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Seismic Response and Liquefaction Analysis by an Approximate Method

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SYNOPSIS Presented is a simplified procedure for performing the dynamic effective stress analysis. An equivalent linear method is applied to the procedure. It is assumed, in this method, that the variations of the shear modulus and damping factor due to strain level and effective stress are independent one onother. That is, firstly the total stress analysis is done in order to obtain the effective strain. Then the effective stress analysis is carried out and the moduli are varied due to the variation of the effective stress only. The accuracy of the result is checked by comparing it with that of nonlinear solution.

INTRODUCTION

There are some methods available for the response analysis of saturated sand ground of which parameters are variable with the development of pore water pressure. These methods, called effective stress analysis, have been mainly applied to the analysis of liquefaction problem. But because such current methods have to consume a lot of calculating time, its practical application seems to have been obstructed. A new simplified method is presented here for computing the dynamic response of the ground, considering the effect of pore water pressure buildup. This proposed method is an extension of the equivalent linear dynamic response analysis which has been used in the total stress analysis. In this way, by applying the simplified stress-strain relation to the effective stress analysis, it becomes possible to reduce plenty of calculating time of the effective stress analysis.

TREATMENT OF MATERIAL NONLINEARITY OF THE GROUND

The stress-strain relation of the soil during the cyclic loading condition should represent the mechanism of nonlinearity and energy dissipation. There have been several attempts to formulate such the soil deformation. Hardin-Drnevich model (1972), one of the fittest model with a few parameters, is used in the present analysis and shown by Eq.(1):

$$\tau - \tau_a = \frac{G_0(\gamma-\gamma_a)}{1+\left|\frac{\gamma-\gamma_a}{n\gamma_r}\right|} \qquad (1)$$

in which G_0 is shear modulus, γ_r is reference strain, τ_a, γ_a are the stress and strain values at the last stress reversal, and n is an index showing whether the curve is skeleton or branch (n=1: sleleton curve, n=2: branch curve). G_0 and γ_r are obtained from the following equations.

$$G_0\ (\mathrm{kN/m^2}) = \frac{3300(2.97-e)^2}{1+e}\sqrt{\sigma_m'} \qquad (2)$$

$$\gamma_r = \frac{\tau_f}{G_0} \qquad (3)$$

$$\tau_f = \frac{\sigma_v'}{2}\sqrt{(1+K_0)^2\sin^2\phi-(1-K_0)^2} \qquad (4)$$

in which e is void ratio, σ_m' is effective mean principal stress, K_0 is coefficient of earth pressure at rest, and ϕ is effective angle of shear resistance. Eq.(2) is given by Richart et al. (1970). In the process of pore water pressure buildup, Eq.(1) is varied by G_0 and γ_r which are in proportion to the square root of effective stress, respectively.

PROCEDURE OF PORE WATER PRESSURE CALCULATION

A procedure is described to calculate the excess pore water pressure due to seismic loading. The procedure is based on the results from stress-controlled tests shown by Seed et al.(1976). The relation between stress ratio τ/σ_v' and number of cycles N_ℓ required to cause liquefaction is rearranged by a line in log-log co-ordinates, as shown in Fig. 1. The equation of the line is:

$$N_\ell = \frac{0.038}{\left(\frac{\tau}{\sigma_v' D_r}\right)^{4.97}} \qquad (5)$$

in which D_r is relative density. Equation of the curve shown in Fig. 2 has been given by Seed et al.(1977) as the following form:

$$\frac{u}{\sigma_v'} = \frac{2}{\pi}\sin^{-1}\left(\frac{N}{N_\ell}\right)^{1/2\delta} \qquad (6)$$

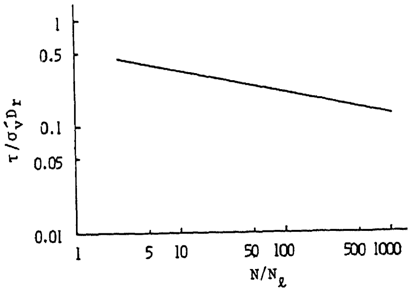


Fig. 1. Relation between Stress Ratio and Number of Cycles

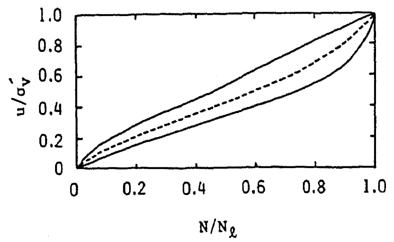


Fig. 2. Rate of Pore Water Pressure Buildup in Cyclic Simple Shear Tests (after Seed et al. (1977))

in which u is excess pore water pressure and δ is coefficient (in the case of average curve, δ =0.7).

