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SCALED MODEL TESTS AND NUMERICAL ANALYSES ON NONLINEAR DYNAMIC RESPONSE OF SOFT GROUNDS

by Takeji Kokusho[I], Takahiro Iwatate[I] and Satoshi Ooaku[II]

ABSTRACT

Shaking table tests have been performed in this research to represent the nonlinear vibration of soft grounds due to strong earthquake motions. Special care has been taken for the design of a soil container in order to reproduce idealized horizontal shear wave propagation in the model ground. Nonlinear acceleration responses to various random motions measured in the model ground are compared with the numerical results obtained by the equivalent linear analysis widely used in engineering practice. This comparative study reveals general efficiency of the analytical technique for engineering prediction of earthquake motion of soft grounds although the analysis gives poorer prediction for smaller amplitudes involved in the random motion.

INTRODUCTORY REMARKS

It was pointed out by Dr. Kanai (1969) that the dynamic response of the ground to major portion of earthquake motions is mostly caused by the vertical propagation of horizontal shear wave motion (S_H-wave). Number of research works followed along this way in which soft ground was idealized as linear elastic material possessing constant shear modulus and damping ratio irrespective of the intensity of earthquake disturbance. The modern development of soil dynamics in this decade have experimentally clarified the dynamic properties of soil materials. Hardin and Drnevich (1970,a) making a thorough experimental study on the subject came to the principal conclusion that the magnitude of strain has predominant effect on dynamic properties of soil; namely for shear strain larger than 10^{-4}, the shear modulus of soil decreases, and the equivalent damping ratio increases with the increase of strain.

Based on the Kanai's theory of multi-reflection of S_H-wave Schnabel and Lysmer (1972) proposed an numerical analysis of earthquake response of ground, what is called "equivalent linear analysis", in which strain-dependency of dynamic properties of soil is taken into account in approximate fashion. According to this analytical scheme shear strain as much as 10^{-3} is to be induced in soft grounds during strong earthquakes implying the significant nonlinear effects should be expected in these occasions. It is therefore of much importance to find out how efficient the equivalent linear analysis is for the prediction of nonlinear earthquake response of soft grounds. The best way to do this is to have simultaneous records of vibrational response to a strong earthquake motion at different levels of a ground, which are not yet available.

The main objective of this research is to prove the applicability of the analysis with the help of scaled model tests. A model ground of one meter in depth made of fine clean sand was set on a shaking table, and various kinds of random waves simulating earthquake motions were applied to it. The change of the acceleration amplification along with the table acceleration was extensively studied. The experiments were compared with the equivalent linear analysis often

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I. Central Research Institute of Electric Power Industry
II KASUMIGASEKI INFORMATION CENTER, NIKKAGIKEN CORPORATION

used in engineering practice, in which material properties extracted in advance from the damped free vibration of the model ground itself were used.

SCALED MODEL VIBRATION TESTS

Model Ground   On the ground that the dominant horizontal motion of earthquakes is caused by the propagation of horizontal shear waves, it is crucial to reproduce the free horizontal motion of the shear mode in the model ground. Conventional soil containers with rigid walls have considerable confinement on the shear vibration. A new type of a soil container shown in Fig. 1 has been developed in this research to satisfy the requirement mentioned above. Its walls consist of 16 rectangular steel hollow frames with 60 mirimeter square cross-sectional area. These frames are set up one by one to form the side walls of one meter in height with ball bearings at each contact to ensure frictionless horizontal movements. The wall is covered by rubber membrane to make the box watertight. Special care was taken to reduce the weight of the wall to 9 to 12 percent of the weight of the contained soil.

Fine clean sand was chosen as a material for the model ground which was dropped by a specially designed apparatus. It was a shallow box mounted on the soil container whose bottom consists of twentyfour separate thin flaps rotating at the same time by pulling a lever. Such high void ratio as 0.89 to 0.91 could be attained by this apparatus for both dry and saturated ground.

