EARTHQUAKE-INDUCED RESIDUAL SETTLEMENTS IN SOFT SOILS

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ABSTRACT

Based on the methods previously presented by the authors (Yasuhara et al., 1992) which are capable of predicting the degradation in strength and stiffness for clays in the course of shaking during earthquakes, a methodology has been developed to estimate the earthquake-induced residual deformation as well as post-earthquake settlement due to dissipation of cyclically induced excess pore pressures in soft soils. Simplified formulae included in the proposed methodology are given as functions of the amplitude of cyclic-induced excess pore pressure ratio, u/σ'_c, and soil properties represented by plasticity index, I_p, and factor of safety, F_s. The results calculated using the proposed methodology are, therefore, presented in the form of a design chart to give the settlement versus normalized excess pore pressure relations including the effects of the above-mentioned parameters, I_p, and, F_s.

INTRODUCTION

In comparison with cyclic behaviour of loose sands, soft cohesive soils is believed to be stable because it is scarecely liquefied even under strong motion of ground from earthquakes. Rather, residual deformation and post-cyclic recompression settlements are important, because of possibly leading to instability and then inducing residual and differential settlement of structures. Extreme cases were experienced in earthquakes of 1957 and 1985 at Mexico and in 1964 at Alaska. Thus, soft cohesive ground can be endangered to meet instability and settlement during and after earthquakes. However, in comparison with sandy soils, there have not been many researches done to investigate the cyclic behaviour of soft cohesive soils from small to large strains. Among the engineering problems on soft soils during earthquakes like both stress amplification and earthquake-induced settlement and instability each pertaining to small and large strain region, respectively, the present paper focusses on earthquake-induced settlements, that is, the immediate or residual settlement during earthqauke and the post-earthquake recompression settlement.

EARTHQUAKE-INDUCED SETTLEMENT IN SOFT GROUND

Earthquake-induced settlement should be considered separately in the both cases, the one with structures and the other without structures on the surface of the ground, as shown in Fig. 1. Since settlement in the former case constitutes both the immeadiate or residual settlement, ΔS_i, which is observed just after earthquake and must sometimes be followed by generation of excess pore pressures, and the post-earhquake recompression settlement, ΔS_{vr}, due to dissipation of cyclic-induced excess pore pressures, total incremental settlement, ΔS_{cy}, caused by earthquake is thus given by:

$$\Delta S_{cy} = \Delta S_i + \Delta S_{vr} \qquad (1)$$

Generally speaking, excess pore pressure generation of soft soil with structures during earthquakes is prevented from the initial shear stress induced by structures. On the contrary, residual settlement is caused by this existing initial shear stress which may make the cyclic strength decrease in clays (Hyodo et al., 1994).

Although, in the latter case, residual settlement can not be observed, post-earthquake recompression settlement is significant because the larger excess pore pressure is generated than in the former with structures on the ground, frequently leading to liquefaction in loose saturated sand. Therefore, large post-earthquake recompression settlement due to immediate dissipation of cyclically induced pore pressure equal to the vertical overburden stress during liquefaction have been observed after earthquakes.

The above-stated situations of loose saturated sand are different from those of soft clay although normally consolidated clay posesses very weak strength and high compressibility. In addition, as will be mentioned later, a very important aspect in relation to cyclic problems is the irreversibility of strength and stiffness of soft clay after degradation due to cyclic loading.

A METHOD FOR EVALUATING THE EARTHQUAKE-INDUCED RESIDUAL SETTLEMENT IN SOFT GROUND

Evaluation of Immediate Settlement

Fig. 2 illustrates a key sketch for imterpretation of the effect of cyclic loading on load intensity versus settlement curves observed in the horizontal soft ground : curve I for static loading with no cyclic loading and curve II for a case with earthquake loading subjected. If this soft ground consists of loose saturated sand, bearing capacity decreases immediately due to liquefaction and large displacement occurs due to failure of ground and then we have curve III since in comparison with soft cohesive ground, loose saturated sandy ground is very sensitive to cyclic loading. However, even if the soft ground constitutes the cohesive soils, the effective stress decreases due to excess pore pressure generation and then it induces instability depending upon the severity of cyclic loads and intial shear stresses.

