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Soil liquefaction potential evaluation with use of the spectrum at depth

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ABSTRACT: A method to evaluate soil liquefaction susceptibility is presented. In this method the response spectrum of the ground was introduced to take into account the significant influence of soft surface layers on the maximum acceleration at the ground surface. Resonant period of a site is estimated from the configuration of horizontal surface layers, which is expressed by shear wave velocities evaluated from the N-value distribution of the standard penetration test. The maximum acceleration at the surface was calculated by multiplying the maximum input acceleration at the bed rock by the dynamic amplification factor proposed by Kanai. A conventional method of liquefaction analysis was adopted to determine the liquefaction susceptibility of sand layers of different N value distribution. Critical N values of SPT and the maximum epicentral distance of liquefaction were derived from the theoretical considerations.

1 INTRODUCTION

Soil liquefaction is well known as one of the major causes of seismic damage to structures and foundations, it is caused by large deformation or cracks in the foundation ground due to subsidence and displacement in the liquefied layers. The evaluation of liquefaction potential at a site is essential to take measures for the prevention of seismic disasters and reduction of damage. Liquefaction is likely to occur when a loose sand is in saturated conditions and shaken by a strong earthquake or shocks which result in a build up of hydrostatic pore pressure and a decrease of the effective stress. The character of ground shaking on the ground surface generated by an earthquake is complexically influenced by the magnitude, the epicentral distance and the soil conditions. Some frequencies of strong shaking may be amplified considerably by low-velocity surface layers and the overall spectral level of ground motion may increase as the seismic velocity of near surface materials decreases, Kanai (1957). Herein, we express the degree of magnification quantitatively by using the spectrum at depth, Kanai (1958), after having selected the appropriate acceleration attenuation law for the site. The basic scheme of this method is similar to that presented by T.Iwasaki et al. (1982) but it differs from them by the point that the maximum acceleration at the ground surface is estimated by introducing the dynamic response of the surface layers using Kanai's method. The method was used to analyze various cases of the sites where liquefaction has taken place and non-liquefied sites were also treated to evaluate the safety factors during past earthquakes in Japan, then the results were compared to those of Seed (1979) and Iwasaki (1982).

2 THEORY OF LIQUEFACTION POTENTIAL EVALUATION

2.1 Spectrum at depth

The character of ground shaking is influenced by the magnitude, the epicentral distance and usually the complex surface geologic conditions. Seismic waves are usually amplified by alluvial surface layers due to the multireflection phenomenon. The amplification is, herein, represented by the dynamic amplification factor (DAF). For the purpose to compute the amplification ratio, Kanai has proposed the following expression (1958):

$$G(T_g, T) = 1 + \frac{1}{\sqrt{\left[\frac{1+\mu}{1-\mu}\left\{1-\left(\frac{T}{T_g}\right)^2\right\}\right]^2 + \left(\frac{0.3}{\sqrt{T_g}}\cdot\frac{T}{T_g}\right)^2}} \qquad (1)$$

where $G(T_g, T)$ is the dynamic amplification factor (DAF), T the period of vibration of seismic wave, T_g the predominant period of surface layer, ρ_1 the density of surface layer, ρ_2 the density of base layer, C_1 the Velocity of seismic wave in surface layer, C_2 the velocity of seismic wave in base ground and

$$\mu = \frac{\rho_1 C_1}{\rho_2 C_2}$$

If we can develop an approximate spectrum of seismic waves at depth in bedrock, then the response spectrum at the ground surface can be easily obtained from the above relation. To analyze the seismic soil liquefaction hazard we should consider the worst condition which is, in this study, the case of resonance $T=T_g$. For the case of resonance, Kanai's method gives the following expression for the dynamic amplification factor (D.A.F).

$$D.A.F = 1 + \frac{\sqrt{T_g}}{0.3} \qquad (2)$$

2.2 Determination of the predominant period T_g

The predominant period T_g can be determined by the structure of the surface layer. By applying the theory of multi-reflection of waves in surface layers, we can obtain the predominant period of an alluvial layer. From the multireflection theory of waves the predominant period T_g is obtained by the following:

$$T = 4\sum \frac{H_i}{V_{si}} \qquad (3)$$

where H is the total thickness of the surface layer, H_i the thickness of the layer i and V_{si} is the velocity of seismic wave at layer i.

