Behaviour of Foundations in Saturated Sand during Earthquakes

by

Shamsher Prakash*

V. Chandrasekaran**

P. Nanda Kumaran**

Introduction

IN aseismic design of structures, vibration of the structure induces additional stresses, which are estimated by the designer. Mathematical models of structures have also been developed for stress analysis corresponding to a desired input ground motion. The structural members designed against these earthquake stresses are regarded as earthquake-proof. But a very important phenomenon, the soil-structure interaction is neglected in such an analysis. The problem is important since the structures, although structurally safe, may settle excessively, tilt and sink into the sub-soil. This important phenomenon was observed during the Nepal-Bihar earthquake of 1934 and more recently in Niigata earthquake of Japan, 1964 and Broach earthquake of Gujarat, 1970.

When a saturated sand mass is subjected to earthquakes, the passage of seismic waves cause pulsating stress applications resulting in readjustment of sand particles. The cyclic stress applications cause increase in pore pressures. If the pore pressure so developed equals the initial effective confining pressure, the resulting effective stresses and thus the strength of the sand are reduced to zero. The sand mass then behaves as a heavy liquid causing the sinking and tilting of structures (Figure 1) or floating up of buried boxlike structures (Figure 2). This is termed liquefaction of sand (Prakash and Gupta 1970).

Damage to engineering structures due to liquefaction has been observed in some of the past earthquakes such as New Madrid (1811), Kansu (1920), Bihar-Nepal (1934), Niigata (1964), Alaska (1964) and Broach (1970). The damage due to liquefaction is generally spread over wide areas as was the case for the Bihar-Nepal earthquake. (Plate 1 of Ref. 8)

Assessment of Liquefaction Potential

Engineers over the world have employed two different test procedures for the assessment of liquefaction potential of a sand. They are (1) large sample tests on a horizontal shaking platform and (2) small sample tests in an oscillatory shear box or a triaxial shear machine with

* Professor of Soil Dynamics, } School of Research and Training in Earthquake Engineering, University of Roorkee, Roorkee.

** Lecturers in Soil Dynamics,

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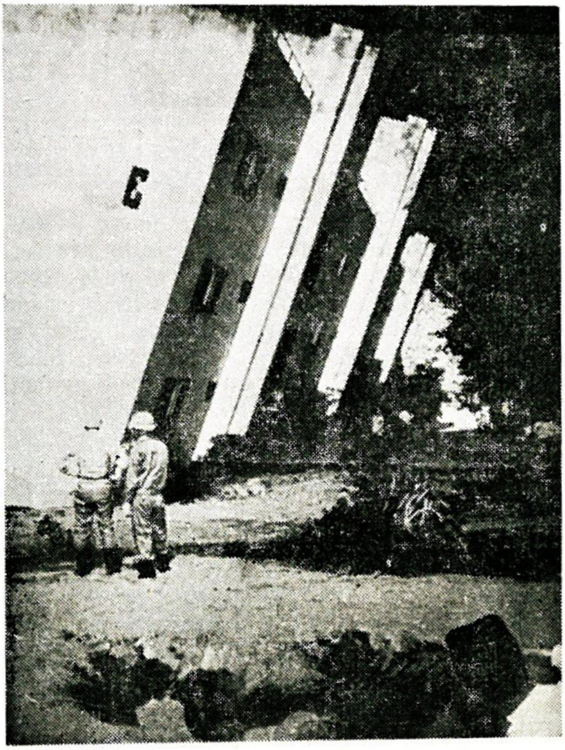

FIGURE 1 : A building tilted during Niigata earthquake.

