EARTHQUAKE INDUCED RESIDUAL SETTLEMENTS OF FOUNDATIONS

U. Holzlöhner(I)

SUMMARY

Model foundations were loaded dynamically in order to investigate the residual settlements induced by earthquakes. The settlements increase logarithmically with the number of load cycles. Other variables considered were contact area of the foundation, static and dynamic load, frequency, void ratio and moisture content of a well-graded gravelly sand. The settlements to be expected due to earthquakes can be found by means of model tests.

INTRODUCTION

Earthquake is known to induce large settlements of foundations. If a structure is founded on loose, watersaturated sand or silt liquefaction is likely to occur. In other cases, failure will also occur at a lower level under dynamic and static load acting simultaneously than under purely static load. This has been experimentally shown by Stavnitser and Karpenko (1). However, there may be considerable residual settlements too, if the loads are far too small to produce failure. The settlements gradually increase with repetitional loading and lead to damages, especially in the case of rigid structures.

TEST PROCEDURE

In order to determine the settlement due to vibration, model foundations at different scales were loaded directly. The tests were conducted at open air and at a smaller scale in the laboratory. At the site, a pit was excavated which formed a spherical calotte of 10 m in diameter and 3 m in depth. The moist, well-graded gravelly sand was filled in by a dredger and was weakly compacted to reach a medium relative density. As the filling followed always the same process, the sand homogeneously settled at a density which could be reproduced. This was controlled by penetration tests and by the sand replacement method. A model foundation is placed at the center of the circular surface of the refilled excavation. The largest diameter of the contact area is 0.75 m. For this size, the vibration of the foundation is not affected by the reflexions from the adjacent soil.

(I) Bundesanstalt für Materialprüfung, Berlin (West), GERMANY

An eccentric-mass vibrator attached to the foundation excites the foundation to vertical translatoric vibrations, the displacement amplitude of which and the phase angle between force and displacement being measured. Using the recorded values, the amplitude of the dynamic force in the contact area can be calculated. By means of high-precision levelling the vertical displacement of the foundation is measured after duration time intervals of 1 min, 2 min, 4 min, etc. In the case of the laboratory tests, the settlement was continuously recorded.

TEST RESULTS

The measured residual settlement can be represented as a function of the number of load cycles n by

$$s = S \ln \left(\frac{n}{n_o} + 1 \right) \qquad (1)$$

where n_o and S, the logarithmic rate of settlement, serve as parameters. Though up to 10^6 load repetitions have been applied, the settlements still continued. A number of authors have reported a similar semi-logarithmic dependancy. The settlements observed by Brumund and Leonards (2) tended to stop entirely at 10^5 cycles whereas the tests conducted by Tschebotarioff (3) exhibited no ultimate residual settlement. The parameter n_o obviously is dependent on the strain history of the sand. The values of n_o were found to be about 500 to 1000. The test results were not sufficient to quantify the dependancy of n_o on the test parameters. However, the influence of different parameters on the logarithmic rate of settlement S could clearly be established.

In Figure 1, S divided by the radius r of the contact area is plotted against the ratio of the dynamic force amplitude P_{dyn} to the static force P_{stat} in the contact area. Only the results of the in-situ tests are included which have been performed on moist sand of medium dry density, $\rho_t = 1{,}84\ Mg/m^3$. The rate of settlement increases with the square of P_{dyn}/P_{stat}. The rate of settlement increases also with increasing frequency parameter $\omega . r/v_R$, but more weakly. ω is the frequency and v_R the velocity of the Rayleigh waves. According to Tschebotarioff (3), the settlement does not depend on frequency. This result has been found, however, with very low values of the frequency parameter.

Another present test series showed that S increased with increasing parameter $P_{stat}/(\rho_s g r^3)$. Here, g denotes acceleration of gravity and ρ_s specific density of the sand.