To compute V_{si}, a relation between the velocity of the seismic shear waves and average N-value of the Standard Penetration Test within a layer was used. The relation proposed by T. Imai and M. Yoshiziwa (1975) is expressed as follows:

$$V_{si} = 97. \cdot N_i^{0.314} \qquad (4)$$

where V_{si} is the velocity of seismic wave at layer i and N_i the SPT average N-value at layer i.

Then, we may write T_g as:

$$T_g = 0.041 \sum H_i \cdot N_i^{-0.314} \qquad (5)$$

2.3 Attenuation law

Various empirical relationships are available in the literature to describe the relation between the magnitude of the event, the distance from the source and the ground motion parameter. After having tested some of these formulas, we have selected the following relation which is used in the computation process of the proposed procedure of liquefaction potential evaluation. The selected attenuation relation is represented by:

$$\log_{10} \frac{(a_g)_{\max}}{640} = \frac{(\Delta + 40)}{100}(-7.604 + 1.7244 \cdot M - 0.1036 \cdot M^2) \qquad (6)$$

where $(a_g)_{\max}$ is the maximum acceleration at bedrock (gals), M the Richter Magnitude and Δ is the epicentral distance (km).

2.4 Outline of the procedure

In this procedure, the ability of a soil element at any depth to resist liquefaction is expressed by the liquefaction resistance factor, F_L, defined as follows:

$$F_L = \frac{R}{L} \qquad (7)$$

where R is the in-situ resistance (undrained cyclic shear strength) of a soil element to dynamic load which can be evaluated by:

$$R = 0.082 \sqrt{\frac{N}{\sigma_v' + 0.7}} + \Delta R \qquad (8)$$

where N is the number of blows of the SPT, σ_v' the effective overburden pressure (in kgf/cm^2) at a depth z and ΔR is the correction by the particule size of sand.

L is the dynamic load factor (or maximum shear stress of the soil element) generated in the soil element by a seismic motion. According to Seed and Idriss (1971) the value of L is estimated by:

$$L = \frac{\tau_{\max}}{\sigma_v'} = \frac{(a_s)_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} r_d \qquad (9)$$

in which $\tau_{\max}$ represents the maximum shear stress (kgf/cm^2) in the soil element, σ_v' and σ_v respectively the effective overburden pressure and total overburden pressure of the layer under consideration, $(a_s)_{\max}$ the maximum acceleration at the ground surface (gals), g the acceleration of gravity and r_d denotes the dynamic stress reduction factor which is a function of the soil depth.

Seed and Idriss (1971) have proposed the following relationship between r_d and depth.

$$r_d = 1. - 0.015 \cdot Z \qquad (10)$$

where z is the depth in meters.

In this paper, we are introducing the dynamic response of the soil which is expressed by the degree of amplification of the acceleration in the bedrock at the ground surface. This degree of amplification is represented by the Dynamic Amplification Factor (DAF), as mentioned earlier. Thus, the estimation of the newly proposed dynamic load factor, L, can be made by the following expression:

$$L = \frac{(a_g)_{\max} \cdot DAF}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot (1 - 0.015 \cdot Z) \qquad (11)$$

where DAF denotes the dynamic amplification factor of the site and $(a_g)_{\max}$ is the maximum acceleration in the bedrock (gals) determined by attenuation laws or expert estimation. Therefore the key factor to evaluate the seismic soil liquefaction potential (liquefaction resistance factor), which is denoted by F_L, can be written as follows:

$$F_L = \frac{g}{(a_g)_{\max} \cdot DAF}\left[0.082\sqrt{\frac{N}{\sigma_v' + 0.7}} + \Delta R\right]\frac{\sigma_v'}{\sigma_v} \cdot \frac{1}{(1 - 0.015 \cdot Z)} \qquad (12)$$

