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Soil Liquefaction and Stability of Foundation

Liquéfaction du Sol et la Stabilité des Fondations

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SYNOPSIS. Two-and one-dimensional problems of consolidation of saturated non-cohesive soils subjected to dynamic and in particular to vibration loads are considered with an occluded gas content and deformation irreversibility of vibrocreep process taken into account. A method of explosive layer-by-layer sounding is suggested to estimate the possibility of liquefaction, density and dynamic stability of loose saturated soils in-situ. Vibrations of a building on two-layered foundation caused by seismic elastic and plastic travelling waves are discussed as well as the bifurcation effect leading to the rocking and even to the general loss of stability and overturning of a building. The criteria ensuring the foundation stability are established. Part I is written by Ivanov and Part II-by Sinitsyn.

Part I. As a result of a failure of loose saturated soils, affected by seismic action either by repeated impulses or by vibration a process of consolidation occurs which leads to either a partial or a complete soil liquefaction. A number of characteristic features of the process of soil structure failure and compaction of loose soils under the influence of dynamic actions necessitate a particular consideration of these phenomena and associated problems which constitute a part of a general theory of soil consolidation.

Under the effect of vibration the process of compaction has its peculiar features one of which is the presence of a strongly pronounced irreversible vibrocreep deformations well described by the exponential relationship (Fig.1) and practically the absence of "instantaneous" deformations as well. Another very characteristic feature of the process of non-cohesive soil compaction due to vibrations is the decrease of soil compactibility with the increase in compressive stresses in the soil skeleton and the absence of compaction at the stresses that are higher than a definite value for each dynamic action intensity and soil density (Fig.2). Thus a vibro-compression relationship at a constant acceleration of vibrations and at a constant compressive stress (σ) may be presented in the following view:

$$\varepsilon(t)=\varepsilon_0-a_1[\sigma(\varepsilon_0)-\sigma](1-e^{-\gamma_1 t})$$

where

$\sigma(\varepsilon_0)$ - stresses at which the soil with an initial void ratio (ε_0) is not compacted at the given accelerations of vibrations;

a_1 and γ_1 - vibrocreep parameters of a soil skeleton.

In case of the progress of a consolidation

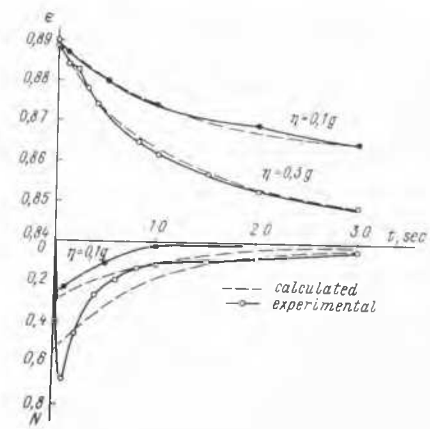

Fig.1. Variation of void ratio (ε) and degree of liquefaction ($N=P/P_{max}$)

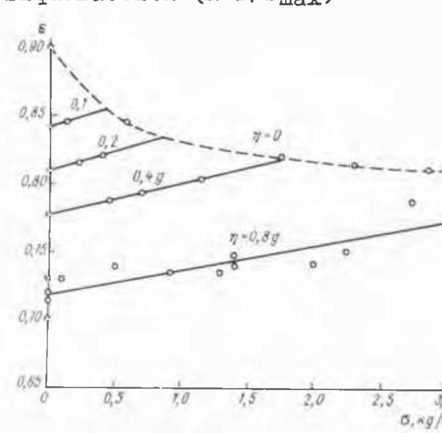

Fig.2. Vibrocompression relationships of a fine-grained sand at different accelerations of vibrations (η)

process and as a consequence of excess porewater pressure (p) we obtain

$$\sigma(\varepsilon_0)-\sigma(t)=\sigma'=\sigma(\varepsilon_0)-(\sigma^{*}-p), \quad (1)$$

where

σ^{*} -stresses in the soil skeleton in a stabilized state. Thus the increase of soil compactibility under vibration effects.

At the decrease of porewater pressure and monotonously decreasing value of σ', a change of void ration may be presented in the following form, a complete irreversibility of vibrocreep deformations being taken into account:

$$\varepsilon(t)=\varepsilon_0-a_1\sigma'(t)(1-e^{-\gamma_1 t})+\int_0^t a_1(1-e^{-\gamma_1 t})\frac{\partial\sigma'}{\partial t}dt \quad (2)$$

Thus, proceeding from ordinary notions about incompressibility of water and solid particles and Darcy-Gersevanov permeability law and taking into account the relationships similar to (1) and (2) the equation for a two-simensional problem of consolidation of a two phase non-cohesive soil may be obtained in the following form:

$$-\left[\frac{\theta(\varepsilon_0)-\theta^{*}}{\gamma_w(1+\xi)}+H\right]e^{-\gamma_1 t}=C\nabla^2 H, \quad (3)$$

where

$$C=\frac{(1+\varepsilon)K}{a_1\gamma_1\gamma_w};$$

K is the permeability coefficient; θ is a total of normal stresses and H is an excess head of porewater.

