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LIQUEFACTION ANALYSES OF LAYERED SAND DEPOSITS CONSIDERING OVERCONSOLIDATED RATIO

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SUMMARY

The second author proposed a liquefaction analysis method of layered sand deposits and constitutive equation of soils under cyclic loading. In the present study, the effects of overconsolidated ratio (OCR) of soil and the type of input ground motion to the response of the ground were investigated. The results of the numerical calculation showed that the larger the value of OCR was, the less the generated excess pore water pressure was, and that the repeating type input motion caused more excess pore water pressure than the shock type did.

INTRODUCTION

In the usual designing procedures, the liquefaction potential of the sandy soils was evaluated by the simplified method (Ref. 1). However, when we need the time histories of the response of the ground, such as excess pore water pressure, acceleration, velocity and displacement; reliable liquefaction analysis method is essential.

Oka has been developing a liquefaction analysis program of layered sand deposit (LAPES-1D), in which the analysis method and the constitutive equations of soils proposed by him are incorporated. In the present study, the following points were investigated by using LAPES-1D.
(1) The sensitivity of the response of the ground to the overconsolidated ratio (OCR) of the soil.
(2) The influence of the types of the input ground motions (shock or repeating type) on the response of the ground.

METHOD OF ANALYSIS

In the analyses, we used the one dimentionally approximated equation of motion of two-phase mixture (Ref. 2). We used the elasto-plastic constitutive equation of sand, which is capable of describing the behavior of sand under cyclic loading (Ref. 3). Fig.1 shows the boundary surface, the plastic potential surface and the loading path of the model. When the stress state is inside of the boundary surface, the shape of the plastic potential surface varies gradually with the increase of the relative stress parameter. In this model, the overconsolidated ratio (OCR) acts as a parameter which defines the shape of the plastic potential surface. Therefore it is reasonable to assume OCR as a parameter which represents the aging effect and the stress-strain history of the sand.

CONDITIONS OF CALCULATION

We selected the Ohama-No.1-quay of the Akita Port which suffered slight damage in the shock of The 1983 Nipponkai-Chubu Earthquake as model ground (Ref. 4). The total thickness of the model ground is 20 m, and the water table is at the depth of -2 m. The base rock is inpermeable and drainage is allowed only in the upward direction. Table 1 shows soil parameters of the model ground, where the parameters are defined as follows:

- e : void ratio
- γ : unit weight
- Vs : shear wave velocity
- k : coefficient of permeability
- G' : parameter for strain hardening
- Mf* : value of stress ratio at the failure state
- Mm* : value of stress ratio at the maximum volumetric compression state during shear deformation process
- κ : swelling index
- K_0 : earth pressure coefficient at rest at the initial stress atate

The soil parameters of the model ground were estimated from the N-value of the site by using the following relations, where ϕ represents internal frictional angle.

ϕ = 0.3N + 27 (°) --- Peck

e = 0.55 / tan ϕ --- Caquot-Kerisel

$$Vs = 68.7\ N^{0.417}\ (\mathrm{m/sec})$$

$$Mf^* = \sqrt{2/3}\cdot\frac{6\sin\phi}{3-\sin\phi}$$

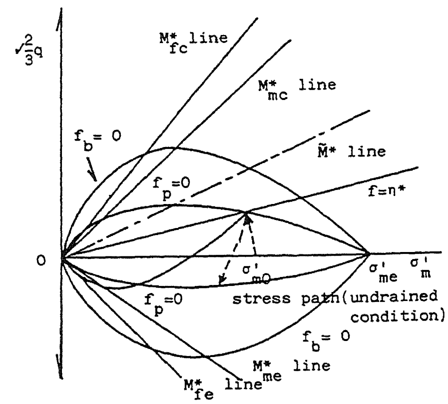


Fig. 1 Schematic Diagram of Boundary Surface (fb) and Plastic Potential Surface(fp)[3)]

$$Mm^* = 0.867\ Mf^*$$

$$G' = 65.9\ \phi - 294.5\ ;\ \phi < 42.4°$$
$$= 18.0\ \phi - 2527\ ;\ \phi > 42.4°$$

