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Two dimensional liquefaction analysis for ground with embankment

H.Abe
Institute of Civil Engineering of Tokyo Metropolitan Government, Japan

ABSTRACT: A two dimensional practical effective stress method EFFCTD for assessing liquefaction potential of ground with embankment is proposed. This program code was verified by shaking table tests for sandy ground with embankment. With EFFCTD, it was possible to reproduce the measurement results obtained in the shaking table tests.

1 INTRODUCTION

Inst. of Civil Eng. of Tokyo Metropolitan Government is in the process of developing the TOTAL system (Total Judgment for Earth Structures in Tokyo Lowland using Static and Seismic Response Analysis including Liquefaction Problems), an evaluation system designed primarily for the assessment of the seismic resistance of earth structures. This development is carried out with a view to streamlining the anti liquefaction measures for structures. Concept of the TOTAL is shown in Fig. 1. In the establishment of the system, allowance was made for the fact that the information of liquefaction damage in Tokyo has been extremely minor so that the system preparation work involved, in the first stage, (1) the development of a liquefaction analysis program based on effective stress method, (2) the implementation of shaking table tests, and (3) the gathering of data concerning precedents of liquefaction damage which has arisen at other cities in Japan. In the second stage, the work involved a comparison of the analysis of the results of (1) and the experiment results of (2) so as to ascertain the performance capability and the suitability range of the analysis program. In the third stage, the task was to create a system capable of reproducing the cases of damage under (3) (precedents) from the analysis program under (1). After this, analysis was performed with the data for the facilities and grounds under the control of the Metropolis of Tokyo to establish a system for predicting liquefaction damage and for examining the appropriate preventive measures. This publication presents the simulation results for the shaking table tests on ground with embankment using the liquefaction analysis program EFFCTD, a relatively convenient analysis program based primarily on experimental data using the laboratory dynamic test, obtained in the second stage of the system construction work.

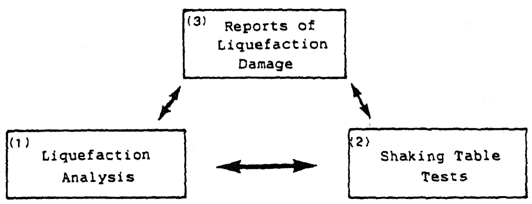


Figure 1. Concept of the TOTAL

2 LIQUEFACTION ANALYSIS PROGRAM EFFCTD

The computer program EFFCTD is seismic response analysis technique based on explicit (or implicit) direct integration method using the two dimensional finite element method. For the relationship between shear stress and shear strain based on the Ramberg-Osgood formula with Masing's rule (modified R-O model). The modified R-O model can calculate hysteretic non-linear behavior of soil. This model, however, cannot directly determine excess pore water pressure (an acronym E.P.W.P.) due to dilatancy. The undrained rate of E.P.W.P. build-up under irregular shear loading has been determined by Kokusho's method (1982). Kokusho et al. established an algorithm for effective stress path by using the empirical E.P.W.P. build

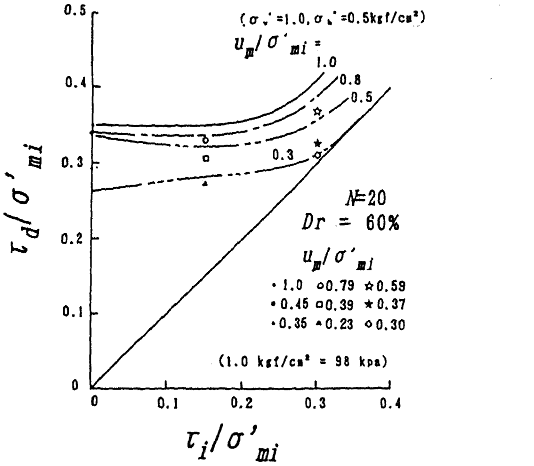
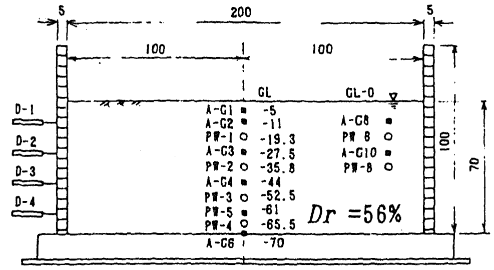


