Liquefaction conditions for saturated cohesionless soils

Les conditions de liquéfaction des sols saturés pulvérulents

N. N. MASLOV, D.Sc., Professor, Road-Transport Institute, Moscow, USSR
P. L. IVANOV, D.Sc., Professor, Polytechnical Institute, Leningrad, USSR

SYNOPSIS In the first part of the report (Maslov) on the basis of "the filtration theory of sand body stability disturbance under dynamic actions" the evaluation of the effect of compression stress due to the structure weight on sand foundation stability is given. The thickness of the load layer ensuring the foundation stability is determined. The second part (Ivanov) of the report deals with the developed field test estimation of the possibility of low cohesive soil liquefaction by means of "explosion sounding procedure".

CONDITIONS OF SAND LIQUEFACTION IN A WATER-SATURATED LAYER DURING EARTHQUAKES

The developed "filtration theory of sand body stability disturbance" (Maslov, 1957, 1958) makes it possible to estimate the degree of sand foundation stability of structures during earthquakes, even before sand soil conversion into a completely liquefied state, starting from the estimation conditions of the value of a sand soil shear strength (s_{din}). According to this theory, the strength fall of a saturated sand soil, being under dynamic actions, arises at the cost of the reduction of normal stresses, acting in the layer thickness under these conditions. This phenomenon occurs under the conditions of the uplift formation ($\gamma_w h_z$) in the layer thickness in connection with the occurence of the dynamic head h_z (fig. 1) during shaking and compacting of the sand.

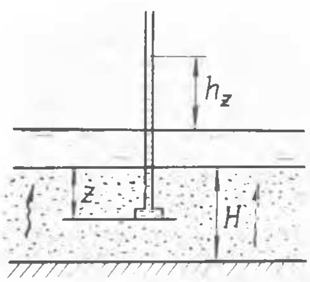

Fig.1 An Operating Diagram to Estimate the Value of Hydrodynamic Head h_z

In the simplest case, at a relatively negligible sand layer thickness, the value of the dynamic head h_z is determined according to the expression:

$$h_z = \frac{\vartheta_n}{k_f} (Hz - \frac{z^2}{2}).$$

Accordingly, the hydrodynamic gradient (I_z), ensuring the upward movement of the water, squeezed out during the layer densification

$$I_z = \frac{\vartheta_n}{k_f} (H - z).$$

Here H - the thickness of a sand layer; z - the depth from the surface; k_f - the sand coefficient of permeability; $\vartheta_n (t^{-1})$ - an experimentally defined coefficient of consolidation.

For the simplest conditions ϑ_n is taken to be independent of time and the depth of the horizon under consideration z. The sand layer shear strength under these conditions will be expressed as follows:

$$s_{din} = (p_z + \gamma_{sd} \ z - \gamma_z \ h_z) \operatorname{tg} \varphi ,$$

where p_z - the vertical compressing normal stress at the depth of z from the structure weight; γ_{sd} - the unit weight of the sand in water; γ_w - the unit weight of water.

The coefficient of dynamic consolidation ϑ_n testifies a fast sand consolidation due to porosity n in given conditions. It depends upon the intensity of dynamic effects (due to α_s acceleration) upon sand density, its grain-size distribution, upon the degree of grain rounding and so on. In a general case ϑ_n=dn/dt.

The dynamic head h_z takes on a substancial value only at $\vartheta_n > 0$, that is in the presence of sand consolidation while the sand layer being under dynamic effects. Such situations occur when the conditions $\alpha_s > \alpha_{cr}$ are observed. In the latter inequality α_s is still the acceleration of seismic oscillation with the oscillation frequency determining it, and α_{cr} is a critical acceleration at the same parameters. The value of critical acceleration α_{cr} is to be determined only experimentally.

Again, refer to the evaluation of the stability of the structure, erected and discussed under the conditions considered. The degree of its stability is determined by: a) the strength of its foundation, that is the relation of forces acting at a given point (tangential stresses) to the sand shear strength s_{din};

b) the effectiveness of the structure deepening, specified in the design. Under the conditions: $\alpha_s > \alpha_{cr}$ and with no load layer, s_{din} will decrease and the more intensively, the greater the dynamic head h_z. Porvided that the condition $\gamma_{sd}\ z = \gamma_w\ h_z$ is implemented the $s_{din} = 0$. In other words, the significance of the structure deepening in this case will be reduced to zero.

