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SIMPLIFIED PROCEDURES FOR THE EVALUATION OF SETTLEMENTS OF STRUCTURES DURING EARTHQUAKES

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SUMMARY

Based on a review of previous studies, simplified methods of analysis are presented for estimating earthquake-induced settlements of structures on both saturated and nonsaturated sand deposits. To investigate the effectiveness of the proposed methods, informations concerning the settlements of structures during earthquakes were compiled, and the observed settlements were compared with those computed by the proposed method.

The computed settlements are generally consistent with the observed values, indicating that the proposed methods can be used as a first approximation to predict earthquake-induced settlements of structures.

INTRODUCTION

It has been recognized that the structure on sand deposits tend to settle when it is subjected to earthquake shaking. The shear strain caused by earthquake shaking in the soil will always be accompanied by some volumetric strain, which will result in the settlement of structure. The decrease in soil modulus with increasing shear strain or decreasing effective confining pressure as a result of excess pore pressure generation, will also result in the settlement of structure.

Although methods for estimating settlements of sands have been proposed by several investigators (e.g., Lee and Albaisa, 1974, Silver and Seed, 1971, and Tokimatsu and Seed, 1986), there seems no work on the prediction of settlement of structures during earthquakes.

The object of this paper is therefore to propose simplified methods of analysis to predict earthquake-induced settlements of structures on both saturated and nonsaturated sand deposits, based on a review of previous studies concerning the settlement of sand. To investigate the effectiveness of the proposed method, information concerning settlements of structures during earthquakes were compiled, and the observed settlements were compared with those computed by the proposed methods.

SETTLEMENT OF STRUCTURE WITHOUT PORE WATER PRESSURE GENERATION

The total settlement of a structure due to earthquake shaking may be given as

$$S_{st} = S_v + S_e \tag{1}$$

in which S_{st} = total settlement of the structure due to earthquake shaking
S_v = settlement due to volumetric strain caused by earthquake shaking
S_e = immediate settlement due to change in soil modulus

The analysis of each component of the total settlement during earthquake will be discussed in some detail.

<u>Settlement due to volumetric strain</u> The primary factor controlling settlements due to volumetric strain without the generation of pore pressure is the cyclic shear strain induced in the soil below the structure. Thus the evaluation of the settlement needs the knowledge of shear strain distribution in the soil during earthquake.

At any given depth in a soil deposit, the effective shear strain, γ_{eff}, induced by earthquake shaking may be estimated from the relationship (Ref. 1):

$$\gamma_{eff} = \frac{\tau_{av}}{G_{eff}} = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_0 \cdot r_d \cdot \frac{1}{G_{eff}} \tag{2}$$

Rearranging the terms leads to:

$$\gamma_{eff} \left(\frac{G_{eff}}{G_{max}} \right) = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_0 \cdot r_d \cdot \frac{1}{G_{max}} \tag{2'}$$

in which γ_{eff} = effective shear strain
G_{eff} = effective shear modulus at induced strain level

