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LARGE SCALE MODEL TESTS AND ANALYSIS OF GRAVEL DRAINS

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SUMMARY

Performance of gravel drains, installed as one of remedial measures against liquefaction, is studied with large scale model tests. The tests are conducted with a container, set on the shaking table, made of a stack of 64 aluminum rings; the rings of 200 cm in diameter being stacked to the total height of 200 cm. The results of the shaking tests and the analysis indicate a simple design procedure for determining the spacing between the gravel drains.

INTRODUCTION

Installing gravel drains, such as shown in Fig.1, is one of remedial measures against liquefaction; the gravel drains will increase the average permeability of the ground and thus reduce the potential damages associated with liquefaction. The gravel drains can be installed without vibrations and noises and, therefore, have an advantage over compaction methods when the ground has to be improved near the existing structures.

Performance of gravel drains was first studied by Seed and Booker (Ref. 1) by solving a kind of consolidation equations. Several studies followed them (Refs. 2 through 4). However, there have never been a large scale model test under horizontal exciations to study the applicability of the equation used by Seed and Booker. Therefore, the present study is carried out.

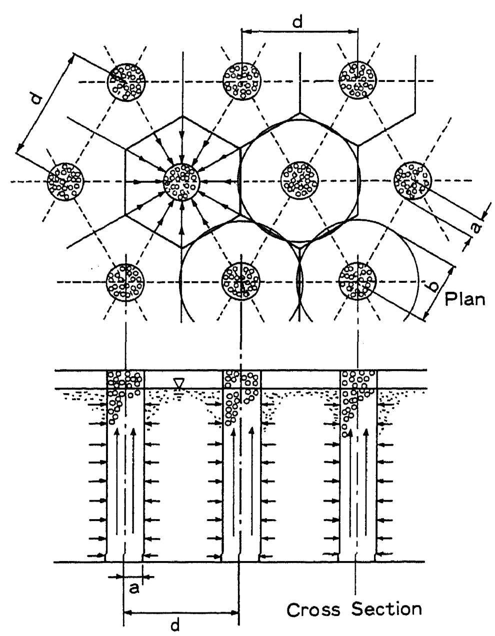


Fig.1 Schematic figure of gravel drains

LARGE SCALE MODEL TESTS

Nine series of shaking table tests

were conducted by using a container which was made of a stack of 64 aluminum rings designed to enforce, in the model ground, pure horizontal cyclic shearing as in the field. The diameter and the thickness of each ring were 200 cm and 2 cm and the total height of the stacked rings amounted to 200 cm. Between the aluminum rings, roller-bearings were inserted to reduce the friction between the rings. The model had the typical cross section shown in Fig.2. The shaking tests were conducted for the model ground with and/or without a gravel drain under sinusoidal and/or earthquake motions. The sands and the gravels used for the tests had grain size accumulation curves shown in Fig.3.

RESULTS OF THE MODEL TESTS

The shaking tests were conducted in various conditions but the typical conditions are shown in Table 1; R-204, R-302, and R-303 are the tests without a gravel drain and R-502 and R-503 are the tests with a gravel drain. When these models were shaken by a sinusoidal input motions of 2 Hz for the duration of 10 seconds, excess pore water pressures gradually rose, giving the maximum excess pore water pressures (the ratio of the maximum excess pore water pressures over the initial vertical effective stress) as shown in Fig.4; i.e., with the gravel drain, excess pore water pressures were smaller than those without the gravel drain.

A closer look in the effect of the gravel drain reveales, as shown in Fig.5, that the gravel drain decreases the rate of generation in pore water pressures and increases the rate of dissipation. However, the maximum value of the excess pore water pressures are sensitive to the level of the earthquake motions.