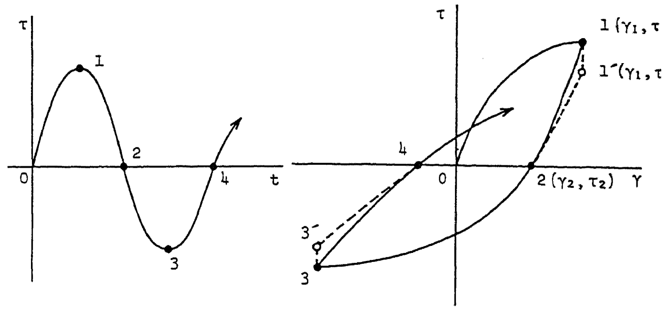
For example, when a soil element in the ground receives the stress-time history with irregular form, it is assumed that the pore water pressure rises in each time the stress crosses the time axis and that its value is calculated by the amplitude of a half stress cycle. Now, let τ_i be the maximum value of the ith stress half cycle. If the soil element receives a half-cycle of stress τ_i, it would be brought close to liquefaction at a rate of $1/2N_{\ell i}$, Where $N_{\ell i}$, obtained by Eq.(5), is the number of cycles required to cause liquefaction under a uniform stress amplitude τ_i. Through the action of shear stress from the first to the mth half cycle, it is considered to be liquefied at a rate of $\sum_{i=1}^{m}(1/2N_{\ell i})$. Substituting this value into N/N_ℓ in Eq.(6), the pore water pressure at the end of the mth half cycle is calculated. Then applying the obtained pore water pressure to the shear rigidity, the modulus is degraded. The pore water pressure grew in the ith half cycle has an influence on the modulus of the (i+1)th half cycle.

EFFECTIVE STRESS ANALYSIS

Nonlinear Analysis

The nonlinear effective stress analysis treated in this study is more simplified one than those of usual methods. The procedure of the method is as the follwing. The concept of stress-strain relation of the soil element is shown in Fig. 3. Stress-strain curve started from the origin is a skeleton curve and represented as:

$$\tau = \frac{G_0 \gamma}{1+\left|\frac{\gamma}{\gamma_r}\right|} \tag{7}$$



(a) (b)

Fig. 3. Schematic Shear Stress and Shear Strain Relation for Nonlinear Analysis, (a): Stress-Time Relation, (b): Stress-Strain Relation

Setting the stress and strain at the first peak point 1, τ_1,γ_1, respectively, the curve returned from this point becomes a branch curve, and is described by the next equation.

$$\tau - \tau_1 = \frac{G_0(\gamma - \gamma_1)}{1+\left|\frac{\gamma - \gamma_1}{2\gamma_r}\right|} \tag{8}$$

When this curve crosses the strain axis, at point 2, the first half cycle is assumed to be the end and the pore water pressure is calculated by the former method. Therefore, G_0 in Eq.(8) is modified to G_0', where $G_0' = G_0\sqrt{\sigma_v' / \sigma_{v0}'}$ (σ_{v0}' = initial effective vertical pressure). Because at the point 2 the slope of the line tangent to the curve changes discontinuously, such behaviour cannot be traced by Eq.(8). Therefore, the following equation is introduced.

$$\tau - \tau_1' = \frac{G_0'(\gamma - \gamma_1)}{1+\left|\frac{\gamma - \gamma_1}{2\gamma_r'}\right|} \tag{9}$$

where

$$\tau_1' = \tau_2 - \frac{G_0'(\gamma_2 - \gamma_1)}{1+\left|\frac{\gamma_2 - \gamma_1}{2\gamma_r'}\right|}$$

in which τ_2, γ_2 are stress and strain at point 2 respectively. Eq.(9) is considered to be the curve which returns from the point 1´. Then at the turning point 3, the slope of the line tangent to the turning curve equals to G_0', and at point 4, the pore water pressure is estimated again, and then the same procedure is conducted continuously.

