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NONLINEAR SEISMIC RESPONSE ANALYSIS OF SOIL DEPOSIT USING STRONG SEISMIC RECORDS

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SUMMARY

Studies using strong seismic records are made in order to investigate the actual nonlinear seismic response of soil deposits, and to clarify the applicability of dynamic models of soil to the real field. It is noted from the identification results obtained by the authors' method that the predominant period of the ground grows longer, and the damping constant becomes greater, than their linear values during the principal motion period. Nonlinear seismic response analyses using strong seismic records are carried out. From the comparison between analytical results and observed records, it is found that the equivalent linear method and the step-by-step method using the modified Ramberg-Osgood model are effective in nonlinear seismic response analysis. In addition, it is concluded that the modified Hardin-Drnevich model gives excessive damping values in the large strain range.

INTRODUCTION

It is very important to make an accurate estimate of the nonlinear seismic response of soft soil deposits because of increasing opportunities for building structures on soft ground, such as the reclaimed land along seashores.

The equivalent linear method and the step-by-step method using the modified Ramberg-Osgood and Hardin-Drnevich models (Refs. 1, 2) are used in nonlinear seismic response analysis. Since there are few records of seismic strong motions obtained by array observation, it is not clear how applicable to the real field these models are.

This paper presents research results obtained by using strong seismic records, dealing with:

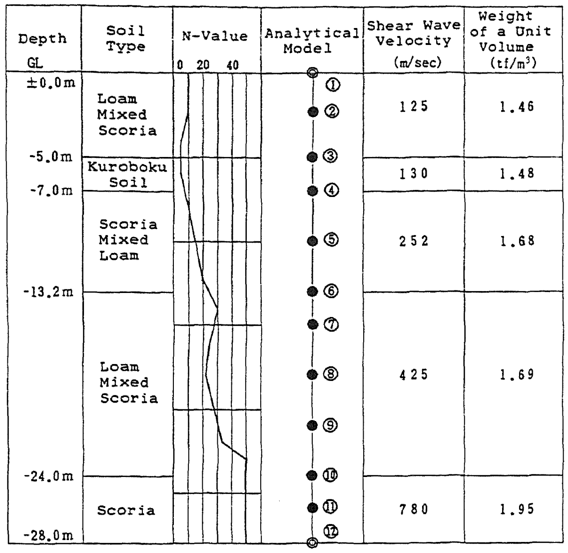
(1) the identification of the time-varying predominant period and the damping constant,

(2) nonlinear seismic response analysis

of soft soil deposits in order to investigate the actual nonlinear seismic response soil deposits, and to clarify the applicability of dynamic models of soil to the real field.

SEISMIC OBSERVATIONS

<u>Soil Profiles at the Seismic Observation Site</u> Seismic observations are made upon the installation of accelerometers as shown in Figure 1 (Ref. 3). Accelerometers



◎ Location of Accelerometers

Fig. 1 Soil Profile and Analysis Model

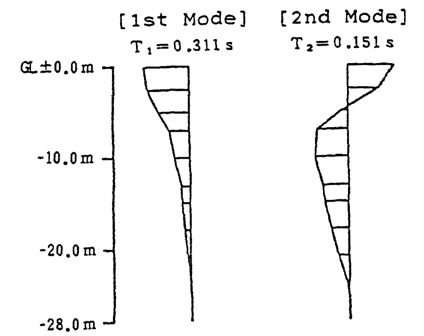


Fig. 2 Eigen Value Analysis Results

were installed at the ground surface and at the base layer (G.L.-28m). The standard penetration N-value of the surface layer (from G.L. to G.L.-7m) is less than 10 and its shear wave velocity is about 130 m/s.

Figure 2 shows the eigenvalue analysis results obtained by the one-dimensional lumped-mass model with 12 lumped-masses, as shown in Figure 1. The predominant period T_1 of this soil deposit is 0.31 seconds.

<u>Seismic Observation Records</u> Figure 3 is the seismic observed records obtained from the Kanagawa-Yamanashi-Kenzakai Earthquake (August 8, 1983, J.M.A. -Japan Meteorological Agency- scale magnitude M=6.0, epicentral distance Δ=18km, focal depth D=22km). The maximum acceleration values at the ground surface and at the base layer are 435 cm/s^2 and 134 cm/s^2, respectively.

Figure 4 shows the transfer functions between the surface and the base. The mean transfer function is calculated by averaging the functions for all observed data except the strong motion shown in Fig. 3. It is found from comparison between both transfer functions that the peak at 0.33 seconds corresponding to the predominant period of the ground grows longer at 0.5 seconds for strong seismic records.