In another situation the work of the structure foundation itself will appear. It is to be noted first of all, that the critical acceleration value for a certain sand soil in this or that state of its density seams to be dependet of the value of the pressure, exerting on the sand, that is, in this case dependent of the thickness of the layer p_z acting over this or that depth.

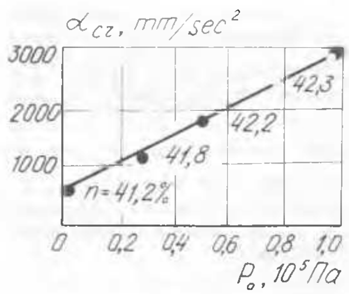

Fig.2 The Dependence of Critical Acceleration (α_{cr}). On the External Load Layer. The Sestroretsk Sand. Fractions: 0.5-0.25mm - 48%; 0.25-0.1 mm - 28%

As experience shows, the relationship $_{cr}$ = $f(p_z)$ is rather important. This assumption is confirmed by the example given in fig. 2. Under this condition, even at $\alpha_s > \alpha_{cr}$ for the layer thickness free of load over the depth z_{cr}, where the critical acceleration α_{crp}, with the allowance for the load effects, will be more than k $_s$, the sand will be in undisturbed state (here k is a safety factor that is to be taken as equal as 1.5). Only at the depths, where the significance of the increase of α_{cr} at the cost of p_z will appear to be negligible to implement the condition $\alpha_{crp} > k\alpha_s$, the effect of the sand strength reduction will also take place under certain conditions, up to a complete sand liquefaction and loss of its stability (fig. 3).

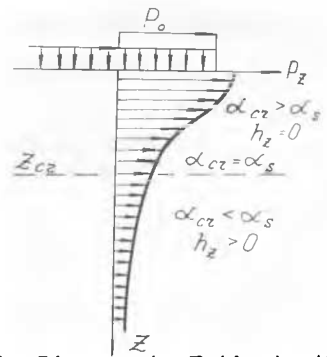

Fig.3 The Diagram to Estimate the Dimensions of the Active Zone, where $\alpha_s > \alpha_{cr}$

As we can see, the effect of the sand stability reduction in the structure foundation is not obligatory in all the cases. Very often,in the relationship considered, under certain conditions the earthquake will not appear destructive for the structure and, in particular, at a relatively low thickness of the sand layer and at a sufficiently high load p_o on the ground in the structure toe. Along with,at small structure dimensions, at a relatively low value of load on the ground and, which is the most important thing, at a high thickness of a sand layer, the situation is likely to become catastrophic. This, perhaps, may be responsible for the phenomenon, paradoxical at the first sight, that in Japan (in Niagata,for example) during the earthquakes light structures appeared to be more sensitive to the earthquakes, while neavy structures suffered less from the destruction.

In certain cases, it appears necessary to increase the seismic stability of fields (areas) of unloaded sand layers. Evidently, in this case, the most promising measure is to compact the sand layer thickness, ensuring the conditions $\alpha_{cr} > \alpha_s$. However, very often this measure,for a number of reasons, may be eliminated. Under similar conditions it may appear reasonable to apply for these aims, looding of the territory with dynamically stable material. Here, it becomes necessary to evaluate the required value of the thickness of that protecting layer (d). This value can be readily found in the terms of the conditions of damping of the hydrodynamic uplift, occuring in the sand layer, by the load layer weight.

In the simplest case we are capable to implement this requirement, at the cost of the load, by means of increasing the required value of critical acceleration, with the following principle being implemented $\alpha_{cr\ p} \geq k\alpha_s$.

Here, again, k is a safety factor. Obviously, in this case, the required thickness of the load layer (d) can be defined from the following condition: γ_{sd} d = p. In other words, it is necessary for the weight (pressure) of the load layer to ensure the needed increase of the critical acceleration α_{cr}.

THE ESTIMATION OF THE FEASIBILITY OF SOIL LIQUEFACTION BY MEANS OF CHARGE EXPLOSION SOUNDING

The methos suggested (Ivanov 1967, 1978) for layer by layer estimation of soil density and a possibility of cohesionless water-saturated soil liquefaction, by means of charge explosion has been further developed (Ivanov, 1983).