Strain-gage type accelerometers were used to measure the vibration at five different levels in the model ground from the surface to the base with approximate intervals of 25 centimeters. The frequency range of the accelerometer was from zero to 70 Hz covering the extent necessary for this experiment. Small seismometer were embedded in the ground at four different levels for seismic explorations which were conducted before shaking table tests to estimate the wave velocities of the ground.

Dynamic Material Properties of Model Ground   The maximum confining pressure in the model ground does not exceed 0.1 kg/cm^2 and laboratory test data corresponding that low stress condition are not available now. The alternative way taken herein was to make direct determination of the material properties by seismic exploration and by the measurement of damped free vibration of the model ground.

The variation of shear moduli determined by seismic explorations is shown in the logarithmic graph of Fig. 2 along with the vertical effective stress. Despite of a rather large scatter of the results, it can be said the shear modulus, G, is approximately proportional to the square root of the effective vertical stress, $\sigma v'$. In order to decide the material properties for larger vibrations than those corresponding to seismic explorations, damped free oscillations of the model ground in its lowest natural frequency were measured. Sinusoidal motion, first given to the model by the shaking table to excite the responance, was suddenly interrupted, and the subsequent damped oscillation was measured to give the average shear modulus G as well as the damping ratio h as the functions of shear strain induced in the ground. Shear strain computed from the division of the displacement at the surface by the ground depth was assumed constant for every portion of the ground, which proved to be reasonable (Kokusho et al. 1978). The dynamic material properties thus obtained, shown in Fig. 3, represent the average values for the ground as a single degree of freedom system. The maximum shear modulus, Gmax, derived in this way is equal to 74 kg/cm^2 for the dry sand, and 56 kg/cm^2 for the saturated sand, undershooting the values determined by seismic explorations as illustrated in Fig. 2. This may be attributable mostly to the fact that the depth to width ratio of the model ground is not large enough to kill the bending mode of oscillation to reproduce the ideal shear vibration.

In order to spot the G vs. γ curve and the h vs. γ curve for larger strains than experimentally determined, theoretical model for strain-dependency of soil properties proposed by Hardin and Drnevich (1970,b) was employed with some modifications. According to the model the G vs. γ relation is written as

$$G/G_{max} = 1/(1 + (\gamma/\gamma_r)). \tag{1}$$

γ_r, called as reference strain, is defined with maximum shear modulus, Gmax, and shear strength, τ_f, by

$$\gamma_r = \tau_f/G_{max}. \tag{2}$$

The same model expresses the h vs. γ relation as

$$h/h_{max} = \gamma/\gamma_r/(1 + (\gamma/\gamma_r)) \tag{3}$$

with maximum damping ratio, hmax, to be assumed. hmax is usually considered to be around 0.3 (Hardin and Drnevich 1970,b). Since it was expected from shaking table tests that the model ground possibly has larger damping ratio than evaluated by Eq.(3), a modified form of the Hardin-Drnevich model proposed by the authors (Kokusho and Sakurai 1978) was used for the h vs. γ relationship. In this modified model, although the shear modulus is expressed by Eq.(2), the damping ratio is differently formulated as

$$h = \frac{4}{\pi}\left[\frac{G_{max}}{G}\left\{\frac{\gamma_r}{\gamma} - \left(\frac{\gamma_r}{\gamma}\right)^2 \ln\left(1 + \frac{\gamma}{\gamma_r}\right)\right\} - \frac{1}{2}\right]. \tag{4}$$

It should be noted that the proposed model gives almost the same damping ratio as that given by Eq.(3) with hmax = 0.3 as long as (γ/γ_r) is less than 2, after that Eq.(4) yields larger damping than Eq.(3). For better agreement with experimental values a constant term of 0.02 was further added to Eq.(4).

<u>Dynamic response to Random Motions with Different Magnitude</u> In order to experimentally simulate the dynamic response of soft grounds to earthquakes, four different random motions were selected to be applied to the model ground, which are named here as Ea, Eb, Ha and Hb. The former two were produced from El Centro motion [1940, NS] while the latter two from Hachinohe motion [1968] by appropriate time shortening and wave filtering processes. As shown in the acceleration response spectra of Fig. 4, Ea and Eb motions have a rather flat spectrum of 8 to 22 Hz and 14 to 30 Hz, whereas Ha and Hb motions have a high concentration of spectrum around 25 Hz. Based on the fact that the first resonant frequency of the model ground for small tremors exists around 18 Hz for the dry ground or 14 Hz for the saturated ground, it is expected that the first two motions are more likely to excite the first resonance than the latter two.

