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A SIMPLIFIED METHOD FOR EVALUATING EARTHQUAKE-INDUCED DIFFERENTIAL SETTLEMENTS OF BUILDINGS ON COHESIVE SUBSOIL

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SUMMARY

In terms of the simple physical feature of the dynamic soil structure interaction, a simplified procedure for calculating the earthquake-induced differential settlement of buildings on natural cohesive subsoil is presented. In the procedure the irregularity of the seismic ground motion and the different distribution of vertical dynamic stresses on the two sides of the building bottom due to the irregular seismic shaking is considered. Instead of FEM the simple method basing on the layer-wise summation is used for calculating the settlement of the subsoil in building existence. Also, the cone model of Meek and wolf is employed to calculate the dynamic stress distribution below the building. Also, the residual strain model of soils under irregular loading and the progressively modified modulus approach are employed to calculate the permanent deformation of soils. The simplified method can consider the combined effects of the soil, structure and seismic wave more reasonably and can describe the process of the differential settlements of the buildings. The comparison results with large shaking table tests and the investigation data of the post earthquake indicate that the new simplified method is reliable and easy to use.

Keywords: Differential settlement; Simplified procedure; Building; Subsoil; Soil structure interaction

INTRODUCTION

During earthquakes, the differential permanent displacements of subsoil usually make structures lose function in service. In the Tangshan Earthquake of China in 1978, for example, many apartment buildings on the soft cohesive subsoil at Tanggu area near Tianjin city have damaged seriously because of the inclination and cracks of the buildings due to the obviously uneven subsidence of the subsoil during the earthquake shock.

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At present, the versatile analytical tool for dynamic analyses is the finite element technique. It was first applied to the dynamic response analyses of earth dam (Clough and Chopra, 1966). Furthermore, the most effective methods currently available for analyzing the large permanent deformation of soils in engineering perhaps are the modified modulus approach and the equivalent nodal point force approach (Lee, 1974; Martin and Seed, 1978). The modified modulus approach is based on the concept that the effect of the dynamic shock is a result of softening of the modulus of material and the permanent deformation of soils is due to the reduced modulus under gravity loads. The equivalent nodal point force approach is that the effect of the earthquake can be represented by a series of equivalent static forces producing the additional permanent displacement in a soil element. In two approaches, the residual strain potential of soils is a key point in determination of the reduced modulus or the equivalent nodal force.

In the previous study (Shi et al, 1988), the finite element analysis considering the soil-structure interaction and the reduced modulus approach are used for simulating the settlements of building and subsoil at Tanggu area during the Tangshan Earthquake of China, 1976. They have obtained good results in estimation of the average settlements of subsoil and buildings. From the calculation, however, the significant differential settlements as the post earthquake investigation can not be obtained even when the asymmetry of the buildings and the transverse non-uniform distribution of the subsoil below the building are supposed to be quite obvious ones.

Generally, the earthquake-induced differential settlement of the buildings on natural subsoil basically depends on three factors: the wave type of seismic ground motion, the property and distribution of soil layer below the building and the weight distribution of the building and foundation. Recently, the effect of asymmetry and irregularity of the inputted seismic waves on the earthquake-induced differential settlement of the buildings on natural subsoil is investigated by Yuan et al (2003) in terms of the earthquake damage phenomena, theoretical analyses, dynamic triaxial tests and shaking table tests. The research has shown that the asymmetry and irregularity of the inputted seismic waves themselves can cause the remarkable differential settlement of the subsoil and structure in some cases and the asymmetrical and irregular character of the inputted seismic waves themselves is a necessary factor to be considered in reasonable evaluation for the problem of the earthquake-induced differential settlements. Other researches (Ishihara et al, 1973, 1984; Nagase et al, 1987) also show that the effect of the asymmetry and irregularity of the seismic loads is significant on the dynamic behavior of the soil in many cases, especially on the permanent deformation of the soft clay soil and the saturated sand.

The existing residual strain potential of soils in the modified modulus approach and the equivalent nodal point force approach, however, results from the triaxial tests of uniform stress cycles and the effect of the asymmetrical and irregular character of the inputted seismic waves themselves on the soil deformation will be neglected. Furthermore, if the seismic loading is turned to the sinusoidal loading of equal-amplitude, the difference of the vertical settlement on the two sides of subsoil and building due to the seismic shaking will vanish. As a result, the effects of the property and distribution of soil layer below the building and the weight distribution of the building and foundation, even when one side of subsoil below the building is soft and the other hard or the building is heavy on one side and light on the other side, on the permanent deformation of subsoil and the differential settlement of building due to the shaking can not be considered reasonably.