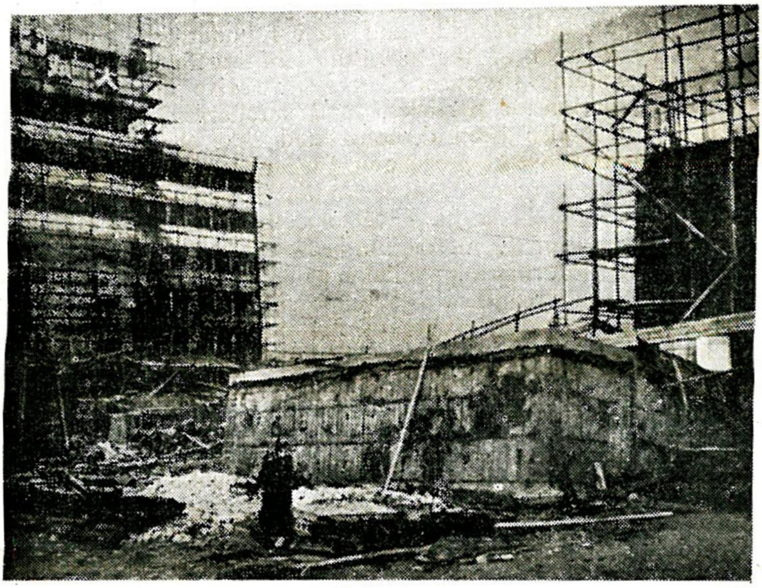

FIGURE 2 : A septic tank floated up—Niigata earthquake.

provision for application of pulsating stresses. In Japan, Russia, India and Canada the first named procedure is popular while in the USA the latter method is in vogue.

LARGE SAMPLE TESTS

The tests on large sized samples [Florin and Ivanov (1961), Yoshimi (1967), Maslov (1957), Prakash and Gupta (1970)] are generally conducted in the following manner. A saturated sand sample is prepared in a container fixed to a shaking platform. A surcharge is then applied to the sand surface corresponding to the confining pressure at the depth which the sample represents. The shaking platform is then vibrated at the desired frequency and acceleration for a desired period. (This table motion is usually a simiplified form of the erratic ground motion at the site). During and after the shaking, pore pressure measurements are made within the sample to obtain data on possible liquefaction.

SMALL SAMPLE TESTS

The development of apparatus for the application of pulsating stresses on soil samples paved the way for the study of liquefaction of sands using small sized samples [Seed and Lee (1966), Lee and Seed (1967)]. A complete investigation of these type requires the analysis of the sand mass subjected to earthquakes (Idriss and Seed 1968) to obtain data on the possible seismic stresses at various points in the sand deposit. The analysis is done by a lumped mass idealisation of the deposit and by choosing the dynamic properties of the soils on the basis of their variation with the values of strain developed at various segments. After the seismic stresses are obtained by means of this analysis, soil samples consolidated at the static stresses are subjected to this additional seismic stress and the loss in strength or the increase in pore pressure is measured.

Studies on large and small sized samples have provided sufficient data on the factors affecting the liquefaction potential of sand deposits. In the case of large sand deposits and dam foundations, therefore, the engineer can be warned of the possibility of liquefaction during earthquakes. In these studies the effect of *in situ* relative density, grain-size, grain shape, gradation of particles, and surcharge has been studied extensively. It has also been demonstrated that the properties of the sand mass can be improved by mechanical means to reduce the possibility of damage. But, unfortunately, the studies are almost exclusively on sand masses. None of the studies have correctly taken into account the very important aspect of the interaction between the soil and a structure resting on it. The information available so far can only be used to see if there is any loss in strength while, the possible settlement or tilt of the structure and the effect of types of foundation on the increase in pore pressure have not been investigated. Therefore, before examining how to take these factors into account, it need be examined how different types of foundations on saturated sand masses have behaved during earthquakes.

Behaviour of Foundations during Niigata Earthquake (Chandrasekaran 1967)

On 16 April 1964 Niigata (Japan) experienced a severe earthquake which was recorded at a magnitude of 7·5 on Richter's scale. The grounds of the Niigata plains around Niigata City and Shonai plains around Sakata City are thick alluvial deposits. They are 80 to 160 m thick in Niigata plains. The lower part of the alluvial deposit consists of many layers of silt and sand. When the earthquake occurred, a number of cracks and fissures appeared in the ground and several sand craters were formed. This obviously was because of the excess pore pressures developed in the alluvial deposit. Extensive damage to Civil Engineering structures in the form of subsidence and tilting occurred, mainly in the Niigata City.

In general, it was observed that buildings resting on the loose alluvial deposit had suffered damage while those resting on compacted ground (like oil refinery structures) were not damaged. This clearly points to the cause of the damage as liquefaction of the foundation soil.

The buildings had in general the following types of foundations :

(1) Shallow foundations

  (*a*) Isolated footings

  (*b*) Strip footings

  (*c*) Mat foundations

(2) Pile foundations.