Equivalent Linear Analysis

The present concept is shown in Fig. 4 which represents the relation between stress and strain of the soil element for the equivalent linear effective stress analysis. It is assumed, in this method, that the degradations of the shear moduli due to an increase of the magnitude of shear strain and reduction of the effective stress are independent one another. Firstly, the total stress analysis is done in order to obtain the shear moduli and damping factors corresponding to the specified strains, called effective strains.

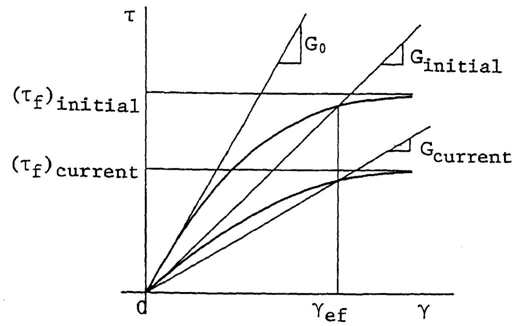


Fig. 4. Schematic Shear Stress and Shear Strain Relation for Equivalent Linear Analysis

These values become the initial constants of the effective stress analysis conducted subsequently. The shear modulus at this stage, $G_{initial}$, is the following form:

$$G_{initial} = \frac{G_0}{1 + \frac{\gamma_{ef}}{\gamma_r}} \qquad (10)$$

in which γ_{ef} is effective strain. Then the effective stress analysis is carried out, and it is assumed that the modification of moduli is caused by not the magnitude of the strain but the rise of pore water pressure only during this analysis. The current shear modulus modified because of pore water pressure buildup, $G_{current}$, is represented by:

$$G_{current} = \frac{G_0\sqrt{\frac{\sigma_v'}{\sigma_{v0}'}}}{1+\frac{\gamma_{ef}}{\gamma_r}\sqrt{\frac{\sigma_{v0}'}{\sigma_v'}}} \qquad (11)$$

In the equivalent linear method, the damping is considered as viscous damping instead of hysteretic damping. The damping matrix is formed as:

$$[C] = h\omega_1[M] + \frac{h}{\omega_1}[K] \qquad (12)$$

in which [M],[K],[C] are mass, stiffness, and damping matrices, respectively, h is damping factor and ω_1 is fundamental natural frequency. The damping factor is formulated by estimating the hysteretic damping of Hardin-Drnevich curve as the following:

$$h = \frac{4}{\pi}\left[\frac{G_0'}{G_{current}}\left\{\frac{\gamma_r'}{\gamma_{ef}} - \left(\frac{\gamma_r'}{\gamma_{ef}}\right)^2 ln\left(1+\frac{\gamma_{ef}}{\gamma_r'}\right)\right\}-\frac{1}{2}\right] \qquad (13)$$

in which $G_0' = G_0\sqrt{\sigma_v' / \sigma_{v0}'}$, $\gamma_r' = \gamma_r\sqrt{\sigma_v' / \sigma_{v0}'}$ and ln is natural logarithm. The form of Eq.(13), is introduced by Kokusho et al.(1978). In Eq.(13), G_0' and γ_r' deteriorate with lowering effective stress. In the present method, the modification of moduli should be done at every half cycle of stress only. Therefore, the calculating time for the effective stress analysis can be reduced excessively.

RESULTS OF NUMERICAL CALCULATION

One dimensional problem is analyzed, making use of the methods, nonlinear and equivalent linear methods mentioned above. The equivalent linear analysis is an approximate method of the nonlinear analysis. Now, it is investigated that to what degree the result of the equivalent linear effective stress analysis is possible to approximate the nonlinear result. This study considered a hypothetical soil site composed of saturated sand overlying a bedrock. The layer is 30 m thick, with the water table at the ground surface. The soil properties are D_r= 50%, G_s= 2.65, e= 0.80 and ϕ = 30°, respectively. The input bedrock motion used is the first 20 sec. of the N-S component of the 1940 El Centro Earthquake. This strong motion record is scaled to have a peak acceleration of 0.05g, 0.1g and 0.15g, respectively. For the response analysis, Newmark-β method is used. In the equivalent linear method, preceding the effctive stress analysis, the effective strain γ_{ef} is obtained by the total stress analysis. It is required in order to estimate the moduli corresponding to the strain level in the response process and given by the following equation:

$$\gamma_{ef} = \alpha\gamma_{max} \qquad (14)$$

in which α is the coefficient called strain reduction factor and γ_{max} is maximum shear strain. The value of α generally takes 0.65 in the total stress analysis. In this study the value of α would be decided by comparing the result of equivalent linear analysis with that of nonlinear one. The calculated results of pore water pressure distribution in the ground are shown in Fig. 5. It is shown the results of the equivalent linear analysis with various values of α and that of nonlinear analysis.