Table 1 Soil Parameters of the Model Ground

Depth (m)	N-value	e	γ (tf/m³)	Vs (m/s)	k (m/s)	G'	Mf*	Mm*
0.0	18	0.866	1.8	229	10^{-5}	1840	1.065	0.924
1.0	19	0.856	1.8	235	10^{-5}	1860	1.076	0.924
2.0	17	0.876	1.8	224	10^{-5}	1820	1.055	0.914
3.0	17	0.876	1.8	224	10^{-5}	1820	1.055	0.914
4.0	32	0.740	1.8	291	10^{-5}	2120	1.215	1.054
5.0	35	0.716	1.8	303	10^{-5}	2180	1.247	1.081
6.0	34	0.724	1.8	299	10^{-5}	2160	1.237	1.072
7.0	20	0.846	1.8	240	10^{-5}	1880	1.087	0.942
8.0	20	0.846	1.8	240	10^{-5}	1880	1.087	0.942
9.0	30	0.757	1.8	284	10^{-5}	2080	1.194	1.035
10.0	31	0.748	1.8	288	10^{-5}	2100	1.204	1.044
11.0	27	0.782	1.8	272	10^{-5}	2020	1.162	1.007
12.0	32	0.740	1.8	291	10^{-5}	2120	1.215	1.054
13.0	38	0.693	1.8	313	10^{-5}	2240	1.279	1.109
14.0	42	0.664	1.8	326	10^{-5}	2320	1.322	1.146
15.0	39	0.686	1.8	317	10^{-5}	2260	1.290	1.118
16.0	41	0.671	1.8	323	10^{-5}	2300	1.311	1.137
17.0	42	0.644	1.8	326	10^{-5}	2320	1.322	1.146
18.0	43	0.657	1.8	330	10^{-5}	2340	1.322	1.155
19.0	44	0.650	1.8	333	10^{-5}	2360	1.343	1.164
20.0	40	0.679	2.29	500	10^{-5}	2280	1.300	1.128

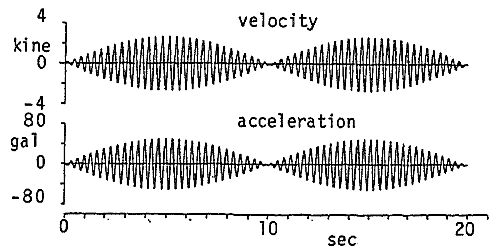
κ = 0.06

K_0 = 1.0

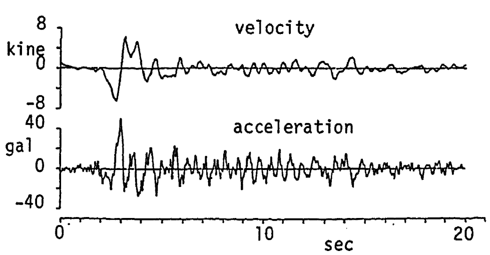
Table 2 shows the condition of calculation. The value of OCR was assumed to be 1.0, 2.0 and 3.0 for a type of input motion. Fig.2 shows the acceleration and the velocity of the input motions. Input motions were treated as insident waves with maximum acceleration of 50 gal.

Table 2 Conditions of Calculation

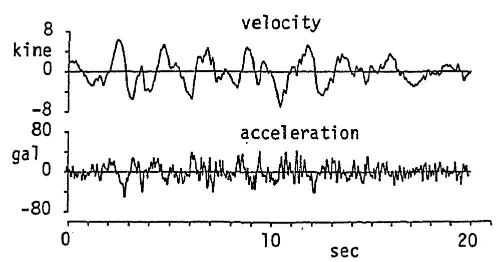
Case	OCR	Input Motion
A1 A2 A3	1.0 2.0 3.0	Sinusoidal Wave with Varying Amplitude (3 Hz) (repeating type)
B1 B2 B3	1.0 2.0 3.0	1980 Chibaken-Chubu Earthquake (Ref. 5) (shock type)
C1 C2 C3	1.0 2.0 3.0	1983 Nipponkai-Chubu Earthquake (Ref. 6) (repeating type)



