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SYNOPSIS: The aim of this work is to establish a quantitative analysis approach of liquefaction hazard. Based on the microscopic field data of liquefaction it is pointed out that non-uniform settlement of building is the primary type of damages caused by sand liquefaction during earthquake. Then a computation algorithem for evaluating settlements is programmed on the basis of "Softening model" idea, and 33 liquefaction settlement events are evaluated by use of this test-computation approach. The results of these evaluation are in good agreement with the observed ones.

INTRODUCTION

It can be noted from the data of earthquake damages of the whole world that the extent and form of ground fracture was and building damages at liquefied sites were quite different. Somewhere sand blowing and water spouting was very serious and accompanied with ground crack, subsidence, or inclination, and crack and differential settlement of buildings, whereas somewhere all those were rather slight. In certain cases though sand blew seriously buildings does not suffer from damages which means that not all liquefable soil deposit can cause damages to ground surface and buildings. Therefore it would be of practical significance to make a further liquefaction hazard analysis and to give prediction method and a seismic measures in addition to the evaluation of liquefaction potential.

The earlist paper which suggested a liquefaction hazard analysis method was presented by Iwasaki (T. Iwasaki et al, 1982) They proposed a liquefaction potential index, P_L, and made calculation P_L for a lot of liquefied and unliquefied sites, and showed that much greater P_L could be obtained for liquefied sites than nonliquefied ones. Therefore, they divided the liquefaction hazard extent into four classes: very low, low, high and very high according to P_L value.

There were similar results in China, using the measured SPT count (Huisan Liu, 1984). However, some remarkable drawbacks would be exhibited for such approachs, for example, 1. P_L account for the bahaviour of liquefied deposit only but nothing on that of the upper buildings. 2. they did not take account of the role of nonliquefied and partialy liquefied deposits in resulted settlement caused by seismic liquefaction. 3. As a relative index, the magnitude and dimension of P_L can only be used for mutual comparision.

Liquefaction hazard analysis should be developed further in such a way that could give out some quantitative indices involving in the effect of both the characteristics of fundation soils and upper buildings. As a further step to this end the writers of this paper are of the opinion that it seems appropriate to use the quantity of seismic settlement for evaluating liquefaction hazard and dividing liquefaction class.

MICROSCOPIC PHENOMENA

The liquefaction settlement stands for the additional settlement of ground surface or building caused by liquefaction of sand during earthquake. A lot of earthquake liquefaction cases have occurred over the world.

A large number of cases of building settlement due to liquefaction has been emerged during several big earthquakes. About 340 of reinforced concrete buildings in Nigata city suffered from liquefaction damage whose main forms were settlement and inclination with maximum settlement of 3.8 m during the 1964 Nigata earthquake. In addition, 33 cases of seismic settlement are included in this paper to have used for analysis. It can be noted from the analysis made that 1. Seismic settlement of buildings was usually accompanied with sand boiling and the stronger the sand boiling the more serious was the seismic settling. 2. In certain cases, seismic settlement of buildings was accompanied yet with ground cracking. 3. Usually, seismic settlement due to liquefaction was nonuniform causing wall crack or whole body inclination of buildings.

CAUSE OF SEISMIC SETTLEMENT

The seismic settlement of buildings due to liquefaction is caused by a variety of factors among which the softening of soil is likely played significant part. Test results available show that many type of soils, especially soft clays and undrained saturated sands, undergo softening under dynamic loading action. For saturated sands under undrained condition such softening is represented as the increase of pore water pressure and decrease of shear resistance, and consequently the loss of stiffness, up to the complete liquefaction. In other words, liquefaction is one form of softening. Therefore, the settlement of saturated sand deposit due to liquefactions during the entire process of earthquake is caused by the softening of sands under almost undrained condition. Moreover, the overburden nonliquefable soil layers become weaker as intrusion and cutting action of sand boiling from the liquefied layer. This may also be regarded as a softening phenomenon but not under dynamic loading condition. It could reduce the retraint action and enhance the seismic settlement.

OUTLINE OF EVALUATION METHOD

In this country, authors (J. F. Xie and Z. J. Shi, 1981) are the earlist to have made a computation of seismic settlement caused by liquefaction softening, and have conducted a series of testing on the seismic settlement characteristics of silts for establishing empirical relationships. Based on the so called "Softening model" concept they compiled a computer program for evaluating settlement of that kind, and succeded in applying it to predict the settlement of Shanghai underground under the locomotive vibration during the coming operation time (S. S. Yu et al, 1986). Recently, the reliability of the method has been tested through a large-scale settlement analysis of Wuanghailo residential district, Tianjin (Z. J. Shi, et al, 1987).

