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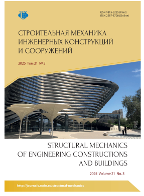

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Behavior of Underground Shell Structure under Seismic Impact from Explosion

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Abstract. The study presents the results of in-situ experimental investigations on the propagation of explosive seismic waves in the ground environment and the behavior of a seismically stressed underground structure in the form of a cylindrical thin-walled shell interacting with the ground during seismic impact from underground instantaneous explosions. It was taken into account that the seismic impact of an underground explosion on an underground structure depends on many factors, especially on the physical and mechanical properties of the soil of the experimental site. The composition of the soil was obtained by drilling holes for explosives from an excavated trench for installing samples of underground structures. Ground vibrations during the explosions were recorded at two points: at the main (*N*1) observation point and at the control (*N*2). Steel samples have been selected as the objects for studying the stress-strain state of underground structures in the form of cylindrical thin-walled shells of a closed section. The kinematic parameters of ground vibration were measured using seismic detectors and an oscilloscope. Ground displacements in three mutually perpendicular directions, which do not follow a linear law, are studied. Mathematical expressions have been selected to describe each of the components of the displacement vector. It is established that the longitudinal component in the equivalent state has a smoother decreasing character. Under the impact of underground explosions, the underground structure vibrates in space in a vertical plane and in two horizontal planes, with an increase in the equivalent distance, the range of vibrations is wider than the others, and the time of action of the waves on the structure increases. The values of logarithmic decrements for each component of the displacement vector of the structure are determined.

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Keywords: underground explosions, field tests, ground environment, explosive seismic waves, underground structures, cylindrical shells, hoop stresses

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Поведение подземного оболочечного сооружения при сейсмовзрывном воздействии

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Аннотация. Приведены результаты натурных экспериментальных исследований по изучению картины распространения сейсмовзрывных волн в грунтовой среде и поведения сейсмонапряженного подземного сооружения, типа цилиндрической тонкостенной оболочки, взаимодействующей с грунтом при сейсмических воздействиях подземных мгновенных взрывов. Было учтено, что сейсмический эффект действия подземного взрыва на подземное сооружение зависит от многих факторов, особенно от физико-механических свойств грунта экспериментальной площадки. Состав грунта был получен при бурении шурфов для взрыва на выброс из вырытой траншеи для укладки образцов подземных сооружений. Колебания грунта при взрывах фиксировались в двух пунктах: на основном ($N1$) пункте наблюдения и на контрольном ($N2$). Объектом изучения напряженно-деформированного состояния подземных сооружений типа цилиндрических тонкостенных оболочек замкнутого профиля выбраны образцы из стали. Измерение кинематических параметров колебания грунта производилось с помощью сейсмоприемников и осциллографа. Изучены перемещения грунта в трех взаимно перпендикулярных направлениях, происходящие не по линейному закону. Подобраны математические выражения для описания каждой из составляющих вектора смещения. Установлено, что продольная составляющая по приведенному состоянию имеет более плавный убывающий характер. При действии подземных взрывов подземное сооружение совершает колебательное движение в пространстве в вертикальной плоскости и в двух горизонтальных плоскостях, с увеличением приведенного расстояния диапазон колебаний шире, чем остальных, время действия волн на сооружение увеличивается. Определены величины логарифмических декрементов затухания для каждой составляющей вектора смещения сооружения.

Ключевые слова: подземные взрывы, натурные испытания, грунтовая среда, сейсмовзрывные волны, подземные сооружения, цилиндрические оболочки, кольцевые напряжения

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1. Introduction

Explosive seismic impacts, depending on the degree of exploration of seismotectonic or man-induced explosive processes and ground conditions of the site, can be determined by any of the methods or by several methods simultaneously: normative, empirical, semi-empirical and analytical. The most probable values of seismic impact parameters and an estimate of their uncertainty should be obtained.

The basis for analytical and experimental evaluation of dynamic properties of soils under such impact are the results of field and laboratory tests of soils. Experimental studies are effectively performed by observing the behavior of a model of the structure or the structure itself in-situ. Extensive measurements of vibration parameters of structures during seismic impact from underground explosions, which most fully reflect natural tectonic earthquakes, can be considered the most perfect experimental method of studying the behavior of the structures. The energy-based evaluation of the process of joint vibration of the underground structure and the environment under explosive seismic waves is of great importance.

The issue of energy-based evaluation of ground behavior during the propagation of seismic and explosive seismic waves was investigated in [1–5].

Deformation characteristics of soils under cyclic loading depend significantly on the level of average stress, porosity and strain amplitude. Dynamic effects of this nature occur during earthquakes. The main parameters used in engineering dynamic analysis of soil base stability at present are the dynamic shear modulus *G* and damping ratio *D*.

Damping ratio (or loss, decay ratio) *D* characterizes the property of materials to absorb dynamic effects.

Russian [6–10] and international scientific literature [11–14] provide systematized results of tests of cohesive and noncohesive soils for measuring the shear modulus and damping ratio under cyclic (low- and high-frequency) loading.

In order to model the conditions of soil under seismic impact, the expected seismic load is calculated before cyclic loading. The methodology for determining its value proposed in [6; 15] is used.