Let us, now, assume to have the structure with q (=q_f/F_s) as an average load intensity on the soft ground in which the load versus settlement relation follows the curve II in Fig. 2 after earthquake. Since both bearing capacity and stiffness degradate, additional settlement, ΔS_i, of soft ground takes place due to shift from A to B in Fig. 2. It is assumed that the load versus settlement curve is given in terms of the following hyperbolic function (Yamaguchi, 1977 : Kusakabe and Kawai, 1989) :

$$S_{NC} = \frac{q_{fNC}\, q}{K_{iNC}(q_{fNC} - q)} \tag{2a}$$

$$S_{cy} = \frac{q_{fcy}\, q}{K_{icy}(q_{fcy} - q)} \tag{2b}$$

Combination of Eq. (2a) with Eq. (2b) leads to :

$$\frac{\Delta S_i}{S_{NC}} = \frac{R_q}{R_K} \cdot \frac{(1 - \frac{1}{F_s})}{(R_q - \frac{1}{F_s})} - 1 \tag{3}$$

where ΔS_i is equal to $S_{NC} - S_{cy}$, R_q : q_{fcy}/q_{fNC}, R_K : K_{icy}/K_{iNC} and F_s : safety factor. By postulating that R_q and R_K are equal to undrained strength ratio, s_{ucy}/s_{uNC} and tangent stiffness ratio, E_{scy}/E_{sNC}, we adopt the following relations previously proposed by the author (1994a, 1994b):

$$R_q = \frac{s_{ucy}}{s_{uNC}} = n_q^{\frac{\Lambda_0}{1 - C_s/C_c} - 1} \tag{4a}$$

$$R_K = \frac{K_{icy}}{K_{iNC}} = \frac{E_{scy}}{E_{sNC}} = \frac{1-\dfrac{C}{1-C_S/C_C}\ln n_q}{n_q} \tag{4b}$$

where Λ_o : strength ratio parameter, C : experimental constant, C_S, C_C : swelling and compression index, and n_q is defined as :

$$n_q = \frac{1}{1-\dfrac{u}{p'_c}} \tag{5}$$

and s_{uNC}, s_{ucy} : undrained strength before and after cyclic loading, E_{sNC}, E_{scy} : undrained secant modulus before and after cyclic loading. The applicability of both Eqs. (4a) and (4b) have been made sure in Figs. 3 and 4. When we combine Eqs. (4a) and (4b) with Eq. (3), Eq. (3) can be a function of u/p'_c and F_s.

The elastic settlement of soft ground, S_{NC}, in Eq. (3), due to self-weight of structures being already experienced before earthquakes, can be calculated using the conventional method:

$$S_{NC} = \mu_1\mu_2\frac{1-\nu^2}{E_t}BI_s \tag{6}$$

where μ_1, μ_2 : corrected facors, I_s : settlement coefficient and E, ν : Young's modulus equal to E_{sNC} in Eq. (4b) and Poisson's ratio, respectively.

Estimation of Settlement due to Dissipation of Cyclic-induced Excess Pore Pressures

As was shown in Fig. 1, immediate settlements are followed by additional settlement due to dissipation of excess pore pressures generated during earthquakes. From the results of cyclic DSS and triaxial tests on clays followed by drainage, authors have proposed the following rleation (Yasuhara and Andersen, 1991):

$$S_{vr} = \Delta\varepsilon_{vr}H = 0.225\,H\frac{C_C}{1+e_o}\log_{10}\left(\frac{1}{1-u/p'_c}\right)$$

$$= 0.225H\frac{C_C}{1+e_o}\log_{10} n_q \tag{7}$$

where C_C is compression index and e_0 is initial void ratio. Those are representative values for the objective soft soil under consideration. It is one of the characteristic features in Eq. (3) and Eq. (7) that both immediate and post-earthquake settlements are given with respect to the excess pore pressure generated by cyclic loading.

