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Characteristics of Soils at Cyclic and Shock Loads

Caractéristiques des Sols du Cycliques et Choc Charges

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SYNOPSIS In the first part written by Ivanov it has been shown that the shape of soil particles has essential influence on compaction and strength of sandy soils. In a wide range of shock and impulsive effects, strength characteristics of soils practically do not change but plate displacements and strains of the soil in a sub-limiting state substantially increase.
The second part written by Sinitsin deals with the solution of a dynamic problem of the two-layer-foundation stress state due to the actions of a falling load. The solution of the problem has been obtained by the method of wave nets taking into account the formation of plastic region, the reflection of stress waves from the elastic rock foundation as well as wave interactions effect.
The third part written by Musaelyan is devoted to the features of slumping strain formation in loess soils under dynamic actions.

I. SOIL INVESTIGATION

As the investigation shows (Ivanov, 1972) the properties of loose soils are strongly influenced by the shape and surface character of soil particles. The following parameters were used as numerical characteristics of particle shapes: rounding coefficients, α, sphericity coefficients, β, and shape coefficients, K, defined by the formulas:

$$\alpha = \frac{\Sigma r}{nR}; \quad \beta = \frac{\omega_p}{\omega_R}; \quad K = \alpha\beta$$

where: R - a critical radius inscribed into the particle contour; r - a radius of all the rest areas of a particle contour; n - the number of contour areas; ω_p - a protection area of each particle; ω_R - a circle area circumscribed round a particle projection.

In the case of ideal sand particle shapes, that is smooth spheres, all these coefficients are equal to 1. In order to obtain mean caracteristics the coefficients were estimated for a hundred particles of a sand, making their photoes by means of a microscope.

The effect of particle shapes on their density may be illustrated by the following examples (Fig.1). In constructing a series of large hydraulic fills fine sands were applied. The sands had virtually the same grain size distribution but it is these different particle shapes thanks to which sufficiently different values of void ratio of aggradated soils, varying from 0,62 for more rounded Shulbin sands up to 0.90 for very sharp-angled rock soils (tails) have been obtained. Characteristic coefficients in a dense (ε min) and loose (ε max) soil states are also displaced. The value of density degree (D), however, was practically the same for all hydraulic fills (0.40-0.46). Thus, the use of the same means of underwater aggradation as it should be expected, has resulted in the same particle density of packing along the area of aggradation.

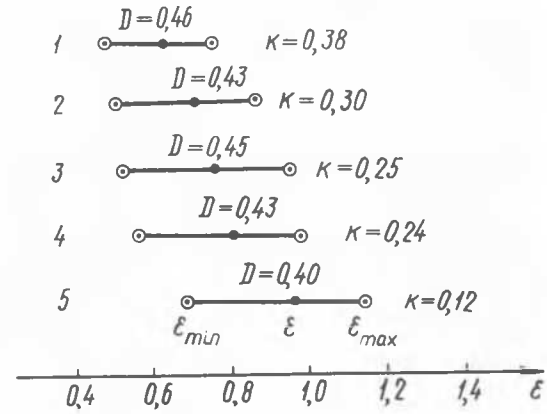

Fig.1 Limiting Void Ratio Variations (ε) for Fine Grained Sands having Various Shape Coefficients (K).
Black circles indicate void ratios obtained during fill aggradation: 1 - Shulbin fill on the river Irtish; 2- the Volga hydraulic fill named after V.I.Lenin on the Volga river; 3 - the Bratsk fill on the river Angara; 4 - the Kairakkum fill on the Syr-Daria river; 5 - the Krasnorechenskij tailings.

With the increase of rounding and sphericity soil particles in loose and dense soil packing as well as during aggradation are filled more compactly, which results in the decrease of characteristic void ratios. Therefore at the one and the same void ratio, soils having the same grain size distribution can have different density. Thus it should be emphasized that void ratio and bulk weight, in particular, can't be taken as characteristic of loose soil density.

