987) based on the mean effective stress and the plasticity index of clays as shown in Fig.5 (a). For sandy deposits, the initial shear modulus, G_0, is evaluated from the SPT blow count using an empirical relation proposed by Imai et al. (1982) as shown in Fig.5 (a). The equivalent linearization method (see Fig.5 (b)) is employed to estimate the current shear modulus, G, taking the non-linearity of the dynamic properties of the ground materials into consideration.

Fig.6 shows the computed distribution of horizontal acceleration. It demonstrates that, when the seismic wave passes through a soft clay layer, the short-period component in the wave is eliminated so that the (horizontal) acceleration acting on reclaimed sandy deposits becomes smaller. The deamplifying effect of the soft clay deposit is found to be far more than that expected intuitively. The results of three dynamic analyses are plotted in the figure, (i) the case where the initial stress state at the time of the start of construction of buildings (event ⑧ in Fig.2) is assumed, (ii) the case where the initial stress state at the time of the earthquake (event ⑨ in Fig.2) is assumed and (iii) the case where the initial stress state at the time of completion of consolidation (event ⑩, in Fig.2) is assumed. The third case is usually assumed in dynamic analyses in current engineering practice. As clay layers get stiffer, the seismic wave passes through the clay layers more easily, resulting in higher horizontal acceleration acting on the reclaimed sandy deposits. As the consolidation proceeds, the deamplifying effect of the soft clay deposit decreases and, as a result, the reclaimed sandy land becomes weaker against an earthquake.

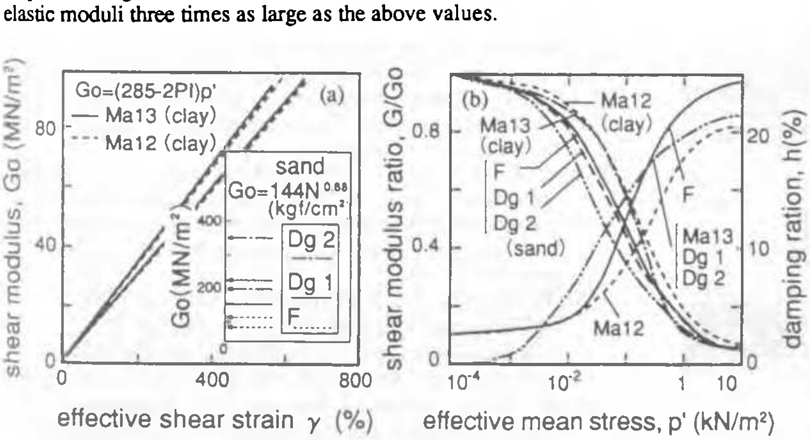


Figure 5. Dynamic soil properties, (a) Initial shear modulus, (b) Non-linear stress and strain characteristics

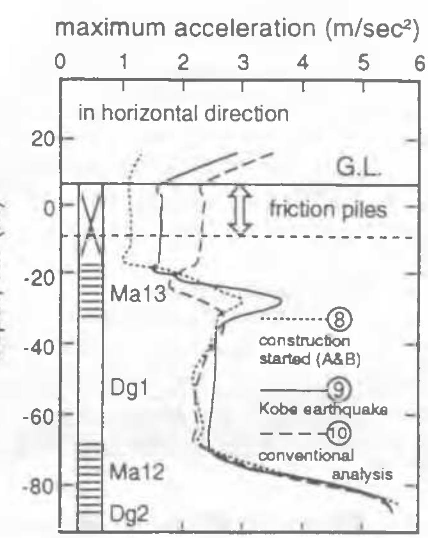


Figure 6. Computed horizontal acceleration

Fig. 7 shows the response spectra at the time of the earthquake (event ⑨ in Fig. 2) and at the time of completion of consolidation (event ⑩, in Fig. 2). The deamplifying effect of the soft clay deposit, Ma13 (see Fig. 2), is most noticeable for the shorter period components in the wave. The peak in the acceleration at the period of around 0.4 seconds indicated by I in Fig. 7 is almost absorbed in the soft clay deposit resulting in less likelihood of resonance in the case of buildings A and B whose natural periods are around 0.1 seconds and 0.3 seconds, respectively. The peak at the period of around 2.0 seconds indicated by II in Fig. 7 (a) is also deamplified by the soft clay deposit. However, as the clay layer continues to consolidate, the seismic wave propagation will increase (see Fig. 7 (b)). This implies that, since the natural period of high rise buildings is around 2.0 seconds, they would have a higher likelihood of resonance with the progress of consolidation.