In case of decreasing σ', that is at the increase of porewater pressures, which can be observed in three-phase soils, the equation (2) takes the form of the equation of a linear creep, and the consolidation equation can be written as follows:

$$\frac{\partial^2 H}{\partial t^2}+\gamma_1(1-A)\frac{\partial H}{\partial t}=B\left(\gamma_1\nabla^2 H-\frac{\partial}{\partial t}\nabla^2 H\right) \quad (4)$$

at the initial conditions

$\frac{\partial H}{\partial t}=\gamma_1 A\left[\theta(\varepsilon_0)-\theta^{*}\right]$ and $H_0=0$, where

$$A=\frac{2a_1}{\beta(1+\xi)(1+\varepsilon)}; \quad B=\frac{K}{\gamma_w\beta};$$

β-gas compressibility coefficient;
ξ- lateral pressure coefficient.

Porewater pressure values obtained from the solution of equations (3) and (4) enable to estimate structure stabilities and the conditions at which a soil turns into a completely liquefied state. The unequality that follows is the condition of the absence of a complete liquefaction and as a consequence of the applicability of equations (3) and (4):

$$2p(x,z,t) < \theta^{*}(x,z,t)$$

In a particular case of a loaded layer (h) of a two-phase soil, located on the impermeable layer by integrating equation (3) we obtain:

$$H(z,t)=\left[\frac{\sigma(\varepsilon_0)-q}{\gamma_w}\right]\left[\frac{\cos(h-z)C_t}{\cos hC_t}-1\right]-\frac{\gamma' z}{\gamma_w}\left[\frac{\sin zC_t}{zC_t\cos hC_t}-1\right],$$

where

$$C_t=\left(\frac{e^{-\gamma_1 t}}{C}\right)^{1/2}; \text{ q- loading}$$

When there is no load, the condition of a complete liquefaction that is

$$\left.\frac{\partial H}{\partial z}\right|_{\substack{z=0\\t=0}}<\frac{\gamma'}{\gamma_w} \text{ becomes } \sigma(\varepsilon_0)\left(\frac{1}{C}\right)^{1/2}\mathrm{tgh}\left(\frac{1}{C}\right)^{1/2}<\gamma'.$$

To estimate a possibility of soil liquefaction in-situ, density and strength of structural bonds in saturated sands, a method of explosive sounding has been elaborated over a number of years. The method is based on the application of a maximum camouflet-charges explosions at different depths over the depth of a test soil layer. As the tests have shown, in accordance with the similarity principle of charge-explosions under conditions of a maximum camouflet, the degree of a soil structure failure and the appearance of the phenomenon of liquefaction are independent of the weight of an explosive charge. The looser the soil, the weaker its structural bonds and the greater the degree of the soil structure disturbance, the greater the subsequent compaction and settlements of the soil surface (Fig.3). As the process of sandy saturated soil compaction takes place basically after the explosive wave the soil particle rearrangement is determined only by structural peculiarities of sandy soils.

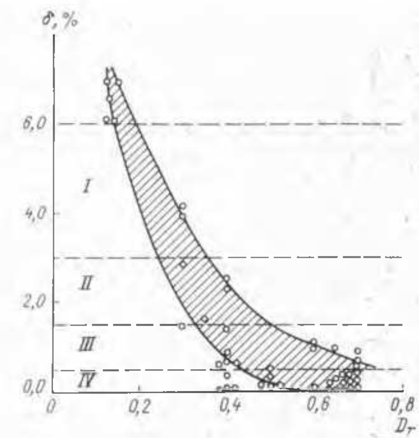

Fig.3. Relative average settlement of sandy soils of various relative density (D_r) during explosions (Field test data).

In fine grained and medium grained sands an explosive charge burial depth (h',m) depending on the thickness of the tested soil layer (H',m) the weight of an explosive-charge (C',kg) and the radius of more uniform settlements are determined from the $h'=0.66H'$, $C'=0.55h'^3$; $R=(2\div5)\sqrt[3]{C'}$.

The average relative settlement (δ) within the R radius is accepted as the main criterion. When it is accepted as the main criterion When it is necessary to find the loosest zones in a layer, charge-explosions at different depths from the ground surface are carried out.