The complete set of model parameters is selected to be:

$$\frac{S}{r}, \frac{P_{dyn}}{P_{stat}}, \frac{\omega . r}{v_R}, \frac{P_{stat}}{\rho_s g r^3}, \xi, \epsilon, \frac{E_{dyn}}{\rho_s g r}, \frac{c}{\rho_s g r} \qquad (2)$$

The symbols ξ, ϵ, E_{dyn} and c, respectively, denote granulometry, void ratio, dynamic modulus of elasticity and apparent cohesion. The dynamic modulus is taken to be independent on the scale though E_{dyn} is slightly affected by the static stress distribution which does not vary similarly. The apparent cohesion c represents the influence of the moisture of the sand, whereas the void ratio ϵ and the granulometry ξ describe the geometric properties of the particles and their mutual spatial arrangement. All the tests, the results of which have been represented in Figure 1, were conducted on the same moist sand. If the scale is varied the last two parameters in (2) cannot be kept constant. They vary inversely proportional to the radius r of the contact area. In spite of that, the results obtained in different scales describe the same relationship, as shown in Figure 1. Consequently, only the ratio of the last two parameters of (2) can affect S/r. Conducting model tests, it is sufficient, therefore, to satisfy simultaneously the model laws:

$$\frac{S}{r}, \quad \frac{P_{dyn}}{P_{stat}}, \quad \frac{\omega . r}{v_R}, \quad \frac{P_{stat}}{\rho_s g r^3}, \quad \xi, \quad \epsilon, \quad \frac{c}{E_{dyn}} \tag{3}$$

This is possible even if the model tests are performed on the same soil on which the prototype shall be constructed. This case is important in practice.

Figure 2 shows the effect of the water content w and the void ratio ϵ on the rate of settlement. These tests have been performed on a confined sand volume of 1 m in height and 1.50 m in diameter. The low water content of 3 - 4 % produce a cohesion of 2.35 kN/m^2. Therefore, moist sand exhibits less settlement than dry one of the same density. No tests were performed on water-saturated or submerged sand. After Gol'dshtein et al. (4) and Tschebotarioff (3) the settlements would then be at least as large as those on dry sand of the same density.

Compaction of the soil immediately below the contact area could not be detected even when the residual settlement of the foundation was large. Meassurements of the settlements at the soil surface showed that only the surface very close to the contact area takes part in the displacements. From these two results, it must be concluded that the residual settlements are produced mostly by shear deformation in the vicinity of the contact area.

APPLICATION TO SETTLEMENTS DUE TO EARTHQUAKES

In the present series of tests, the foundations have been loaded directly whereas they are excited through the contact area by seismic forces in the case of an earthquake. If a rigid foundation is considered the stiffness of the soil can be represented by a 6 by 6 matrix [A] the elements of which being frequency dependant, see

Ref. (5). The force $\{P\}$ in the contact area is

$$\{P\} = [A]\ (\{U_B\} - \{U_o\}) \tag{4}$$

in which $\{U_B\}$ are the displacements of the foundation and $\{U_o\}$ the free-field displacements. The force in the contact area of a directly loaded foundation is related to the displacement $\{U\}$ by

$$P = [A]\ \{U\} \tag{5}$$

Consider two equal structures, one excited by an earthquake the other loaded directly in such a way that the forces in the contact areas are the same in both cases. Then Eqs. (4) and (5) yield:

$$\{U_B\} - \{U_o\} = \{U\} \tag{6}$$

If $\{U_o\}$ is small compared to $\{U_B\}$ then, obviously, both loading cases are nearly identical. If $\{U_o\}$ and $\{U_B\}$ are of the same order of magnitude, the difference $\{U_B\} - \{U_o\}$ will be of the same order because of the phase angle between the two displacements. In the soil close to the contact area, the strains, stresses and displacements will then be of the same order of magnitude in both loading cases. The stresses and displacements due to static loads are equal, anyway. Therefore, tests of the type described here are appropriate to predict the differential residual settlements due to earthquakes.

The following procedure is proposed. If the static and dynamic forces in the contact area are known, the second to the fourth parameters in (3) can be evaluated for the prototype. A model test is conducted with the same values for these parameters. The value S/r obtained at the model foundation is valid for the prototype, too. The parameter n_o can be determined by the test also. Though the soil in seismic areas has been subjected already to a large number of load cycles, a new stress history begins after the construction of the structure because now larger static and dynamic shear stresses occur in the top soil layer than before.