COMPUTATION METHOD

1. Concept of Softening Model

Fig. 1 shows the model in which the stiffness of soil consists of two parts, namely

$$K_{1p} = \frac{1}{\frac{1}{K_1} + \frac{1}{K_p}} \tag{1}$$

Before earthquake the initial displacement u_i is governed by unit A because $K_p \gg K_i$ and $K_{ip} \approx K_i$. Under earthquake loading $f_p(t)$, K_i keeps constant and K_p is decreasing which means that an unrecoverable displacement u_p would be governed by unit B with the total stiffness k_{ip} decreased. Therefore, the concept of softening model is that soils become soften, under repeated earthquake loading action, represented by the reduction of stiffness. In two-dimensional FEM the stiffness in Eq. (1) is replaced by the deformation modulus so that

$$E_{1p} = \frac{1}{\frac{1}{E_1} + \frac{1}{E_p}}, \qquad E_p = \frac{\sigma_d}{\varepsilon_p} \tag{2}$$

where E_{ip} - deformation modulus associated with softening, E_i-initial modulus, E_p - psuedo secant molulus, σ_d -dynamic stress, ε_p- residual strain, determined by experiment.

The settlement analysis consists of two sets of static FEM computation. In the first one use of E_i is made and in the second, E_{ip} instead. The displacement difference between the results of these two computation is regarded as the seismic settlement.

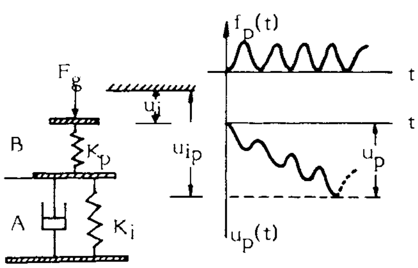

Fig. 1 Stetch of softening mode

2. Computation of Dynamic Response

In the seismic settlement analysis the residual strain must be determined by the dynamic stress in elements. Thus, a dynamic FEM analysis has to be proceeded as sketched in Fig.2. Earthquake accelerogram A corresponding to a given intensity is selected first, and then the accelerogram B at bedrock or computational bedrock is obtained by the inverse transfer technique. Using B as the input and making dynamic FEM analysis for the soil and building system, dynamic stress in soil elements may be obtained. The invese thransfer program of the authors has been used in the analysis (J. F. Xie and Z. J. Shi, 1981).

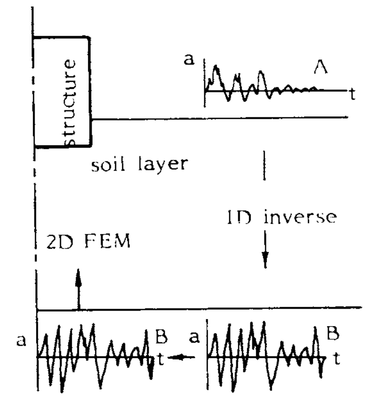

Fig. 2 Computation procedure

3. Mesh Division

As the system is symmetric so only a half of it is taken where there are 10 elements for the structure ans 340 elements for soils. Results of computation analysis available have shown (Z. J. Shi et al, 1987) that a maximum soil depth that effects the settlement of building on natural foundation soils is generally two times of the building breadth. Therefore 50 m in depth and 100 m in breadth of soils are taken to minimize the end influence. The stiffness of structure elements is taken the same, equal to 6×10^4 KPa. The equvalent unit weight of structure elements which depends on the bearing capacity of soils is taken different for different cases to be analized. The height of building is taken as the practical height if it is known or otherwise the height of four storey buildings.

4. Selection of Seismic Ground Motion History

Three ground motion histories are selected according to the following proinciples.

1. The recorded station should be as close as possible to the site of practical example to be analized. 2. Do best to select the very strong motion record or aftershock record which causes the damages to the building to be analized.

In Table 1 three records selected are listed.

Table 1 Selected ground motion history

Site of station	Date	Earthquake	Magnitude	Oritation	Peak accel. (gal)
Tangshan airport	1976,7,31	Tangshan aftershock	5.4	West-East	33.7
Tianjin hospital	1976,11,15	Ningho earthquake	6.9	North-South	149.98
Yingko great hose	1975,2,12	Haicheng aftershock	5.3	North-South	20.8

5. Determination of Soil Parameters

Because of not possible to perform static and dynamic soil test for the several tens of cases selected the parameters needed in this study are determined from the existed data empirical expression, namely as follows:

(1) Doncan's paprameter

The secant modulus E_s of soil element in static FEM analysis is determined by Eq. 3.

$$E_s = K_s(\sigma_3)^{ns} \left[1 - \frac{Rf\ (1 - \sin\phi)\ (\sigma_1 - \sigma_3)}{2 \cdot C \cdot \cos\phi + 2\ \sigma_3\ \sin\phi} \right] \quad (3)$$

where $\sigma_1 . \sigma_3$ -minimum and maximum principal stresses in soil element, respectively, C- cohesion of soil, ϕ - internal friction angle, K_s - maximum static secant modulus, Rf, n_s - constants.