Although the finite element technique is a versatile tool, the efforts at modeling the simple approach are still made considering the application in engineering. In analysis of dynamic interaction of soil and structure, one of typical works is made by Wolf (1994). However, his method is mainly for building and foundation vibration analysis and can not give the permanent settlement evaluation of the building and foundation. By using the modified modulus approach, Yang et al (1997) present a simplified procedure to estimate the settlement of subsoil and building due to seismic shaking. Takada et al (1988) present an empirical formula for calculating the earthquake-induced subsidence of clay layer using the data from post earthquake investigation. However, these procedures are used in estimating the average settlement of subsoil and are not suitable for the differential settlement of subsoil and building due to earthquake motion.

BASIC PRINCIPLES OF THE PROCEDURE

In presenting a procedure, the first thing is to identify the generating mechanism of the earthquake-induced differential settlement of the buildings on the natural cohesive subsoil. From the knowledge available so far, three factors, the wave type of seismic ground motion, the property and distribution of soil layer below the building and the weight distribution of the building and foundation, should be considered in a proper way. In the analysis, the basic principles are:

(1) Divide the settlement into two parts as shown in Fig.1, S_T, the soil layer settlement without the dynamic interaction of soil-structure as well as S_{J1} and S_{J2}, the settlement of building resulting from the dynamic SSI. The settlement of the foundation and building in the paper is referred to the relative settlement, S_{J1} and S_{J2}, i.e. the settlement of the building considering dynamic SSI minus the settlement of soil layer without the dynamic SSI.

(2) Employ the modified modulus approach to obtain the settlements due to the seismic shock.

(3) Using the simplified method of Seed-Idriss (1971) to obtain the horizontal dynamic shear stress without dynamic SSI.

(4) Consider the basic feature of the dynamic stress histories on the two sides of subsoil below the building due to the seismic shock and its effect on the settlement. Take the effect of actual seismic history and the anisotropic property of soil on the developing of permanent deformation into account.

(5) Employ the cone model of Meek and Wolf (1992) to get the dynamic stresses of subsoil considering dynamic SSI.

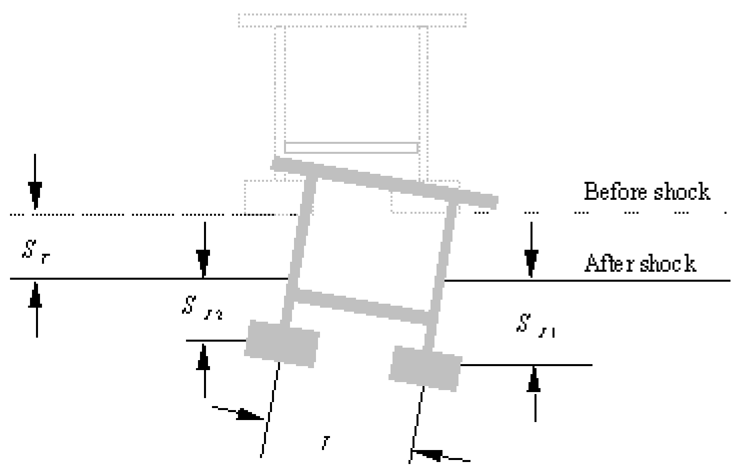


Fig 1 Settlements of building before and after earthquake

SOIL LAYER SETTLEMENT WITHOUT DYNAMIC SSI

In employing the modified modulus approach to obtain the settlement of the soil layer without considering the dynamic SSI due to the seismic shock, the static stress in soil layer in building existence and dynamic stress in soil layer without building are needed.

The stress from weight of the soil layer can be written as

$$\begin{cases} \sigma_{zn} = \sum_{i=1}^{n} \gamma_i \cdot h_i \\ \sigma_{xn} = k_0 \cdot \sigma_{zn} \end{cases} \tag{1}$$

where σ_{zn} and σ_{xn} are the vertical and horizontal positive stress of the soil layer n, separately, and k_0 is the lateral pressure coefficient of soils.

Considering existence of the building, the static additional stress can be expressed by the affecting coefficients as following

$$\begin{cases} \mu_z = 0.17 + 0.96 \cdot e^{-\left(\frac{z}{A}\right)} \\ \mu_x = -0.42 + 1.21 \cdot e^{-\left(\frac{z}{A}\right)} \end{cases} \tag{2}$$

where μ_z and μ_x are the vertical and horizontal affecting coefficients. Z is the depth of the soil layer and A is the width of the foundation.

The vertical and horizontal total stresses, σ_z and σ_x, can be written as

$$\begin{cases} \sigma_z = \sigma_{zn} + \mu_z \left(\frac{G}{A}\right) \\ \sigma_x = \sigma_{xn} + \mu_x \left(\frac{G}{A}\right) \end{cases} \tag{3}$$

where G is the weight per width from the building and foundation.