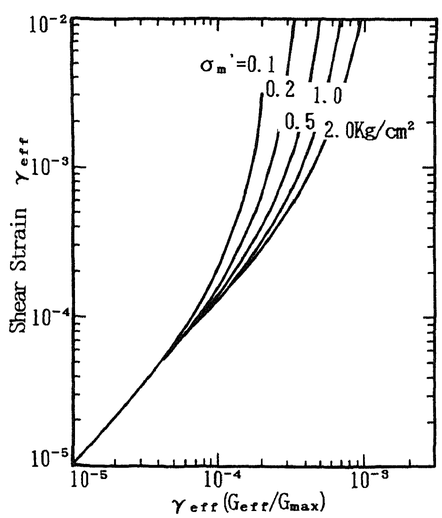

Fig.-1 Determination of Induced Shear Strain

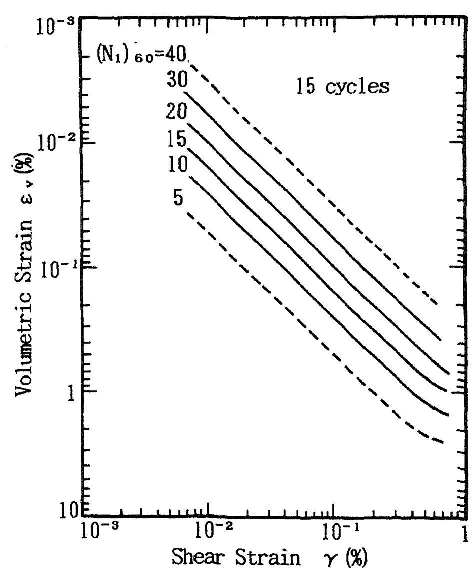

Fig.-2 Cyclic Shear Strain, $(N_1)_{60}$ vs. Volumetric Strain

G_{max} = shear modulus at low strain level
α_{max} = maximum horizontal acceleration at the ground surface
σ_o = total overburden pressure at the depth considered
r_d = stress reduction varying from a value of 1 at the ground surface to a value of about 0.9 at a depth of 30 ft.

Since the value for the product $(G_{eff}/G_{max})\cdot\gamma_{eff}$ in Eq. (2') can readily be evaluated for any given depth, the corresponding shear strain, γ_{eff}, can be read off from the curves presented in Fig. 1 (Ref. 1).

Having thus known the value of effective shear strain, the volumetric strain due to earthquake shaking can be determined as follows. Based on the studies of Silver and Seed (Ref. 2), relationships between volumetric strain and shear strain after 15 cycles for sands at different normalized SPT N-value; $(N_1)_{60}$, are summarized in Fig. 2. The relation shown in the figure can be extended to different magnitude events by multiplying volumetric strain, which can be read from Fig. 2, with scaling factor for a volumetric strain r_v shown in column 3 of Table 1.

Once knowing the effective shear strain from Fig. 1, the volumetric strain to be caused by cyclic loading can be estimated from Fig. 2, and the integration of those volumetric strains with depth results in the settlement of structure due to volumetric strain.

Immediate settlements The immediate settlement S_e can be evaluated using modified Steinbrenner's equation given by Eq. (3), which gives elastic settlement of soil deposit under rectangular load.

$$S_e = q \cdot B \cdot I_p \left(\frac{1}{E_2} - \frac{1}{E_1} \right) \qquad (3)$$

in which q = contact pressure of the structure
B = width of the structure
I_p = coefficient concerning the dimension of the structure, thickness of soil layer and Poisson's ratio of soil
E_1 and E_2 = Young's modulus of soil before and during earthquake shaking respectively.

The reduction in Young's modulus of soil during earthquake shaking may be estimated based on the effective shear strain determined by Eq. (2').

Table 1 Scaling Factor for Effect of Earthquake Magnitude

Earthquake Magnitude M	Scaling Factor for Stress Ratio r_m	Scaling Factor for Volumetric Strain r_v
8-1/2	1.12	1.25
7-1/2	1.0	1.0
6-3/4	0.88	0.85
6	0.76	0.60
5-1/4	0.67	0.40

SETTLEMENT OF STRUCTURE WITH PORE PRESSURE GENERATION

Earthquake induced settlements of a structure on a sand deposit with pore pressure generation can also be predicted by Eq. (1) when the sand deposit does not suffer heavy liquefaction. S_v and S_e are calculated as follows.

Settlement due to volumetric strain It has been shown that the primary factors controlling the volumetric strain followed by the pore pressure generation are the maximum pore pressure generated before initial liquefaction and the maximum shear strain after initial liquefaction. Based on these findings, Tokimatsu and Seed (Ref. 1) proposed a relationship between cyclic stress ratio, $(N_1)_{60}$ and volumetric strain as shown in Fig. 3. The cyclic stress ratio developed in the soil during earthquakes can be estimated by:

$$\left(\frac{\tau_{av}}{\sigma_0'}\right)_{7.5} = \left\{ 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \frac{\sigma_0}{\sigma_0'} \cdot r_d \right\} \cdot r_m \tag{4}$$

in which $\left(\frac{\tau_{av}}{\sigma_0'}\right)_{7.5}$ = equivalent shear stress ratio induced by the earthquake shaking of $M = 7.5$

