EVALUATION OF GROUND SETTLEMENTS DUE TO SOIL LIQUEFACTION

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INTRODUCTION

Saturated sand deposits tend to liquefy and subside when subjected to earthquake shaking which in many cases causes severe damage to overlying structures. The cyclic shear strains induced in the soil under undrained conditions result in build-up of excess pore water pressures. When these pore pressures dissipate, volumetric strains occur resulting in observable settlements at the ground surface.

This paper aims to propose a simplified and practical procedure for predicting liquefaction-induced settlements in sand deposits during earthquakes. Case studies are presented to show the capability of the proposed method.

REVIEW OF PREVIOUS LABORATORY STUDIES

Experimental data obtained by Dobry (1985), shown in Fig. 1, suggest that the pore pressure increase depends mainly on the magnitude of cyclic shear strain. The average curve will be adopted here as the model of pore pressure build-up. Based on laboratory tests, Lee and Albaisa (1974) found that the amount of settlement for non-liquefaction conditions increases with increasing pore pressures, and with decreasing relative density. Their test results are summarized in Fig. 2. Tatsuoka, et al (1984) and Nagase and Ishihara (1988) examined the amount of settlement after initial liquefaction. They concluded that the amount of settlement is significantly controlled by the maximum shear strain induced in the soil. Their data are plotted in Fig. 3. In both cases, the volumetric strain is expressed in terms of settlement index defined here as the product of volumetric strain and relative density. Average curves obtained from least-squares analyses are also shown.

PROCEDURE FOR EVALUATING GROUND SETTLEMENTS IN THE FIELD

The prediction of liquefaction-induced settlements in sand deposits involves the following steps:

a. Calculate the maximum cyclic shear strains induced in all sand layers. Assuming that the soil follows the hyperbolic stress-strain relation, these can be computed from the equation:

$$\gamma_{max} = \frac{\tau_{max}\ \tau_f}{G_{max}\ (\tau_f - \tau_{max})} \times 100\ \%$$

where τ_{max} equals maximum shear stress developed during an earthquake. τ_f equals soil strength and G_{max} equals the initial tangent shear modulus of the soil.

b. If $\gamma_{max} < 1.0$ %, estimate the settlements from Fig. 2 where r_u is computed using Fig. 1. If $\gamma_{max} > 1.0$ %, i.e. when $r_u=1.0$, use Fig. 3 to calculate the settlements.

In this paper, τ_{max} is computed using the formula proposed by Seed and Idriss (1971) while the parameters τ_f, G_{max} and D_r are evaluated using empirical correlations with SPT N-values measured in the field.

COMPARISON OF COMPUTED SETTLEMENTS WITH FIELD OBSERVATIONS

Table 1 presents the computations of settlements for three sites in Japan which were observed to have liquefied during major earthquakes. The soil profiles of these sites are shown in Fig. 4. The N-values of silty sand layers, indicated by asterisks, are corrected by increasing by 7.5 the actual measured values knowing that silty sands are considerably less vulnerable to liquefaction than clean sands with similar penetration resistance values. In all the cases presented, the computed settlements are in good agreement with the observed values. Note that, for the same site, calculations carried out using profiles surveyed after earthquakes oftentimes yield lower computed values than those using profiles surveyed before earthquakes. The case of Showa Bridge 2, presented in Table 1(C), illustrates this point wherein the computed total settlement has been reduced by about 30 %. Assuming that the same percentage of reduction can be applied to the case of Hachinohe, the total settlement will be about 44 cm which is very close to the maximum observed value of 50 cm.

CONCLUSION

Based on empirical equations obtained from various laboratory tests, a simplified method for predicting ground settlements due to soil liquefaction during earthquakes is proposed in this paper. Comparison of the computed results with actual field observations suggests that the proposed method can be useful in many cases.