On the basis of the laboratory and insitu tests, during which soil density and a degree of its liquefaction were being determined, a rough classification is suggested (zones I through IV in Fig.3). Very loose sands with a great probability of occurrence of liquefaction phenomena fall into the first zone, and dense and loose sands that have structural bonds and where liquefaction phenomena are unlikely to occur, fall into zone III.

Part II. The liquefaction exert a considerable influence on the general dynamical stability of the building and foundation. As result of interaction of seismic waves with foundations the parameters of waves field change it's values.The volume deformations increases and the compression stresses in soil skeleton changes.Now the soil dynamic attract the engineers attention more and more. Determination of the dynamic loads applied to the foundation through the soil is a more complicated problem of the soil mechanics. General methodes of the soil dynamics were exposed in many scientific papers of the soviet scientist:S.V.Medvedev /1/,D.D.Barkan /2/,O.A.Savinov /3/,V.A.Iliichev /4/ and others; as well as foreign-S.Prakash /5/, H.Seed /6/, I.Ioshimi and others. Dynamical stability of the dams,retaining walls and slopes was investigated by A.P.Sinitsyn /8/. But this problem was actual again when during Nijgata earthquake (Japan 1964)the multistory buildings of aseismic structure were overturned. This phenomenon was explaned as a result of liquifaction of the saturated sandy foundation.Detailed investigation performed by Sinitsyn /9/ showed that the liquefaction effect of the foundation cause the irregular displacements of the building but cannot evoke its complete overturning.Considering seismometric data and using general methods of the wave mechanics it is possible to determine the specific conditions of the building vibrations when it is placed on twolayered foundation and is caused by the travelling elastic or plastic seismic waves.The problem should be nonlinear;the bifurcation that occurs is connected with the rocking of the building due to the travelling seismic wave. The plastic waves interacts with foundation structure and the general stability of the building may be lost by overturning.The design method for determination of the dynamical stability of the structures placed on twolayered foundation is elaborated by using of general methods of the soil mechanics.The upper layer consist of soft sedimentaly soil and is placed on the rock foundation.The weak and strong seismic sources are investigated by which the elastic and plastic waves occure.The plastic waves are propagated with a small velocity and are carring the main part of the seismic energy flux.The travelling plastic wave cause the asimmetric model of seismic load transmission to the structure. As result of solving the nonlinear equations of the foundation-structure vibrations due to the travelling seismic wave the formuls are obtained for the critical dynamic load evaluation. The criteria of stability of the foundations were established such as the rate of physical parameters of the foundation including deformation modulus,density,velocities, accelerations and the length of the seismic waves.Now the liquification occure and together with the rocking of the building due to the travelling wave cause the lost of the general stability of the foundation and the building.For estimation the foundation stability designing scheme of the waves field from earthquake hearth to the building was investigated. This scheme include three main areas:1-the hearth,2-the energy transmission area;as it is shown in fig.4. The travelling wave create the asimmetric load on the foundation. As result the building begin to rocking For the soft soils the bilinear diagram is chosen,now for compression stresses a lower modulus of deformation is valid.But the unloading occure with a higher modulus.And the nil point of the sugrade reactions diagram during the vibrations of structure change its place. The motion of the structure is describeb by nonlinear equations and the instability effect of motion with increased amplitudes occure.Rocking of the building create the bifurcation and the two formes of possible instability arise.

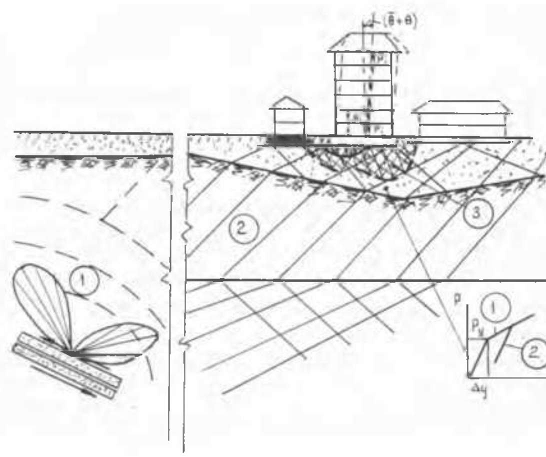

Fig.4.Designing scheme of the wave field

If the inclination of the building is absent the lost of stability accure as result of the liquification and the equilibrium equation is obtained if the projections of all forces on the vertical axis are compared to nil.By the inclination of the building due to the rocking two equilibrium equations must be kept in consideration. Namely the above mentioned remain and the second-equality to nil of the moments is addet. Now we have two values of the critical load by which the lost of the general stability of the building occure. For evaluation of critical loads the building is considered as a system with many degrees of freedom and the equation of motion in matrix form after decomposition on simmetric and asimmetric modes is:

$$[J_o]\frac{\partial^2\bar{\theta}}{\partial t^2}+[C_i]\frac{\partial\bar{\theta}}{\partial t}+[r_{ik}]\bar{\theta}=\left[1\left(\frac{b}{2}-vt\right)\right] \quad (5)$$

