Soil Dynamics and Earthquake Engineering 75 (2015) 130–146

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Liquefaction potential evaluations by energy-based method and stress-based method for various ground motions

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ARTICLE INFO

Article history:
Received 30 June 2014
Received in revised form
9 March 2015
Accepted 2 April 2015
Available online 28 April 2015

Keywords:
Liquefaction potential evaluation
Energy-based method
Case history
Stress-based method
Seismic wave energy

ABSTRACT

An energy-based liquefaction potential evaluation method (EBM) previously developed was applied to a uniform sand model shaken by seismic motions recorded at different sites during different magnitude earthquakes. It was also applied to actual liquefaction case histories in Urayasu city during the 2011 M9.0 Tohoku earthquake and in Tanno-cho during the 2003 M8.0 Tokachi-oki earthquake. In all these evaluations, the results were compared with those by the currently used stress-based method (SBM) under exactly the same seismic and geotechnical conditions. It was found that EBM yields similar results with SBM for several ground motions of recent earthquakes but has easier applicability without considering associated parameters. In Urayasu city, the two methods yielded nearly consistent results by using an appropriate coefficient in SBM for the M9.0 earthquake, though both overestimated the actual liquefaction performance, probably because effects of plasticity and aging on in situ liquefaction strength were not taken into account. In Tanno-cho, EBM could evaluate actual liquefaction performance due to a small-acceleration motion during a far-field large magnitude earthquake while SBM could not.

1. Introduction

The energy-based liquefaction potential evaluation method (EBM) was first proposed by Davis and Berrill [1] based on an assumption that the pore-pressure buildup is directly related to the amount of seismic energy dissipated in a unit volume of soil, in which seismic energy arriving at a site was estimated from the earthquake magnitude and the distance from the center of energy release. In a different approach for defining the seismic energy, Kayen and Mitchell [2] used Arias Intensity [3] as a measure of earthquake-shaking severity for assessment of liquefaction potential. On the other hand, undrained cyclic loading tests focusing on the dissipated energy in soil specimens were conducted by Towhata and Ishihara [4], Yanagisawa and Sugano [5], Figueroa et al. [6] and Jafarian et al. [7], and a unique relationship was found between the dissipated energy and excess pore-pressure independent of cyclic shear stress history. Nevertheless, the application of EBM in engineering practice has been very limited so far in contrast to the stress-based method (SBM). It is probably because concrete and delineated procedures for EBM and its application results were not provided in relation to corresponding SBM results, yet.

In a previous paper by Kokusho [8], an energy-based liquefaction evaluation method (EBM) was proposed together with experimental data and delineated evaluation steps. First, a data set of undrained cyclic triaxial tests with parametrically changing relative density and fines content was interpreted in scope of energy, demonstrating that the strain amplitude or pore-pressure buildup during cyclic loading is uniquely correlated not only to energy dissipated in soil specimens but also to strain energy imposed upon the specimen from outside. Hence, EBM was developed in which liquefaction potential can be evaluated by comparing an energy capacity for liquefaction in a sand layer with upcoming seismic energy E_u without regard to the differences in seismic ground motions. Then, a liquefaction potential of a uniform sand deposit of 10 m thick was evaluated by EBM to compare with SBM. An input motion recorded during the Tohoku earthquake (seismic magnitude $M=9.0$) was given at the ground surface in a real time scale. It was found that all the saturated sand deposit tends to liquefy both in EBM and SBM almost consistently if the stress reduction coefficient $r_n=0.80$ corresponding to the M9.0 earthquake is used in SBM, while no such consideration is needed in EBM. If the same acceleration motion was given to the model in a compressed half-time scale, however, the evaluation by EBM was very different from that by SBM because the upward energy E_u reduces to about 1/8, whereas the effect on the induced stress is minor. Furthermore, a significant qualitative difference was recognized that a liquefaction potential in a uniform sand deposit is evaluated higher in a shallower portion than in a deeper portion in EBM but vice versa in SBM for the M9.0 earthquake motion.

Thus, a basic procedure of EBM was already developed and applied to simplified soil models to compare with SBM [8]. As

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http://dx.doi.org/10.1016/j.soildyn.2015.04.002
0267-7261/

pointed out in the previous paper however, further studies on its applicability to case histories with detailed soil investigations and seismic motions are needed to show its reliability in actual site conditions.

In order to study the applicability of EBM further in more realistic setting in this paper, the uniform sand deposit is shaken at first by various ground motions recorded at different sites during different earthquakes having almost the same peak ground acceleration (*PGA*), and the liquefaction susceptibility evaluated by EBM is compared with that by SBM. Then, a case history in Urayasu, Japan, where extensive liquefaction occurred during the M9.0 Tohoku earthquake and another case history in Hokkaido, where a M8.0 earthquake triggered liquefaction-induced flow slide at a site 230 km from the hypocenter and the *PGA* was only about 0.05 g, are investigated by the two evaluation methods to examine differences in their applicability to actual performance in actual site conditions.

2. Evaluation procedures by SBM and EBM

Procedures of SBM and EBM employed in this paper to evaluate liquefaction potentials in a hypothetical uniform sand as well as actual soil profiles in case studies are briefly reviewed here based on the previous publication by Kokusho [8].