Using the simplified method of Seed-Idriss to obtain the horizontal dynamic shear stress in the soil layer without building. The dynamic shear stress in the soil layer n, τ_{tn}, can be expressed as

$$\begin{cases} \tau_{tn} = 0.65 \cdot \gamma_d \cdot \sum_{i=1}^{n} \gamma_i \cdot h_i \cdot \frac{a_{\max}}{g} \\ \gamma_d = 1 - 0.0133 \cdot Z \end{cases} \tag{4}$$

where $a_{\max}$ is the maximum acceleration of surface motion, and γ_d is the modified coefficient concerning with depth Z given by Seed-Idriss.

Assuming the earthquake-induced settlements of soil layer without dynamic SSI are uniform, the residual strain potential of soils can be calculated by (Yu et al, 1988)

$$\begin{cases} \varepsilon_{pn} = 10\left(\dfrac{\sigma_{dn}}{\sigma_{3n} \cdot C_{5n}}\right)^{\frac{1}{S_{5n}}}\left(\dfrac{N}{10}\right)^{-\frac{S_{1n}}{S_{5n}}} \\ \sigma_{dn} = \tau_{tn} \cdot \alpha \\ C_{5n} = C_{6n} + S_{6n}\left(K_{cn} - 1\right) \\ S_{5n} = C_{7n} + S_{7n}\left(K_{cn} - 1\right) \end{cases} \tag{5}$$

where, ε_{pn}: residual stain of the layer n; N: the equivalent cyclic number of seismic loading; α: the empirical coefficient; σ_{dn}: the vertical stress of the layer n; σ_{3n}: the confining stress of the layer n; K_{cn}: the consolidation ratio of the layer n; C_{6n} ¡¢ C_{7n} ¡¢ S_{6n} ¡¢ S_{7n} ¡¢ S_{1n}: the parameters of the layer n connected to soil type.

Using the Duncan-Chang model, the initial modulus of the soil layer before the earthquake can be expressed as

$$E_0 = K_s\left(\sigma_3\right)^{n_s}\left[1 - \frac{R_f\left(1 - \sin\phi\right)\left(\sigma_1 - \sigma_3\right)}{2C \cdot \cos\phi + 2\sigma_3 \cdot \sin\phi}\right] \tag{6}$$

In terms of the modified modulus approach, final modulus of the soil layer is given by

$$E_n = \frac{1}{\left(\dfrac{\varepsilon_{pn}}{\sigma_{dn}}\right) + \dfrac{1}{E_{0n}}} \tag{7}$$

From the following stress-strain relation

$$\varepsilon_z = \frac{1}{E}\left[\sigma_z\left(1 - \mu^2\right) - \mu\left(1 + \mu\right)\sigma_x\right] \tag{8}$$

the residual strain of the each soil layer is

$$\Delta\varepsilon_{zn} = \left[\frac{1}{E_n} - \frac{1}{E_0}\right]\left[\sigma_z\left(1 - \mu^2\right) - \mu\left(1 + \mu\right)\sigma_x\right] \tag{9}$$

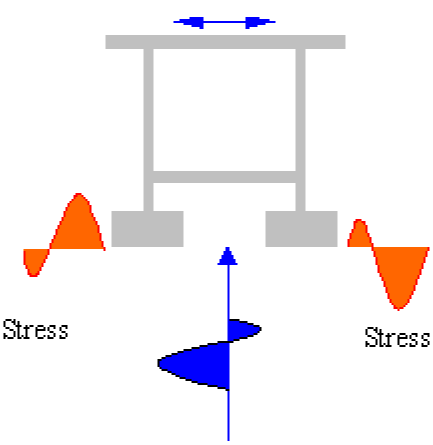
According to the layer-wise summation, the total settlement of the soils is constructed by

$$\begin{cases} \Delta S_n = \Delta\varepsilon_{zn} \cdot h_n \\ S_T = \displaystyle\sum_{n=1}^{m} \Delta S_n \end{cases} \tag{10}$$

where ΔS_n is the settlement in the layer n and m is the number of the soil layers.