In accordance with this methodology, which has already become conventional, the seismic load is characterized by the value of cyclic shear stress ratio (CSR) for an earthquake of a given frequency:

$$\mathrm{C}SR = \frac{\tau_{av}}{\sigma_v}, \tag{1}$$

where τ_{av} is the average expected cyclic shear stress at the specified magnitude; σ_v' is the vertical overburden stress.

Before the earthquake, the soil element located under the horizontal surface is subjected to consolidation for a long time in state K_0 (K_0 is the ratio of the horizontal and vertical stresses under consolidation in natural conditions). During an earthquake, a series of successive cyclic shear stresses act on this soil element under undrained conditions (Figure 1). These stresses are applied in the absence of lateral deformation, because the flat earth surface is assumed to extend infinitely in the horizontal direction.

The determination of τ_d is based on the idea that seismic shear stresses at a particular point in the soil body arise due to the propagation of mainly transverse waves. Using the technique of its evaluation proposed in [6; 15; 16], the procedure of determining τ_d reduces to the following:

a) assuming that the soil column under the selected elementary volume at depth *h* vibrates as a completely rigid body, the maximum shear stress is represented as

$$\left(\tau_{\max}\right)_i = \frac{\gamma h}{g} \cdot a_{\max}, \tag{2}$$

where $a_{\max}$ is the maximum acceleration on the ground surface; γ is the soil density;

b) the considered soil column actually behaves as a deformable body, so the real shear stress at depth h, $(\tau_{\max})_d$, is lower, and this difference increases with depth:

$$\left(\tau_{\max}\right)_d = r_d \left(\tau_{\max}\right)_i, \tag{3}$$

where $r_d \leq 1$ is the coefficient of stress reduction with depth, adopted according to [6; 15; 16].

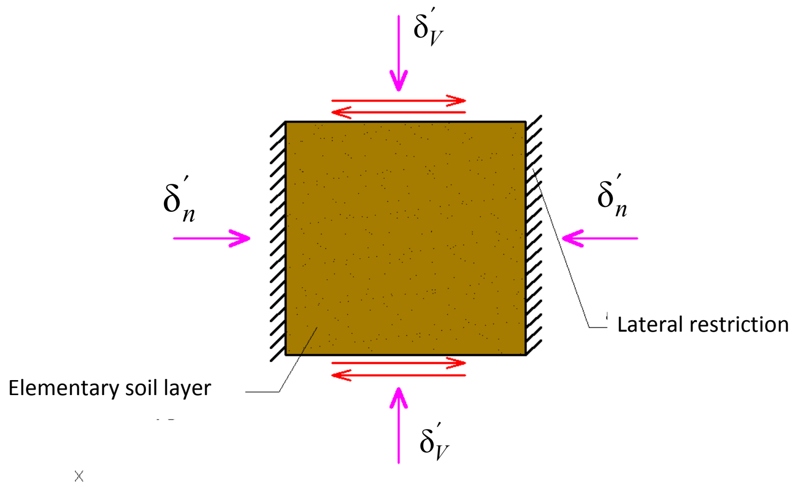

Figure 1. Stress state of soil in field conditions during an earthquake
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

Based on the above, for the depth interval up to 30 m, the maximum dynamic shear stress is assumed to be equal to:

$$\left(\tau_{\max}\right)_d = \frac{\gamma h}{g} \cdot a_{\max} \cdot r_d, \tag{4}$$

The real dynamic loading of the ground during an earthquake is random in nature (Figure 2), and in practical calculations is reduced to a force-equivalent periodic (usually sinusoidal) law.

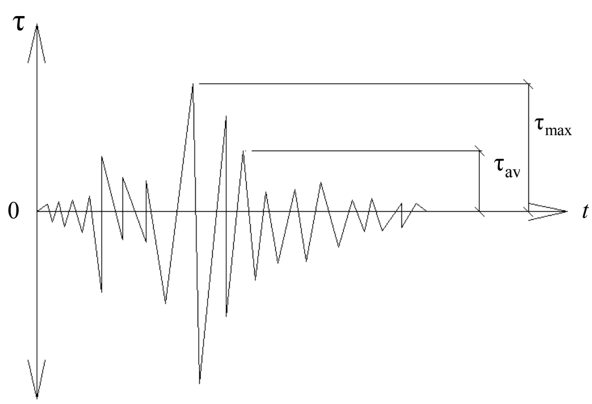

Figure 2. Irregular pattern of shear stress changes during an earthquake
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

The change of shear stress in time caused by the vertical propagation of seismic shear waves through a soil body with a flat surface will be non-periodic and multidirectional when considered in the horizontal plane. In order to quantify the liquefaction characteristics of water-saturated sand layers under complex loading conditions, several correction factors are usually introduced for the cyclic strength obtained under periodic cyclic loading (C_2 and C_5) [15; 16]. Coefficient C_2 takes into account the influence of non-periodic load, which acts in one direction only, coefficient C_5 — the influence of multi-directionality of seismic loading. The product $C_2 \cdot C_5$ reflects the joint influence of non-periodicity and multi-directionality of the loading. Using these coefficients, the relative maximum shear stress $\tau_{max,I}$ / σ_0', causing a particular shear strain under multidirected non-periodic loading, in terms of the relative cyclic stress (τ_{de} / σ_0'), causing the same strain for the same number of cycles of periodic unidirectional loading, can be expressed as

$$\frac{\tau_{\max,i}}{\sigma_0'} = C_2\, C_5 \left(\frac{\tau_{d\,e}}{\sigma_0'}\right) / \sigma_0, \tag{5}$$

where τ_{de} is the amplitude of the periodic cyclic shear stress, causing a particular shear strain; $\tau_{\max,i}$ is the peak value of the non-periodic shear stress under seismic loading, causing the same strain.