The particle shapes and as a cosequence pore configurations of soils essentially affect their water permeability (Fig.2 a). Particle

shape exerts still greater influence on strength characteristics of loose soils (Fig.2 b).

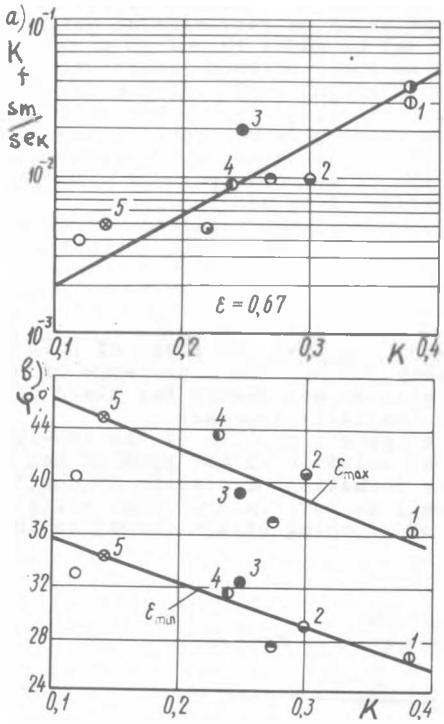

Fig.2 The effect of the Shape Coefficient K on the filtration coefficient K_f(a) and the angle of internal friction φ (b) (numbers on Fig1).

In a number of cases big porosity due to a small rounding degree in a particle shape may serve as an indication of bigger soil strength. Thus,for example,the Kairakkum sand aggradated at a void ratio equal to 0.80 (Fig.1) and a bulk weight equal to 1.49 t/m3,exibits greater strength (i.e.has greater value of φ ,see Fig 2 b) and possesses more stable structure under dynamic actions thanks to the angled shape as compared to Shulbin fill sand aggradated at a void ratio equal to 0.62 (1.64t/m3) and with less stable structure,composed of rounded particles approaching a spherical shape. Therefore, as it may seem paradoxical for constructions in a seismic region,the choice of Kairakkum sand having a less bulk weight appeared to be more successful.

The effect of density degree on the process of compaction may be observed in the experiments with a structure failure due to liquefaction and compaction of water-saturated sands under shock effects in a shear-compressive device, placed on a vibro-shock table. The test data analysis shows that for each density degree of a soil (except D = 1) there is a certain critical value of intensity shock effects at which the structure failure and soil compaction occur With the increase of the shock intensity,volume strains increase but to a definite limit, beyond which, further increase of impact (shock) effect intensity slightly changes the compaction value. Thus,each initial density degree has its own corresponding limiting maximum feasible value of volume strains. In other

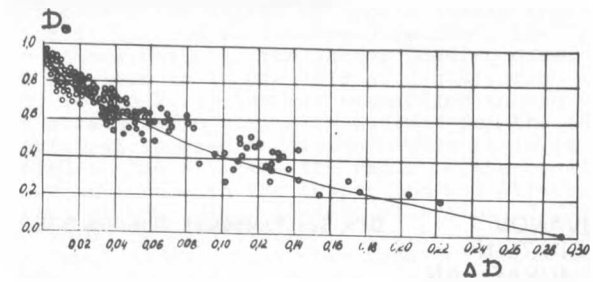

Fig.3 Limiting Variations of the density degree (Δ D) depending on the initial Value of the Density Degree (D_o) under shock effects.

words, limiting volume strains are defined only by the initial soil density degree and the soil structure at the liquefaction due to shock effects transforms from one state into another only in a certain manner. The curves of limiting density degree variations (Δ D) for soils having variable grain size distributions practically coincide, (Fig.3,sands from fine grained to coarse grained structures) and are satisfactorily described by empirical relationship:

$$\Delta D = 0{,}33\,(1 - D_o)^2$$

where D_o- the initial density degree. Then, the obtained limiting void ratio (ε) depending on the initial value of (ε_o) will take on the form of:

$$\varepsilon = \varepsilon_o - (\varepsilon_{max} - \varepsilon_{min})\,\Delta D$$

Numerous tests using plates with vibration loads as well as stabilometers with impulsive loads have shown,that in a wide range of oscillation accelerations (up to 1g) there is every reason to consider that strength characteristics of loose soils,that is the angle of internal friction,doesn't change 1,3 and is equal a static angle.