SETTLEMENT DUE TO DYNAMIC SSI

Excited by the vertically propagating shear wave, the vertical stress histories on the two sides of subsoil below the building basically control the differential settlement of the buildings on natural subsoil due to the seismic shock, and this has been proved in the shaking table tests (Yuan et al, 2003). The form of the actual vertical stress histories on the two sides of subsoil is similar to the acceleration history on the surface nearby the building and the stress histories on the symmetrical two sides of the subsoil are asymmetrical as shown in Fig.2 when the soil layer and building are uniform. The asymmetrical behavior of the couple vertical stresses on the symmetrical two sides of the subsoil can be seen in the numerical calculation and the shaking table tests. Also, the asymmetrical distribution of the couple vertical stresses on the two sides of subsoil is approximately true when the soil layer or the building is non-uniform in a certain degree. Furthermore, The above tests and analysis show that the earthquake-induced differential settlement of subsoil and building is basically controlled by the couple vertical stress histories on the two sides of subsoil below the building.



Seismic ground acceleration

Fig 2 Sketch of relation between the surface ground acceleration and the vertical stress histories below the foundation of building

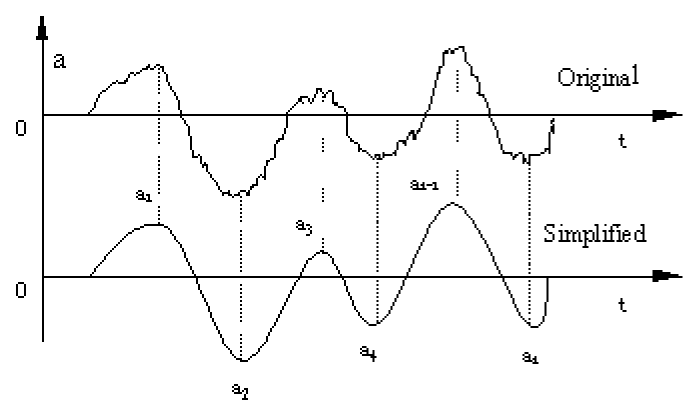


Fig 3 Simplification of the seismic acceleration

The random seismic acceleration on the surface can be simplified to a series of cyclic waves with different amplitudes as shown in Fig.3. The additional stresses resulting from the dynamic interaction of soil-structure can be obtained by the cone model of Meek and Wolf as shown in Fig.4. In the figure, Z_0^τ ¡¢ Z_0^σ are the height of shear and press cones determined by the Poison ratio and the foundation width, respectively, and b_0 is half width of the foundation.

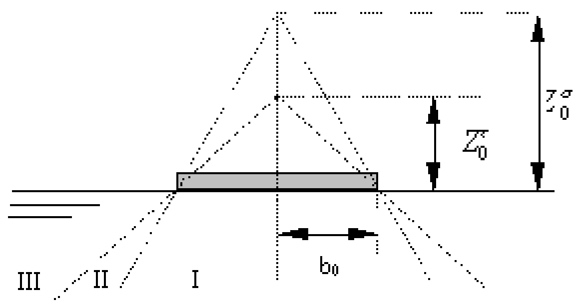


Fig 4 The cone model of Meek and Wolf

The horizontal shear stress in region ¢ñ is

$$\begin{cases} \tau_j = \left(\dfrac{Z_0^\tau}{Z_0^\tau + Z} \right) \dfrac{G}{A} \cdot \dfrac{a_{\max}}{g} \\ \tau_{ji} = \left(\dfrac{Z_0^\tau}{Z_0^\tau + Z} \right) \dfrac{G}{A} \cdot \dfrac{a_i}{g} \\ Z_0^\tau = \dfrac{\pi(2-\mu)}{8} \cdot \dfrac{A}{2} \end{cases} \tag{11}$$

where a_i is the acceleration amplitude of the *i*th cycle. The vertical positive stress in region ¢ñ is

$$\begin{cases} \sigma_j = \left(\dfrac{Z_0^\sigma}{Z_0^\sigma + Z} \right) \dfrac{G}{A} \cdot \dfrac{a_{\max}}{2 \cdot g} \\ \sigma_{ji} = \left(\dfrac{Z_0^\sigma}{Z_0^\sigma + Z} \right) \dfrac{G}{A} \cdot \dfrac{a_i}{2 \cdot g} \\ Z_0^\sigma = \pi(1-\mu) \left(\dfrac{C}{C_s} \right)^2 \cdot \dfrac{A}{8} \end{cases} \tag{12}$$

where

$$\left(\frac{C}{C_s}\right)^2 = \begin{cases} 3 & \mu < \frac{1}{3} \\ 4 & \frac{1}{3} \le \mu < \frac{1}{2} \end{cases} \tag{13}$$

The total dynamic stress σ_{din} in the layer n for the acceleration amplitude of the *i*th cycle can be expressed as

$$\sigma_{din} = \sigma_{ji} + \tau_{Sn} \cdot \alpha \tag{14}$$

where

$$\tau_{sn} = \tau_{ji} + \tau_{tni} \tag{15}$$

$$\begin{cases} \tau_{tni} = 0.65 \cdot \gamma_d \cdot \sum_{k=1}^{n} \gamma_k \cdot h_k \cdot \frac{a_i}{g} \\ \gamma_d = 1 - 0.0133 \cdot Z \end{cases} \tag{16}$$