Shear stress amplitude τ_{de} under periodic load in equation (5) corresponds to half of the full amplitude of axial stress σ_{de}, which causes liquefaction under triaxial test. Therefore, using the relation $\sigma_0' = \frac{1+2K}{3}\sigma_v' = C_1 \sigma_v'$, formula (5) can be expressed as

$$\frac{\tau_{\max,i}}{\sigma_v'} = C_1 C_2\, C_5 \left(\frac{\tau_{d\,e}}{2\sigma_0'}\right). \tag{6}$$

On the other hand, the relative cyclic stress, defined as the amplitude of periodic shear stress τ_{de}, divided by the effective vertical stress (τ_{de}/σ_v') is related to $\tau_{max,I}/\sigma_v$' as

$$\frac{\tau_{\max,i}}{\sigma_v'} = C_2\, C_5 \left(\frac{\tau_{d\,l}}{\sigma_v'}\right). \tag{7}$$

Generalized results of numerous tests on non-periodic loading [6; 15] show that the value of coefficient $C_2 C_5$ for sandy soils with relative density J_d less than 0.7 is approximately 1.55. Therefore, if taking $K_0 = 1$, and coefficient C_1, according to

$$\sigma_0' = \frac{1+2K}{3}\sigma_v' = C_1 \sigma_v'$$

is equal to 1, by substituting these values into (6) and (7), the following is obtained:

$$\frac{\tau_{\max,i}}{\sigma_v'} = \left(\frac{\sigma_{d\,l}}{2\sigma_0'}\right) = \frac{1}{0.65}\left(\frac{\tau_{d\,l}}{\sigma_v'}\right). \tag{8}$$

This equation establishes relationships between the cyclic strength values, determined using different methods for the cyclic shear stress amplitudes, at which liquefaction begins. They are also valid for the determination of any amplitude of cyclic shear stress regardless of whether its value is sufficient for the liquefaction process. Therefore, equation (8) can be written in a more general form

$$\frac{\tau_{\max,i}}{\sigma_v{}'}=\left(\frac{\sigma_d}{2\sigma_0{}'}\right)=\frac{1}{0.65}\left(\frac{\tau_d}{\sigma_v{}'}\right). \tag{9}$$

Thus, in practical calculations for evaluating the liquefaction potential of clayey and sandy soils with different degrees of water saturation, the average values of shear stresses caused by an earthquake at depth h are determined from the expression

$$\tau_{av}=\left(0.65\frac{\gamma h}{g}\right)a_{\max}r_d. \tag{10}$$

The value of $a_{\max}$ is selected according to the peak horizontal accelerations on the earthquake accelerogram.

Peak vertical accelerations in a scenario earthquake are much smaller than the horizontal components and may not be taken into account in the evaluation of soil liquefaction.

The number of loading cycles (N) in a laboratory experiment modeling seismic effects depends on the duration of the earthquake and, therefore, on the magnitude of the earthquake and is presented in [6; 15–17]. The calculation described above gives the maximum value of the expected cyclic shear stress due to earthquake (τ_{av}), which corresponds to half of the axial dynamic load in triaxial dynamic tests.

2. Purpose and Objective

The purpose of the study is to conduct a full-scale experimental investigation to analyze the propagation pattern of explosive seismic waves in the ground medium and the behavior of a seismically stressed underground structure in the form of a cylindrical thin-walled shell, interacting with the ground under the seismic impact from instantaneous underground explosions.

The objective of the study:

1. Determine the ground stress in three mutually perpendicular directions from the explosion point;
2. Obtain experimental data of seismic vibrations of the underground structure;
3. Determine the values of logarithmic decrements for each component of the displacement vector of the structure;
4. Establish empirical formulas for the relationship between the displacement of the underground structure and the charge weight for the excavation explosion (EE) and the epicentral distance in three mutually perpendicular directions for each displacement component;
5. Determine the maximum values of longitudinal and hoop stresses in the underground cylindrical structures.

3. Materials and Methods

It is commonly known that the seismic impact of an underground explosion on an underground structure depends on many factors, especially on the physical and mechanical properties of the soil of the experimental site, both at the explosion point and in places where the underground structure is installed. For the study, the experimental site was equipped on a relatively flat terrain, one side of which bordered with low hills. The composition of the soil was studied using samples obtained by drilling holes for EE from excavated trenches for installing samples of underground structures. The soil conditions of the experimental site are characterized as silty, of hard consistency, with rare cement-colored calcareous inclusions. The content of fine-grained fraction in the soil body is: fine sand — 27%, silty particles — 53%, clay — 20%. The maximum soil moisture retention capacity depending on depth varies within 17–22 %, the highest value of plasticity is 15–18, specific gravity varies within 1.72 — 2.05 T/m^3.