PARAMETRIC STUDY FOR RESIDUAL SETTLEMENTS OF SOFT GROUND

Immediate Settlement

Following the procedure for predicting the changes in undrained strength and stiffness, the numerical experiments were conducted for soft ground consisting of several kinds of cohesive soil, including peat whose index properties are summarized in Table 1. The results are shown in Fig. 5 and Fig. 6, indicating that:

1) The settlement ratio increases with increasing the cyclically induced excess pore pressure. In particular, settlement ratio increases dramatically when u/p'_c reaches 0.75 approximately when we assume F_s equal to 3.0. This tendency is in good correspondence with the fact that soft clays in cyclic triaxial or DSS tests reach failure once the excess pore pressure ratio becomes 0.70 through 0.75 (Yasuhara et al., 1992).

2) In case of very highly plastic clays, settlement ratio accelerates with increasing the pore pressure ratio and decreasing the safety factor. On the contrary, the relation between settlement ratio versus pore pressure ratio in other clays with low to high plasticity is not influenced by the safety factor.

3) In the case of highly plastic Mexico city clay as shown in Fig. 6, the settlement ratio is strongly influenced by the safety factor. In case of Fs = 1.5, settlement is devastatingly increased, probably leading to failure even when the excess pore pressure ratio is below 0.5.

Fig. 7 shows the calculated variations of settlement ratio with the load intensity by structures equal to the inverse value of safety factor (=q/qf), as a function of excess pore pressures generated during earthquakes. If we assume that the earthquake-induced excess pore pressure ratio u/p'_c is equal to 0.3 for Mexico city clay ground, the following is pointed out from Fig. 8:

1) Although the settlement ratio is almost constant, nearly 1.0 through 2.0 at most, until the load intensity ($=1/F_s$) becomes 0.75, it suddenly increases beyond this critical value of 0.75 for 1/Fs.

2) In other words, it is suggested that we should take the value of safety factor larger than 1.33 to avoid the devastating settlement.

By replotting the settlement ratio $\Delta S_i/S_{NC}$ versus log. u/p'_c, as an example is shown in Fig. 9 it is possible to determine the inflection point which must show the critical excess pore pressure ratio, designated by $(u/p'_c)_{CRT}$, where residual settlement accelerates as is shown in Fig. 9. This critical value of pore pressure ratio given by $(u/p'_c)_{CPT}$ is plotted against plasticity index, I_p, of each soft soil in Fig. 10 for different safety factor, F_S. This critical value tends to decrease from a certain value of plasticity index (nearly 70 to 100 of I_p). This tendency is more eminent for the smaller value of safety factor.

Post-earthquake Long-term Settlement

From Eq. (7), void ratio decrement, Δe_{vr}, due to dissipation of cyclic-induced excess pore pressures is denoted by:

$$\Delta e_{vr} = 0.225 \, C_c \log n_q \qquad (8)$$

The results calculatred using Eq. (8) are presented in Fig. 11 in the relation between Δe_{vr} and u/p'_c, for representative existing clays with different plasticity index. From Fig. 11, we learn the same trend as in immediate settlement that void ratio decrement due to dissipation of cyclically induced excess pore pressures increases with increasing the plasticity index.

7. CONCLUSIONS

In the present paper, a relatively simplified procedure being taken into consideration the degradation characteristics of strength and stiffness is proposed for evaluating the earthquake-induced settlements of soft ground. This procedure is characterized by :

1) Earthquake-induced settlement of soft ground with structures is divided into two categories : immediate and post-cyclic time dependent settlements.

2) Both settlements induced by earthquakes is a function of excess pore pressures generated during earthquakes.

3) Immediate settlement is additionally dependent on the plasticity index and safety factor by structures.

4) Post-earthquake settlement due to dissipation of cyclically induced pore pressures is exhibited markedly in soft soil with the large plasticity index.