The increment of the residual strain of soils under random earthquake loads can be obtained by (Yuan et al, 2003)

$$\varepsilon_{pin}^{i} = \varepsilon_{pin}^{i-1} + \Delta\varepsilon_{pin}^{i} \qquad i\text{=1 to } M \tag{17}$$

where

$$\Delta\varepsilon_{pin}^{i} = \left(\frac{\sigma_{din}}{\sigma_3 c_5}\right)^{\frac{1}{s_5}} \left(-\frac{s_1}{s_5}\right)\left(\frac{i-1}{10}\right)^{\left(-\frac{s_1}{s_5}-1\right)} \qquad i\text{=2 to } M \tag{18}$$

$$\Delta\varepsilon_{pin}^{1} = 10\left[\frac{1}{c_5} \cdot \frac{\sigma_{d1n}}{\sigma_3}\right]^{\frac{1}{s_5}} \left(\frac{1}{10}\right)^{-\frac{s_1}{s_5}} \tag{19}$$

where M is the total number of cycles. The modified modulus in the layer n for the acceleration amplitude of the *i*th cycle can be written as

$$E_{in} = \left(\frac{1}{E_0} + \sum_{i=1}^{M} \frac{\Delta\varepsilon_{pin}}{\sigma_{din}}\right)^{-1} \tag{20}$$

The settlement considering the dynamic SSI is attained by

$$\begin{cases} \Delta S_{in} = \Delta \varepsilon_{zin} \cdot h_n \\ S_J = \sum_{i=1}^{M} \sum_{n=1}^{m} \Delta S_{in} \end{cases} \tag{21}$$

$$\Delta \varepsilon_{zin} = \left(\frac{1}{E_{in}} - \frac{1}{E_{0n}} \right) \left[\sigma_{zn} \left(1 - \mu^2 \right) - \mu (1 + \mu) \sigma_{xn} \right] \tag{22}$$

where S_J is the final settlement of whole soil layers under the earthquake shock and ΔS_{in} is the settlement due to the dynamic stress σ_{din} in the layer n for the acceleration amplitude of the *i*th cycle.

DIFFERENTIAL SETTLEMENT OF SUBSOIL AND BUILDING

The settlements of the two foundations, S_{J1} and S_{J2}, can be calculated by using the asymmetrical couple vertical stresses on the sides of the subsoil according to the above procedure and the relative settlements of the two foundations to the settlement of soil layer are given separately by

$$\begin{cases} \Delta S_1 = S_{J1} - S_T \\ \Delta S_2 = S_{J2} - S_T \end{cases} \tag{23}$$

Finally, the differential settlement is obtained by

$$\Delta S = \left| \Delta S_1 - \Delta S_2 \right| \tag{24}$$

COMPARISON WITH THE SHAKING TABLE TESTS

To check the validation of the simplified method, the shaking table tests as shown in Figs.5 and 6 are conducted in the shaking table of 5m×5m at the Institute of Engineering Mechanics, CEA, China.

