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Full Length Article

A comparative study between simplified and nonlinear dynamic methods for estimating liquefaction potential

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ABSTRACT

This paper estimated the liquefaction potential of a saturated soil deposit subjected to a horizontal seismic excitation at its base using the total stress approach. A comparative analysis between the simplified and the nonlinear dynamic methods was used to verify to what extent the simplified method could be reliable. In order to generalise the reliability of the simplified method for any value of the maximum acceleration for the used earthquakes, a correction for the maximum acceleration less than 0.3*g* was proposed based on the comparison of safety factor values determined by the dynamic method illustrated by the equivalent linear model with lumped masses and the simplified method for a given profile of soil subjected to 38 earthquakes. The nonlinear behaviour of soil was represented by two hyperbolic models: Hardin and Drnevich, and Masing. To determine the cyclic resistance ratio (CRR), the cone penetration test (CPT) based method, the standard penetration test (SPT) based method, and the shear wave velocity based method were used. The safety factor was calculated as the ratio of CRR/CSR, where CSR represents the cyclic stress ratio. The results of the proposed correction have given smaller values of the safety factor compared to the nonlinear dynamic methods for the maximum acceleration less than 0.3*g*. In other words, by considering this correction, the most unfavourable case is always given by the modified simplified method.

1. Introduction

After the Alaska and Niigata earthquakes in Japan in 1964, Seed and Idriss (1971) developed a simplified procedure based on in situ tests to evaluate the liquefaction potential, which was defined by a safety factor calculated by the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). Thereafter, this procedure was modified and improved, in particular by Seed and Idriss (1982), Seed et al. (1983, 1985) and Youd et al. (2001).

A simplified procedure based on empirical formulations deduced from the study of a limited number of sites sometimes underestimates the risk of liquefaction unfortunately compared with exact methods (dynamic methods). For this reason, it is necessary to make a comparative analysis between dynamic and simplified methods in order to verify to what extent the simplified procedure (Seed and Idriss, 1971) is reliable. Therefore, the first goal of this study is to estimate the seismic response of soil by performing a dynamic analysis to deduce the cyclic stress ratio (i.e. CSRD), which is calculated as the ratio of the maximum shear stress deduced from the dynamic analysis to the effective vertical overburden stress. The second goal is to compute the CSR by the simplified procedure (Seed and Idriss, 1971). Finally, the study aims to evaluate the liquefaction potential in terms of safety factor (F_s) which is defined as the ratio of CRR to CSR (or CSRD). In the dynamic analysis, the soil behaviour is nonlinear with energy dissipation due to hysteresis. To calculate the maximum shear stress caused by the earthquake in each layer, we use the equivalent linear method with lumped masses associated with the hyperbolic models of Hardin and Drnevich (1972) and Masing (1926) to simulate this behaviour and to deduce the CSRD.

2. Equivalent linear analysis with lumped mass

The equivalent linear analysis with lumped mass is an iterative procedure used to estimate the nonlinear dynamic response of a

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soil deposit subjected to an accelerogram at rocks. Assuming that the rocks and interfaces between different layers of soil deposits are essentially horizontal, we can consider each layer to be linear elastic and then develop a model with lumped mass in order to analyse the nonlinear dynamic response of the soil deposit during an earthquake. Krylov and Bogoliubov (1943) and Bogoliubov and Mitropolskii (1961) proposed to use an equivalent linear spring constant and an equivalent damping ratio for a single-degree-of-freedom system with nonlinear characteristics.

Idriss and Seed (1968) suggested an equivalent linear scheme where the shear modulus and damping were modelled by a linear spring and a dashpot, respectively. The parameters of the spring and the dashpot were computed based on the secant shear modulus and damping ratio for a given level of shear strain. The shear modulus and damping ratio values were iteratively calculated based on the computed strain. For earthquake motion, Idriss and Seed (1968) proposed that the properties must be computed for a strain equal to 2/3 of the maximum strain level in a given layer. Currently, an expression suggested by Idriss and Sun (1992) that relates the ratio of the effective shear strain to the maximum shear strain (R_γ) with the earthquake magnitude (M) is mostly used:

$$R_\gamma = \frac{M-1}{10} \tag{1}$$

This analysis can be conducted in the following steps:

(1) Step 1: Choice of soil layer number required for analysis

The layer number required for analysing the response of soil deposit is chosen as follows:

(i) The deposit is divided into several layers with uniform material proprieties, and the period $(T_1)_i$ is computed by the following equation:

$$(T_1)_i = \frac{h_i}{\sqrt{gG_{\max(i)}/\gamma_i}} \tag{2}$$

where h_i, $G_{\max(i)}$ and γ_i are the thickness, maximum shear modulus and unit weight of the i-th layer, respectively; and g is the acceleration of gravity.