2.1. SBM-procedure

The basic idea of the SBM evaluation procedures are almost the same internationally in comparing cyclic resistance ratios (CRR) of in situ sands with seismically induced cyclic stress ratios (CSR) as explained by Idriss and Boulanger [9]. In the SBM employed in Japan, liquefaction is to occur if a factor F_L expressed in Eq. (1) is lower than unity.

$$F_L = R/L \tag{1}$$

Here, in situ cyclic resistance ratio (CRR) R may be obtained from R_{L20} corresponding to a stress ratio for the onset of liquefaction (the double amplitude axial strain $\varepsilon_{DA} = 5\%$ and the number of cycles N_c=20) of isotropically consolidated triaxial test specimens, using earth-pressure coefficient at rest K_0.

$$R = R_{L20}(1+2K_0)/3 \tag{2}$$

The value R_{L20}, which was originally investigated on intact soils sampled by an in situ soil freezing technique, is actually determined from SPT N_1-value in engineering practice using the empirical formula Eqs. (3)–(5) [10].

$$R_{L20} = \begin{cases} 0.0882\sqrt{N_a/1.7} & : \ N_a \geq 14 \\ 0.0882\sqrt{N_a/1.7 + 1.6\times10^{-6}(N_a-14)^{4.5}} & : \ 14 \leq N_a \end{cases} \tag{3}$$

where N_a is given by the next equation:

$$N_a = c_1N_1 + c_2 \tag{4}$$

N_1 is SPT blow-counts normalized by effective overburden stress σ_v' as:

$$N_1 = 1.7N/(\sigma_v'/p_0 + 0.7) \tag{5}$$

here p_0=98 kPa, and c_1, c_2 are parameters depending on fines content F_c as follows:

$$c_1 = \begin{cases} 1.0 & (0\% \leq F_c < 10\%) \\ (F_c+40)/50 & (10\% \leq F_c < 60\%) \\ F_c/20-1 & (60\% \leq F_c) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \leq F_c < 10\%) \\ (F_c-10)/18 & (10\% \leq F_c) \end{cases} \tag{6}$$

Seismically induced cyclic stress ratio (CSR) L in Eq. (1) is evaluated as:

$$L = r_nL_{max} = r_n\tau_{max}/\sigma_v' = \tau_0/\sigma_v' \tag{7}$$

where $L_{max} = \tau_{max}/\sigma_v'$ is the maximum seismic stress ratio, τ_{max} =maximum seismic shear stress and σ_v'=effective overburden stress. Here, τ_{max} is calculated by one dimensional equivalent linear response analysis [11]. The equivalent cyclic stress amplitude τ_0 is determined by modifying the maximum stress τ_{max} as $\tau_0 = r_n\tau_{max}$, where r_n represents a stress reduction coefficient to replace an irregular motions with an equivalent sinusoidal motion of a given number of cycles. Here, the coefficient r_n is given in an empirical formula by Tokimatsu and Yoshimi [12] using an earthquake magnitude M.

$$r_n = 0.1(M-1) = 0.65/MSF \tag{8}$$

The r_n-value has a correlation with a magnitude scaling factor (*MSF*) used in North American practice [9] as indicated in Eq. (8). M=7.5 in Eq. (8) gives r_n=0.65, which is the case normally used as a default value in Japan.

2.2. EBM-procedures

In EBM, it is first necessary to determine dissipated energy for liquefaction in soils. According to a series of cyclic loading undrained triaxial tests described in the previous publication by Kokusho [8], normalized dissipated energy $\Delta W/\sigma_c'$ was correlated almost uniquely with R_{L20} for various soils with different relative densities and fines contents;

$$\Delta W/\sigma_c' = 0.032 - 0.48R_{L20} + 2.40R_{L20}^2 \tag{9}$$

where σ_c' is the effective confining stress. As explained in the previous paper [8], it is assumed that this lab-based relationship also holds in in situ soils with natural soil fabrics, though the absolute values of $\Delta W/\sigma_c'$ and R_{L20} may vary individually reflecting field conditions. Then, Eq. (9) can readily be converted to a relationship between the normalized dissipated energy $\Delta W/\sigma_c'$ and the normalized N-value, N_1, considering the effect of F_c by incorporating Eqs. (3)–(6) used in SBM practically.

Another experimental finding obtained in the previous research [8] was that a certain amount of energy ΔW is dissipated internally corresponding to the elastic strain energy W given from outside in the cyclic loading laboratory test, and the normalized strain energy W/σ_c' given from outside to a soil specimen (normalized by σ_c') is uniquely correlated with the normalized energy dissipated inside $\Delta W/\sigma_c'$ with no regard to relative density and fines content as:

$$W/\sigma_c' = 5.4\times10^{1.25\times\log(\Delta W/\sigma_c')} \tag{10}$$

This indicates that not ΔW directly but W calculated by Eq. (10) should be compared with seismic wave energy given from outside in the field.

The seismic wave energy in the field is imparted to a given soil layer as the wave propagates not only upward but also downward. It is clear that the downward energy constitutes a part of the original upward energy. This indicates that the upward energy is the maximum possible seismic wave energy to be compared, though the downward energy may somewhat decrease during its transmission due to insignificant energy dissipation in non-liquefiable upper layers in some cases. A simplified liquefaction evaluation, both conventional stress-based and energy based, cannot consider all possible details of site-specific soil conditions and liquefaction process, which inevitably necessitates some sort of approximations. In a simplified EBM procedure considering various site conditions, it seems possible to use only the upward energy to compare with the strain energy W for liquefaction calculated in Eq. (10). It is also important to note that the energy is imparted to liquefiable layers not at once but gradually and concurrently during the process of liquefaction.

Some methodologies on how to evaluate the upward energy in EBM were already discussed in the previous paper [8], though energy evaluations in design motions are not yet included in current