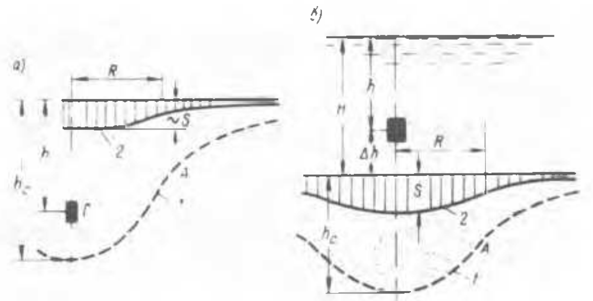

Fig.4 Zones of Settlements (1) and Soil Structure Failure (2) during Explosions of: a) Deeply Burried Charges; b) Underwater Charges

During explosions of deepy burried charges and underwater charges (fig. 4) in sufficiently loose soils there occur a zone (1) of the soil liquefaction and subsequent densification and also a corresponding soil surface settlement (2). The charge mass (C in kg) is chosen from the condition of camouflet of the charge explosion in the ground and in the water according to the relationships:

$C = 0.055\ h^3$ and $C = 0.1\ H^{2.4}$, where h and H are the depths (in m) of a charge burrying into the ground or water.

The dimensions of the liquefaction and compaction zones may be preliminary estimated according to the empirical relationships (Ivanov, 1967, 1978). In field tests the depth of the liquefaction zone (h_c) may be refined due to the settlements of marks and bench marks, installed over the depth of the layer being investigated. In each test explosion it should be necessary to measure a surface soil settlement. The distance, within which a relatively uniform compaction can occur (settlement S) is taken as the radius of propagation of the main zone of soil structure destruction during explosions, that is the radius of the effective charge action (2) which is near the radius of the location of the characteristic zone A of the graph bend of the soil surface settlement (fig. 4).

As disclosed by tests, when explosions are performed under conditions of a maximum camouflet, the degree of the sand soil structure failure and the onset of the liquefaction phenomena are independent of the charge weight of the same type of the explosive. In this case the charge weight is responsible only for the dimensions of the zone of the structural destruction. This characteristic feature of camouflet explosions conforms well with the similarity principle. The looser the soil, the weaker the structural bonds and the greater its structural disturbance, the higher the degree of subsequent compaction and settlement of the soil surface. It is of importance to note, that the compaction of sand and fine-grained saturated soils generally occurs after the explosive waves cease to act in the soil and the particle rearrangement is controlled only by specific characteristics of the soil structure and the soil dead weight. At a complete destruction of the structure and at a complete soil liquefaction, the value of the soil relative settlement is unequivocally related with the value of the initial density. All these determine the main principles of application of explosions to evaluate saturated loose soil density and a feasibility of soil liquefaction.

The principle criterium of the structure dynamic stability, density and liquefaction probability for saturated loose soils in the technique of charge explosion sounding is a mean relative settlement of the soil surface within the distance "R" from the charge burial site, $\delta = (S/h_c)\ 100\%$. Field and laboratory tests accompanied by the soil density inspections allowed to obtain a table of a tentative estimation of dynamic stability and density of fine and medium grained water-saturated sands (Table).

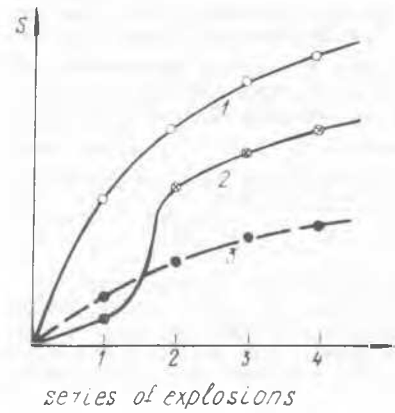

Fig.5 The Accumulation Character of Mean Settlements during Subsequent Explosions of Charges Having the Same Weight: 1 - in Loose Soils; 2 - in Soils Having Substantial Structure Strengths; 3 - in Soils of Mean Density

The ratio between the soil settlements, occured after a number of consequitive explosions of charges of the same weight in one and the same place, also characterises the initial density of cohesionless soils. The greater the the difference between the mean settlements of the soil surfaces after two or three consequitive explosions, the looser the soil (see Table) and the higher the probability of the appearance of large liquefaction zones. In the presence of pronounced structural bonds, the settlement after the second explosion as a result of bond destructions may be greater than the one after the first explosion (fig. 5).