(2) Seismic settlement parameter

The following empirical expression are used

$$\varepsilon_p = 10 \left[\frac{\sigma_d}{\sigma_3} \frac{1}{C_5} \right]^{\frac{1}{s_5}} \left(\frac{N}{10} \right)^{-\frac{s_1}{s_5}}$$

$$C_5 = C_6 + S_6(K - 1) \quad (4)$$

$$S_5 = C_7 + S_7(K - 1)$$

where N - number of cycles, taken to be 20 in computations, S_1, C_6, C_7 and S_7 - test parameters.

(3) Density of soils

If the dry weight of silts is less than 1.6 ton/m^3 they are in loose state and otherwise they are in dense state. The density of sand may also be determined by standard penetration blow count and shear wave velocity. Density of sands may be evaluated by the following relation

$$D = \left(\frac{N_{63.5}}{5.22\sigma_v'} \right)^{1/2} \quad (5)$$

where σ_v - effective overburden pressure.

(4) Shear wave velocity (V_s) versus depth (z)

The maximum shear modulus of soil needed in dynamic response analysis can usually be deduced from the shear wave velocity. In this study the shear wave velocity is evaluated by expression

$$V_s = A + B\ Z \quad (6)$$

for Tianjin area, where A and B are test coefficients varied with soil type,-and for other area by expression.

$$V_s = 75.^{67}\ N_{63.5}^{0.1436}\ Z^{0.2563} \quad (7)$$

for other area.

COMPUTATION RESULTS AND DISCUSSION

In this study, computed are 33 cases, of which 3, 26, and 4 in order located at area of intensity 7, 8, and 9. Some viewpoints may be formed after a detailed analysis is made of the computation results obtained.

1. The computation results obtained by use of the method presented in this paper are quite well close to that of practical situations both qualitatively and quantitatively. Some typical results are shown in Table 2, it can be noted that if the computed seismic settlement is less than 3 to 4 cm it can be neglected that is to say the foundation soil is sound enough, without settlement and fracture, because such small settlement does not endanger oridinaly buildings.

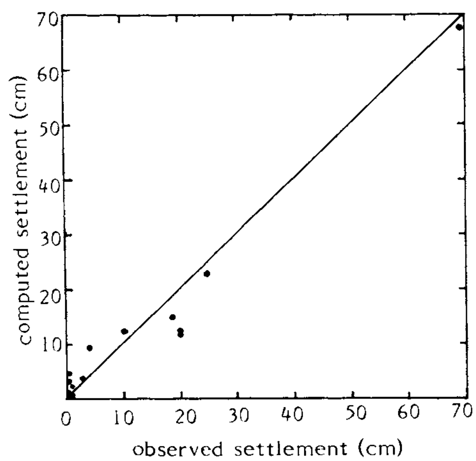

Fig. 3 Computed and observed steelement

2. The seismic settlement due to liquefaction depends not only on the buried depth, thickness and intensity of the liquefaction layer but also on the characteristics of non-liquefable layers and upper buildings. The liquefaction potential index, P_L can not represent quantitatively the damages extent of building and sometimes even give wrong information. For example, Shihuajian area has P_L of which is within slight and medium damages extent but the actual damage in the area belonged to serious damages extent, where units of 4 to 5 story residential building commonly settled 20 to 30 cm, with maximum of 38 cm and all inclined. And the computed settlement of 5 story building by using the method given in this paper is 36.6 cm close to the actually measured value.

3. The characteristics of input ground motion affects the quantity of seismic settlement. For comparision the seismic settlement of building in Yingko area are computed under two different earthquake records, which are shown in Table 2. It can been seen that twofold difference may reach for some buildings.