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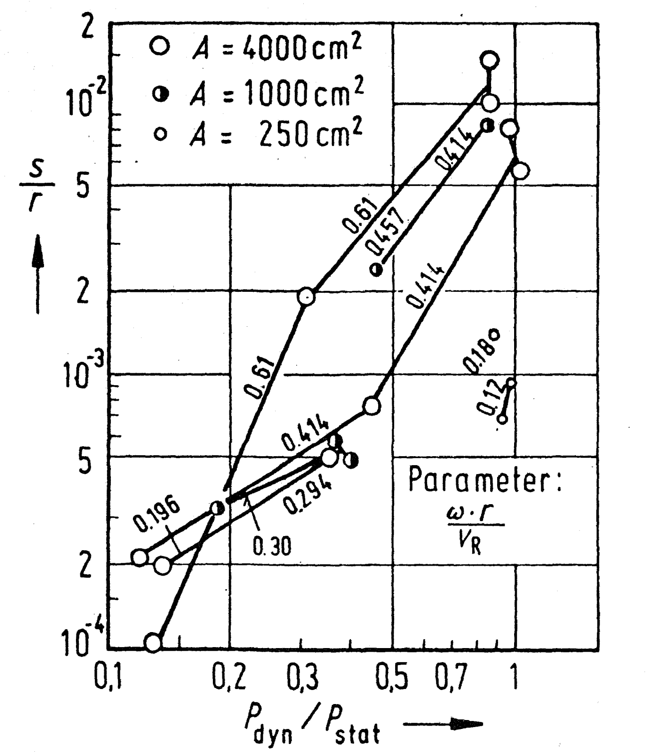

Figure 1. Logarithmic rate of settlement divided by the radius of the contact area vs. ratio of the forces in the contact area (A = contact area).

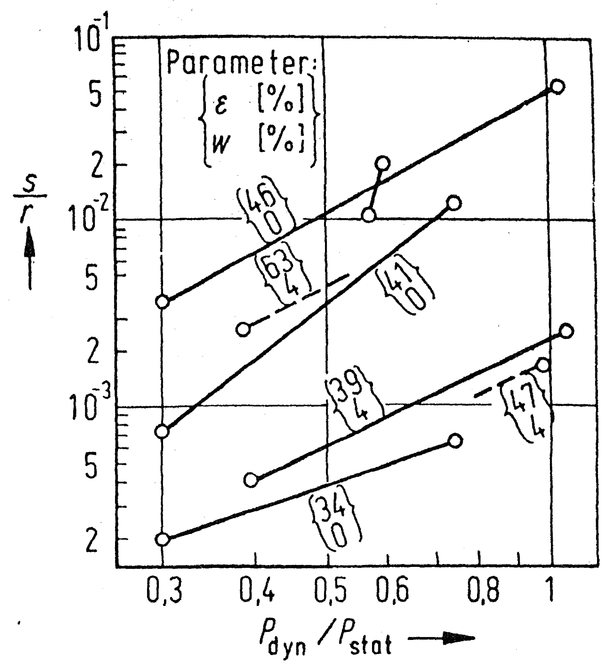

Figure 2. Effect of void ratio and water content on logarithmic rate of settlement.

DISCUSSION

S.C. Sharda (India)

Will these observations and conclusions by the author be applicable to the two layer system say the upper layer being medium dense and the lower being loose ? If yes to what extent ?

Author's Closure

With regard to the question of Mr. Sharda, we wish to state that there are always difficulties arising with inhomogeneous or layered soil if model tests are to be applied. However, the method proposed here is applicable to a two layer system, at least in principle. The model tests then have to be conducted in such a way, that the ratio of the radius of the contact area to the depth of the upper layer has the same value in model test and prototype. That can be achieved either by removing part of the upper layer at the site of the model test or, if the thickness of the upper layer varies, by conducting the model test at a site where the layer has the appropriate depth.