Ground vibrations due to explosions were recorded at two points:

– at the main (*N*1) observation point: displacements of ground particles in three mutually perpendicular directions (u_o, v_o, w_o), velocity and acceleration of ground motion in longitudinal directions $(\dot{u}_0,\ \ddot{u}_0)$;

– at the control (*N*2) observation point, which is located at a distance of 100 m from the main observation point towards the explosion points, the same parameters of ground vibrations were recorded as at the main point, but only in radial directions.

The object of study of the stress-strain state of underground structures in the form of cylindrical thin-walled shells of closed section are small-thickness steel specimens with the following geometric dimensions:

- type 1 — D_H = 720 mm, L = 6.0; δ = 8 mm;
- type 2 — D_H = 920 mm, δ = 8 mm, L = 6.0;
- type 3 — D_H = 1050 mm, δ = 12 mm, L = 6.0.

These specimens were placed at a depth of 5.0 m from the daylight ground surface. Seismometers were installed in three sections of the structures to measure ground displacement, velocity and acceleration.

To determine the interaction parameters, in addition to the kinematic parameters of vibration of the structure, the incident loads on the structure in three mutually perpendicular directions were measured simultaneously. In this case, membrane-type sensors were used, installed in 7 points, which were appropriately calibrated before the experiments.

The kinematic parameters of ground vibrations were measured using seismometers and oscilloscopes. Seismometers of the following types were used: VEGIK, S-5-S, OSP-2M, SM-3 and galvanometers of types GB-III-B-5, GB-III-B-10 and GB-IV-V-3, installed in six H-700(H-041) loop oscilloscopes.

To measure the soil pressure on the structures, sensors with membrane thickness of (1-4)10^{-3} m, radius of $22\cdot10^{-3}$m and "FPKA-20" strain-gauge elements, which recorded axial (ε_{ix}) and hoop (ε_{iy}) strains, were used. And in the system of sensors measuring the incident load on the structures, galvanometers of type M1005 (Figure 4) were used.

Signals from the strain gauges and pressure sensors were recorded by four H-117/1(H-115) oscilloscopes using M1005 and M017 type galvanometers.

The oscilloscopes were initiated with the help of special triggering devices providing simultaneous start of all oscilloscopes and the explosion, as well as automatic stop after the decay of the vibration process. This allowed to determine the propagation velocity of explosive seismic waves.

In the explosions, grammonite 79/21 was used as the charge, which in all cases was initiated by a detonating cord and an instantaneous electric detonator. Explosions were performed through the oscilloscopes by breaking the loop, which was wrapped around the detonator. Explosions with the EE charge weight of 420–7000 kg were made. The explosions were carried out approaching the specimens under study.

Since the conducted explosions differed significantly in weight, and the recorded ground and structure vibrations are associated with different distances from the explosion point, parameter R_{eq} (equivalent distance) was used for comparing observations with each other:

$$R_{eq}= R/\sqrt[3]{C}\ \ [\text{m*kg}^{-1/3}], \tag{11}$$

where *R* is the epicentral distance (m); *C* is the EE charge weight (kg).

4. Results and Discussion

The results of experiments for studying the ground displacement in three mutually perpendicular directions, depending on the equivalent distance, are shown in the form of a graph in Figure 3.

It can be seen from the figure that the components of the displacement vector are comparable with each other, moreover, the change of these parameters does not follow a linear law. From the curves of the displacement components, it can be observed that the longitudinal component curve has a smoother decreasing character with the increase in equivalent distance than the other ones.

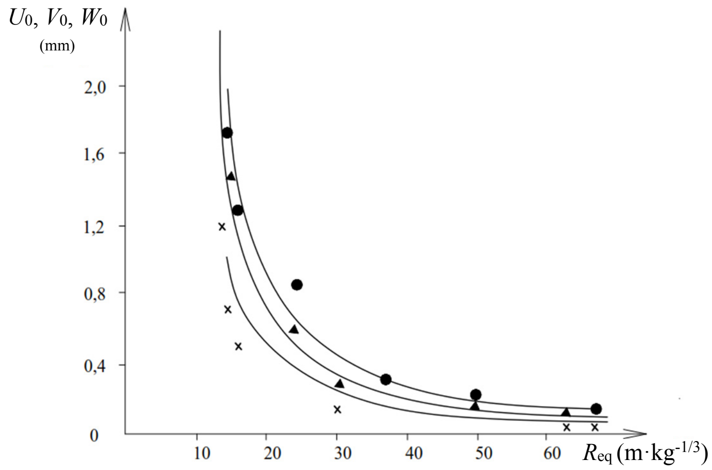

Figure 3. Relationship between the absolute maximum ground displacements in three mutually perpendicular directions and the equivalent distance:
Δ — vertical displacement; • — longitudinal displacement; *X* — transverse displacement
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

Each component of the ground displacement vector was approximated. Despite some scatter of experimentally obtained points, it can be stated that the experiments with different equivalent distances are sufficiently described by the following relationships [18]:

- for the longitudinal component:

$$A_{\text{lon}} = 6.21\mathrm{e}^{-0.0976\,R\text{eq}} \quad \text{or} \quad A_{\text{lon}} = 101.1\left(\frac{\sqrt[3]{C}}{R}\right)^{1.57}, \tag{12}$$

