THE PROBLEMS OF THE RELIABILITY AND OPTIMALITY OF THE EARTHQUAKE-PROOF STRUCTURES

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SUMMARY

The report is devoted to the problem of the optimum designing of the earthquake-proof structures. Some considerations of the authors on the principal questions, connected with this problem are regarded here as well as some particular results are given here concerning this problem.

1. Some Principal Theses.

The problems of optimality and closely connected with them problems of reliability are of great importance in all spheres of human activity. In technique these problems are connected with the development of such constructions or schemes, which at a minimal expense should possess the maximal effectivness of their design reliability degree.

The solution of these problems come across the fundamental difficulties, even in those cases, when the task, as a whole, can be settled to the purely deterministic scheme when we have full initial information. These difficulties grow rapidly when instead of deterministic scheme, a probabilistic or stochastic scheme are to be introduced. Naturally it is supposed that we have sufficient initial probabilistic information for the solution of this task. And, at last, the task grows to be more complicated, and in some sense, it becomes even infinite if initial probabilistic information is uncomplete and is not sufficient. Namely, with the situation given above we should come across in the theory of earthquake-proof structures. Therefore, the development of optimality and reliability of earthquake-proof constructions cannot be based only on the pure theoretical and experimental investigations.

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Here the engineering investigation of the earthquake results and the engineering intuition are of great importance. Along with this same peculiarities in the behaviour of the structures during the earthquake, to which up to the present time, nobody payed special attention, should be properly investigated. Some items of this report are devoted to such problems. In general, the solution of the problem of optimality and reliability upon designing some types of structures should be presented as follows:

A. Reasonable Choice of Structure Design Model.

It should be taken into account to consider the really important factors, determining the behaviour of structure and ignore all unimportant ones so as not to complicate the whole process of the designing of a structure.

In many cases such factors as elastic-plastic behaviour, the development of one-sided ties (cracks) accumilation of damages, etc. play an important role when an attempt is taken to choose the design model.

It should be noted that the reasonable choice of a design model of a structure or a building is a very difficult task and, for its right decision, it needs profound knowledge of regularity of the work of structures during the earthquake and pure engineering intuition. The right choice of design models has a decisive importance for a ground approach to the problem of optimality and reliability in the construction of earthquake-proof structures.

B. The Choice of Statistic Hypothesis About the Character of Earthquake Ground Vibration.

As it is known we have not sufficient initial probabilistic information for a stochastic designing of a structure under the earthquake. Nevertheless, according to the data we possess (accelerdgrammes of earthquakes, having taken place in different hydrogeological conditions) we can offer in many cases reasonable statistic hypothesis concerning the character of earthquake vibration. These hypotheses can be checked by the method given below.

C. The Designing of an Optimal Structure and the Analysis of its Reliability in the Supposition that the Structure works on given above Hypothetic Stochastic Vibration.

The results of many authors, having investigated this problem in the recent years, can be used for the development of optimum models of structures. However the work should go on. As far as the designing of the reliability of structures is concerned, here the general methods of the theory of reliability can be used, as it is supposed that the external stochastic excitation is given. All these tasks, however, can be solved, as a rule, with the use of electronic computers.

D. The Testing of the accepted Stochastic Hypotheses on the Base of Integral Analysis of the Consequences of Earthquakes.

After the calculations of a great number of different structures and buildings are performed, the results of these calculations can be compared with the integral analysis of the consequences of earthquake. This comparison can be presented as a ground for the accepted stochastic hypotheses about the character of external forces during an earthquake.

That is a rather complicated way of solving the problem of optimality and reliability of an earthquake-proof structure under conditions of unsufficient initial stochastic information. We cannot give more detailed information on this problem as the space of the report is restricted. But it is clear, the decision of this problem requires great efforts of large groups of scientists for many years. Below, as it has already been noticed, the results of some particular investigations, connected with the problems given above, will be shortly regarded.