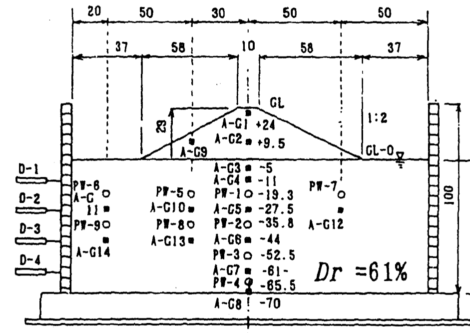
Figure 2. The effect of the initial shear stress in the E.P.W.P. changes

up formula proposed by Seed et al.(1976) and the cumulative damage concept proposed by Annaki and Lee (1977). The R-O model is one dimensional stress-strain equation. For the two dimensional problems, assuming the shear stress τ_{xy} and shear modulus G_{xy} in the X-Y plane are equivalent to one dimensional shear stress τ_d and shear modulus G_0. These values were installed in the plain strain elasticity D matrix proposed by Clough and Woodward (1967).

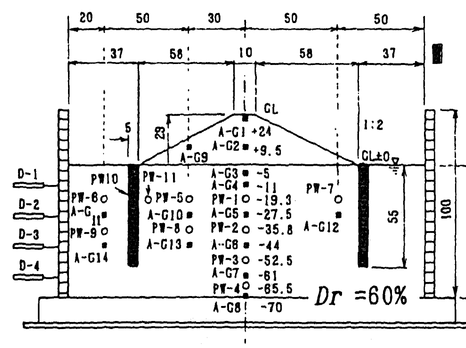
For soil elements under the slope of embankment, initial static shear stresses will exit on the horizontal plane. To carry out the estimation of the E.P.W.P. build up of the ground with embankment subject to the action of the initial shear stress τ_i, the results of undrained cyclic hollow cylinder torsion tests were used because of the excellent reproducibility of the stress conditions they offer. The test data were arranged so as to determine the stress ratio (divided by the initial effective mean stress σ'_{mi}) at a cyclic number N=20, that is, the cyclic shear stress ratio τ_d/σ'_{mi} and the initial shear stress ratio τ_i/σ'_{mi}, and the E.P.W.P. ratio in term of u_p/σ'_{mi}. For the Toyoura sand (Japanese standard soil, G_s = 2.643, D_{max} = 0.42 mm, U_c = 1.73, D_{50} = 0.2096 mm, e_{max} =0.974, e_{min} = 0.610) , the test result is shown in Fig. 2. For the Toyoura sand (relative density Dr = 60%), it was observed that the greater the initial shear stress ratio was, it became for the E.P.W.P. ratio to decrease. The test results were incorporated into the EFFCTD so that it became possible to reflect the effect of the initial shear stress in the E.P.W.P. changes.