- for the transverse component:

$$A_{\text{tr}} = 2.37\mathrm{e}^{-0.062R\text{eq}} \quad \text{or} \quad A_{\text{tr}} = 5.43\left(\frac{\sqrt[3]{C}}{R}\right)^{0.72}, \tag{13}$$

- for the vertical component:

$$A_{\text{vert}} = 2.62\mathrm{e}^{-0.058R\text{eq}} \quad \text{or} \quad A_{\text{vert}} = 216.4\left(\frac{\sqrt[3]{C}}{R}\right)^{1.84}. \tag{14}$$

Below, some experimentally obtained records of seismic vibrations of the underground structure, as well as its absolute displacements, are presented. The records of seismic vibrations of the underground structure in three mutually perpendicular directions in the form of oscillograms are shown in Figure 4.

Figure 4 shows the vibrations of the underground structure in different directions. As a result of underground explosions, the underground structure vibrates in space in the vertical plane and in two horizontal planes. It should be noted that the natural vibrations of the underground structure here do not have an obvious form.

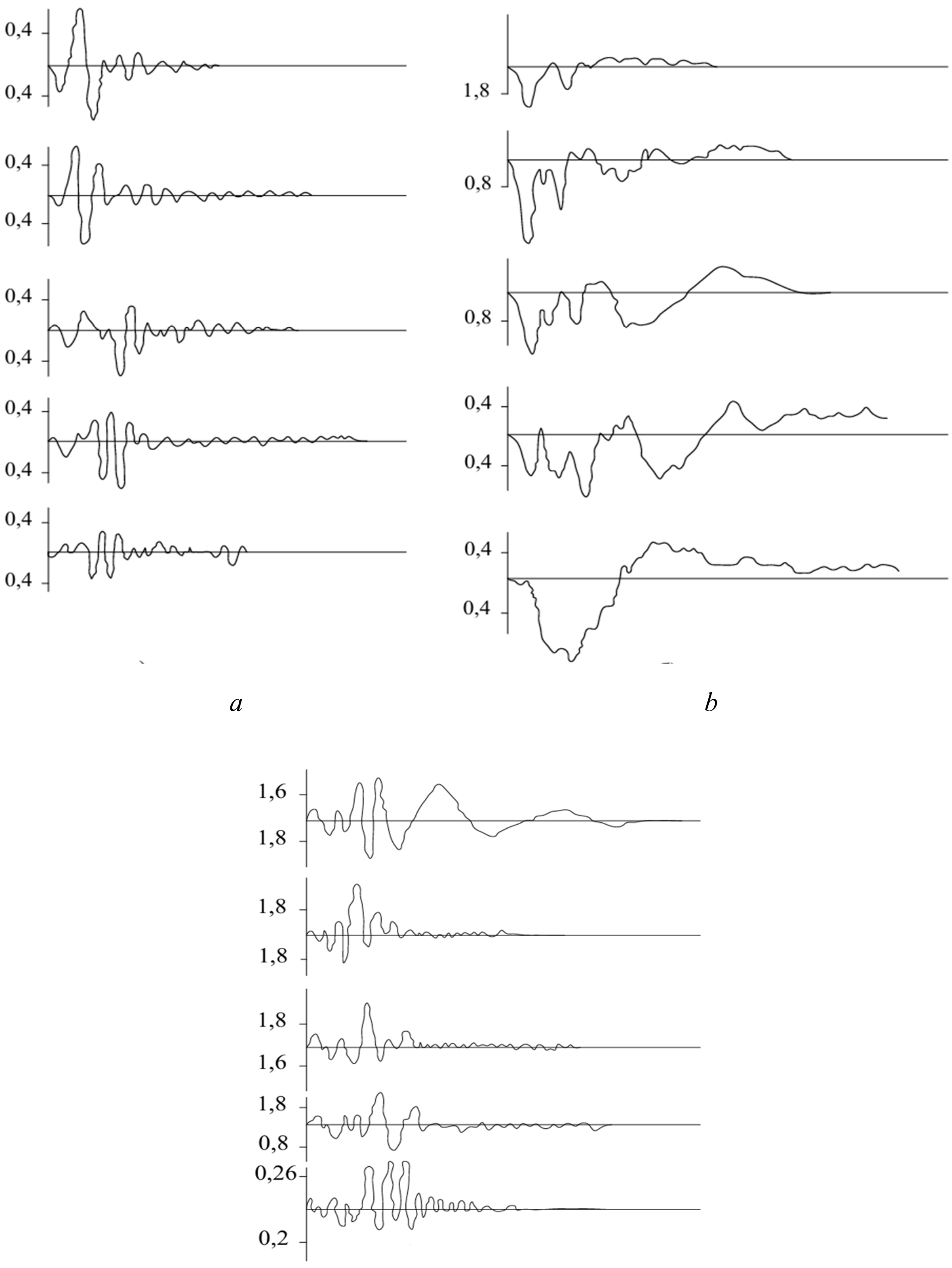

Figure 4. Records of vibrations of the underground structure:
a — in the longitudinal direction; *b* — in the transverse direction; *c* — in the vertical direction
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

Some asynchrony is observed in the motion of the structure. The asynchrony in motion, apparently, indicates that some total motion was recorded in the experiments. By parallel consideration of the records of wave patterns obtained for large equivalent distances, similarities were observed between all records. It is can be easily seen that in the record regions with maximum amplitudes, the peak corresponding to the maximum moves towards the end of the record as the equivalent distance increases. Therefore, they are carried over by waves having a relatively low propagation velocity. Hence, it can be said that with the increase of the equivalent distance, the vibration range is wider than for others, the time of impact of the waves on the structure increases.