σ_0' = effective overburden pressure at the depth considered

r_m = scaling factor for a stress ratio concerning the magnitude of earthquake shown in column 2 of Table 1

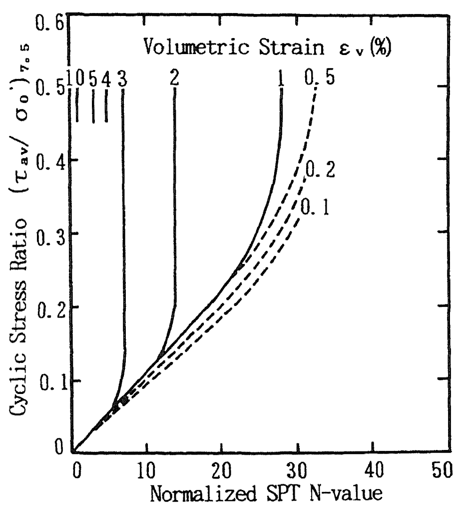

Fig.-3 Cyclic Stress Ratio, $(N_1)_{60}$ vs. Volumetric Strain

Thus by knowing the $(N_1)_{60}$ value, the volumetric strain below the structure can be determined from Fig. 3. The settlement of the structure due to volumetric strain is then determined by integrating the volumetric strains.

Immediate settlement The immediate settlement S_e with pore pressure generation may also be evaluated from Eq. (3). To determine Young's modulus during pore pressure generation for E_2 in Eq. (3), the change in effective stress due to pore pressure generation as well as the shear strain level developed in the soil should be taken into account.

When the sand deposit heavily liquefies, Eq. (1) can not be used, because the settlement of the structure is affected by the shear deformation of the ground, and because Young's modulus of the soil can not accurately be determined.

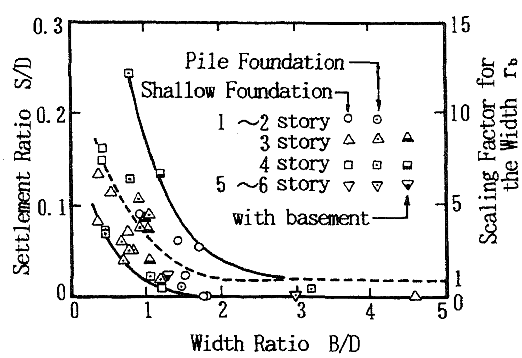

Fig.-4 Scaling Factor vs. Width Ratio

In this case, based on the field observation, earthquake-induced settlements of structure may be approximately estimated by:

$$S_{st} = S_v \cdot r_b \qquad (5)$$

in which r_b = scaling factor concerning the shear deformation

The scaling factor r_b can be evaluated as follows.

Field observation of the settlement of reinforced concrete buildings during the Niigata earthquake (1964) was summarized by Yoshimi et al. (Ref. 4) as Fig. 4. Fig. 4 shows that the number of stories, the presence of basement or the pile does not appear to have a significant effect on the settlement ratio. Despite the scatter, however, there is a definite relationship between the settlement ratio and the width ratio, i.e. appreciable settlement occurred where the width ratio was less than 2 whereas the settlement was small and constant where the width ratio exceed 2 or 3.

This means that, when the width of structure is sufficiently large compared to the thickness of liquefied layer, the settlement of structure is nearly equal to that of the ground without a structure, and can be treated as a one-dimensional problem. In contrast, when the width of structure is not so large, primary cause of the settlement of structure is the shear deformation of the ground and can not be treated as a one-dimensional problem. Based on the findings, the effect of shear deformation is approximately represented by the scaling factor r_b, that is the settlement ratio normalized by the settlement ratio of width ratio equal 3. The value of r_b is also shown on the right hand of Fig. 4.