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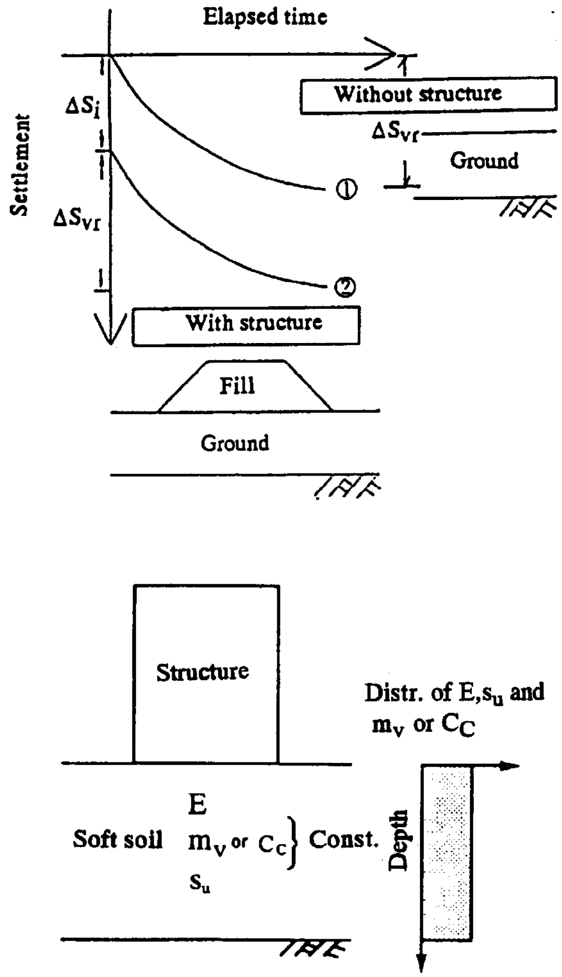

Fig. 1 A key sketch for earthquake-induced settlement

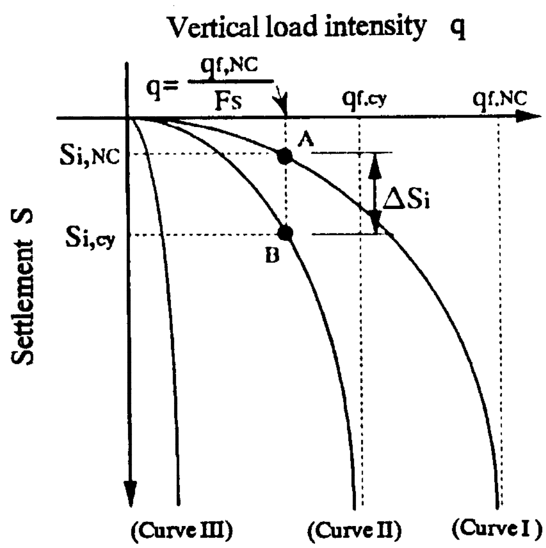

Fig. 2 Settlement versus load intensity relations before and after earthquakes

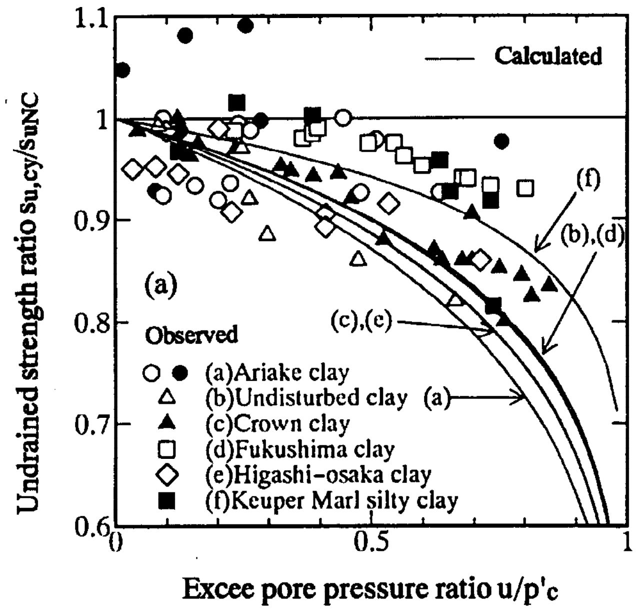

Fig. 3 Undrained strength degradation for various clays and peat plotted against cyclic-induced pore pressure