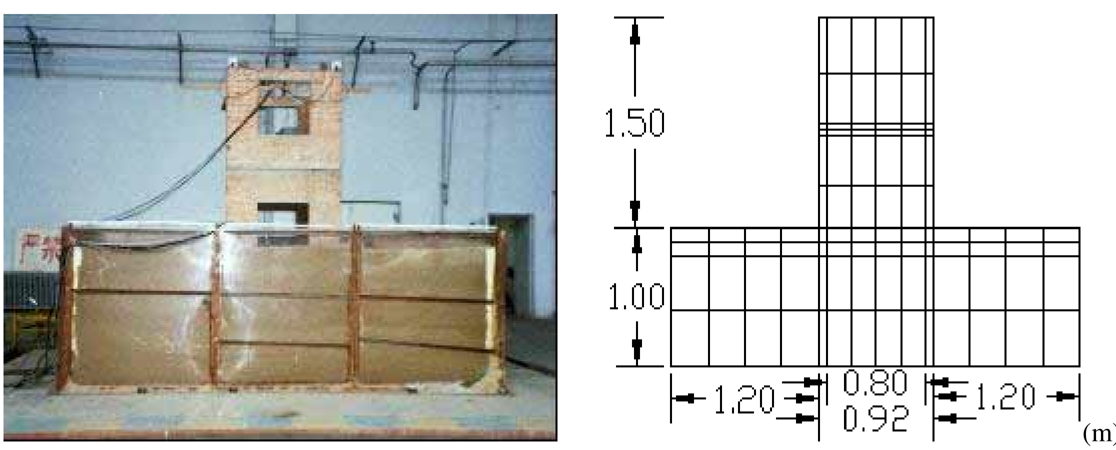
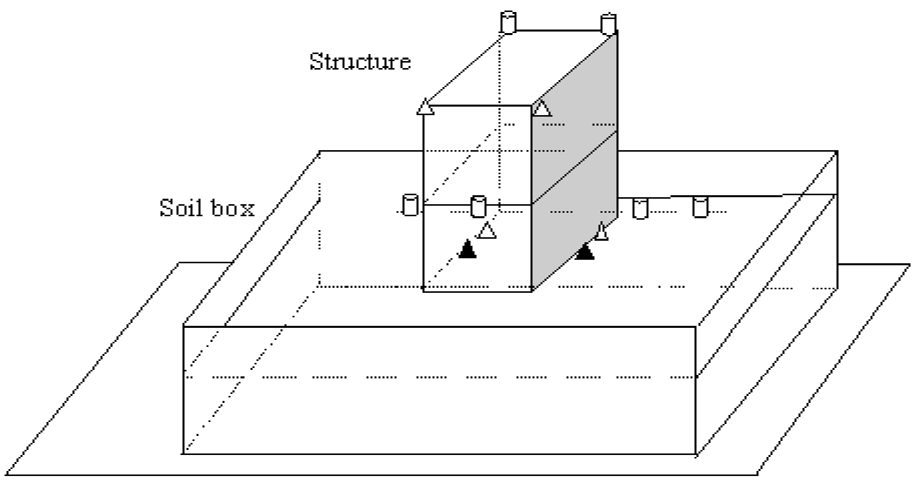


Fig.5 Sketch of the shaking table tests

The parameters of the Duncan-Chang and dynamic calculation of the subsoil as well as the building model in the tests are shown as Tables 1-3. The inputting waves used in the tests are shown as Fig.7 and Fig.8. First is the EL-Centro wave recorded in the Imperical Valley Earthquake of the Unite State of America in 1940 and second is the acceleration obtained acceleration record at the Tianjing Hospital during aftershock of the Tangshan Earthquake of China in 1976 (simply named as Tianjing wave). The amplitude of the peak amplitude of the two waves is adjusted to 0.2g and only one direction of horizontal shaking motion along the length is employed in the tests.



Vertical displacement meter ¡÷ Vertical accelerometer ¡ø Vertical pressure gauge

Fig.6 Model of soil-structure dynamic interaction in the shaking table tests

Table 1 The parameters of the Duncan-Chang model for the subsoil

K_a/kPa	n_s	φ	C/kPa	R_f
4800	0.5	32	0	0.84

Table 2 Parameters of settlements

S_1	C_6	S_6	C_7	S_7
-0.1	0.45	0.5	0.1	0.05

Table 3 The parameters of model

Foundation unit pressure /kN	Foundation width /m	Foundation depth /m	Subsoil depth /m
1.67	0.07	0.1	1.2