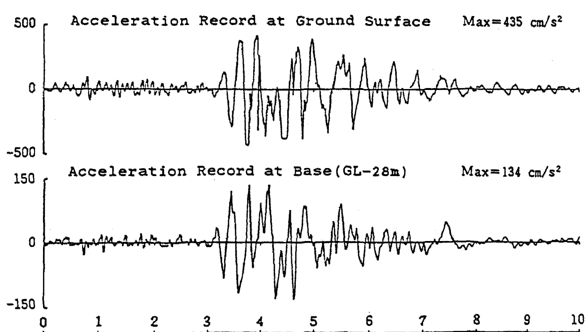


Fig. 3 Strong Seismic Motion Records
(M=6.0,D=22km,Δ=18km)

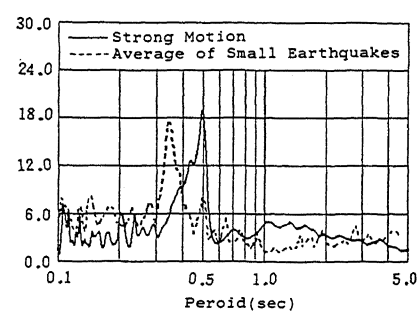


Fig. 4 Transfer Functions
(Ground Surface/Base)

IDENTIFICATION OF TIME-VARYING PREDOMINANT PERIOD AND DAMPING CONSTANT

Typical phenomena of a nonlinear seismic response of soft soil deposits are the growth of the predominant period and an increase in the damping constant. The authors have proposed a time-variant identification method (Ref. 4) which is an extension of the technique originally proposed by J.L. Beck (Ref. 5) to apply to nonlinear seismic response problems.

<u>Results of Identification</u> Table 1 shows the identification results for the predominant period T_1, the damping constant h_1, and the participation factor β_1, of the ground using strong seismic records for 2.0 second specific period ranges. The J-value shown in Table 1 is the minimum values of measure-of-fit defined as Equation (1) which indicates the degree of agreement between the observed records x(t) and the analytical results u(t) (Ref. 4).

$$J = \int_{t0}^{t1} \{ x(t)-u(t) \}^2 \, dt \Big/ \int_{t0}^{t1} x(t)^2 \, dt \qquad (1)$$

Where, t0 and t1 are the starting and the ending times of a specific period range. A smaller J-value means the identification results are more accurate. It is found from the identification results that the predominant period of 0.35 seconds corresponding to the elastic seismic response becomes 0.43 seconds, and that the damping constant of 0.02 - 0.03 increases to 0.07 - 0.08 in the principal motion period of 4 - 6 seconds.

Figure 5 shows contour lines of the J-values for the relative acceleration at the ground surface. The horizontal and vertical axes in Figure 5 represent the predominant period T_1 and the damping constant h_1 of the ground. It is found that the oval shaped contour lines have their centers at the minimum points of the J-value and these centers move to longer periods and higher damping as the time history progresses.

Table 1 Identification Results

Motion Item	Examined Values	Specific Period Ranges (sec) 0 ~ 2	2 ~ 4	4 ~ 6	6 ~ 8	8 ~10
Displ.	T_1(sec)	0.352	0.409	0.426	0.421	0.407
	h_1	0.032	0.107	0.072	0.028	0.021
	β_1	2.855	2.889	2.822	2.435	2.332
	J_{11}	0.214	0.024	0.009	0.010	0.019
Vel.	T_1(sec)	0.346	0.409	0.431	0.423	0.405
	h_1	0.026	0.127	0.089	0.031	0.031
	β_1	2.208	2.740	2.860	2.310	2.015
	J_{11}	0.166	0.015	0.003	0.010	0.022
Acc.	T_1(sec)	0.347	0.436	0.427	0.423	0.405
	h_1	0.025	0.133	0.080	0.032	0.031
	β_1	2.076	2.826	2.728	2.297	1.987
	J_{11}	0.044	0.009	0.006	0.020	0.024

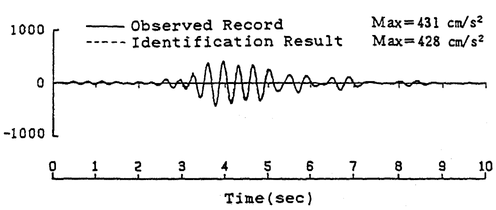


Fig. 6 Comparison between Observed Record and Response Acceleration Based on Idendification Results