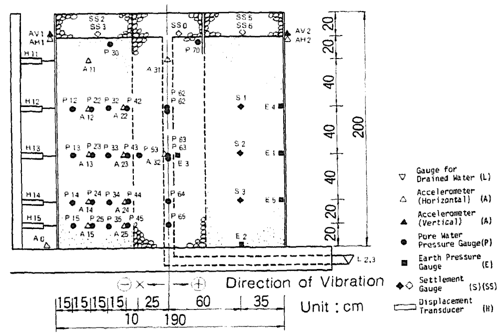


Fig.2 Cross section of Model

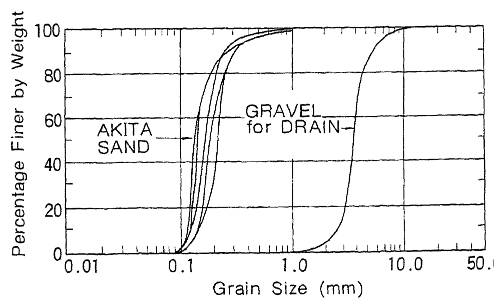
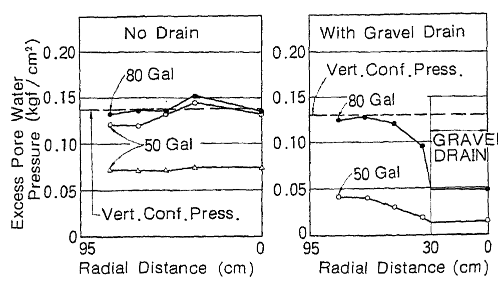


Fig.3 Grain size accumulation curves of the sand and gravel



(a) Without a drain (b) With a drain
Fig.4 Maximum excess pore water pressures

Table 1 Conditions for the experiments

Case Number	R-204	R-302	R-303	R-502	R-503
Relative Density (%)	29	31	31	33	35
Acceleration of Shaking Table (Gal)	43	50	81	50	69

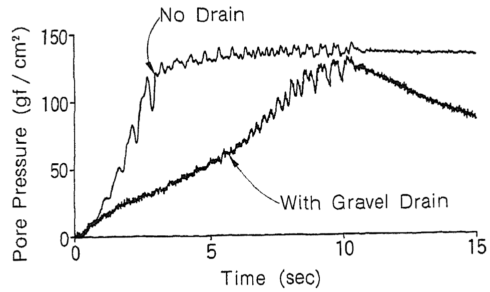
COMPARISON BETWEEN MEASURED AND COMPUTED RESULTS

In order to evaluate the performance of the gravel drain, Seed and Booker's approach (Rel. 1) was adopted because of its simplicity. Their approach is based on Terzaghi's consolidation equation with an additional term for excess pore water pressure generation. In their study, this equation was solved under simplifying assumptions. However, in the present study, this equation was solved directly by the finite element method with the constants obtained by the backfitting technique with the test data obtained without the gravel drain.

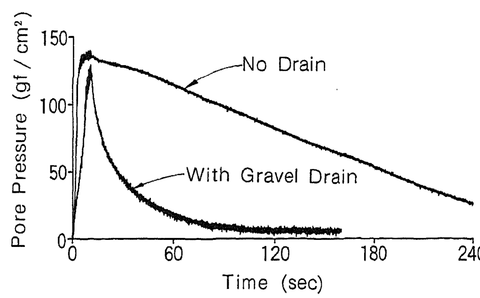
Comparison between the measured and the computed results indicates, as shown in Figs. 6 and 7, that the above-mentioned approach has acceptable applicability. However, the coefficient of compressibility obtained by the back-fitting technique was, as shown in Fig.8, not constant when the excess pore pressure ratio exceeded 0.5.

PERFORMANCE OF GRAVEL DRAINS UNDER SINUSOIDAL CYCLIC LOADINGS

The above-mentioned approach indicates that the pore water pressure ratio u/σ_{vo}' depends on the following