II.TWO-LAYERED SOIL FOUNDATION

This foundation consists of the upper sedimentary layer and a rigid rock. The falling weight causes a wave field which may be beyond elastic limit,now in soil foundation the elastic and plastic waves arise. These waves propagate with different velocities. The problem is complicated because the wave field in layered soil depends upon the reflection of stress waves from the elastic rock foundation and of the waves interactions. For solution of this dynamic problem is used the method of waves nets elaborated before for aseismic design of the structures (Sinitsin 1967,1975). At first the conditions of longitudinal wave propagation near the point of blow strike are investigated. The blow is fulfilled by throwing down load from a given height. During the energy transmission from the falling load to the foundation the compaction of the soil and the plastic areas is formed and the velocity of the waves in the areas is changed. The plastic waves have much lower velocity than the elastic The simple nets may be obta-

ined for elastic waves because their propagation velocity depends upon the elasticity modul of the foundation and of the density of the soil. These waves are reflected from rock foundation and the reflection coefficient is a function of time. Taking into consideration the conditions on the contact plane as the equality of particle velocities on the boundary of the soft upper layer and rock foundation the formula (1) for reflection coefficient is obtained:

$$K = 2\left(1 - \frac{\alpha}{\omega_1} e^{-\alpha t} \sin \omega_1 t\right) \qquad (1)$$

$$\alpha = 0{,}5\,\frac{\rho_0 c_0}{m}\,;\quad \omega^2 = \frac{r_{11}}{m}\,;\quad \omega_1 = \sqrt{\omega^2 - \alpha^2}$$

ρ_0 -density of soft layer; c_0-velocity of incident wave; r_{11} unit reaction of rock; m -mass of rock unit. For the rigid rock K=2 as $\omega = 0$ and $\alpha = 0$. The influence of rock deformability causes that the reflection coefficient at the beginning of the motion should be less than two, but after sin should be negative the item in brackets of formula (1) is greater than unit, and the coefficient is greater than two. As a result the pressure in the reflected wave increases and two waves arise there - elastic and plastic. The plastic wave falls behind the elastic one. If on the contact plane between falling load and foundation the pressure is great than in the layer the two waves propagate at once. Ahead is the elastic wave and beyond it is the plastic wave which has greater pressures than elastic wave. After reflection the interaction between these waves arise. The pressure on the contact plane between falling load and soil is distributed irragularly. For covex contact surface the pressure intensity is increasing to the symmetry axis, but for plane surface the intensity increases to the borders. The diagram of pressure distribution on the contact surface depends on the physique soil parameters too. For the saturated soil the pressure diagram smoothes out. The blow of great intensity causes formation of a compacted layer with a higher wave velocity, now the distortion of the waves propagation beam on the x-t plane takes place. The physical parameters of soft soil layer on which the blow is done may be approximately presented by a threelinear diagram (Fig.4). The part OA of diagram belongs to the elastic conditions of the layer. The part AB describes transition beyond elastic limit, now the deformation modul decreases rapidly and the plastic waves arise, but the unloading goes with elastic law. If the intensity of the blow is great the compaction of soil arises. On the part BC of diagram the soil deformation modul increases. In limit case the part BC should be vertical and the hardening of the upper layer of the soil appears. The following three cases arise as a result of the stress values: 1) If $\sigma < \sigma_A$ in the layer propagates the elastic wave only with velocity c_0 .2) If $\sigma_A < \sigma < \sigma_B$ there are two waves - the elastic and the plastic which have velocity $c_1 < c_0$ 3) If $\sigma > \sigma_B$, than the compaction wave arises and propagates with high velocity c_c and the density of soil increases $\rho_c > \rho_0$ On this part of a diagram there are two cases: a) $c_c < c_0$ and ahead of compaction front the elastic wave propagates; b) if $c_c > c_0$ there is no elastic wave. The limit pressure $(\sigma_c)_{cr}$ by which the elastic wave is absent may be obtained by using the equation of mass preservation:

$$\sigma_c = \sigma_0 \frac{\varepsilon_c}{\varepsilon_0} \frac{1}{(1-\varepsilon_0)} \qquad (2)$$