Fig. 5 shows acceleration response of the ground measured at five levels due to the Ea motion with three different magnitudes. It is evidently seen that the surface motion varies quite a bit with increasing base acceleration in spite of the similarity of the base motions. The decrease of amplification with increasing base acceleration is also obviously seen. The change of amplification along with the magnitude of base motion is more clearly illustrated in Fig. 6 where the ratio of the maximum acceleration of the ground of four different levels to that of the base is taken versus the maximum base acceleration for the four random motions. It is noteworthy here that the amplification scattered for smaller base accelerations than 100 gal shows consistent decline for larger accelerations irrespective of the random motions, and at the surface it even decreases less than one for larger accelerations than 300 gal. Fig. 7 is the same graph of amplification vs. base acceleration in the saturated case for Ea and Ha motions. For the saturated ground the amplification decreases more rapidly than for the dry ground and becomes smaller than one for base acceleration around 80 gal. Liquefaction which did not evidently take place under 80 gal can not be the principal cause for this rapid decline of amplification. Instead, this may be explained by the difference of the soil density and the effective vertical stress.

Thus, the shaking table tests have demonstrated that the acceleration amplification of the model ground due to strong base motions is much smaller than that due to small motions. In the light of new concept of simulitude (Kokusho et al. 1978) which may possibly be applicable to model tests using real soil materials, a similar decline of acceleration amplification is very likely to occur for soft grounds of prototype. The increase of damping with increasing shear strain will be the main cause of this consistent reduction of amplification, although the decrease of the shear modulus with increasing strain may possibly fluctuate this by the change of the resonant frequency of the ground.

EQUIVALENT LINEAR ANALYSIS

The frequently used way to estimate the acceleration response of soft ground is the equivalent linear analysis in which, instead of performing complete nonlinear response analysis, a series of linear analyses are iteratively conducted employing appropriate modulus and damping corresponding to induced strain. The problem hereby arises how to choose the most appropriate values in order to enable the linear analysis to best approximate the nonlinear response. According to Schnabel and Lysmer (1972) the effective strain, γ_{eff}, determined by the maximum strain, γ_{max}, as

$$\gamma_{eff} = \delta\gamma_{max} \tag{5}$$

can be used to choose the corresponding soil modulus and damping. The same authors contended that the constant δ may take 0.65 although such values as 0.5 to 0.7 will not make great difference in the results. The discussion on δ is not the subject of this research, hence γ_{eff} = 0.65 γ_{max} is adopted in this analysis since it is most widely used in engineering practice today.

The ground was first assumed to be uniform with single values of the shear modulus, G, and the damping ratio, h. In this case the $G \sim \gamma$ relationships of Fig. 3 were directly used. Next a four layers model was employed to represent the model ground. In that case the maximum shear modulus, G_{max}, and the reference strain, γ_r, for each layer were determined by using the assumption that they are proportional to the square root of the effective vertical stress (Kokusho et al. 1978).