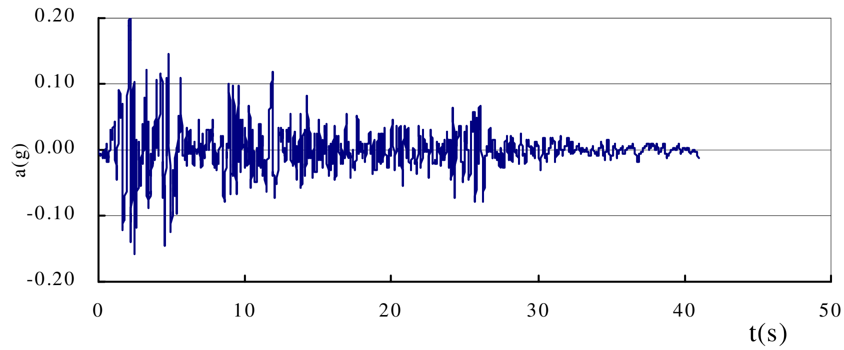


Fig.7 The EL-Centro wave used in the shaking table tests

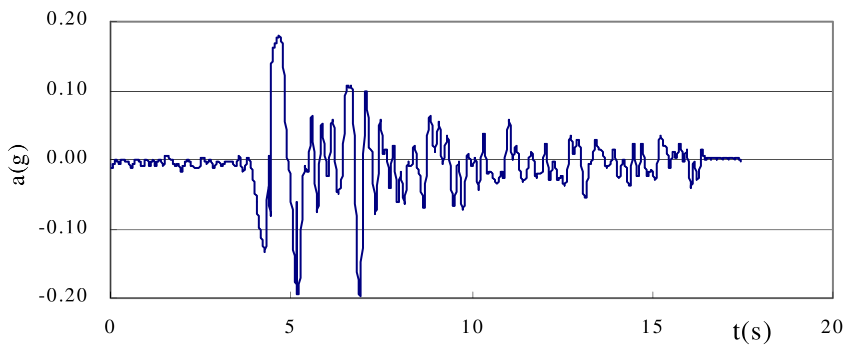


Fig.8 The Tianjing wave used in the shaking table tests

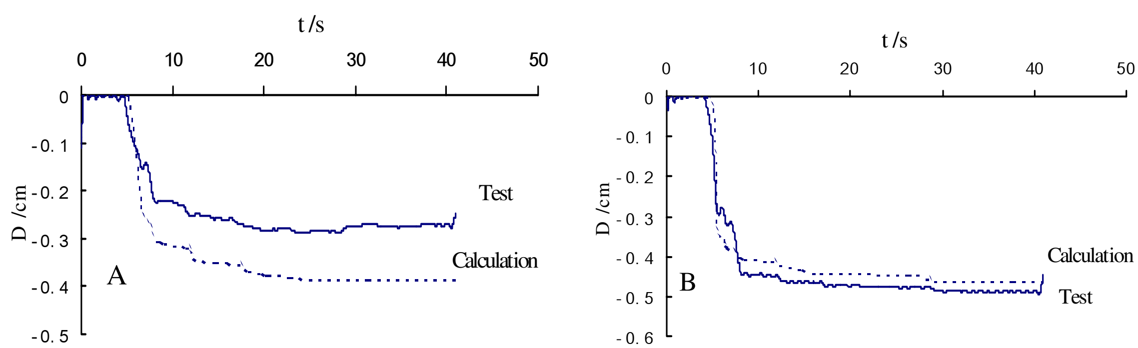


Fig.9 The calculated and tested permanent displacements on two sides of the building for incidence of the EL-Centro wave

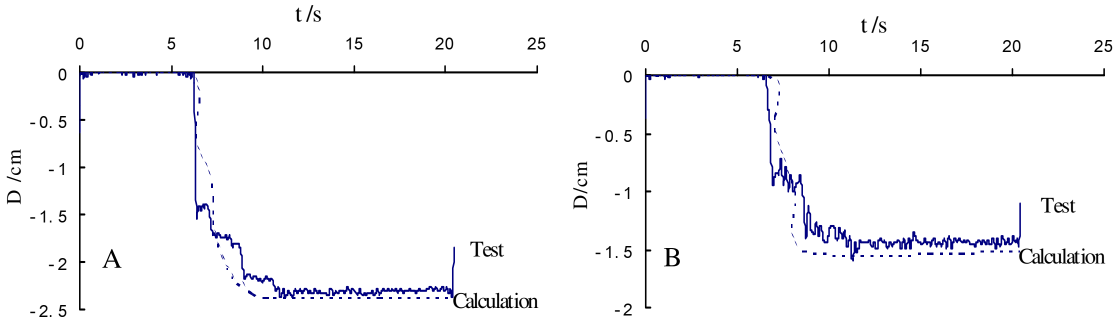


Fig.10 The calculated and tested permanent displacements on two sides of the building for incidence of the Tianjing wave

The comparison results between the calculation based on the simplified procedure and the tests are illustrated in Figs.9-10 and Table 4, in which the A and B separately represent record locations of the permanent displacements on one side and the other side of the bottom the structure in the tests. The comparison results show that the calculated inclination directions of the building are the same as the tested ones. For excitation of the EL-Centro wave, the inclinations are both to the side B and for excitation of the Tianjing wave, are both to the side A. Also, the settlements from calculation and tests are in the same dimension and, the histories and the time of appearing of the obvious displacements from the calculation and tests are similar. Therefore, the calculated differential settlements are quite agreeable with the tested results in the general trend.

Table 4 The comparison of the calculated and tested permanent displacements

Inputting wave	EL-Centro wave		Tianjing wave	
a_{max}/g	0.2		0.2	
Location	A	B	A	B
Test/cm	0.28	0.50	1.45	2.30
Calculation/cm	0.37	0.47	1.50	2.36

COMPARISON WITH THE DAMAGE PHENOMENA IN THE TANGSHAN EARTHQUAKE

As the shaking table is the scaled model tests, the values of displacements are not enough to form the large permanent deformation of soils. To check the efficiency of the simplified method in the practical problem, the investigation data on the apartment buildings on the soft cohesive subsoil at Tanggu area near Tianjin city in the Tangshan Earthquake of China in 1978 are compared to the calculation.