$[J_o]$-matrix of momentum inertia of the masses $[r_{ik}]$-rigidity matrix,$/c_i/$-attenuation matrix $\bar{\theta}$ -inclination angle of the building.
The solution of equation (5) is obtained by means of the finite elements method and by influence function φ_i obtained from consideration the motion of a load $p_i=1$ which is travelling along foundation structure.

$$\bar{\theta}=\Sigma\int_0^t\varphi_i(t_1)f_i(t-t_1)dt_1$$

After some simplifications the following formul for evaluation of the inclination angle of the building is obtained:

$$\bar{\theta}_i=\Sigma\int_0^t\frac{1}{J_o\omega_i^2}\left[b\sin\frac{2\omega_i t_1}{2}-v\left(t-\frac{\sin\omega_i t_1}{2}\right)\right]f_i(t-t_1)dt_1 \quad (6)$$

$f_1(t)$-the accelerogram of travelling seismic wave
I_o -Inertia momentum of the buildings masses
ω_i^o -frequencies of the vibrations of the masses
b --length of the foundation structure
v -velocity of the seismic wave

The equilibrium equation corresponding to the second mode of the instability, when the vertical axis of the building have the inclination angle $(\theta+\bar{\theta})$ is written as follow:

$$\sum P_i(\theta+\bar{\theta})H_i+\int_{-b/2}^{+b/2} p\,x\,dx=\beta H\theta^2 \qquad (7)$$

P_i-weight of the buildings elements.
H_i-distance from the foundation to the load P_i
p -ordinats of the subgrade reactions diagram.
$\beta H\theta^2$ -restoring momentum of subgrade reactions diagram.

The solution of the equation (7) is:

$$P_{kp}=P^o_{kp}\left[1-\left(\frac{4\beta}{Eb/2}\cdot\frac{H}{b}\cdot\bar{\theta}\right)^{1/2}+\dots\right]=P^o_{kp}\,\eta \qquad (8)$$

P_{kp} and P^o_{kp}-are the critical loads with bifurcation consideration and without it.
E-deformation modul of the foundation
β-charakteristic rate of the restoring momentum.
η-diminuation coefficient of P^o_{kp} with consideration of buildings rocking

In fig.5 the diagram for evaluation of the stability criteria of the buildings is drawn.

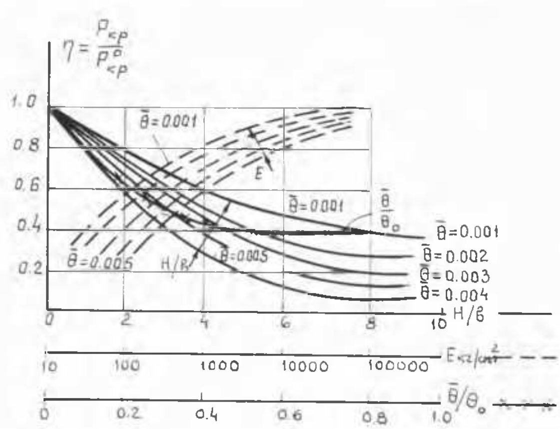

Fig.5.Diagram of diminuation coefficient.
For example if H/b=6 this correspond to the 7-8 story building and if $\bar{\theta}$=0.005 the coefficient of stability decrease almost 10 times. There are established three criteria of the buildings instability. The first one is the value of the inclination angle $\bar{\theta}$ of the building when the rocking occure as result of the travelling seismic wave. The second criterion is the ratio of buildings height to its width (H/b),and the threed is the value of deformation modul of the foundation. As result of liquefaction the deformation modul of foundation decrease to 100 kg/cm2 and the value of critical load reduces now the safety coefficient decrease eight times. Let us cheking the value of critical load for the buildings which have lost the stability by Nijgata earthquake. For calculation we take the following mean quantities:

$H=30m$, $b=10m$, $\omega^2=10\ 1/sec^2$, $v=300$ m/sec
$I_o=180$ tn.m.sec^2, $P^o_{kp}=500$ tn/m, $\bar{\theta}=0.04$, $E=100$ kg/cm^2

From diagram of fig.5 we obtain :

$\eta=0.15$ and $P_{kp}=0.15\cdot500=75$ tn/m.

The building must lose the stability if it's weight per one meter of the length was more than 75 tn/m and this was happen in reality.

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