Fig. 8 shows acceleration time histories at the surface due to Ea and Hb motions with base accelerations of almost 200 gal analysed with the single layer model. Their agreements with the measured performance are quite satisfactory. Fig. 9 again shows analysed acceleration responses at the surface using the four layers model. Analytical results in Figs. 8 and 9 well coincide with each other implying that the single uniform model is almost sufficient to numerically analyse the dynamic response of the ground. Similar comparisons of the analyses and the experiments are shown in Fig. 10 for the saturated ground. Although generally good agreements of the analyses and real performances are quite visible in Figs. 8 to 10, a careful look at the figures reveals that the degree of coincidence differs from one portion to another in each random motion. This can be more obviously seen in Fig. 11(a) where each amplitude of measured random motions shown in Figs. 8 and 10 is taken along the abscissa against the amplitude ratio of analysis to experiment along the ordinate. For larger amplitudes involved in random motions the ratio is about one meaning that the analysis makes good estimate of real performance, while for smaller amplitudes the ratio takes smaller values than one implying that the analysis somewhat underestimates the experiment. In Fig. 11(b) some of the measured amplitudes of the motions are again taken against the corresponding phase lags in seconds between the two acceleration motions of experiment and analysis. The phase lag relatively small for larger accelerations grows larger for small accelerations. The two graphs described above indicate that the equivalent linear analysis where the constant δ in Eq.(5) is equal to 0.65 makes a good prediction of acceleration motion for relatively larger amplitudes involved in random motions, whereas poorer prediction should be expected for the remainder. This comparative study has, therefore, confirmed a vague expectation that the nonlinear vibration of a ground is only partially simulated by the equivalent linear analysis.

It can be maintained, however, this fact will not greatly hinder the efficiency of the analysis in engineering sense, since larger amplitudes of earthquake response are most significant to engineering design. Maximum accelerations at the surface of the model ground due to the random motions predicted by the analyses are plotted with the star marks in Figs. 6 and 7 for the dry and saturated grounds. Generally good agreements between the measurements and the analyses are quite visible for wide ranges of base acceleration. Furthermore acceleration response spectra shown in Fig. 12 also demonstrate a satisfactory agreement between the experimental measurement and the analysis hence the effectiveness of the analysis.

CONCLUSIVE REMARKS

The combined study of the model vibration tests and the numerical analyses has drawn a conclusion that the equivalent linear analysis offers a good prediction of the nonlinear dynamic response of soft grounds for engineering purposes.

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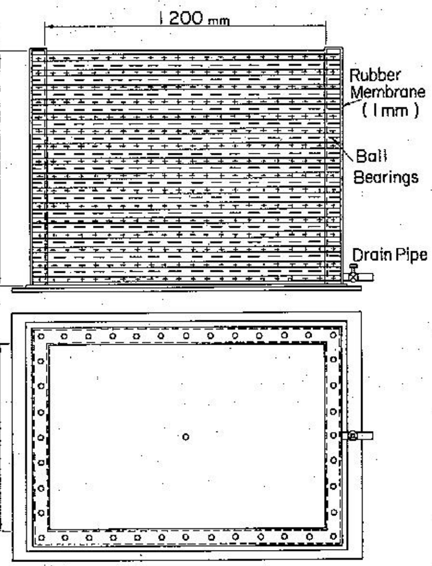