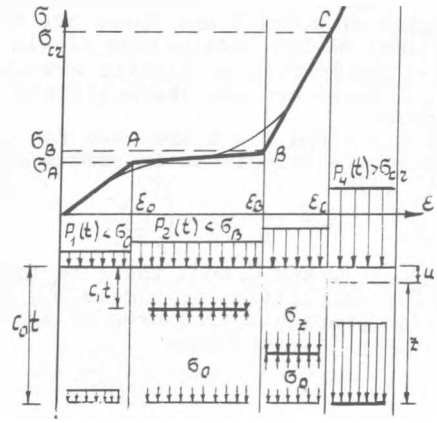

Fig.4 Scheme of waves propagation.

The elastic and plastic waves propagation is studied before and may be used here too. More detailed investigations should be fulfilled for compaction layer when the pressure on soil surface p(t) and the part BC of diagram should be vertical. The equation of motion is:

$$\frac{d}{dt}\left[\rho_c (z-u)\,\dot{u}\right] = \mu(t) \qquad (3)$$

$$z-u = (1-\varepsilon_c)z\,;\quad \dot{u} = \varepsilon_c \dot{z}\,;\quad \rho_0 = \rho_c(1-\varepsilon_c)\,;\quad \frac{\sigma_0}{\varepsilon_0 \rho_0} = c_0^2$$

Equation (3) is nonlinear and the solution is obtained by some simplifications. The pressure of falling load is transmitted to the soil so that the maximum p_0 arises rapidly and than the pressure decreases. As a result the supersound velocity C_c in compacted layer decreases to $\dot{z} = C_c$ and from this equation the time value $t=t_s$ is calculated by which the supersounding motion is finished. The thickness of compacted layer is obtained by formula (4):

$$z_s = c_c t_s \int_0^{t_s} \mu(t)\,dt \Big/ \mu_{cr} t_s \qquad (4)$$

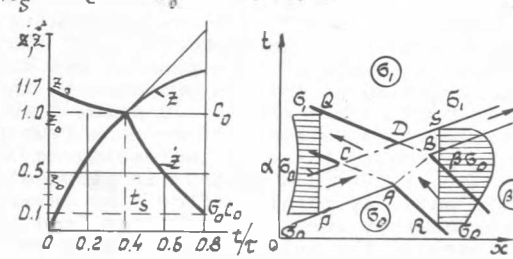

Fig.5 Scheme of compaction. Fig.6 Scheme of waves.

Scheme of compaction front motion and separation of different areas is shown in Fig.5 for sandy layer with following data: $\mu(t) = \mu_0(1 - \frac{10}{\tau})$

$$\frac{\sigma_0}{\mu_0} = 0{,}50\,;\quad \varepsilon_0 = 0{,}10\,;\quad \frac{\mu_{cr}}{\mu_0} = 0{,}75$$

By $t_s = 0{,}4\frac{t}{\tau}$ the supersounding motion is transformed to sounding and the thickness of compacted layer is $z_s = z_0$ if $t_s \geq 0{,}8\frac{t}{\tau}$ there is the elastic wave only. For investigation

of waves interaction the wave nets on x - t plane are drawn. There is a more interesting case when the elastic wave meets the plastic as it is shown in Fig.6. The stresses change: in elastic wave from σ_o to $\alpha\sigma_o$; in plastic wave from σ_o to $\beta\sigma_o$; and $\alpha<1$; $\beta>1$. Interaction of the waves begins at point A and there are two characteristics: AC has inclination of elastic wave and AB - inclination of plastic wave. From points B and C there are two characteristics of elastic wave.
The contours P - Q and R - S are used for evaluation of particle velocities in points Q and S by folmula (5):

$$v_Q = v_P + \int_P^Q \frac{\Delta\sigma}{\rho_o c_o} \quad \text{and} \quad v_S = v_R + \int_R^S \frac{\Delta\sigma}{\rho c_1} \qquad (5)$$