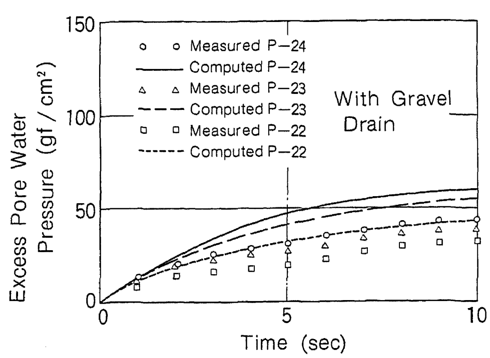


(a) During the first 15 seconds

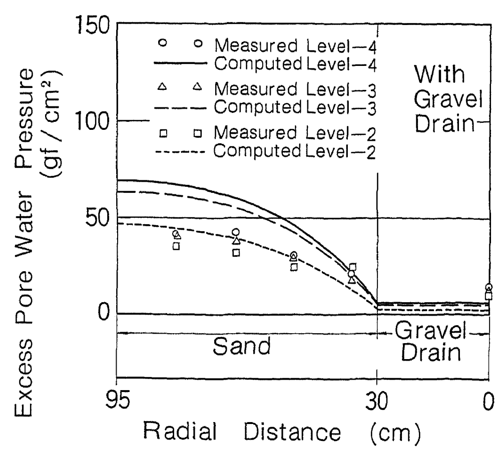


(b) During the 240 seconds

Fig.5 Time histories of the excess pore water pressure at P24 with/without the gravel drain



(a) Time histories of excess pore water pressures



(b) Maximum values of excess pore water pressures in the cross section

Fig.6 Comparison between the measured and the computed results dimensionless parameters; a/b = a ratio characterizing the geometric configuration in which a,b = radii of the gravel drain and the equivalent circle of the tributary area shown in Fig.1, t_d/t_l = a ratio characterizing the duration of the earthquake shaking in relation to the duration of the earthquake shaking required to cause initial liquefaction, $T_l = k_s t_l/(m_v \gamma_w a^2)$ = a factor characterizing the duration of earthquake shaking required to cause initial liquefaction in relation to the consolidation properties of the ground, $R = (8/\pi^2)(k_s/k_d)(h/a)^2$ = a ratio characterizing the resistance of the drain in relation to the permeability of the sand in which k_d = the coefficient of permeability of the gravel drain and h = height of the gravel drain, and α = a parameter characterizing the shape of the pore water pressure generation curve and taking the typical value of 0.7.

Among these parameters, there are two parameters which were not used by Seed and Booker. One is T_l which will later be shown a more appropriate parameter than $T_d (= T_l t_d/t_l)$ used by Seed and Booker. The other is R which was derived by Yoshikuni and Nakanodo (Ref. 5). The parameter t_d/t_l will be shown to have very minor influence upon the pore water pressure ratio for most of the cases. As shown in Fig. 9, when the maximum value of u/σ_{vo}' is less than about 0.5, u/σ_{vo}' reaches its steady state value before $t = t_l$ $(=0.5t_d)$. Therefore, the maximum value of u/σ_{vo}' is determined only by T_l and that the maximum value does not depend on t_d/t_l if $t_d/t_l > 1.0$. Similar conclusions can be drawn for various values of a/b, T_l, and R as shown in Fig.10, in which u/σ_{vo}' reaches 90 % of the steady state value at the time t_{90}. t_{90} is always less than $2t_l$ if $(u/\sigma_{vo}')_{max} < 0.5$. Thus the parameter t_d/t_l has very minor influence upon $(u/\sigma_{vo}')_{max}$ for most of the cases.

Based on the above considerations, the maximum excess pore water pressure ratios averaged over the horizontal section of the ground are given by the nondimensional parameters a/b, T_l, and R as shown in Fig.11.

DESIGN OF SPACING BETWEEN THE GRAVEL DRAINS AGAINST EARTHQUAKES

Once the results shown in Fig.11 are in hand, the spacing between the gravel drains can be determined by the procedure illustrated in Fig.12. In determining the spacing, the value of $(u/\sigma_{vo}')_{max}$, the earthquake ground motions, and the constants of the soils have to be given.