The first model



The second model



The third model

○ Pore pressure gauge

▪ Accelerometer

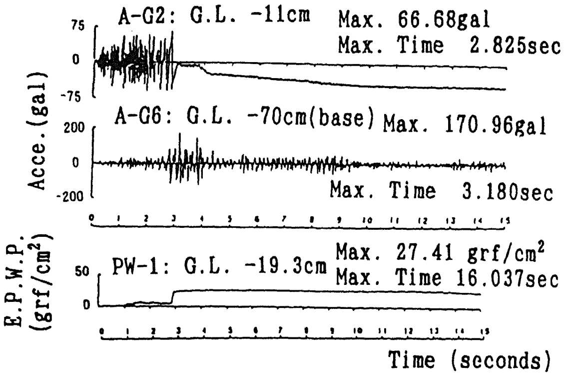
⟹ Displacement transducer

Unit cm

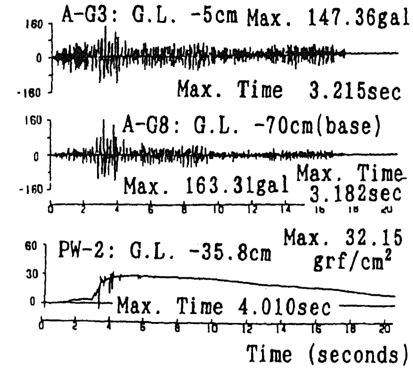
Figure 3. Experimental models

3 SHAKING TABLE TESTS

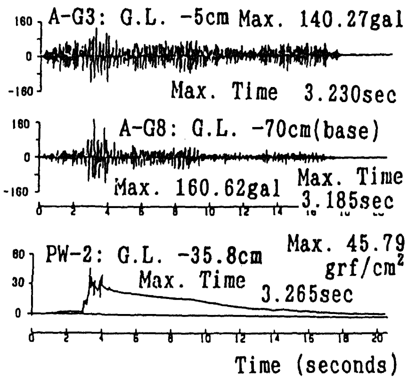
The authors have performed shaking table tests using observed seismic wave on sandy ground and embankment models in order to establish countermeasures against the liquefaction of earth structure due to



The first model



The second model



The third model

Figure 4. Observed acceleration and excess pore water pressure time histories

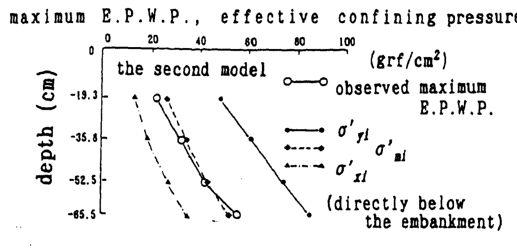


Figure 5 Maximum excess pore water pressure

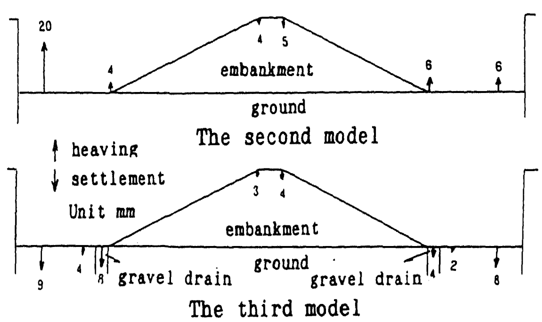


Figure 6 Displacement after experiments

earthquakes. The tests were carried out on a 4m in length and 4m in width shaking table at Inst. of Technology, Shimizu Corporation in Tokyo. The authors prepared three types of models with sandy soil to perform the shaking table tests. Three models are shown in Fig.3. The soil container (2m long, 1m high, and 1.5 m wide) was made by a pile of 18 aluminum frames. The first model (70 cm in depth, approximately Dr = 60%,) consisted of saturated Toyoura sand ground only (G_s = 2.645, D_{max} = 0.42 mm, U_c = 1.77, D_{50} = 0.1979 mm, e_{max} =0.952, e_{min} = 0.595) and was exposed to vibration, acceleration motion known as Hachinohe E-W and recorded during the Tokachi-oki earthquake of 1968, at a maximum acceleration amplitude of 171 gal. Seismic wave excitation direction was horizontal only, and the time scale factor by considering similitude was 1/4. The second model, obtained by filling an embankment using unsaturated Toyoura sand on the unimproved ground, was subjected to vibration at a maximum acceleration of 163 gal. The third model, placing gravel drains (G_s = 2.714, D_{max} = 4.75 mm, U_c = 2.29, D_{50} = 2.17 mm, e_{max} =0.780, e_{min} = 0.549) immediately at the toe of the slope of the embankment, was submitted to the shaking test at a maximum acceleration of 161 gal.

