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RESULTS OF DETERMINING LIMITING DYNAMIC COMPRESSION DIAGRAMS FOR SANDY SOILS AND CLAY

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UDC 624.131+539.215

A method was described in [1, 2] for determining limiting dynamic compression diagrams corresponding to instantaneous loading ($\dot{\varepsilon} = \infty$) for soils and porous materials sensitive to deformation rate. The method is based on the relationship of weak-disturbance propagation rates with the limiting dynamic diagram $\varphi(\varepsilon)$ with compression of a viscoplastic material. However, actual data for determining $\varphi(\varepsilon)$ in [1, 2] was only obtained for air-dried sandy soil. Given below are the results of experimental studies for determining these diagrams for sandy soils with different moisture contents, and also for dense clays.

It is assumed in the same way as in [1, 2] that the main properties of sandy and clay soils under short-term dynamic loads with sufficient accuracy are described with uniaxial compression (under plane strain conditions) by a deformation rule

$$\frac{\partial\varepsilon}{\partial t} = \frac{1}{E(\sigma_1, \varepsilon)}\frac{\partial\sigma_1}{\partial t} + g(\sigma_1 - f(\varepsilon)), \quad E(\sigma_1, \varepsilon) = \begin{cases} E(\varepsilon), \dfrac{\partial\sigma_1}{\partial t} \geqslant 0, \\ E_*(\sigma_1, \varepsilon), \dfrac{\partial\sigma_1}{\partial t} < 0, \end{cases} \tag{1}$$

where σ_1 is the greatest principal stress; $E(\varepsilon)$, $E_*(\sigma_1, \varepsilon)$ are functions determined by experiment with loading ($\partial\sigma_1/\partial t \geq 0$) and with unloading ($\partial\sigma_1/\partial t < 0$); $g(z) > 0$ with $z = \sigma_1 - f(\varepsilon) > 0$ and $g(z) \equiv 0$ with $z \leq 0$; $f(\varepsilon)$ is the static compression diagram for the material with $\dot{\varepsilon} \to \infty$.

As shown in [2], for a material of type (1) the relationship for a small-disturbance propagation rate $c(\varepsilon)$ and limiting dynamic diagram $\varphi(\varepsilon)$ ($\dot{\varepsilon} = \infty$) under load is determined by the relationship

$$E(\varepsilon) = \frac{d\varphi(\varepsilon)}{d\varepsilon} = \rho_0 c^2(\varepsilon) \tag{2}$$

(ρ_0 is initial material density). By integrating, from (2) we obtain the limiting dynamic diagram

$$\varphi(\varepsilon) = \int_0^\varepsilon E(\xi)\, d\xi, \quad \dot{\varepsilon} = \infty. \tag{3}$$

Thus, by knowing from an experiment the relationship $c(\varepsilon)$ it is possible to plot the limiting dynamic diagram $\varphi(\varepsilon)$ ($\dot{\varepsilon} = \infty$) with loading.

Testing was carried out in a UDN-150 unit [1, 2] fitted with a system for measuring weak-disturbance propagation rates in a compressed material. As in [1], compression was created as

Moscow. Translated from Zhurnal Prikladnoi Mekhaniki i Tekhnicheskoi Fiziki, No. 6, pp. 166-169, November-December, 1988. Original article submitted October 6, 1987.