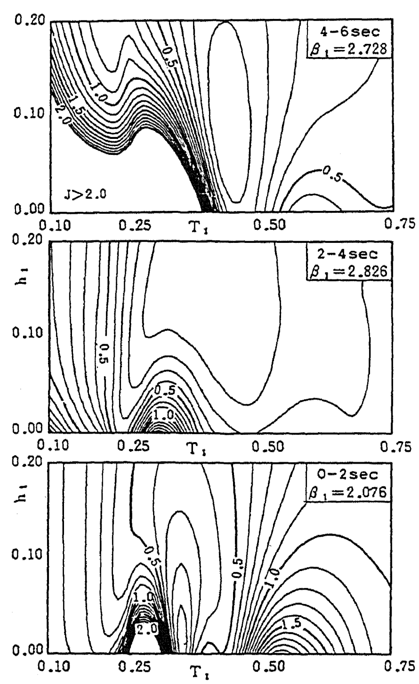


Fig. 5 Contour Line of J_{11} for Acceleration at Ground Surface

Seismic Response Analysis Based on the Identification Results Figure 6 shows a comparison between observed records and analytical results based on the identification results obtained for the relative acceleration. Both waves are calculated by the band pass filter of 2.0 - 3.3 Hz. Since both waves are in quite good agreement, it is considered that the time-variant identification for the predominant period T_1 and the damping constant h_1 of the ground is sufficient. Furthermore, it was confirmed the fact that a growth of the predominant period and an increase in the damping constant of the ground determined using strong seismic records are caused by nonlinear seismic behavior.

NONLINEAR SEISMIC RESPONSE ANALYSIS USING STRONG SEISMIC RECORDS

Analytical Model and Soil Properties The model used for nonlinear seismic response analysis is the same as the eigenvalue analysis model shown in Figure 1. Lumped-mass number 12 corresponds to the location of the accelerometer at the base layer (G.L. −28m), similarly lumped-mass number 1 corresponds to that at the ground surface (G.L. ±0m).

For each soil layer, the initial shear modulus G_0 was calculated using $G_0=\rho Vs^2$. Where ρ is density of soil and Vs is shear velocity of soil layer. Figure 7 shows the shear modulus ratio $G/G_0 \sim$ strain γ and the damping constant heq $\sim$ strain γ curves of the soil which are obtained by the dynamic tests, and by the theoretically defined modified Hardin-Drnevich model (modified H-D model) and the Ramberg-Osgood model (modified R-O model). The constants hmax of the modified R-O model were all determined by the authors' procedure (Ref. 6), the $G/G_0 \sim \gamma$ curve of the modified R-O model and the experimental curve were in good agreement under strain level of 10^{-3} at each layer of the soil. The reference strain γr which is the strain corresponding to $G/G_0=0.5$, was taken based on the $G/G_0 \sim \gamma$ curve obtained from the dynamic test. In the equivalent linear analysis, the coefficient used in estimating the effective strain from the maximum strain was decided as 0.65.

To clarify the characteristics of the nonlinear seismic response method, a linear seismic response analysis was also carried out. The linear analysis was performed by the modal analysis method, the damping constants were decided from the mean transfer function as follows; $h_1=0.07$, $h_2=0.06$, and $h_3 \sim h_{11}=0.05$.

Nonlinear Seismic Response Analysis Results In Figure 8, the results obtained by these analyses are compared with observed records at the ground surface. In all cases, the solid lines are the calculated waves and the dashed lines are the observed records. There are significant differences between the results of these models in the principal motion period of 3 - 8 seconds.

Although in linear seismic response analysis considerable phase lags are produced between observed and calculated waves, in nonlinear seismic response analysis including the equivalent linear analysis, the phases of the calculated