The results of these experiments were used to examine the seismic behavior the models. Fig. 4 shows the typical results of the acceleration and the excess pore water pressure (E.P.W.P.) time histories in these experiments. Fig.5 shows the maximum E.P.W.P. in the second model ground. In the first experiment on the ground alone, the E.P.W.P. in the model ground as a whole reached the effective overburden pressure σ'_{yi} and the

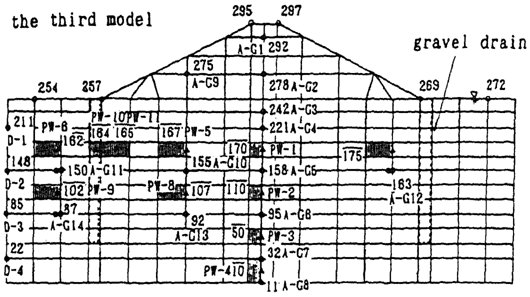


Figure 7 Finite element model

ground was no longer able to accommodate the response acceleration motion so that liquefaction occurred. In the second experiment, the E.P.W.P. at the directly below the embankment rose to σ'_{mi}, without giving rise to liquefaction, so that the amount of drop of the acceleration amplitude was also small. At the more remote locations on the periphery of the ground model, however, liquefaction did take place since the maximum E.P.W.P. reached σ'_{yi}. There was no major difference between the accelerations and the water pressures measures for the