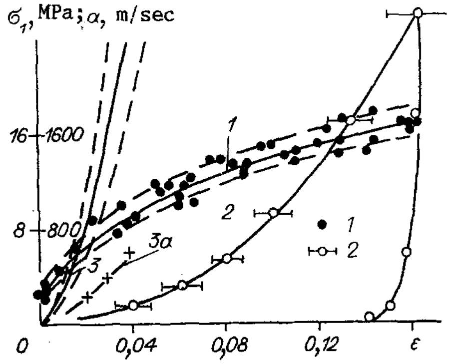

Fig. 1

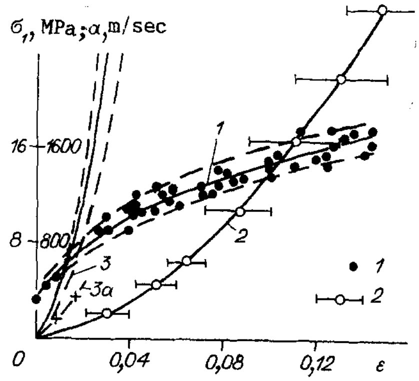

Fig. 2

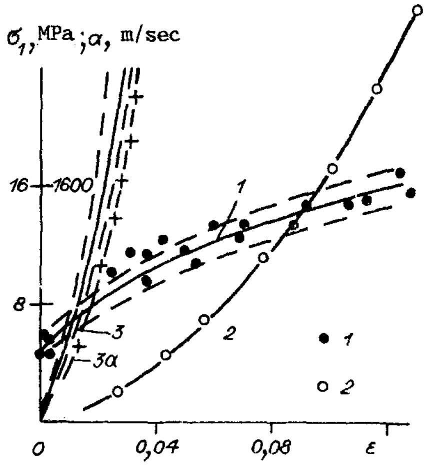

Fig. 3

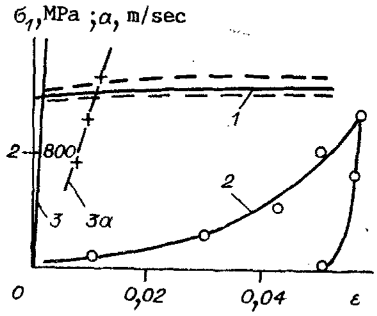

Fig. 4

TABLE 1

Name of soil or material	c_0, m/sec	$\varphi(\varepsilon)$		$f(\varepsilon)$	
		$E_1 \cdot 10^{-2}$, MPa	$\frac{m_1}{\nu_1}$	$K_1 \cdot 10^{-2}$, MPa	$\frac{m_2}{\nu_2}$
Clay $\rho_0 = 1{,}70$—$1{,}75$ g/cm^3 $w = 0{,}20$—$0{,}22$	1189	31,5	$\frac{0{,}60}{1{,}66(\pm 0{,}23)}$	0,100	$\frac{2{,}6 \cdot 10^3}{3{,}0}$
Sand $\rho_0 = 1{,}50$ g/cm^3 $w = 0{,}05$	250	1,00	$\frac{137{,}3}{1{,}96(\pm 0{,}084)}$	0,150	$\frac{116{,}6\binom{+5{,}9}{-6{,}3}}{2{,}37(\pm 0{,}11)}$
$w = 0{,}10$	350	1,838	$\frac{70{,}00}{1{,}89(\pm 0{,}077)}$	0,350	$\frac{19{,}1\binom{+1{,}8}{-1{,}7}}{1{,}77(\pm 0{,}18)}$
$w = 0{,}15$	460	3,216	$\frac{35{,}1}{1{,}83(\pm 0{,}081)}$	0,400	$\frac{15{,}6\binom{+3{,}8}{-3{,}0}}{1{,}67(\pm 0{,}37)}$

a result of impact of a falling load weighing 100 g onto an immovable block in a piston device. The immovable block included a piezoelectric transducer recording the instant of impact over the upper surface of the specimen. Arrival of a weak disturbance at the lower surface of the specimen was recorded by a piezoelectric transducer set up on the bottom part of the unit.

In order to determine the relationship $c(\varepsilon)$ a soil specimen was subjected to a static loading with a load σ_{1i} changing in steps ($i = 1, 2, \ldots, n$). In this way a record was made of static strain ε_i and the time corresponding to it for travel of a weak-discontinuity wave over a distance equal to the height of the specimen after loading $h_i = h(\varepsilon_i)$. With quite small specimen height the propagation rate for a weak discontinuity $c(\varepsilon_i)$ was determined as the average $c(\varepsilon_i) = h_i/t_i$, $i = 1, 2, \ldots, n$.

Testing was carried out on sandy soil with a density in the air-dried condition ρ_c = 1.48-1.50 g/cm^3 with moisture content w = 0.05, 0.10, 0.15, and also dense clay with ρ_0 = 1.70-1.75 g/cm^3 and w = 0.20-0.22. The particle-size composition of the soils was the same as that in [2]. For testing specimens of sandy soils a weighed sample corresponding to the prescribed density and moisture content was packed into a special ring of the UDN-150 unit [2]. Clay was supplied to the laboratory in the form unbroken paraffinized core samples of cubic shape with an edge of 20 cm. Five speciments were cut simultaneously from this core sample by means of a special circular knife with five cutters, and the specimens were tested under identical conditions. In all cases soils were brought from the same test sites where field experiments were carried out [2].

Test results for sandy soil are presented in Figs. 1-3 for w = 0.05, 0.10, 0.15, respectively, and for clay in Fig. 4. Here curves 1-3 and points 1, 2 relate to $c(\varepsilon)$, $f(\varepsilon)$, $\varphi(\varepsilon)$. Given for comparison in Figs. 1-4 are curves 3a, i.e., limiting dynamic diagrams $\varphi(\varepsilon)$ ($\dot{\varepsilon} = \infty$) obtained with relationships at the shock-wave front used in field tests [2].

Experimental relationships $c(\varepsilon)$ and $f(\varepsilon)$ for all cases may with sufficient accuracy be approximated in the form

$$c(\varepsilon) = c_0\left(1 + m_1'\varepsilon^{\nu_1}\right)^{1/2}, \quad m_1' = m_1\nu_1; \tag{4}$$

$$f(\varepsilon) = K_1\left(\varepsilon + m_2\varepsilon^{\nu_2}\right) \tag{5}$$

[K_1, m_1, ν_1, m_2, ν_2 are experimental coefficients (see Table 1)]. The range of applicability for relationships (5) and (6) is for sand $0 \le \varepsilon \le 0.12$-$0.14$ and for clay $0 \le \varepsilon < 0.06$.

Limiting dynamic diagrams (ε) are determined from (3) and (4) in the form $\varphi(\varepsilon) = E_1 \cdot (\varepsilon + m_1\varepsilon^{\nu_1})$, $E_1 = \rho_0 c_0^2$ [E_1 is dynamic elasticity modulus for the soil (see Table 1)].

Comparison of the limiting dynamic diagrams $\varphi(\varepsilon)$ (curves 3) obtained in experiments with similar diagrams obtained for the same soils in field experiments using the relationships at the shock-wave front (curves 3*a*) point to the presence of a marked difference between them (with the exception of data of sand with w = 0.15). These differences are mainly connected with insufficient accuracy for the method of determing $\varphi(\varepsilon)$ diagrams under field conditions. Here accuracy depends not only on the accuracy of direct measurements of field parameters for stresses, but also to a significant extent on the accuracy of determining the shock-wave front propagation rate in the soil from experimental results for the path of its movement.

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