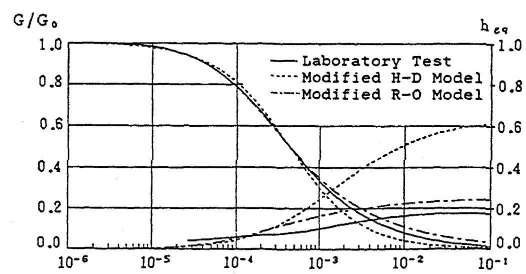


Fig. 7 $G/Go \sim \gamma$ and heq$\sim \gamma$ Relation

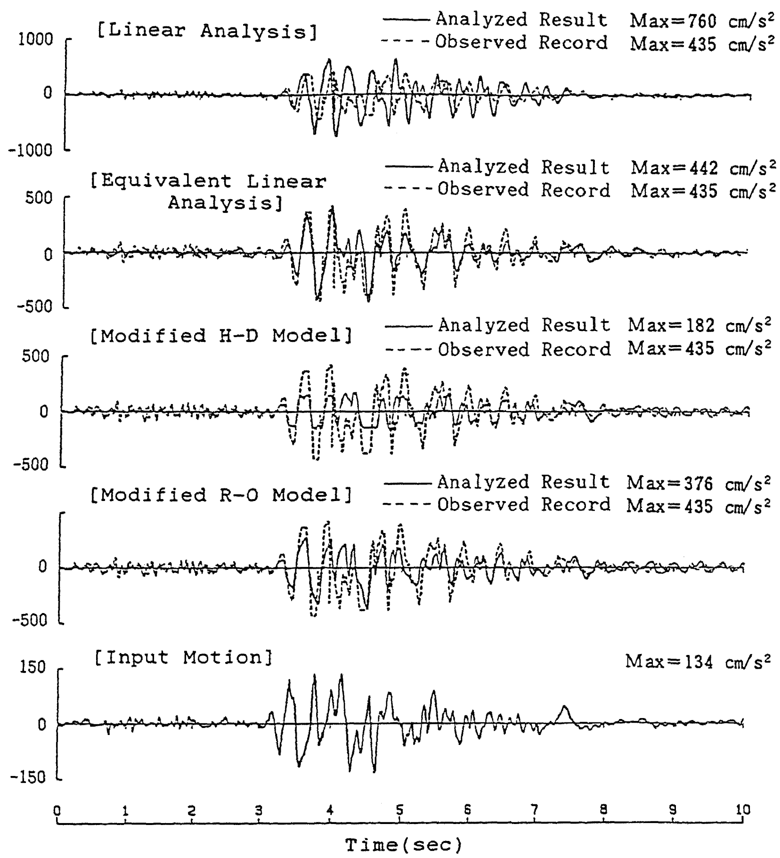


Fig. 8 Analyzed Results and Observed Records at Ground Surface

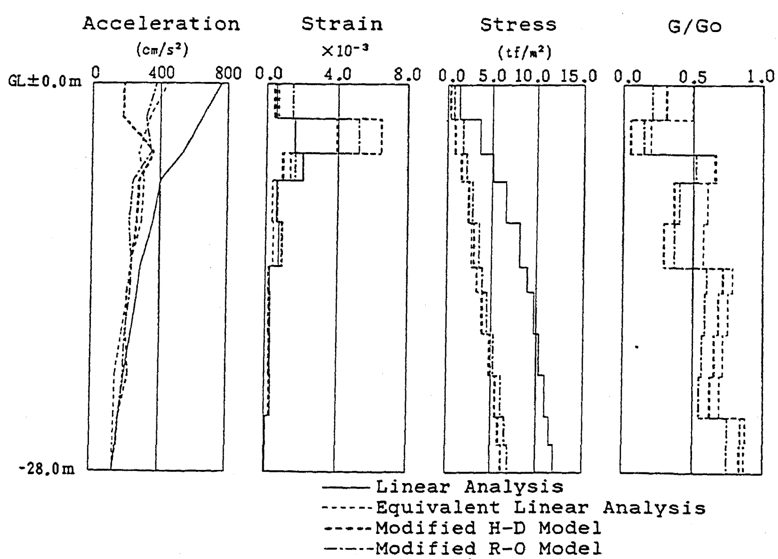


Fig. 9 Maximum Response Distribution

waves are in good agreement with those of the observed waves. However, the amplitude of the calculated waves for the modified H-D model is greatly suppressed. This is caused by the uniqueness of the modified H-D model in which the hysteresis curve is determined using Masing's law, that is the model which gives excessive damping values ($hmax=2/\pi=0.637$) for large strain. On the other hand, the calculated waves obtained by the equivalent linear analysis and the modified R-O model for which the value of hmax was determined by the authors' procedure, agree quite well with the observed records.

It can be seen from the maximum response distribution diagram shown in Figure 9 that the maximum strain is of the order of 10^{-3} and G/G_0 is less than 0.5 at the surface layer.

CONCLUSIONS

The conclusions obtained through the study are as follows:

(1) The results of the identification clearly indicate that the predominant period grows longer and the damping constant becomes greater than their linear values during the principal motion period caused by the nonlinear seismic behavior of the soil deposits.

(2) In the analytical results using the linear model, a phase lag appears between the analytical values and observed records.

(3) Because the modified Hardin-Drnevich model gives excessive damping values in the range of large strain, the amplitude of the calculated waves is greatly suppressed.

(4) Analytical results, obtained by the equivalent linear method and the step-by-step integration method using the modified Ramberg-Osgood model for which the value of hmax was determined by the authors' procedure, agree quite well with observed records.

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