It can be seen from the figure that the vibration pattern of the underground structure is not quite a correct increasingly-decaying sinusoid, without any impulsive superimpositions. The peak corresponding to the maximum at small equivalent distances is observed at the beginning of the motion, i.e., the vibration of the structure begins with a very sharp entry of large amplitude. The similarity of the pattern between the longitudinal and vertical components is that both oscillograms do not start with a sharp entry of maximum amplitude. The special aspect of the structure vibration in the transverse direction is that it starts with a sharply pronounced peak, which corresponds in value to the maximum. On the basis of the structure vibration records, the values of decrements for each component of the displacement vector of the structure were determined. The average value of the decrement for the longitudinal component is $\lambda_{lon} = 0.54$; for the transverse — $\lambda_{tr} = 0.68$; and, finally, for the vertical component — $\lambda_{vert} = 0.79$.

By paying attention to the average values of the decrements for each component and comparing them with each other, the following inequality can be established: $\lambda_{lon} < \lambda_{tr} < \lambda_{vert}$, i.e., the average value of the decrement of the vertical component is larger than the others. If vibration decay in the horizontal direction (along the *O-X* axis) is mainly due to the compliance of the ground or, in other words, in this direction, damping occurs due to the interaction or overcoming the cohesion energy of contact between the body of the structure and the ground, then for the vertical component the structure vibration decay is associated with the dissipation of energy as a result of significant strains of the ground [19].

From the comparison of experimental data shown in Figures 3 and 5, it follows that the maximum values of longitudinal and transverse horizontal displacements of the structure are smaller than those of the ground medium.

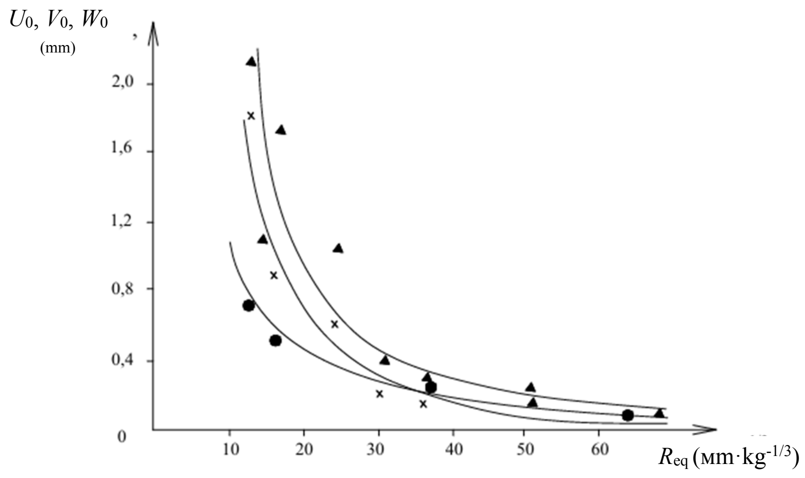

Figure 5. Relationship between the absolute maximum displacements of the underground structure in three mutually perpendicular directions and the equivalent distance: Δ — vertical displacement; • — longitudinal displacement; *X* — transverse displacement

Source: made by B.S. Rakhmonov, I.T. Mirsayapov

The curves of the relationship between the absolute displacement of the structure in three mutually perpendicular directions and the equivalent distance were approximated (Figure 6).

Empirical formulas for the relationship between the displacement of the underground structure and the weight of the EE charge and the epicentral distance in three mutually perpendicular directions for each displacement component were derived:

- for the longitudinal displacement:

$$U = 9.29\left(\sqrt[3]{C} / R\right)^{-0.988} \quad \text{or} \quad U = 2.16\, e^{-0.069R\text{eq}};$$

- for the vertical displacement:

$$W = 141\left(\sqrt[3]{C} / R\right)^{-1.67} \quad \text{or} \quad W = 4.11 e^{-0.067R\text{eq}}; \tag{15}$$

- for the transverse displacement:

$$V = 700\left(\sqrt[3]{C} / R\right)^{-2.44} \quad \text{or} \quad V = 7.8 e^{-0.13R_{\text{eq}}}. \tag{16}$$

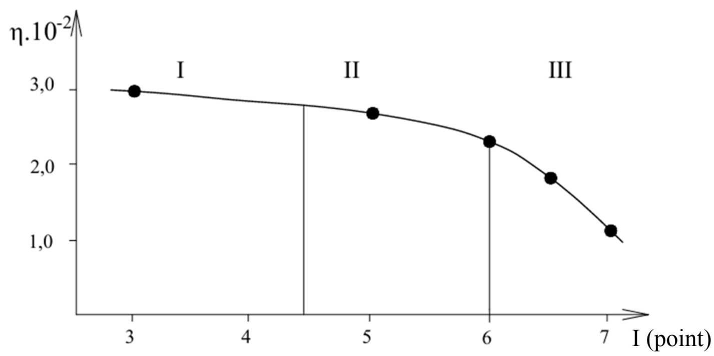

Figure 6. Relationship between the η coefficient and the intensity of explosive seismic vibrations of the ground environment; I, II and III — regions
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