The typical soil layer and the parameters at Tanggu area are listed in Table 5 and Table 6. The typical building at Tanggu area generally has 9m width and 1m depth of the raft foundation and leads 120kN pressure per length to the bottom. There is no earthquake record for the main shock at Tanggu area. The horizontal acceleration record of NS component of the main shock available at Beijing Hotel is used here as shown in Fig.11 because Beijing Hotel and Tanggu area are both located in the west of the epicenter of the earthquake. The peak amplitude of the incident acceleration is adjusted to 0.2g, the same seismic density at Tanggu area in the Tangshan earthquake.

Table 5 The distribution and static parameters of the soil layer at Tanggu area

Soil type	Depth/m	K_S/kPa	n_S	φ	C/kPa	R_f
Silt clay	1¡«3	152	1.0	23.8	5	0.364
Mucky soil	3¡«6	396	0.729	22.9	20	0.402
Mucky soil	6¡«10	1237	0.465	23.2	38	0.478
Muck	10¡«50	422	0.665	21.5	10	0.447

Table 6 The dynamic parameters of the soil layer at Tanggu area

Soil type	S_1	C_6	C_7	S_6	S_7
Silt clay	-0.128	0.61	0.186	0.208	0.0
Mucky soil	-0.145	0.53	0.178	0.240	0.0
Mucky soil	-0.203	0.38	0.150	0.350	0.0
Muck	-0.159	0.49	0.160	0.220	0.0

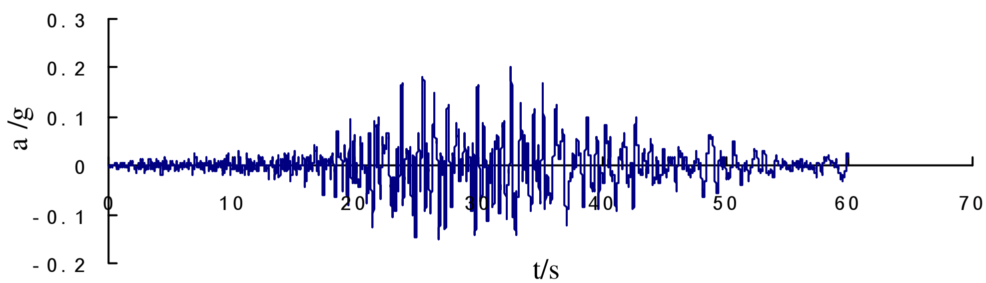


Fig.11 The Beijing Hotel wave used for calculation

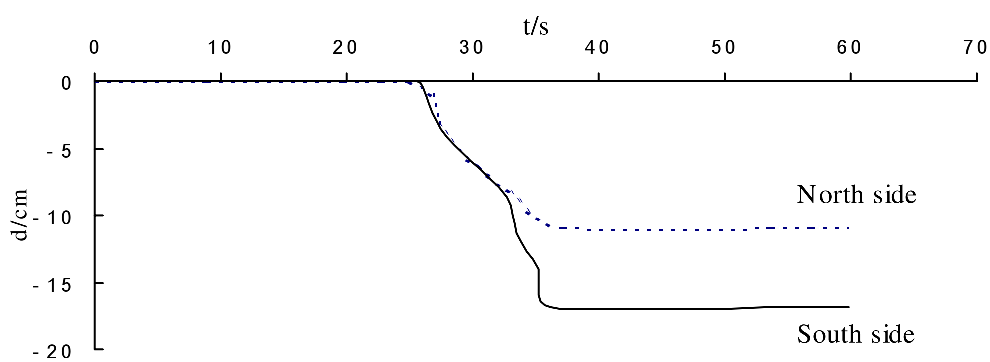


Fig.12 The calculated differential settlement of the building at Tanggu area

The calculated results based on the simplified procedure are illustrated in Fig. 12. The results between the calculation and the investigation data for the actual situation are agreeable in a great extent. The calculated inclination direction of the building is the same as the actual one, to the south side. Also, the settlement from calculation and tests is in the same dimension and the calculated differential settlement is about 6 cm, near to the actual value of 8 cm.

CONCLUSIONS

A simplified procedure for calculating the earthquake-induced differential settlement of buildings on natural cohesive subsoil is presented. The procedure can consider the dynamic interaction of soil-structure and the action of the actual seismic ground motion. The comparison results with large shaking table tests and the investigation data of the post earthquake indicate that the new simplified method is reliable. The simplified procedure has features of simple principle, engineering accuracy and is easy to use.

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