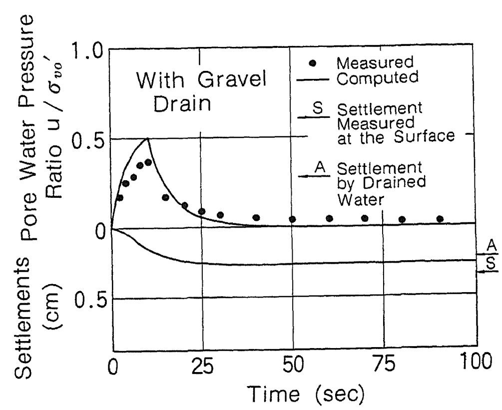


Fig.7 Comparison between the measured and the computed settlements and excess pore water pressure ratio

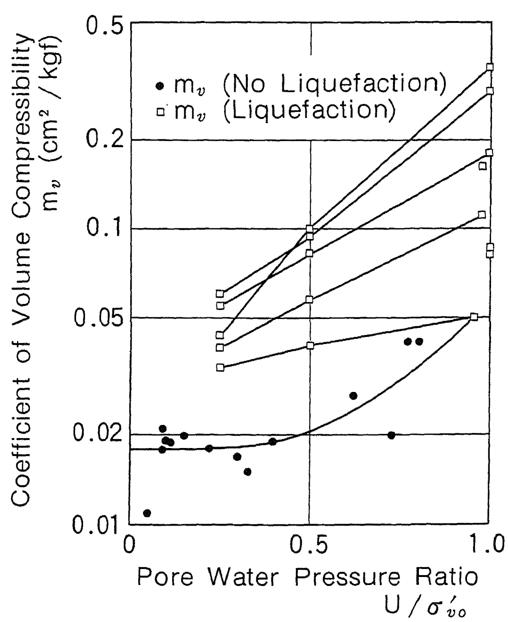


Fig.8 Coefficient of volume compressibility

The value of $(u/\sigma_{vo}')_{max}$ seems to be, in the current practice, often determined as of 0.5. However, the test results suggest that $(u/\sigma_{vo}')_{max}$ is very sensitive to the level of the earthquake motions. Thus, it is necessary to allow a large safety factor upon the value of $(u/\sigma_{vo}')_{max}$. At present, it is difficult to specify the exact value of the safety factor. However, if the authors, who have conducted the large scale model tests, were requested to use an engineering judgement, the authors would suggest the safety factor of two as a standard value;

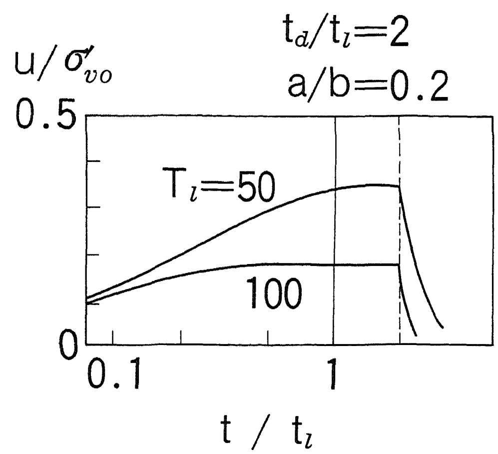


Fig.9 Typical excess pore water pressure change

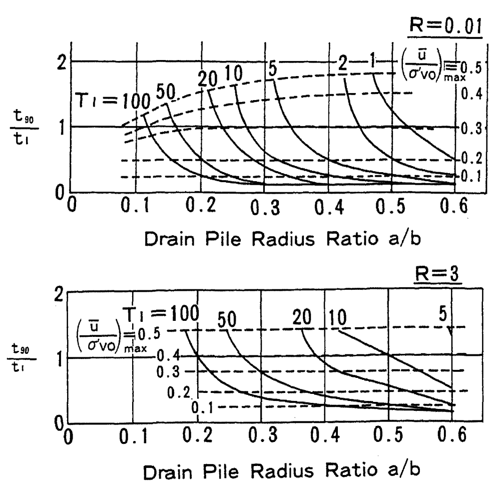


Fig.10 Duration of shaking required to attain the steady state pore water pressures