When it is necessary to separate the loosest areas of the cohesionless soil layer and to estimate differentially the soil structure stability over the layer depth, sounding explosions are performed with charges of differ-

Characteristics of the Structure Dynamic Stability and Soil Density		Probability of Liquefaction Phenomena	Criteria of Explosion Sounding		Criteria of Dynamic Sounding	
			Mean Relative Settlement - δ in the Limits of R, %	Relationship of Settlements during the 1st and the 2nd Explosions	Mean Conventional Dynamic Strength P_d, MPa	Changes of the Value P_d after Explosion, %
I	unstable, very loose	high probability of occurence	> 3	> 1.5	< 2.0	> +30
II	slightly stable, loose and mean density	feasible	3-1.5	1.5-1.2	2.0-3.5	+30-(+10)
III	stable, mean density and loose with structural bonds	scarcely probably	1.5-0.5	1.2-1	3.5-5.0	±10
IV	highly stable, mean density and dense, with strong structural bonds	practically impossible	< 0.5	< 1	> 5.0	-10-(-20)

ent weight over the corresponding depths. In order to estimate a noticeable influence of the explosion of the previous charge (C_i) upon the test results obtained from the explosion of the following charge, larger by weight, C_{i+1}, the explosions should be performed at the points separated from one another in a plan by a distance surpassing $2(R_i+R_{i+1})$. Estimating the settlements S_i and S_{i+1} after each explosion one can calculate the relative settlement of a layer portion of $h_{c,i+1} - h_{c,i}$ as being equal to $\delta = (S_{i+1} - S_i)/(h_{c,i+1} - h_{c,i})$.

Increasing the weight of successively exploded sounding charges and calculating relative settlements one can make an explosion sounding plot and estimate the structure stability over the depth. For example (fig. 6) similar sounding plot (Smirnov, Falk) of underwater explosion of 5, 7, 13, 24 and 40 kg charges testifies that the sand bottom of the water area (the water depth being 17-19 m) is composed mainly of two layers: the upper layer, having a low structure dynamic stability, and the lower layer, having a highly stable structure.

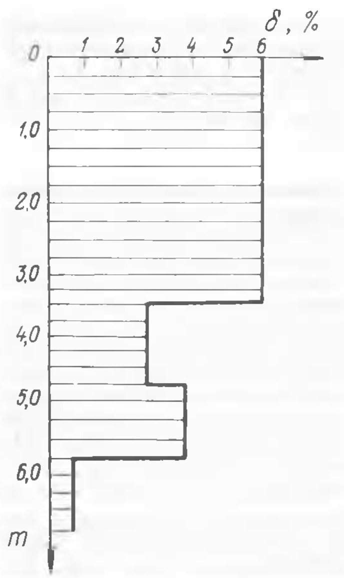

Fig.6 A Diagram of a Charge Explosion Sounding of the Bottom of the Water Area

Charge explosion sounding may be performed along with other techniques of field tests, carried out on sand soils, particularly, using a technique of impact sounding (dynamic penetration). On carrying out dynamic soundings before and after explosions it was observed that the looser the soil and the weaker the structural bonds, - the greater the increase of conventional dynamic strength P_d; as to dense sand soils and soils of mean density, having pronounced structural bonds, their conventional dynamic strength decreases. In this connection, the table of dynamic stability characteristics was supplemented (Dudler) with the criterion of the relative increment of the conventional dynamic strength.

REFERENCES

Maslov, N.N. (1957). Proc. 4th Conf. Soil Mech. Found. Eng., London.

Maslov, N.N. (1958). Cong. des Grands barrages, New York, 12115.

Ivanov, P.L. (1967). Compaction of cohesionless soils by explosions. Stroyizdat, p. 160, Leningrad, Translated and Published (1972) for the US Department of Commerce, National Technical Information Service, Springfield, Va 22161.

Ivanov, P.L., Krasnikov, N.D. (1978). Charge explosion sounding of saturated cohesionless soils. 6th Symp. on Earthquake Eng., University of Roorkee, vol. 1, pp. 151-156.

Ivanov, P.L. (1983). Compaction of loose soils by explosion technique. 230 pp. Nedra, Moscow.