COMPARISON OF COMPUTED AND OBSERVED SETTLEMENTS OF STRUCTURE DURING EARTHQUAKES

To investigate the effectiveness of the proposed method, information concerning the settlements of structures during earthquakes were compiled, and the observed settlements are compared to those computed by the proposed method. The settlements of structure are computed for the case in which a

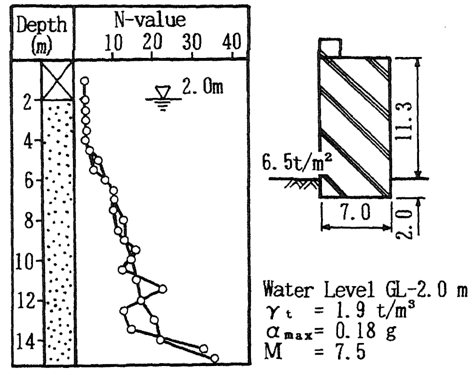

Fig.-5 Soil Profile of Kawagishi-cho

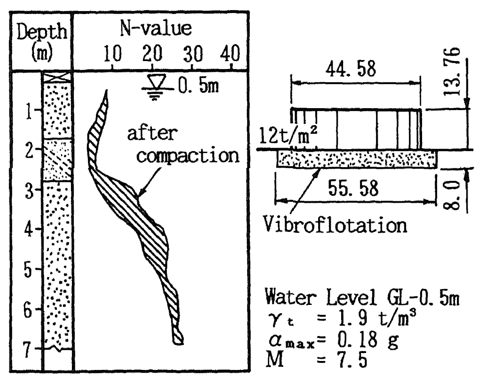

Fig.-6 Soil Profile of Ohse

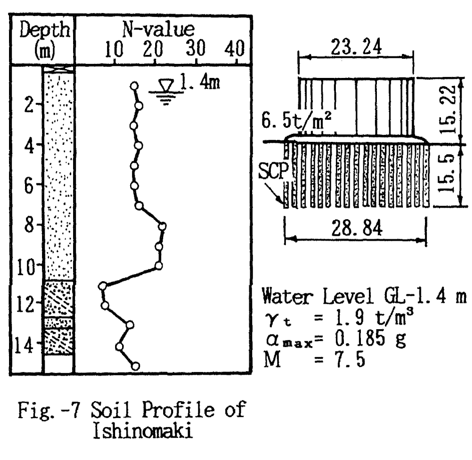

Fig.-7 Soil Profile of Ishinomaki

building settled more than 1 m due to liquefaction during Niigata earthquake (1964), and for the cases in which oil tanks settled without pore pressure generation during Niigata earthquake and Miyagiken-oki earthquake (1987). There site characteristics are shown in Figs. 5-7.

The observed and computed settlements of the structures during earthquake in both cases are summarized in Table 2 and Table 3 respectively. Although the number of field cases for which data are available is quite limited, the computed settlements are generally consistent with the observed values, indicating that the proposed method is effective.

Table 2 Comparison between Computed and Observed Settlements for Liquefied Deposit shown in Fig. 5 (Kawagishi-cho)

Computed (cm)		Measured
S_v	S_{st}	(cm)
30	160	46 ～300 Ave. 130

Table 3 Comparison between Computed and Observed Settlements for Non-Liquefied Deposits (Ohse, Ishinomaki)

	Computed (mm)			Measured
	S_v	S_e	S_{st}	(mm)
Ohse Fig. 6	4	1	5	Tank-A 20～30 mm Tank-B, C No Damage
Ishinomaki Fig. 7	8	2	10	5.3～8.0 Ave. 7 mm

CONCLUSION

Simplified method of analysis have been proposed for estimating probable settlements of structure on both saturated and nonsaturated sand deposits subjected to earthquake shaking.

Comparison of the numerical results with several case histories indicates that the methods presented herein can be used in many cases as a first approximation for evaluating the settlements of structure due to earthquake shaking. In the application of the methods, it is of course essential to check that the final results are reasonable in the light of available experience.

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