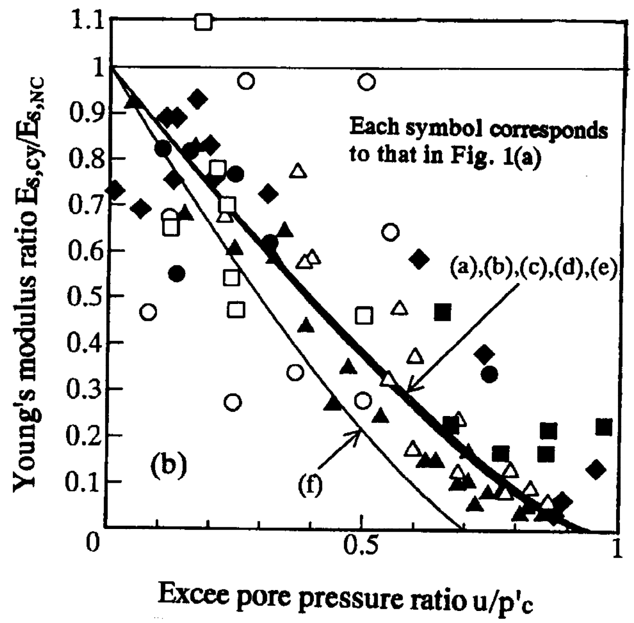

Fig. 4 Stiffness degradation for various clays plotted against cyclic-induced pore pressure

Table 1 Experimental parameters used for calculating earthquake-induced settlements

	I_p	Λ_0	$1-C_s/C_c$	C_c
Bootlegger clay	16	0.768	0.783	0.294
Drammen clay	25	0.769	0.765	0.440
San Francisco Bay mud	41	0.769	0.733	0.699
Bangkok clay	53	0.767	0.709	0.893
Ariake clay	69	0.763	0.677	1.153
Mexico city clay	240	0.595	0.335	3.923
Akita peat	295	0.491	0.225	4.814

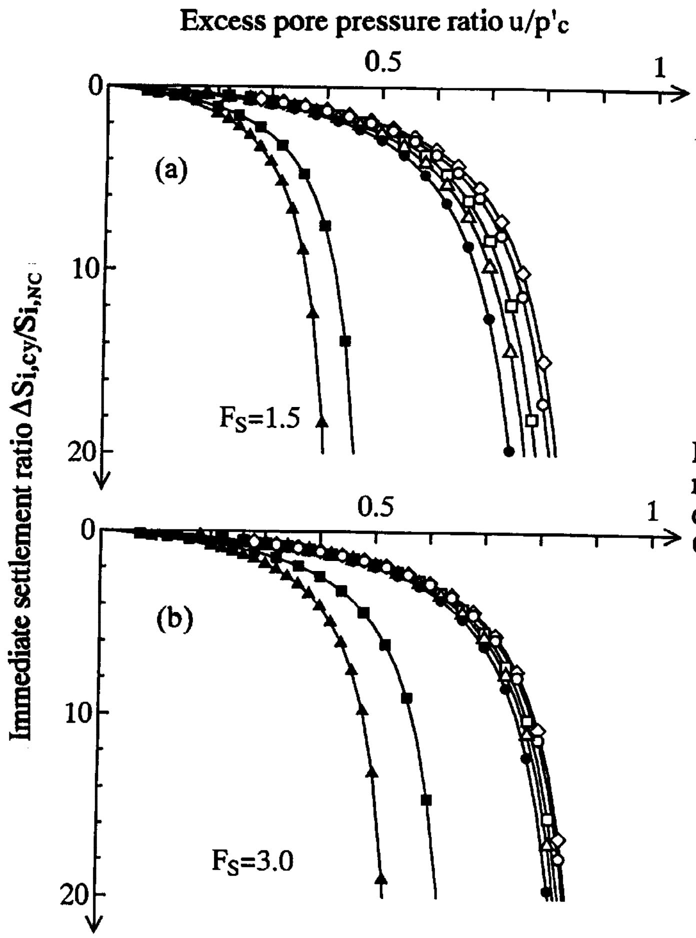

Fig. 5 Calculated earthquake-induced immediate settlement ratio versus cyclic-induced pore pressure ratio relations for different safety factors in cohesive soils with different plasticity index