Table 2 Computed Seismic Settlement under Different Input Ground Motion

Input ground motion	Yingko glass fiber plant	Yingko paper plant, club	Pangjin chem fetilizer plant
No.3 Earthquake	8.2	5.5	6.0
Record at panlung Mt	3.7	3.8	3.3

4. Pressure on foudation base affects the settlement. This is concluded from the settlement tests of soil specimen under vibration, which showed that the initial deviator

stress in specimen was primary factor influencing the settlement. Although a quite large vibrational deformation occurred also in isotropic consolidated specimen under dynamic loading condition, almost no residual deformation was caused after removal of the dynamic loading. In other words, the dynamic loading made soil soften but not seismic settlement which could only be resulted when a static deviator stress was existed at the same time. It can be predicted, therefore, that pressure on foundation base will also affect the quantity of seismic settlement, the greater the pressure, the greater being the settlement. Fig. 4 gives a part of computed results for Tianjin area where the allowable bearing capacity all equals 12 t/m^2 The settlement of 4 story buildings are computed under base pressure of 8.4 and 12 t/m^2, respectively. It is seen from the figure that the settlement is decreased as the base pressure decreases. The extent of decreament of settlement depends on locations and soil properties.

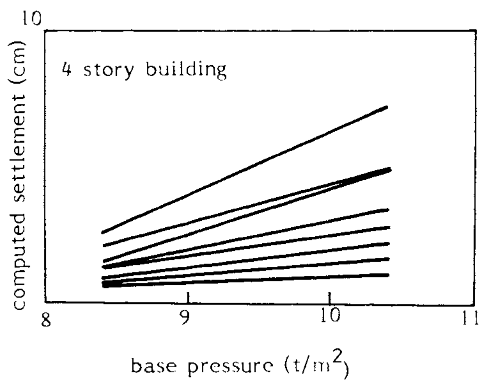

Fig. 4 Computed settlement and base pressure

5. Height of buildings affects the settlement. The heigher the building, the larger is the settlement. Results of Fig. 5 confirmed this trend.

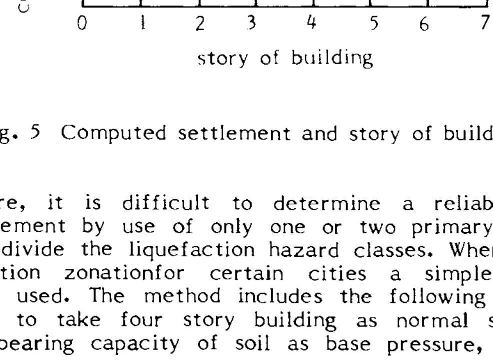

Fig. 5 Computed settlement and story of building

Therefore, it is difficult to determine a reliable range of settlement by use of only one or two primary factors, and to divide the liquefaction hazard classes. When making liquefaction zonationfor certain cities a simple method may be used. The method includes the following contents namely, to take four story building as normal structure, actual bearing capacity of soil as base pressure, to make input earthquake time history based on earthquake risk analysis and artificial earthquake wave composition technique.The relationship between liquefaction class and liquefaction damage extent presented in Table 3 may be used.

For more important buildings the entire settlement analysis procedures presented in this paper must be conducted rigorously where the static and dynamic properties of each layer must be determined by experiment, and the building height as well as the base pressure should use the practical value.

Table 3 Liquefaction class and hazard extent

No	Computed seismic settlement(cm)	Liquefaction class	Liquefaction damage extent
I	< 4	slight	no base failure
II	4-8	medium	crack in building
III	> 8	serious	serious crack in building, non-uniform settlement and inclination

CONCLUSION

A. Seismic settlement due to liquefaction is caused by a variety of factors among which softening is the main cause of damages.

B. The procedure presented in this paper including a set of testing and analysis gives results in good agreement with practical observed ones. It has been shown that the seismic settlement of buildings depended of four main factors, namely, liquefied layer, non-liquefied layer, characteristics of building and of input earthquake record.

C. For important buildings the more rigorous procedure presented in this paper should be followed while for liquefaction zonation and damage prediction the simplified procedure may be used.

REFERENCES

1. Liu H. S. et al., Liquefaction Hazard Analysis for Sites, EEEV, No. 4, 1984.

2. Shi Zhaoji, Yu Shoushun and Wuang Lunnian, Computational Analaysis of Seismic Settlement for Tanggou New Harbour, Res. Report, IEM, 1987.

3. Toshio Iwasaki, Tadashi Akawa et al., Simplified Procedure for Assesing Soil Liquefaction during Earthquake, Soil Dynamics and Earthquake Engineering Conference, 1984

4. Xie J. F. and Shi Z. J., Priliminary Evaluation of Seismic Settlement for Foundation Soil of Shento Power Plant, 1981.

5. Xie Junfei, Shi Zhaoji, Inversion of Ground Motion and Its Allication to Earthquake Hazard Analysis, EEEV, No. 2, 1981.

6. Yu Shaoshung, Shi Zhaoji, Xie Junfei, and Feng Wanling, Computational analysis of Vibration Settlement for Shanghai Underground, EEEV, No. 1, 1986.