Fig. 1 Soil Container used in the Experiment

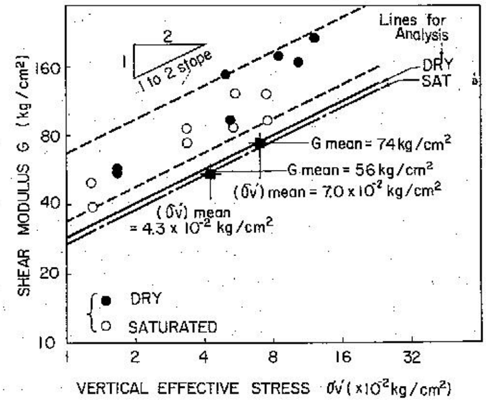

Fig. 2 The Shear Modulus, G, Obtained by Seismic Explorations

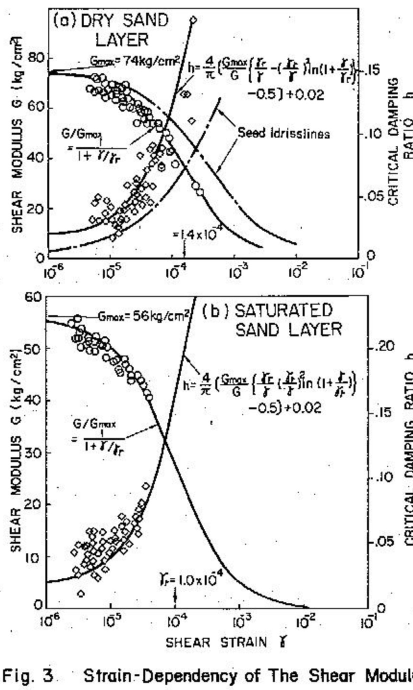

Fig. 3 Strain-Dependency of The Shear Modulus and The Damping Ratio

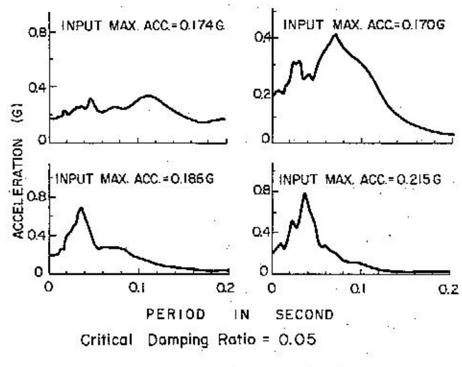

Fig. 4 Response Spectra of Applied Random Motions

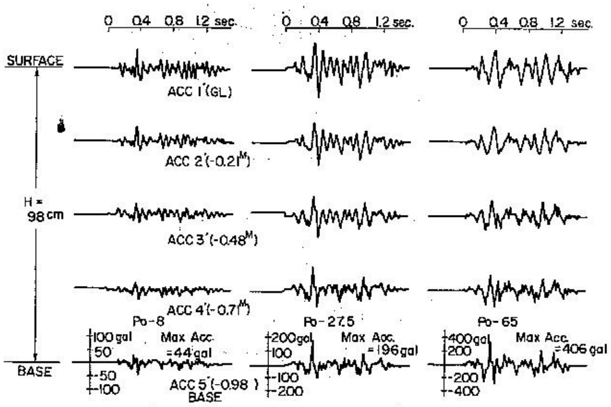

Fig. 5 Acceleration Motions of the Model Ground

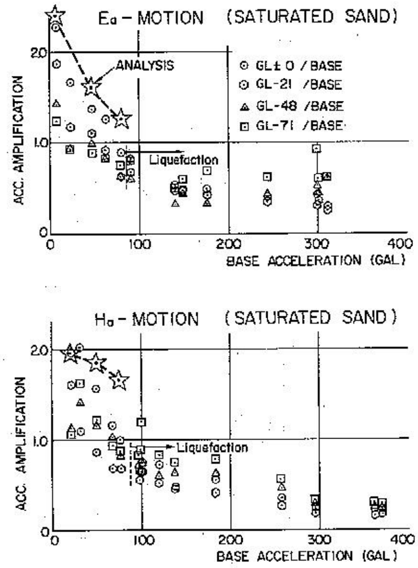

Fig. 7 Variation of Amplification with Base Acceleration (Saturated Sand)

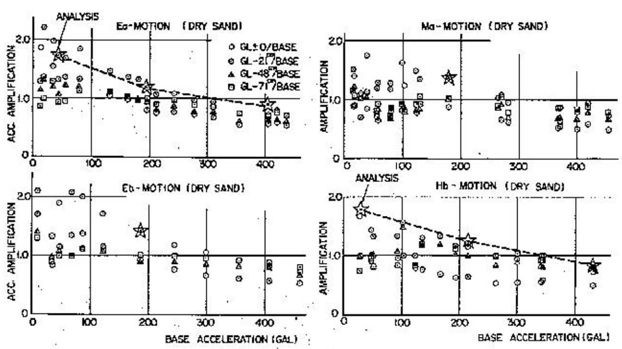

Fig. 6 Variation of Amplification with Base Acceleration (Dry Sand)