(a) Sinusoidal Wave with Varying Amplitude



(b) 1980 Chibaken-Chubu Earthquake



(c) 1983 Nipponkai-Chubu Earthquake

Fig.2 Input Ground Motions

RESULTS OF CALCULATION

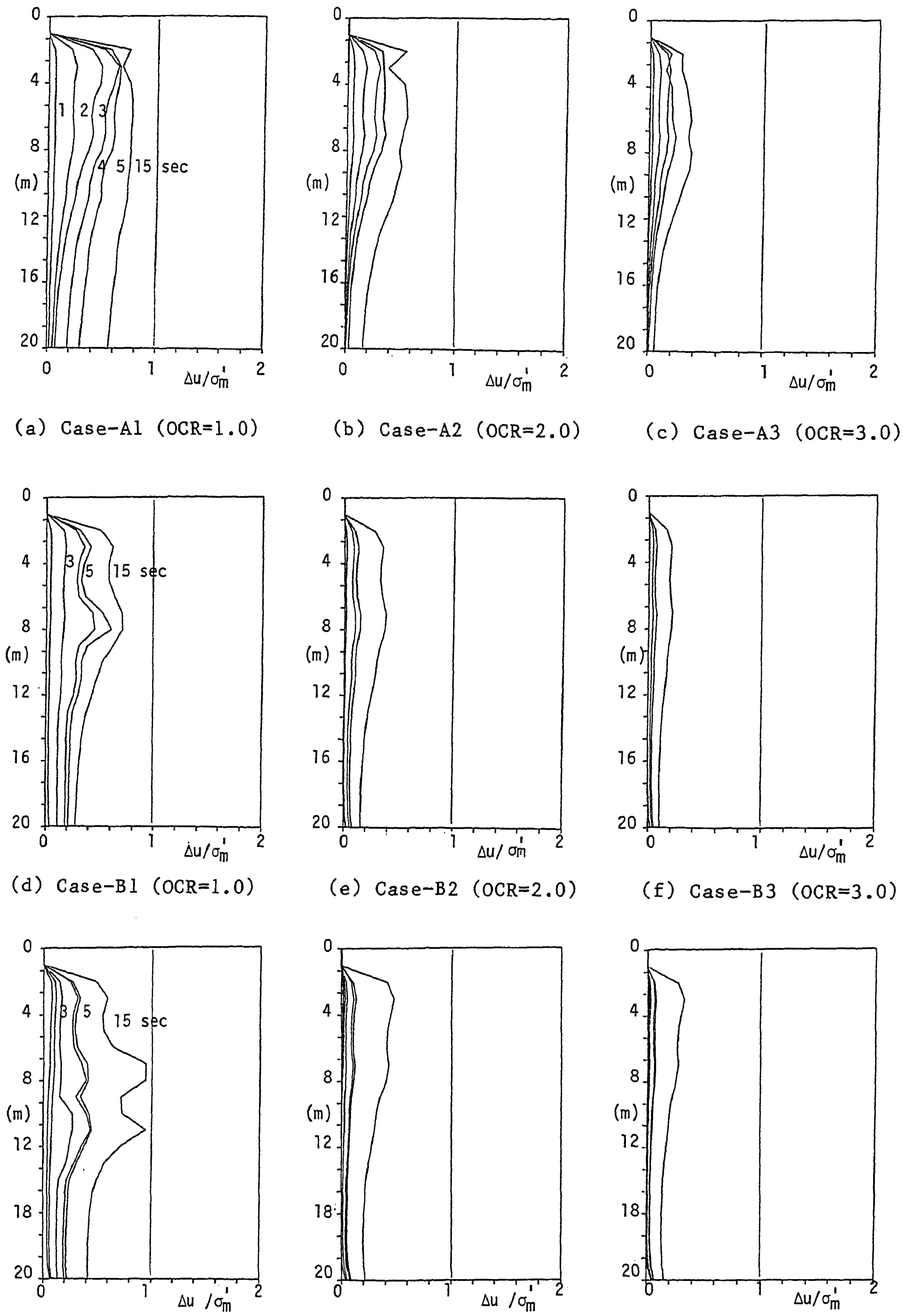
Fig.2 shows the distributions of the calculated exess pore water pressures. From Fig.2 , it can be said as follows:

(1) The larger the value of OCR was, the less the generated excess pore water pressure was.