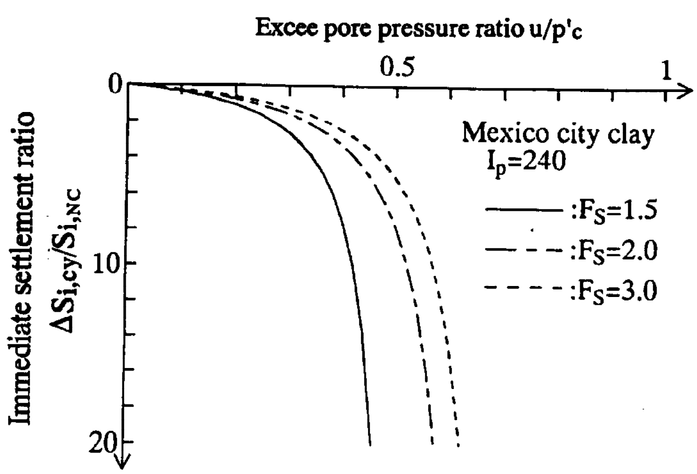

Fig. 6 Influence of safety factor on immediate settlement ratio versus earthquake-induced pore pressure ratio relations for Mexico City clay

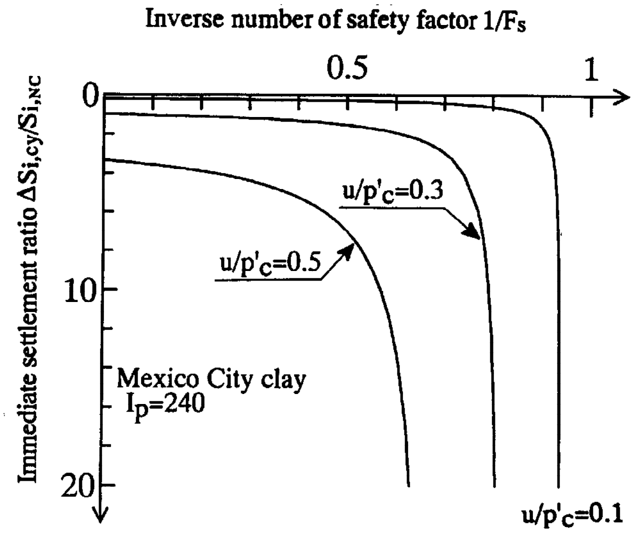

Fig. 7 Immediate settlement ratio versus inverse number of safety factor relations for different earthquake-induced pore pressure ratio in Mexico City clay.

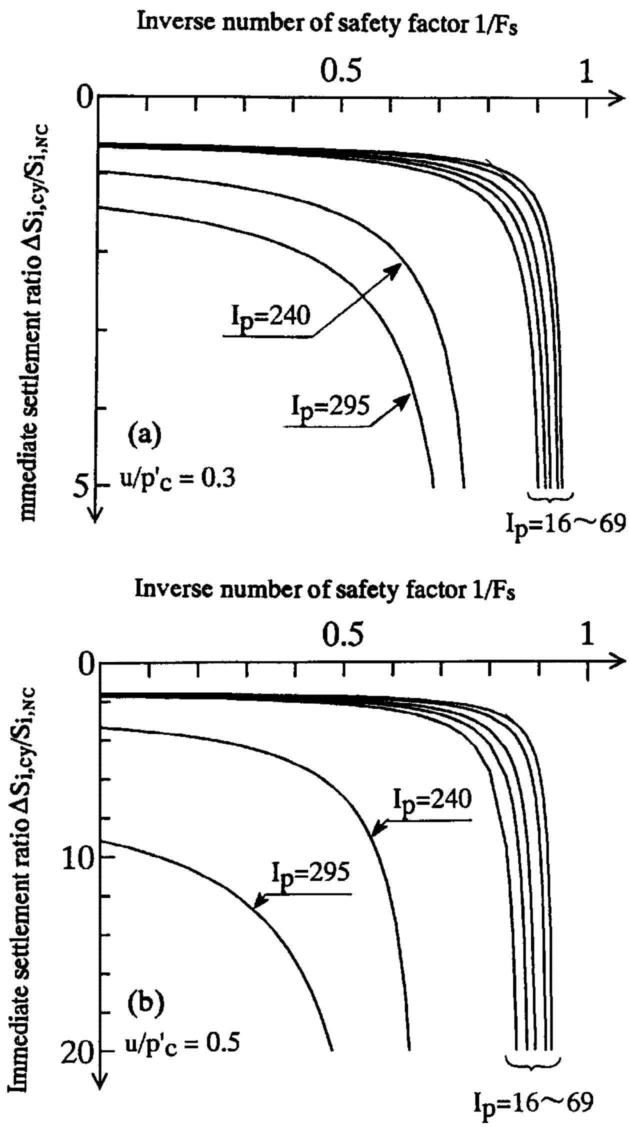

Fig. 8 Immediate settlement ratio versus inverse number of safety factor relations for cohesive soils with different plasticity index