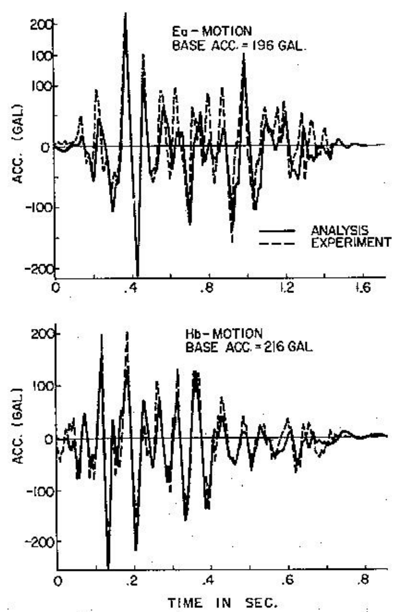

Fig. 8 Comparison of Acceleration Time History (Single Layer Model, Dry Sand)

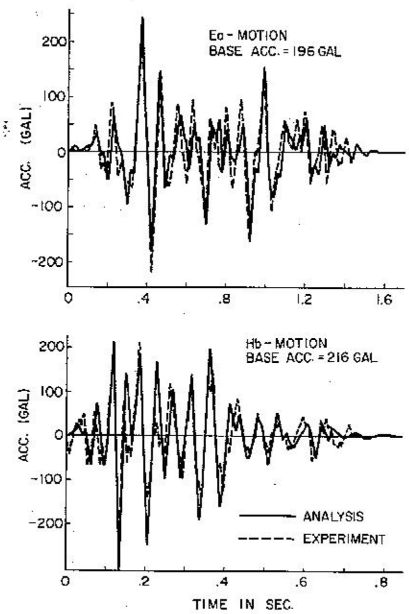

Fig. 9 Comparison of Acceleration Time History
(Four Layers Model, Dry Sand)

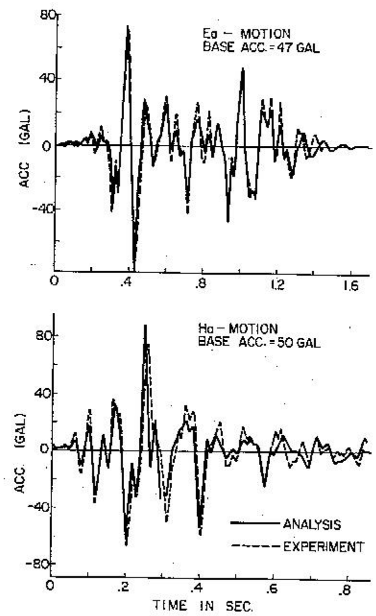

Fig. 10 Comparison of Acceleration Time History
(Single Layer Model, Saturated Sand)

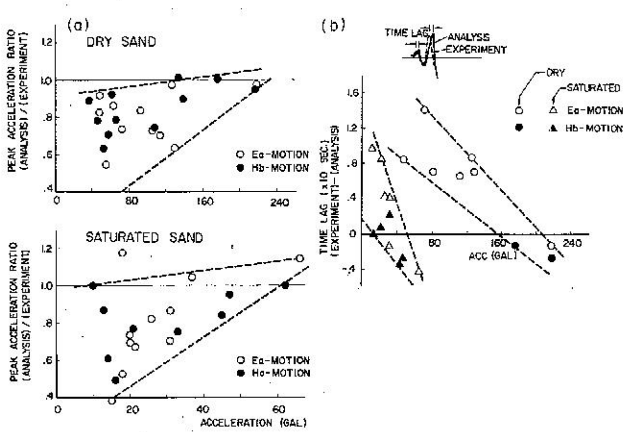

Fig. 11 Comparison of Accelerations between the Equivalent Linear Analysis and the Experimental Measurement

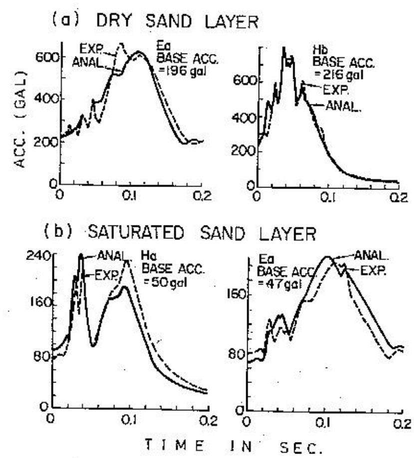

Fig. 12 Acceleration Response Spectra of the Analytical Results Compared with the Experiments