REFERENCES

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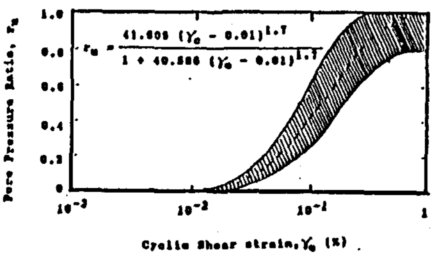

FIGURE 1 - Summary of results from strain-controlled cyclic triaxial tests on seven different sands with different specimen preparation techniques, densities and confining pressures (Dobry, 1986)

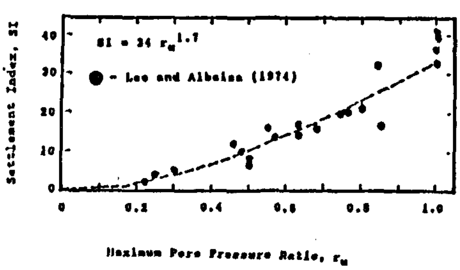

FIGURE 2 - Relationship between settlement index and maximum pore pressure ratio

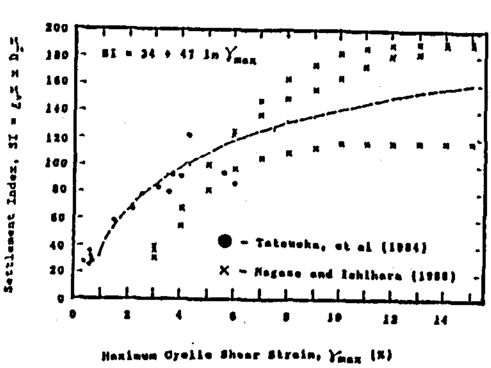

FIGURE 3 - Relationship between settlement index and maximum cyclic shear strain

TABLE 1 - COMPUTATION OF SETTLEMENTS AND COMPARISON WITH FIELD OBSERVATIONS

A. Rifu Marshalling Yard - Miyagi-ken Oki Earthquake, 1978 ($M = 7.4$)
a_{max} = 185 gals, Survey Time = Before earthquake

Layer No.	N-value	γ_{max} (%)	r_u	ε_v (%)	Settlement (cm)
1	13.5 *	0.173	0.667	0.249	0.50
2	7.0	0.525	0.953	0.699	1.06
3	9.0	0.538	0.955	0.655	0.44
4	7.5 *	0.680	0.978	0.765	0.88
5	7.5 *	0.734	0.983	0.791	0.79
6	7.5 *	0.785	0.988	0.816	0.81
7	7.5 *	0.832	0.991	0.839	0.62
8	12.5 *	0.586	0.964	0.634	0.64
9	5.0	1.263	1.000	1.440	1.78
10	4.0	1.566	1.000	2.010	1.24
11	22.5 *	0.421	0.922	0.464	0.86
12	23.0	0.442	0.930	0.483	0.72
13	8.0	1.026	1.000	0.976	1.17

Observed Settlement: 10 cm — Total: 11.51 cm

B. Hachinohe Paper Co. - Tokachi Oki Earthquake, 1968 ($M = 7.9$)
a_{max} = 235 gals, Survey Time = After earthquake

Layer No.	N-value	γ_{max} (%)	r_u	ε_v (%)	Settlement (cm)
1	2.0	0.540	0.956	0.986	1.72
2	0.6	2.994	1.000	5.164	5.42
3	0.4	5.869	1.000	9.058	9.05
4	0.3	9.318	1.000	12.916	12.91
5	12.0	0.418	0.921	0.452	0.45
6	36.0	0.191	0.707	0.188	0.18
7	15.0	0.452	0.933	0.442	0.39
8	24.0	0.331	0.876	0.322	0.28
9	16.0	0.510	0.949	0.465	0.46
10	27.0	0.356	0.892	0.331	0.33
11	18.0	0.535	0.955	0.467	0.46
12	25.0	0.433	0.927	0.387	0.38
13	20.0	0.550	0.958	0.468	0.49
14	37.0	0.351	0.889	0.311	0.31
15	25.0	0.502	0.947	0.429	0.42
16	28.0	0.477	0.941	0.410	0.43
17	33.0	0.433	0.927	0.376	0.48

Observed Settlement: 50 cm (maximum) — Total: 34.16 cm

C. Showa Bridge 2 - Niigata Earthquake, 1964 ($M = 7.5$)
a_{max} = 170 gals, Observed Settlements = 1.1 - 9.2 cm at piers

Survey Time	Computed Total Settlement (cm)
Before earthquake	10.82
After earthquake	7.90

FIGURE 4 - SOIL PROFILES OF INVESTIGATED SITES