Analysis of damages in Niigata City based on the types of foundations of various damaged buildings yielded a satisfactory correlation, thereby, indicating their importance. Table I shows the number of buildings analysed in a particular area and the damage of buildings with shallow and deep foundations (Kishida, 1966). It is apparent that the buildings having spread foundations suffered more damage than those having pile foundations.

TABLE I

Damage extent and type of foundation in Niigata Earthquake.

Type of Foundation	Damage	
	No and Slight	Intermediate and Heavy
Shallow Foundation (63 buildings)	36%	64%
Piled Foundation (122 buildings)	45%	55%

Among the buildings on shallow foundations, those on mat foundations suffered the least damage and buildings on strip footings seem to have performed slightly better than those on isolated footings. Damage to buildings on pile foundations depended on the strength of the soil at the tip of the pile and its embedded depth.

From the above observations it is apparent that the "foundation type" is equally important as compared to the strength of the soil in assessing damage to a building during an earthquake. From the analysis of damage during a particular earthquake information on qualitative behaviour of different types of foundations can be obtained, while from the test data on soils, the liquefaction potential of the soils can be ascertained. It is impossible at the present (1970) state of knowledge to account for the foundation–soil interaction during earthquakes. As a consequence, the current design procedures are largely empirical. In order to fill the gap in that direction, model studies of the following type are proposed.

Proposed Model Studies

To be representative of the prototype a model has essentially to satisfy the three conditions listed below (Clough and Pirtz 1958) :

(1) Similarity of lengths

(2) Similarity of forces

(3) Similarity of time periods.

The first condition calls for reduction of all linear dimensions by the scale ratio $\lambda\left(\frac{\text{Length of model}}{\text{Length of prototype}}=\frac{L_m}{L_p}\right)$.

The dead weight forces in the model and the prototype are in the ratio $\frac{\gamma_m}{\gamma_p}.\lambda^3$ where γ_m and γ_p are the unit weights of materials in the model and prototype respectively. Since no control is possible in this force once a geometrically similar model is desired, all the forces in the model and prototype must be in the same ratio for perfect similitude. Therefore it follows (Clough and Pirtz 1958).

$$\text{(1)}\quad \frac{\text{(Unit weight of fluid) model}}{\text{(Unit weight of fluid) prototype}}=\frac{\gamma_m}{\gamma_p} \qquad ...(1)$$

$$\text{(2)}\quad \frac{\text{(Acceleration) model}}{\text{(Acceleration) prototype}}=1 \qquad ...(2)$$

$$\text{(3)}\quad \frac{\text{(Time periods) model}}{\text{(Time periods) prototype}}=\sqrt{\lambda} \qquad ...(3)$$

Considering a pile foundation as a typical example, the relation between the elastic properties of materials in the model and prototype can be derived. For a laterally loaded long pile the ground deflection Y_g at a lateral load Q_g acting at the ground level is given by (Reese and Matlock 1956)

$$Y_g=\frac{Q_g\,T^3}{EI} \qquad ...(4)$$

where

T = Relative stiffness factor $=\sqrt[5]{\frac{EI}{n_h}}$.

E = Young's modulus of the pile material.

I = Moment of inertia of the pile section about the axis of bending.

n_h = rate of change of soil modulus with depth.

$$\frac{T_{model}}{T_{prototype}}=\sqrt[5]{\frac{E_m\,I_m\,(n_h)_p}{E_p\,I_p\,(n_h)_m}}$$

n_h has dimensions $F\,L^{-3}$

$$\therefore \quad \frac{(n_h)_m}{(n_h)_p} = \frac{F_m}{F_p}\left(\frac{L_p}{L_m}\right)^3 = \frac{\gamma_m}{\gamma_p}$$

$$\frac{I_m}{I_p} = \lambda^4$$

$$\frac{T_m}{T_p} = \sqrt[5]{\frac{\gamma_m\, E_m\, \lambda^4}{\gamma_p\, E_p}}$$

$$\frac{(Y_g)_m}{(Y_g)_p} = \frac{(Q_g)_m}{(Q_g)_p} \cdot \left(\frac{\gamma_m \;\; E_m}{\gamma_p \;\; E_p} . \lambda^4\right)^{\frac{3}{5}} \; \frac{E_p\, I_p}{E_m\, I_m} = \lambda \text{ For similarity}$$