The likelihood of liquefaction increasing with the progress of consolidation is shown in Fig. 8, in which FL is the factor of safety against the liquefaction. The results in this figure indicate that the damage due to liquefaction would have been more serious at the site if the 1995 Kobe earthquake had occurred after completion of consolidation of the soft clay layer. This is consistent with the observed phenomena: the damage due to the liquefaction of sandy fill was more concentrated in the older part (higher degree of consolidation) of Port Island and Rokko Island (see Fig. 1). Besides, the dynamic analysis indicates that the maximum bending moment on the foundation beams of building A at the time of the 1995 Kobe earthquake is 0.2 MNm. This is much smaller than the maximum bending moment (1.5 MNm) due to the differential settlement.

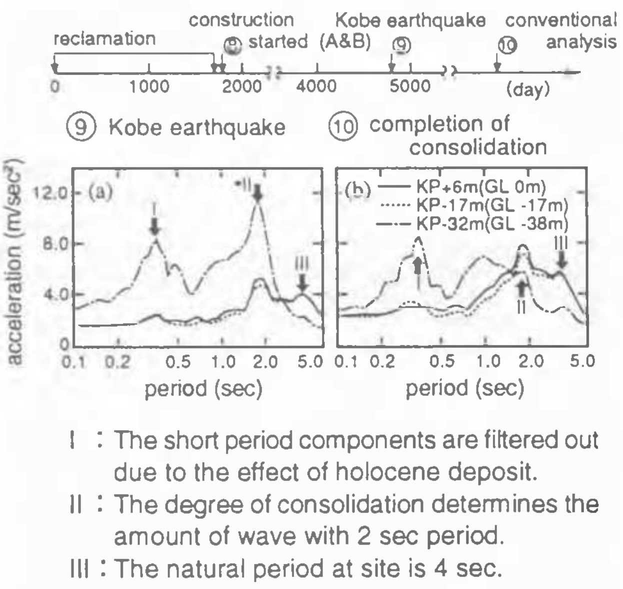


Figure 7. Response spectra of horizontal acceleration, (a) At the earthquake, (b) After completion of consolidation

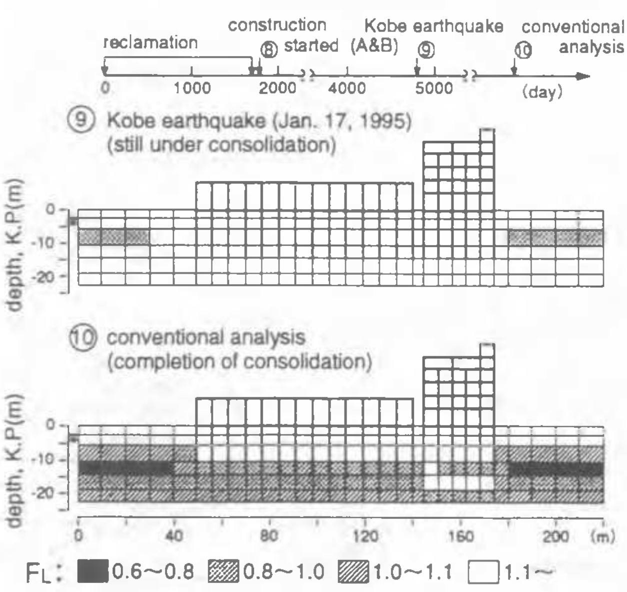


Figure 8. Factor of safety against liquefaction, F_L

5 CONCLUSION

It was demonstrated that the soft clay layer would help prevent the short period component in the horizontal wave from passing through the soft clay layer and the soft clay layer would deamplify the earthquake motion in which the short period (high frequency) component is dominant such as in the case of a near-field earthquake. Such a deamplifying effect would not have been found to be greatly influenced by the degree of consolidation of the soft clay layer. This paper reveals that the deamplifying effect is far more than that intuitively expected but it decreases with the progress of consolidation of the soft clay layer. It is predicted that the damage at the site due to liquefaction would have been more widespread if the consolidation of the soft clay layer had been completed at the time of the 1995 Kobe earthquake. These results seem to be consistent with the overall phenomena observed in Port Island and Rokko Island (see Fig. 1). Therefore, the results obtained in this paper should not be regarded as special cases but should be considered as relevant for any soft clayey ground. In addition they lead to the necessity of the aseismatic design work of foundation considering the dependency of deamplifying effect on the degree of consolidation of the soft clayey ground.

The soil properties of the soft clay layers shown in Fig. 3 are the data at the time before reclamation of man-made island. The dynamic soil properties at the time of the start of construction of the buildings (event ⑧ in Fig. 2), at the time of the 1995 Kobe earthquake (event ⑨ in Fig. 2) and at the time of completion of consolidation (event ⑩ in Fig. 2) were estimated by carrying out static soil/water coupled analyses. The subsequent dynamic analyses were then performed, based on data obtained from the static analyses. Without employing such a two-step procedure, consisting of static and dynamic analyses, no meaningful solution would be derived for this type of seismic problems concerning with the soft clayey ground. Finally, the authors emphasize the important role of static analyses which should be carried out prior to the dynamic analyses.

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