3 Case studies

Severe liquefaction at many sites was induced by the past strong earthquakes, and many structures were damaged due to soil liquefaction. Preliminary studies are executed on numerous sites where soil liquefaction has occurred and non-liquefied sites are also studied. The calculation results indicate that in general the values of the liquefaction resistance factor F_L are less than unity at liquefied layers and larger than unity for non-liquified layers. It should be mentioned that about 11% of F_L-values are larger than unity at liquified layers, and about 13% of F_L-values are less than unity at non-liquified layers. From this study we can conclude that this procedure is valid and effective and should be adopted for liquefaction potential evaluation. Since the proposed procedure has proved its validity, a theoretical study has been conducted by use of the method with different distributions of SPT N-values and different seismic loadings. Different soil conditions are given by the N value distribution functions designated by the depth z, such as N = C, N = Kz, N = Kz+C, etc..where K and C are constants. In order to evaluate the liquefaction potential of the site, the geotechnical parameters are used as the input of the soil characteristics such as soil profile, density, water table level, surface layer thickness and N values of SPT. The seismic parameters used are the site location, the magnitude, the epicentral distance and the regional attenuation constants.

The output will be the predominant period T_g of the ground, the dynamic amplification factor (DAF), the acceleration at the bedrock $(a_g)_{max}$ and at the ground surface $(a_s)_{max}$, the resistance factor F_L and also the cyclic shear stress (liquefaction strength R) τ_l/σ_v'. The results of the analysis are presented and discussed in the next chapter.

4 Results

Figure-1 shows a chart obtained by the proposed method to evaluate the seismic soil liquefaction potential in terms of critical SPT N-values versus depth for Richter magnitude 6.5 and for the epicentral distance of 100 km. One can see from these charts that there are three particular epicentral windows:

1. The first window is in which liquefaction is very likely to take place (LL);
2. The second window is in which liquefaction may or may not take place (ML);
3. The third window is in which liquefaction is very unlikely to occur (NL).

This chart was developed for the case where the water table stays at ground surface (h_w). Similar ones for other different magnitude and depths of water table or any other conditions may easily be developed using the peocedure described previously. For engineering purposes it should be noted that the estimation of seismic soil liquefaction is given by simple reading on the charts for the first and third windows. But for the second window where the soil liquefaction may or may not take place depending on the earthquake magnitude and local soil conditions. It must be recommended to conduct a precise investigation of the soil liquefaction potential for further evaluations. Figure-2 indicates the relationship between the critical corrected N values and the cyclic stress ratio causing liquefaction or liquefaction strength. In this chart corrected SPT N-values N_c is defined as the measured penetration resistance corrected by an effective overburden pressure of 1 kgf/cm^2, since it is well known that the SPT N-values are very much influenced by the effective confining pressure in the soil as well as the density which may reflect the undrained strength of the soil. Based on the results of Tokimatsu and Yoshimi (1983), N_c, which is adopted hereafter, may be expressed by the following relation:

$$N_c = C_N \cdot N \tag{13}$$

where

$$C_N = \frac{1.7}{\sigma_v' + 0.7} \tag{14}$$

σ_v' the effective overburden pressure in kgf/cm^2 and N the penetration resistance.