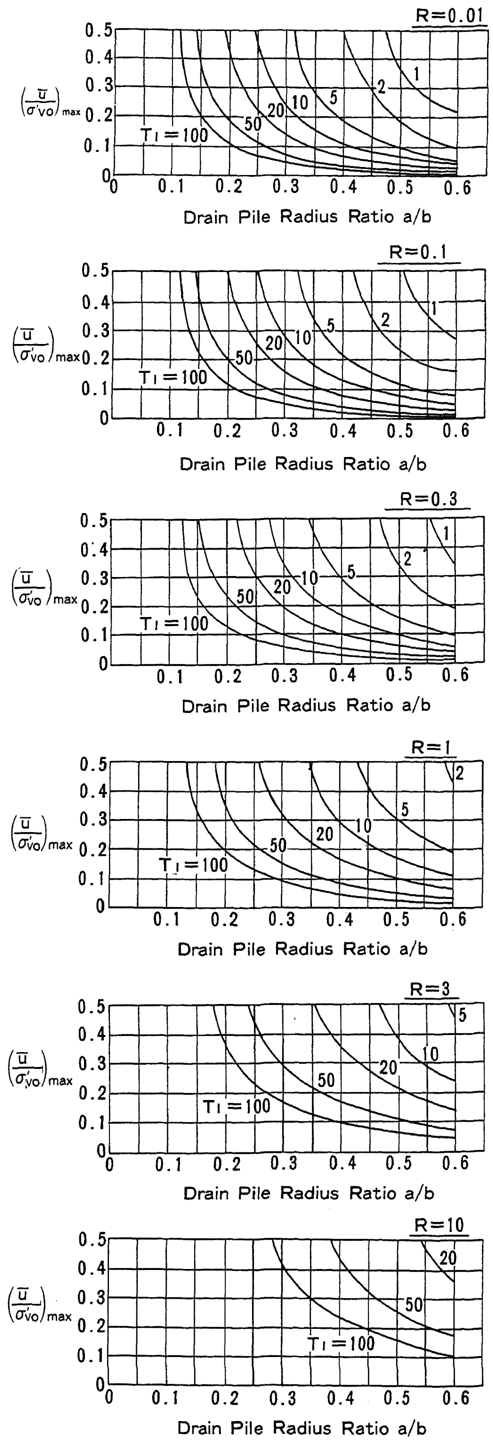


Fig.11 Relation between maximum pore pressure ratio, time factor T_l, well resistance R, and radius ratio a/b

i.e. suggesting 0.25 as a standard value of $(u/\sigma_{vo}')_{max}$. The details in the procedure for determining the spacing between the gravel drains will be found in Ref. 6.

CONCLUSIONS

Following conclusions are derived from the present study:

(1) Terzaghi's consolidation equation with additional term for excess pore water pressure generation has acceptable applicability for analysing the behaviour of the gravel drains.

(2) The following nondimensional parameters should be considered for the design of the gravel drains; a/b = a ratio of the radius of the gravel drain over that of the tributary area, T_l = a factor characterizing the duration of earthquake shaking required to cause initial liquefaction in relation to the consolidation properties of the ground, and R = a ratio characterizing the resistance of the gravel drain in relation to the permeability of the ground.

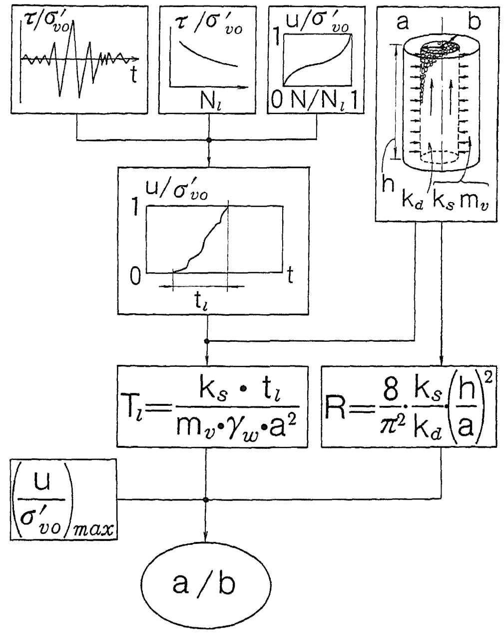


Fig.12 Procedure for determining the spacing between the gravel drains

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