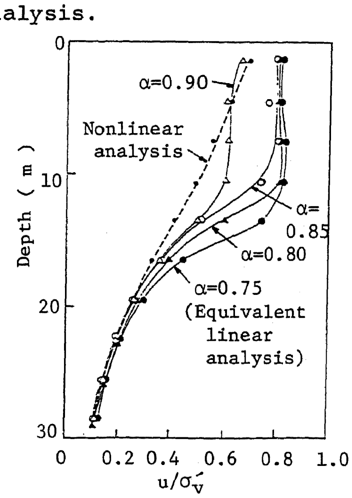


Fig. 5. Distribution of Pore Water Pressure Analyzed by Nonlinear and Equivalent Linear Methods (a_{max}= 0.1g)

It is recognized that in the case that α= 0.9, the curve of equivalent linear analysis seems to agree well with that of nonlinear one. Therefore, α= 0.9 is used in this study. In Fig. 6, 7, there are the results of the maximum shear stress and the maximum acceleration distributions in the ground. In both figures, the results are shown in the case of input maximum acceleration 0.05g, 0.1g and 0.15g, respectively. It is recognized by these figures that the results of equivalent linear analysis agree well with those of nonlinear analysis regardless of the magnitude of input acceleration. Fig. 8 shows the results of pore water pressure buildup in the top element of the

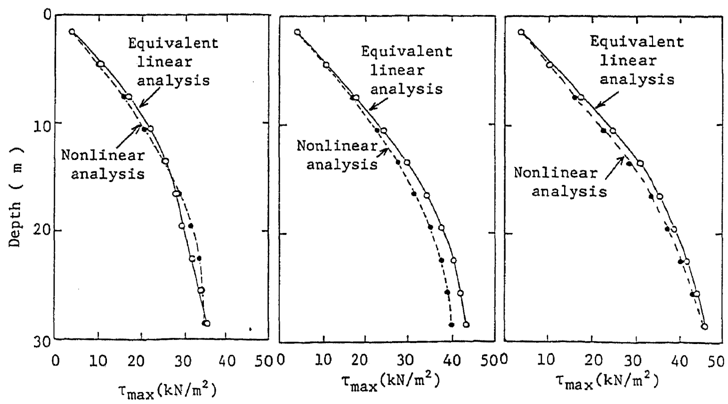


Fig. 6. Distributions of Maximum Response Shear Stress

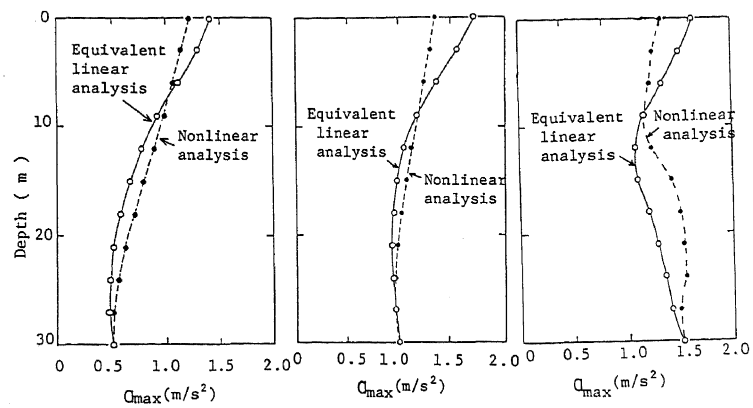


Fig. 7. Distributions of Maximum Response Acceleration

ground using the equivalent linear and nonlinear method, respectively. It is shown that the curve of former result rises rapidly at a certain time whereas the curve of latter rises gradually. The both results coincide at the end, but are different considerably on the midway.