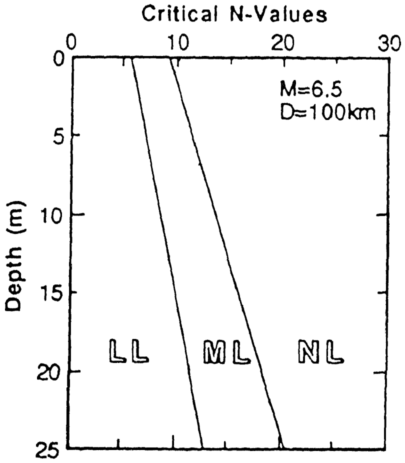

Figure-1: Critical N Values to Cause Liquefaction

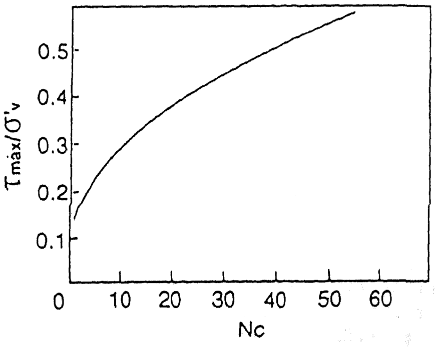

Figure-2: Relation between Cyclic Stress Ratio and Corrected N Value

From historical seismicity, it can be said that the most probable ground surface acceleration may range from 0.16 to 0.20g, which is usually used in designing structures as horizontal earthquake force. For the full range of the ground surface acceleration mentioned above, the possibility of liquefaction was checked and critical N values for the different soil conditions are obtained. Figure-3 indicates the critical N value obtained from the study. Figure-4 illustrates a relationship between the earthquake Richter magnitude M and the maximum epicentral distance R where soil liquefaction may occur. It shows the upper bound of the maximum epicentral distance at site where seismic induced soil liquefaction may take place for any given earthquake of Richter magnitude M. From our study described earlier, the following relationship between the maximum epicentral distance D(km) and Richter magnitude is proposed:

$$\log D = 0.31 \cdot M + 0.02 \qquad (15)$$

where D is the epicentral distance in km and M the Richter magnitude.
This relationship will be compared with field data and other proposed relations in the next chapter.

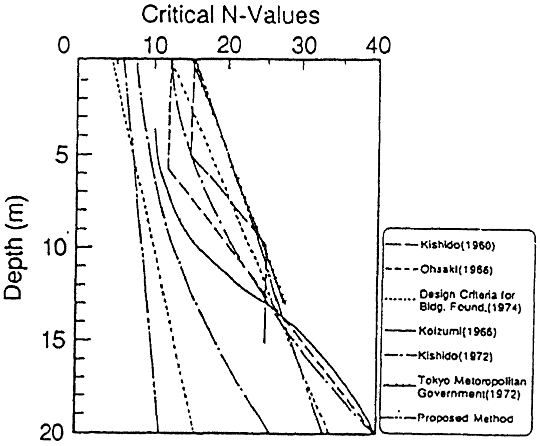

Figure-3: Comparison of Critical N Values

5 COMPARISON AND DISCUSSION OF THE RESULTS

Herein we are comparing the critical SPT N-values versus depth to cause liquefaction with other charts proposed by Kishida, Koizumi, Ohsaki, Tokyo Metropolitan Government, and the one adopted for design criteria for building foundations in Japanese code as seen in Figure-3. It is easily recognized that the proposed line is very close to the others and presents a good agreement with the chart used in Design Criteria for Building Foundations (Japanese Code). It is also seen in Figure-3 that critical SPT N-values are similar to among the other various proposals. A large amount of data of liquefaction sites is taken from Kuribayashi and Tatsuoka (1975). The data compiled in Figure-4 represents the maximum epicentral distance to a site at which soil liquefaction was observed after certain earthquakes. These data when published were representing only Japanese earthquakes. After publication of Kuribayashi (1975), Youd (1977) added some case histories from non-Japanese earthquakes. These data are included in Figure-5. In the frame of the proposed method described earlier we assumed the relation between the maximum epicentral distance D and Richter magnitude M as:

$$\log D = 0.31 \cdot M + 0.02$$

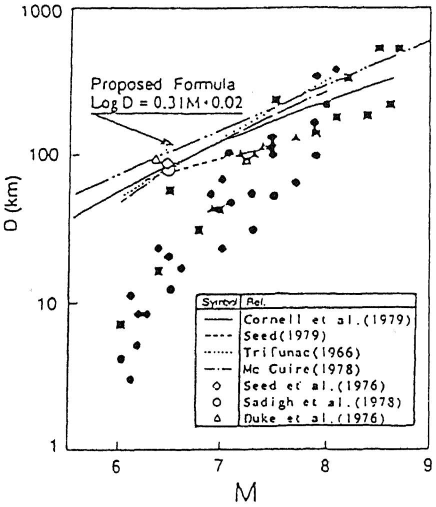

Figure-4: The Maximum Distance to A Site of Liquefaction

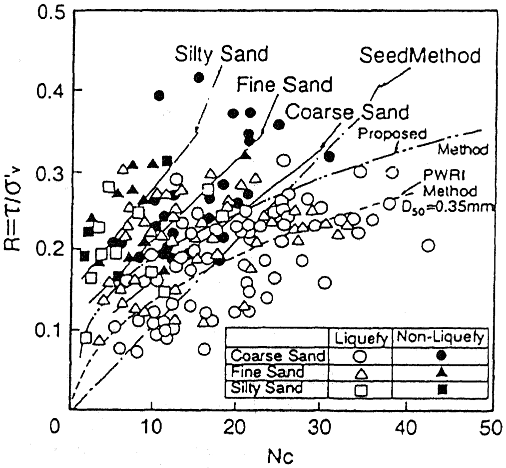

Figure-5: Comparison with Other Proposed Methods

From Figure-4 one can see that for values of M less than 6.5 the proposed relation tends to overestimate the maximum epicentral distance where liquefaction may occur. However, it can be seen for M larger than 6.5, it fits quite well with the data described previously. As far as these data are concerned, we can suggest the use of this relation mentioned above to obtain the maximum epicentral distance for given earthquakes with magnitudes equal or greater than 6.5. One can remark from Figure-4 that the relation proposed fits well the data for larger magnitudes (M>6.5). So comparing it with the other proposals shown on the same figure, we may say that the proposed relation lies in the same range as those in Cornell et al.(1979), Duke et al.(1976), Mc Guire(1978), Sadigh et al.(1978), Seed et al.(1976), Shannon et al.(1979) and Trifunac(1966), but represents much better the field data for magnitude M>7. The major difference between all these proposed lines relating to the correlation of maximum epicentral distance and magnitude seems to be mainly caused by the coefficient of M. In Figure-5 we compare the data of sites where liquefaction has or has not occurred; according to Seed (1979), PWRI (Public Works research Institute, Japan) and the proposed methods. At first sight, we can see the good agreement of the proposed method (evaluated for D_{50}=0.15mm) with the relation adopted in the Japanese code of Bridge design (PWRI method) on Figures 5 and 6. As far as these data are concerned and from these last two figures we remark that the Shibata (1981), Seed (1983), Kokusho (1983) and Tokimatsu and Yoshimi (1983) methods tend to underestimate the soil resistance for small N-values and particularly for silty sands whereas the proposed method and the one of Iwasaki et al. (1982) tend to underestimate the soil resistance for larger N-Values.

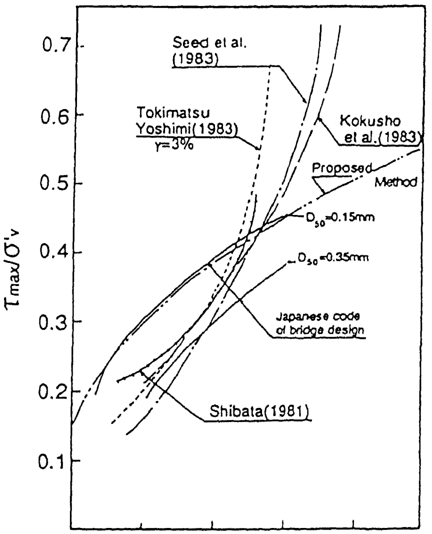

Figure-6: Comparison with Other Proposed Methods

6 CONCLUSION

From a seismic point of view, the thickness and the characteristics of the soil play a significant role in controlling the resonant frequency component and acceleration levels which may develop at or near the ground surface for a given seismic event. A new analytical approach to evaluate seismic soil liquefaction is proposed by using the spectrum at depth which takes into account the influence of the characteristics of the alluvial deposits, surface layers conditions and the earthquake characteristics. A good agreement is obtained between the proposed method ant the Iwasaki- Tatuoka method. As far as the available data are concerned, this procedure can be used to evaluate the general views of seismic-induced soil liquefaction.

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