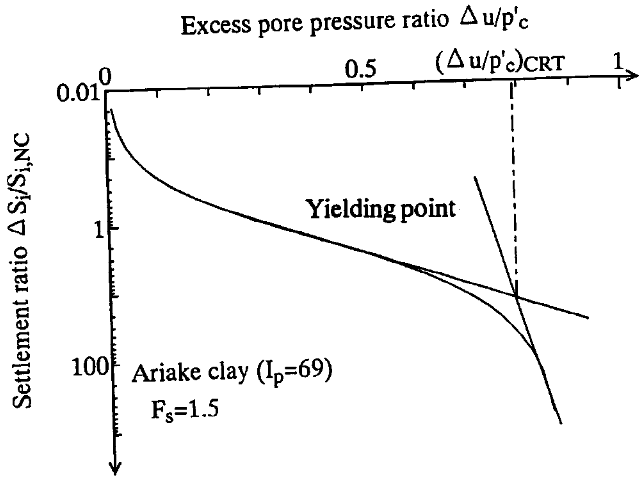

Fig. 9 Definition of critical excess pore pressure

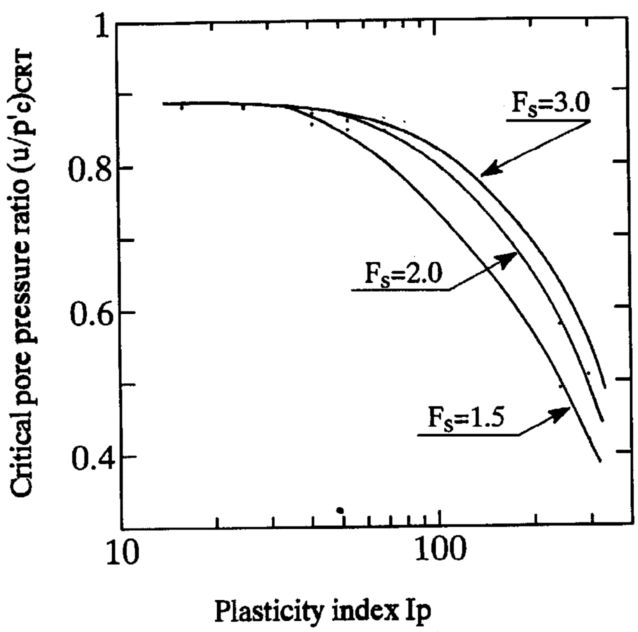

Fig. 10 Dependency of critical pore pressure on plasticity index as a function of safety factor

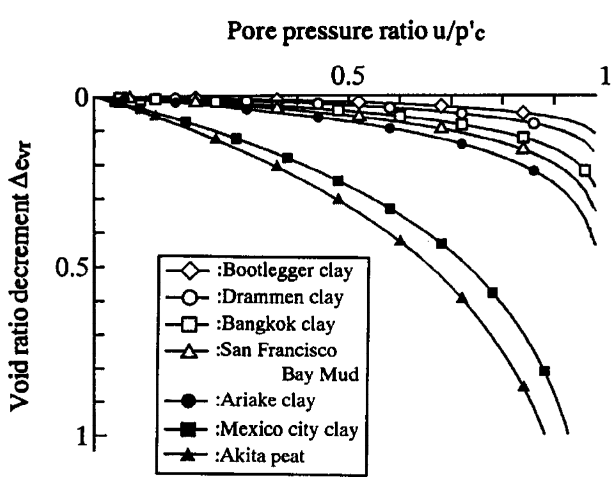

Fig. 11 Void ratio decrement due to dissipation of earthquake-induced pore pressure in coheisve soils with different plasticity index