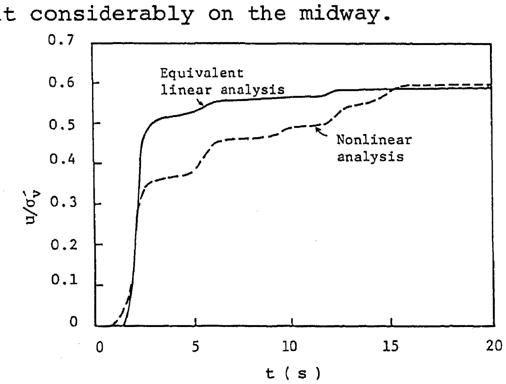


Fig. 8. Time History of Pore Water Pressure in the Top Element of the Ground

Fig. 9 shows a comparison of the surface acceleration computed by the both methods. There are definite points of similarity, but it seems that the equivalent linear approximation does not reproduce the short-period components of motion present in the nonlinear solution. The shear stress and strain relation of the top element are plotted in Fig. 10. In the equivalent linear solution, the slope of the line drawn through

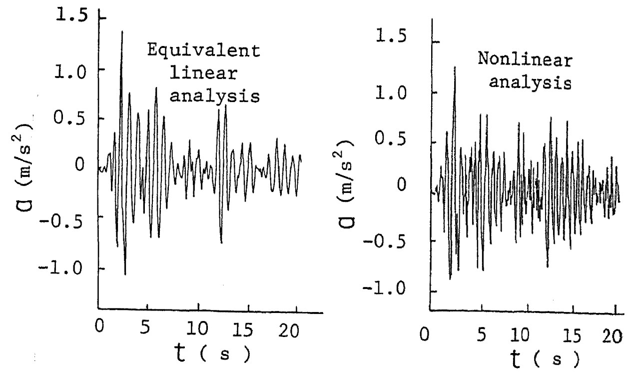


Fig. 9. Time History of Ground Surface Acceleration (a_{max} = 0.05g)

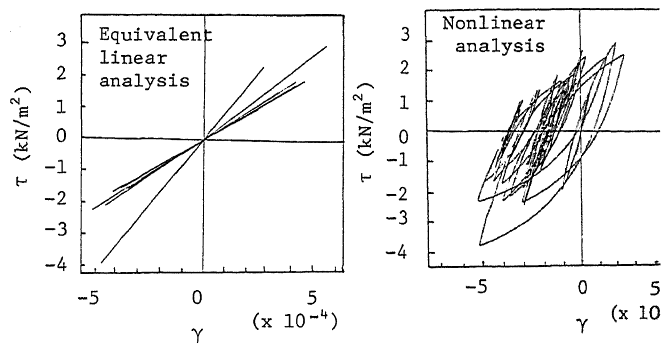


Fig. 10. Relation between Shear Stress and Shear Strain in the Top Element

the origin changes gradually as the pore water pressure buildup. In the nonlinear solution, the shape of loop changes with time and loop shifts to the direction of the residual strain.

CONCLUSIONS

As an approximate method for dynamic effective stress analysis, the equivalent linear effective stress method was proposed. Comparing its result with that of the nonlinear solution which is a nearly exact numerical solution, we saw much bet agreement. It was indicated that the equivalent linear approximation was adequate with respect t the maximum values of the responses namely, acce eration, shear stress and pore water pressure.

REFFERENCES

Hardin, B. O., & V. P. Drnevich, (1972) Shear Modulus and Damping in Soils: Design Equations and Curves, Proc. ASCE, Vol. 98, No.SM7, pp. 667-691.

Kokusho, G., & A. Sakurai, (1978) Modified Hardi Drnevich Model, The 33th Annual Convention of JSCE, Part 3, pp. 253-254. (in Japanese)

Richart, F. E., J. R. Hall, & R. D. Wood, (1970) Vibrations of Soil Foundations, Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Seed, H. B., P. P. Martin, & J. Lysmer, (1976) Pore-water Pressure Changes during Soil Liquefaction, Proc. ASCE, Vol. 102, No.GT4, pp. 32 346.

Seed, H. B., & J. R. Booker, (1977) Stabilizatic of Potentially Liquefiable Sand Deposits Usinc Gravel Drains, Proc. ASCE, Vol. 103, No.GT7, p 757-768.