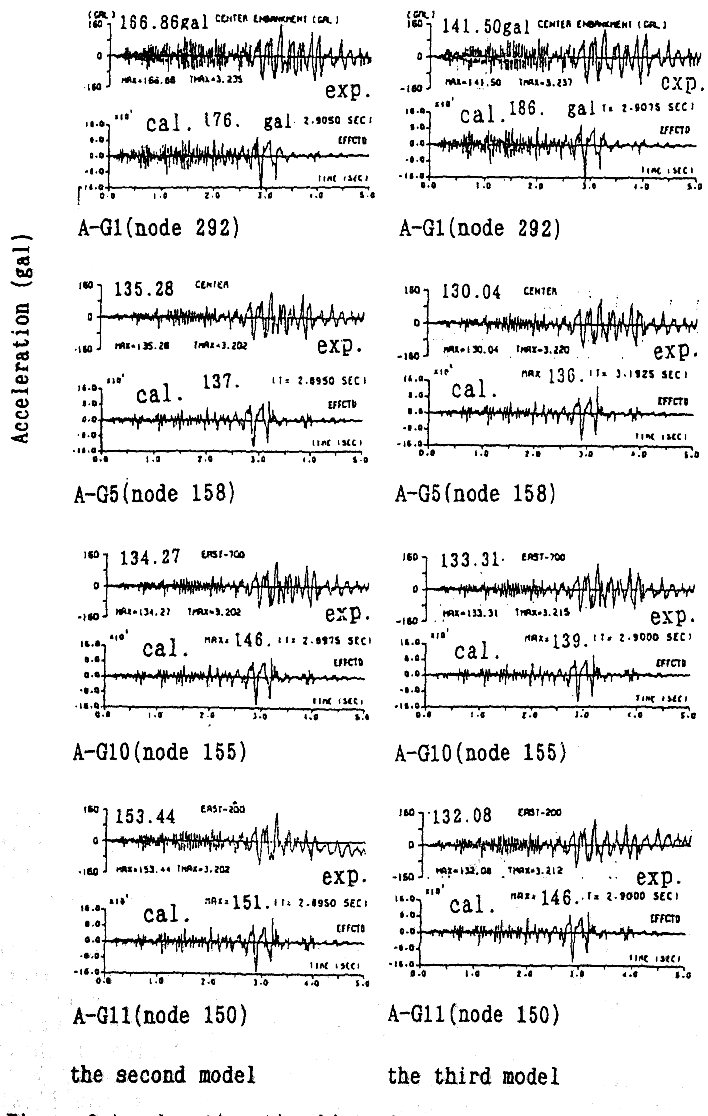


Figure 8 Acceleration time histories

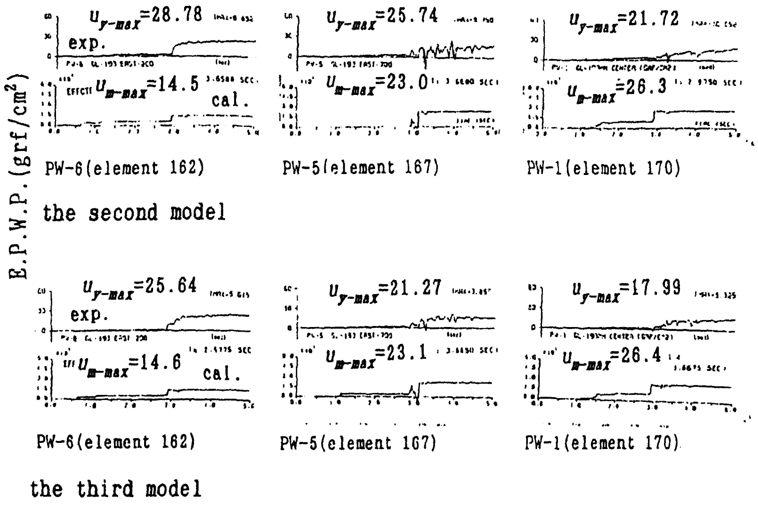


Figure 9 Excess pore water pressure time histories

improved ground used in the third experiment and the unimproved ground in the second experiment. In the improved ground, the E.P.W.P. was dissipated at a very early stage. Fig. 6 shows the displacement in the second and third models. After the experiment, the height of the ground was measured when it was found that in the case of the improved ground, both the embankment and the peripheral ground had subsided. By contrast, the experiment on the unimproved ground had resulted in subsidence of the embankment but to a heaving of peripheral ground at the toe of the embankment slope. The unimproved ground immediately underneath the embankment, however, is very vulnerable to lateral deformation so that the rise in lateral pressure was small and typical liquefaction is difficult to occur as was found in the case of the sandy ground used in the first experiment. In the improved soil (installing gravel drains) obtained by filling an embankment on the ground, lateral displacement of the sandy ground is suppressed to as greater extent than in case of the unimproved ground so that this can reasonably be expected to be effective as an anti-liquefaction measure.

4 COMPARISON BETWEEN CALCULATIONS AND EXPERIMENTS

Fig. 7 shows finite element mesh for the third model. A step by step analysis was carried out by central difference method. The effective mean stress σ'_m, tangential shear modulus G'_t, and shear strength τ'_f were changed from the excess pore water pressure (E.P.W.P.) which was generated at each time step. The EFFCTD parameters for the second and third models are shown below. Assuming the R-O model's parameters α' =(saturated sand (S.S.) 3.18, unsaturated sand (U.S.) 4.05, and saturated gravel(S.G.) 2.77) and β =(S.S.1.67, U.S. 2.02, and S.G. 1.47), Seed's empirical parameter =0.5, liquefaction stress parameters R_{120} = (S.S 0.165 and S.G. 0.33) and gradient of $Log\ R_l$ - $Log\ N$ relation (S.S. -9.8 and S.G. -1.9), internal friction angle ϕ' =(sand 39° and gravel 62°), transformation angle ϕ'_{CM} = 28°, and shear modulus G'_0 =(S.S. 16180.0 grf/cm², U.S. 18783.0, and S.G. 23736.0 at σ'_{ml} = 1.0 grf/cm²), the comparison between calculated and measured acceleration time histories as shown in Fig. 8. Similar to Fig.9 shows the comparison the E.P.W.P. time histories. From simulation performed on ground including fairly loose unimproved sandy soil ground with embankment and gravel drains, they were possible to obtain results showing practical agreement between the values for the maximum acceleration amplitudes, measured in locations such as directly below the embankment, under slope of the embankment, and on the periphery of the model ground, and the analysis results. The E.P.W.P. values obtained from the analysis led to results suggesting a somewhat greater vulnerability to liquefaction than the measurement values. The substantial drop in the E.P.W.P. seen in the measurements on ground containing gravel drains was not a reproducible phenomenon because of the lack of a dissipation analysis code.

5 CONCLUSIONS

With EFFCTD, it was possible to reproduce the measurement results obtained in the shaking table tests. The results obtained for the installing gravel drains require a dissipation analysis for proper evaluation.

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