(2) The repeating type input motion caused more excess pore water pressure than shock type did.

These results correspond well with the results of laboratory test.

Hereafter, Case-C1(OCR=1.0) and Case-C3(OCR=3.0) will be compared in order to investigate the influence of OCR on the response of the ground. Fig.4 (Case-C1) and Fig.5(Case-C3) show the acceleration response of the ground. In Case-C1, relatively long period waves are predominant after 9 second at the depth of 0 m and -4 m. On the other hand, long period waves can not been seen in Case-C3.



(a) Case-A1 (OCR=1.0) (b) Case-A2 (OCR=2.0) (c) Case-A3 (OCR=3.0)

(d) Case-B1 (OCR=1.0) (e) Case-B2 (OCR=2.0) (f) Case-B3 (OCR=3.0)

(g) Case-C1 (OCR=1.0) (h) Case-C2 (OCR=2.0) (i) Case-C3 (OCR=3.0)

Fig.3 Distributions of Pore Water Pressures
(Normalized by the initial mean effective stress, Time=1,2,3,4,5 and 15 sec)

Fig.6(Case-C1) and Fig.7(Case-C3) show the shear stress, the shear strain and the excess pore water pressure at the depth of -4 m. In Case-C1, the excess pore water pressure increased during the first 9 seconds and kept almost constant value after 9 seconds. After 9 seconds the predominant period of the shear stress and shear strain are relatively long. The elongation of the predominant period of the acceleration, the shear stress and the shear strain is caused by the reduction of the shear modulus of soils due to liquefaction. During the Niigata Earthquake in 1964, the same type of acceleration was actually recorded.

Fig.8(Case-C1) and Fig.9(Case-C3) show the effective stress path at the depth of -4 m. Fig.10(Case-C1) and Fig.11(Case-C3) show the relation between the shear stress and the shear strain at the depth of -4 m. In both the figures, the shear modulus of the soils, which is represented by the tangential coefficient of the stress-strain relations, are decreasing gradually with cyclic loadings and unloadings.

CONCLUDING REMARKS

It was found that the response of the ground is very sensitive to the overconsolidated ratio (OCR) which represents the aging effect and the stress-strain history of the sand. In this study, how to determine OCR was not discussed. It will be convenient if OCR can be determined from simple soil parameters, for example N-value or relative density. More research which includes laboratory tests and numerical calculation is necessary in order to find these kinds of relations.

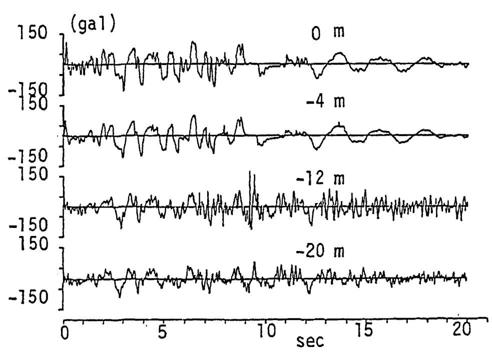


Fig.4 Time Histories of Acceleration (Case-C1, OCR=1.0)

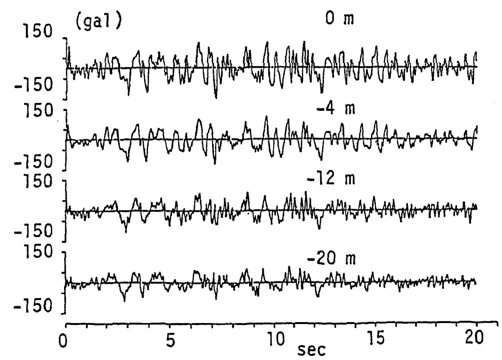


Fig.5 Time Histories of Acceleration (Case-C3, OCR=3.0)