The analysis of the experimental results allowed to obtain the relationship between the displacement of the underground structure and the weight of the EE charge and the epicentral distance to the explosion points in the following form:

$$A = C^{0.392} e^{-0.012R}.$$

In seismic vibrations caused by underground explosions, the stress-strain state of the underground structure is determined by the amount of energy received by the structure. Therefore, in this study, special attention was paid to the kinetic energy imparted by the explosive seismic wave to the structure. To estimate the ratio between the energy propagating in the ground and the energy received by the underground structure during their interaction, the following expression is used:

$$\eta = E_{\text{k.struct}} / E_{\text{gr}}, \tag{17}$$

where $E_{\text{k.struct}}$ is the kinetic energy received by the structure; E_{gr} is the energy propagating over the cross-sectional area of the underground structure. This expression can be written in the following form:

$$\eta = \rho_c\, \pi\, (R^2 - r^2) L v^2 / 0.35\, S\, \rho c_p \sum v^2 T_i\,. \tag{18}$$

The curve of the relationship between η and the intensity of explosive seismic vibration, obtained on the basis of experimental results and calculated using formula (18) is shown in Figure 6.

It can be seen in the figure that with increasing intensity, ratio η decreases slightly. With increasing intensity of vibration due to underground explosions, the total amount of kinetic energy received by the underground structure increases, and the ratio (η) decreases.

Three regions can be distinguished in the general qualitative characteristic of the relationship between contact forces of the structure with the soil and their relative displacement, according to the experimental diagrams of the test results. The first one corresponds to the stage of loading the structure, when the relationship between the forces and relative displacement of the structure is linear. In this case, the soil is being compacted, and elastic and viscous properties of the body, but not plastic ones, are revealed.

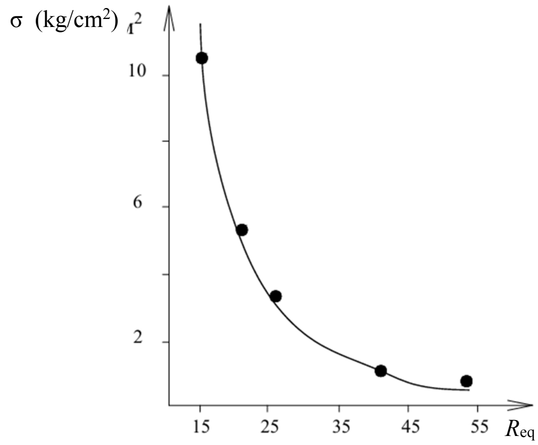

Figure 7. Relationship between radial stresses at the explosive wave front and the equivalent distance
S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

At the second stage, the proportionality between the interaction forces and the displacement of the structure is violated and the elastic character of the interaction is lost. With the increase of the external load it is possible to observe sliding of the underground structure relative to the ground in the third region [1; 20]. By revisiting the graph in Figure 5, it can be concluded that with increasing intensity (external load), the share of energy transferred from the ground to the structure decreases.

The curve of the relationship between the ground stress and the equivalent distance is presented in Figure 7.

It can be seen from the figure that the change in the stress with distance has a non-linear character.

As a result of approximation of the experimental data, the formula for determining the stress in the ground is obtained in the following form:

$$\sigma(R_o) = B\,R_o^{-\eta}\,, \tag{19}$$

where $R_0 = R/0.054\sqrt[3]{C}$ is the equivalent distance, m; C is the EE charge weight, kg; B is a dimensionless coefficient, $B = 10^4$; η is the degree of ground stress decay; $\eta = 2.5$; $15 \le R_0 \le 55$.

To estimate the maximum values of longitudinal bending and hoop stresses in the underground cylindrical structure, where, using the obtained experimental results, similar to works [20; 21], the following empirical expressions are proposed:

$$\sigma_x = k_1\left(\sqrt{C}/R\right)\cdot 1/4\cdot\sqrt[3]{ED^5{}_H\left(\kappa_z L/J\right)^2}\ ; \tag{20}$$

$$\sigma_y = k_1\left(\sqrt{C}/R\right)\cdot 4.25\cdot\left(Eh/D_c^{\,2}\right), \tag{21}$$

where E is the modulus of elasticity of the material of the structure (kg/cm^2); D_H, D_b are the external and internal diameters of the cylindrical structure (cm), $D_c = 1/2\cdot(D_H + D_b)$; k_z is the transverse interaction coefficient (kg/cm^3); L is the length of the cylindrical structure (cm); J is the moment of inertia of the cylindrical structure (cm^4); h is the structure wall thickness (cm); k_1 is the coefficient, which depends on the properties of soil.

To estimate the maximum values of the longitudinal and hoop stresses in the underground cylindrical structures, the above formulas are recommended for use with the values of EE charge weight from 500 to 700 kg and the distance to the structure of 30 to 150 m. Figures 8 and 9 show the relationship between the longitudinal and hoop stresses in cylindrical steel pipes of different diameters and the distance according to the formulas.