(ii) Each layer is then divided into N_i sub-layers. The value of N_i is obtained graphically (Idriss and Seed, 1968) according to the computed value of $(T_1)_i$. The entire deposit is divided into N sub-layers where $N = \sum N_i$. The model with lumped mass is shown in Fig. 1.

(2) Step 2: Calculation of the dynamic response

When the soil deposit is subjected to a horizontal seismic excitation at its base, the equation of motion of all soil layers can be represented in matrix form considering the dynamic equilibrium for each layer as follows:

$$\boldsymbol{m}\ddot{\boldsymbol{x}} + \boldsymbol{c}\dot{\boldsymbol{x}} + \boldsymbol{k}\boldsymbol{x} = -\boldsymbol{m}\ddot{\boldsymbol{x}}_{\mathrm{r}} \tag{3}$$

where $\boldsymbol{m}$ is the mass matrix; $\boldsymbol{c}$ is the dumping matrix; $\boldsymbol{k}$ is the stiffness matrix; $\boldsymbol{x}$, $\dot{\boldsymbol{x}}$ and $\ddot{\boldsymbol{x}}$ are the displacement, velocity and acceleration vectors, respectively; and $\ddot{\boldsymbol{x}}_{\mathrm{r}}$ is the acceleration of rock.

The mass matrix is diagonal ($m_{ij} = 0$ if $i \neq j$) and can be obtained by lumping half the mass of each of two consecutive layers at their common boundary, then we have

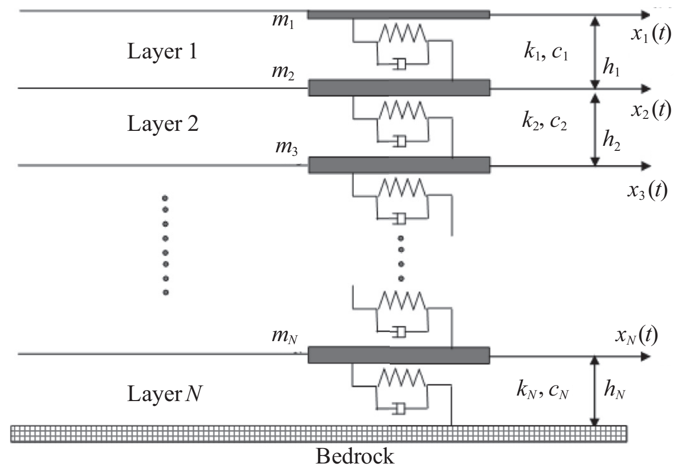

Fig. 1. Model with lumped mass.

$$m_1 = \frac{\rho_1 h_1}{2},\ m_i = \frac{\rho_{i-1}h_{i-1} + \rho_i h_i}{2} \quad (i = 2, 3, \cdots, N) \tag{4}$$

where ρ_i and m_i are the density and lumped mass of the layer i, respectively.

The stiffness matrix is constant for a linear-elastic material in each iteration and is defined as

$$\boldsymbol{k} = \begin{bmatrix} k_1 & -k_1 & 0 & \cdots & 0 \\ -k_1 & k_1+k_2 & -k_2 & \cdots & 0 \\ 0 & -k_2 & k_2+k_3 & \cdots & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & \cdots & -k_{N-1} \\ 0 & 0 & 0 & -k_{N-1} & k_{N-1}+k_N \end{bmatrix}_{N\times N} \tag{5}$$

where $k_i = G_i/h_i$, and G_i is the shear modulus.

Viscous damping is added in the form of damping matrix $\boldsymbol{c}$. In the original damping formulation proposed by Rayleigh and Lindsay (1945), the matrix $\boldsymbol{c}$ is assumed to be proportional to the mass and stiffness matrices:

$$\boldsymbol{c} = \alpha\boldsymbol{m} + \beta\boldsymbol{k} \tag{6}$$

where α (in s^{-1}) and β (in s) are the real scalars.

The damping ratio for the n-th mode of such a system is

$$\xi_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \tag{7}$$

where ω_n is the natural frequency.

The coefficients α and β can be computed using two significant natural modes i and j. The system of differential equations is solved numerically at each time step by using the Newmark-Beta algorithm (Newmark, 1959) which connects the accelerations, velocities, and displacements at the moments $t + \Delta t$ and t.

(3) Step 3: Calculation of the maximum shear strain

For each layer i, the unit maximum shear strain, $\gamma_{\max(i)}$, is calculated according to the maximum interlayers displacements:

$$\gamma_{\max(i)} = \frac{|x_i(t) - x_{i+1}(t)|_{\max}}{h_i} \tag{8}$$