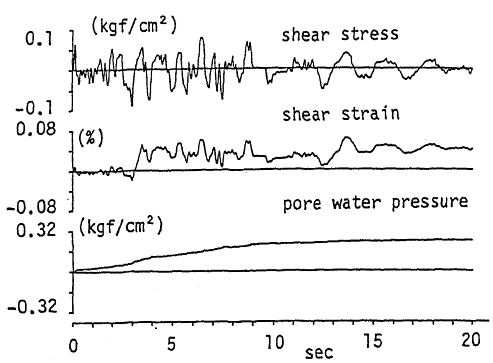


Fig.6 Time Histories of Liquefaction (Case-C1, at the depth of -4 m)

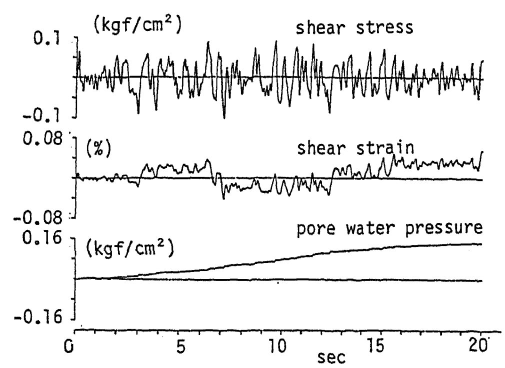


Fig.7 Time Histories of Liquefaction (Case-C3, at the depth of -4 m)

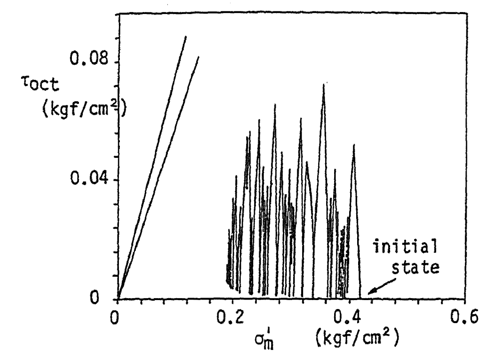


Fig.8 Effective Stress Path
Case-C1, at the depth of -4 m)

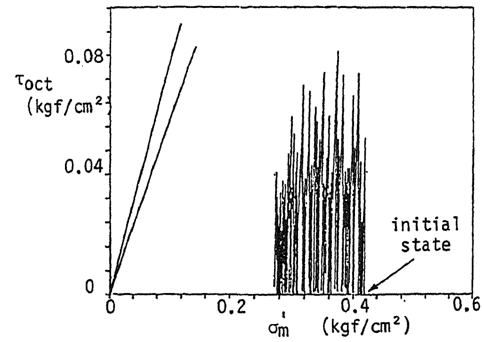


Fig.9 Effective Stress Path
(Case-C3, at the depth of -4 m)

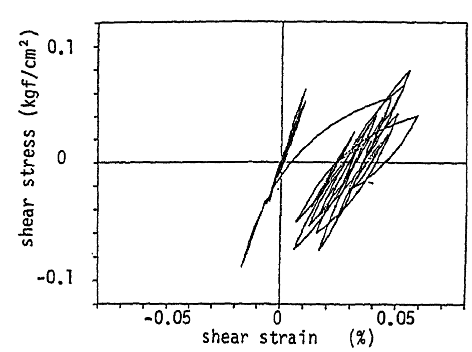


Fig.10 The Relation between the Shear Stress and the Shear Strain (Case-C1, at the depth of -4 m)

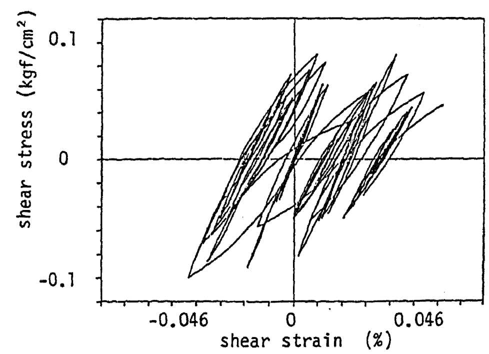


Fig.11 The Relation between the Shear Stress and the Shear Strain (Case-C3, at the depth of -4 m)

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