$$\therefore \quad \lambda = \frac{\gamma_m}{\gamma_p}\; \lambda^3 \left(\frac{\gamma_m}{\gamma_p}\right)^{\frac{3}{5}} \left(\frac{E_m}{E_p}\right)^{-\frac{2}{5}} \lambda^{-\frac{12}{5}} . \frac{1}{\lambda^4}$$

$$= \left(\frac{\gamma_m}{\gamma_p}\right)^{\frac{8}{5}} . \lambda^{\frac{7}{5}} \left(\frac{E_m}{E_p}\right)^{-\frac{2}{5}}$$

or

$$\frac{E_m}{E_p} = \left[\; \lambda^{-\frac{2}{5}} \left(\frac{\gamma_m}{\gamma_p}\right)^{-\frac{8}{5}} \;\right]^{-\frac{5}{2}}$$

$$= \lambda \left(\frac{\gamma_m}{\gamma_p}\right)^4 \qquad \ldots(5)$$

Thus the modulus of elasticity of the model pile should be related to that of the prototype in the ratio $\lambda\left(\frac{\gamma_m}{\gamma_p}\right)^4$. Now, for the shear strains in the soil for the model and prototype to be the same it can be shown the modulus of rigidity of the soil in the model and the prototype should be in the ratio $\frac{\gamma_m}{\gamma_p} . \lambda$. In order that the ratio of Youngs moduli for the foundation and shear moduli for the soil in the model and prototype be in the same ratio, $\frac{\gamma_m}{\gamma_p} = 1$, *i.e.*, the unit weights of the model and prototype materials needs be the same. This is not a difficult condition to achieve in practice.

Finally we have the forces connected with the ultimate strength of the soil to be simulated. Since ϕ is a non-dimensional quantity and since the stresses are simulated in the ratio $\frac{\gamma_m}{\gamma_p} . \lambda$, it follows that

$$\frac{\phi_{model}}{\phi_{prototype}} = 1.$$

Thus, if a geometrically similar model of the sand deposit and the foundation is made with proper care in choosing the model materials to confirm to the requirements described above, the behaviour of the model foundation during a motion conforming to the design earthquake as regards the time periods and acceleration should give quantitative data on the prototype behaviour.

Simple, though it may appear, model materials cannot be easily selected. To obtain a material having a reduced modulus of rigidity but the same angle of internal friction and preferably the same unit weight is extremely difficult, if not impossible. This, therefore, calls for some compromise regarding either the strength characteristics or the modulus of rigidity. Or in other words, one of the quantities can be scaled properly and the effect of the other can be considered while analysing the results.

To choose between the quantities mentioned above, the phenomenon of liquefaction should be looked into.

The strength of a saturated sand can be written as

$$S = (\sigma - u) \tan \phi \qquad \ldots(6)$$

S = shear strength per unit area

σ = total normal stress

u = pore pressure or neutral stress

ϕ = angle of internal friction.

The Equation (6) can be written as

$$S = \bar{\sigma} \tan \phi \qquad \ldots(7)$$

where $\bar{\sigma}$ = the effective normal stress.

During an earthquake the pore pressures increase further say by u' and then the strength of the soil will be given by

$$S = (\bar{\sigma} - u') \tan \phi \qquad \ldots(8)$$

The increase in the pore pressure u' is because of the strains in the sand mass and the increase mainly takes place in the elastic range. Thus it becomes essential that the modulus of rigidity be properly scaled. Since the angle of internal friction cannot be scaled in this case, settlement of material in the model will not be representative of the prototype material. But undoubtedly the pore pressures in the model will be in direct proportion to the pore pressures in the prototype (in the ratio $\frac{\gamma_m}{\gamma_p} . \lambda$).

If on the other hand the value of the angle of internal friction is simulated without regard to the modulus of rigidity the pore pressures developed will have no relation to the actual pore pressures and the results may be misleading.

Therefore, a suitable pore pressure transducer embedded in the model material can give extremely useful data on the development and dissipation of pore pressure in the sand mass represented by the model.

Conclusions

The behaviour of structures founded on saturated sand masses during earthquakes, has shown the influence of the type of foundation on possible damage.

The current method of determining the liquefaction potential of sand masses does not take into account the very important phenomenon of soil-structure interaction.

Model studies of the foundation soil system with proper scaling of the modulus of rigidity and with pore pressure measurements seems to be the best solution of the problem.

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