By comparing v_Q to v_S and v_P to v_R there is the formula (6) for calculation of stress σ_1, in the waves which propagate from area of elastic and plastic waves interactions.

$$\sigma_1 = \frac{1}{2}\left[(2\alpha-1) + (\beta-1)\frac{c_o}{c_1} + \beta\right]\sigma_o \qquad (6)$$

When the elastic wave runs against a plastic wave the inner reflaction effect occurs and in straight direction a weaken plastic wave is pro pagated and there are following stresses:

$$\sigma_1 = \frac{(c_o/c_1 - 1)\beta}{(c_o/c_1 + 1)}\sigma_o \qquad (7)$$

The wave net is now established for $p(t)=3\sigma_o$ from the falling load and is distributed equally on the surfase plane, as it is shown in Fig7.

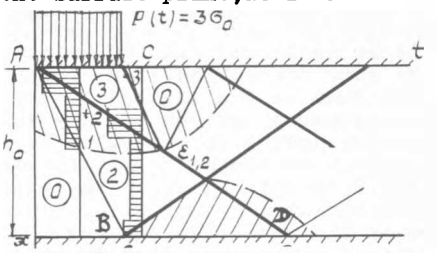

Fig.7 The wave net.

This pressure acted during time interval τ and then the unloading goes on and the pressure on the boundary of the layer is equal to nil. From point A (Fig.4) there are propagated two waves. The wave AB has velocity C_o and the pressure $\sigma_o=1$. The wave AD is a plastic one and has velocity $C=0.25\ C_o$ but the pressure in this wave is $\sigma_1=3\sigma_o-\sigma_o=2\sigma_o$. In point B the elastic wave is reflected from rock foundation and the pressure there is increasing to σ_2 $\sigma_2=(1+\frac{c_1}{c_o})\sigma_o=1,25\sigma_o$ now upwards is propagated a plastic wave. At point C after time interval τ from contact surface of falling load there is propagated the unloading wave with velocity C_o. At point E the unloading wave catches up with the plastic wave and as result of interaction of waves is weakening of pressure σ_3 in pla stic wave $\sigma_3=1,2\sigma_o$. From scheme it is clear that the plastic area in upper layer of the soil is developed gradually and occupies the upper part of the layer with depth less then one half of the layer thickness as it is shown in Fig.4. The formation of plastic area in a soil layer as it is clear from wave net, in general, depends on longitudinal waves as the velocity of these waves is greater than the velo city of transverse waves. The detailed analysis of wave nets drawn so as on Fig.3 for a layer of small thickness the plastic areas arise simultaneously as near of the blow point as on the boundary of rock foundation. But the width of plastic area on the rock is more greater as near to the blow point. In the course of time when the blow duration is large the two plastic areas drawn together and form an area like a cone with cutting of head. If the soil Thick ness is large the influence of reflections from rock foundation decreases and a spherical plastic area arises near blow point.

III. LOESS SOIL

The data of the macroseismic investigations of the earthquake consequences in loess development regions as well as the results of numerous laboratory tests at dynamic loads have shown that under definite conditions the dynamic loads cause changes of loess soil deformation. The above mentioned changes the soil properties (moisture, density and therefore dynamic load parameters :frequency, amplitude) and in some cases they can reach such values take them into acount while designing of buildings and constructions in seismic regions.
The branch has worked out investigations of characteristics for cyclic loads which allow to approximate the collapse deformation develop ment by cyclic loads.
Different methods of applying of cycling loads and the development of the collapse deformations display as well as after stabilisation of collapse deformation and the wide range of soil collapse characteristics and clear out their influense on the additional deformation.

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