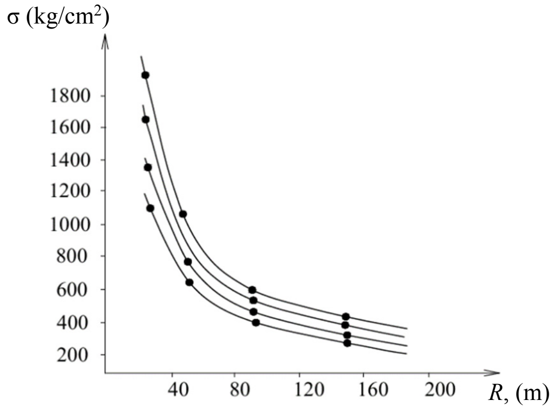

Figure 8. Relationship between longitudinal stresses in steel pipes of various diameters and the distance (The weight of the explosive charge is 400 kg, where 1, 2, 3, 4 are the curves corresponding to:
D_H =720 mm, δ = 8 mm , J =113500 cm^4;
D_H = 920 mm, δ = 8 mm , J = 238500 cm^4;
D_H = 1050 mm, δ = 12 mm, J = 1316000 cm^4;
at values E = 2.1106 kg/cm^2, L = 6.0;)

S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

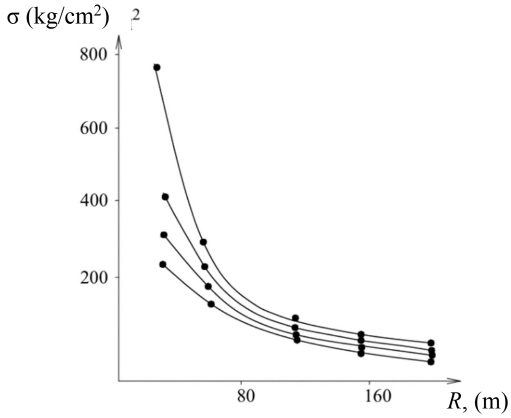

Figure 9. Relationship between hoop stresses in steel pipes of various diameters and the distance. (The weight of the explosive charge is 400 kg, where 1, 2, 3, 4 are the curves corresponding to:
D_H = 720 mm, δ = 8 mm, J = 113500 cm^4;
D_H = 920 mm, δ = 8 mm, J = 238500 cm^4;
D_H=1050mm, δ = 12 mm, J = 1316000 cm^4;
at values of E = 2.1×106 kg/cm^2, L = 6.0;)

S o u r c e: made by B.S. Rakhmonov, I.T. Mirsayapov

The above experimental results can be used to predict the behavior of underground thin-walled structures subjected to explosive seismic waves. The calculations show that the relationships can be used with sufficient accuracy in estimating the seismic intensity of the explosive seismic waves.

5. Conclusion

1. The non-linear explosion-induced ground displacements occurring in three mutually perpendicular directions were studied. Mathematical expressions for describing each component of the displacement vector were established. It was found that the longitudinal component of the equivalent state has a smoother decreasing character.

2. It was established that as a result of underground explosions, the underground structure vibrates in space in the vertical plane and in two horizontal planes. The range of vibrations is wider than the other ones with the increase of the equivalent distance, the time of action of waves on the structure increases.

On the basis of the structure vibration records, the values of the logarithmic decrements for each component of the displacement vector of the structure were determined and the following inequality was established: $\lambda_{lon} < \lambda_{tr} < \lambda_{vert}$, i.e., the average value of the decrement of the vertical component is larger than the others.

3. It was established that if the vibration decay in the horizontal direction (along the *O-X* axis) is mainly due to the compliance of the ground or, in other words, in this direction, damping occurs due to the interaction or overcoming the cohesion energy of contact between the body of the structure and the ground, then for the vertical component the structure vibration decay is associated with the dissipation of energy as a result of significant strains of the ground.

4. During seismic vibrations caused by underground explosions, the stress-strain state of the underground structure is determined by the amount of energy received by the structure. To estimate the ratio between the energy propagating in the ground and the energy received by the underground structure during their interaction, the use of coefficient η was proposed.

It was found that as the intensity of explosion-induced vibration increases, the total amount of kinetic energy received by the underground structure increases, but the η ratio decreases.

5. Three regions can be distinguished in the general qualitative characteristic of the relationship between contact forces of the structure with the soil and their relative displacement, according to the experimental diagrams of the test results. The first one corresponds to the stage of loading the structure, when the relationship between the forces and relative displacement of the structure is linear. In this case, the soil is being compacted, and elastic and viscous properties of the body, but not plastic ones, are revealed. At the second stage, the proportionality between the interaction forces and the displacement of the structure is violated and the elastic character of the interaction is lost. With the increase of the external load it is possible to observe sliding of the underground structure relative to the ground in the third region.

6. As a result of approximation of experimental data, a formula for determining the stress in the ground was obtained. The experimental and theoretical results are suitable for predicting the behavior of underground thin-walled structures under the impact of explosive seismic waves. Calculations show that they can be used in estimating the seismic intensity of explosive seismic waves with sufficient accuracy.

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