2. The Dynamical Response of the Structure with the Brittle Local Damages, Developing during the Process of an Earthquake.

Estimating the reliability of the structure which is under the earthquake, the local damages are permitted. These damages are usually taken into account by the way of regarding of the vibrations of elastic-plastic systems /3, 5, 12/. Some of the recent experimental results /3, 4, 7, 13/ point out on the essential changes of the rigidity of the structures and their natural frequences during the accumulation of damages. In some cases the magnitudes of the rigidity are decreasing up to 10-20 times

/4, 7, 13/. Some magnitudes can not be explained taking into account the plastic type of damages. The plastic deformations caused sometimes certain decreasing of the rigidity. According to V.V.Moskbitin this decreasing may be 30-40 per cent /10/. But the decreasing of the rigidity by ten times and even more is not due to the plastic deformations, but owing to the development of brittle more exactly quasi-brittle damages /4, 11/. The peculiarity of the brittle damages, which is quite different of the plastic ones, is in the absence of the residual displacements and in the development of the essential nonreversible changes of the rigidity of system. Moreover, the great quantity of energy is spent on the damage /6, 8/. Particularly essential changes of the rigidity, as a result of the brittle damages, take place in the case when the structure is a combination of plane elements, for example, a frame with a brick masonry filling or a box-type structure.

Many building constructions are a combination of brittle and ductile materials, for example, reinforced concrete, a metal frame with a brick filling etc. It is particularly reasonable to take into account the brittle local damages for the analysis of such structures.

Some problems concerning different aspects of this complicated question were regarded. The aim of this investigation was to come nearer to the development of the practical earthquake analysis of structures with brittle local damages taken into account.

A. Earthquake Response of a System with a Random Spring Constant, Decreasing as the Brittle Local Damages Develop.

The problem concerns the earthquake response of a linear, more exactly step-linear, oscillator with ideal brittle cut-out ties. The effect of the change of random spring-constant on the earthquake response of the system was considered. The possibility of the replacement of the random law change of spring-constant by the equivalent deterministic law (see art./4/) was also considered.

The movement of such a system is described by the equation:

$$\ddot{x} + p\dot{x} + [q_0 + \bar{a}(t)]x = f(t) \tag{2.1}$$

or

$$x + p\dot{x} + q_0 x = f(t) - \bar{a}(t)x \tag{2.2}$$

$x(t)$ - is a non-stationary random process and can be given as

$$x(t)=\langle x\rangle(t)+x_c(t) \tag{2.3}$$

We shall believe, the ground acceleration $f(t)$ and the random process $\bar{a}(t)$ are connected by mutual correlation function $R_{fa}(\tau)$. The solution of the given problem was obtained by the using of the methods of the theory of the functions of sensibility. Dropping the intermediate stages of the solution we shall give the obtained expression for the dispersion of the process of the earthquake response

$$R_{x_c}(0)=R_{x_o}(0)-2\langle x\rangle\int_0^\infty \kappa(\sigma)d\sigma\int_0^\infty R_{fa}(\sigma-\theta)\kappa(\theta)d\theta+ \tag{2.4}$$
$$+\int_0^\infty \kappa(\sigma)d\sigma\int_0^\infty\{\langle x\rangle^2 2R_a(\sigma-\theta)+R_{x_o}(\sigma-\theta)+R_{ax_o}(\sigma-\theta)R_{x_oa}(\sigma-\theta)\}\kappa(\theta)d\theta$$

where

$$R_{x_o}(\tau)=\int_0^\infty \kappa(\tau)d\tau\int_0^\infty R_f(\tau-\theta)\kappa(\theta)d\theta \tag{2.5}$$
$$R_{ax_o}(\tau)=\int_0^\infty R_{fa}(\tau-\theta)\kappa(\theta)d\theta$$

The solutions were obtained for two models of ground acceleration. One of them was white-noise, the other the Gaussian random process with a cosinus-exponential correlation function /1/. Two variants of the random process of the spring constant change were also considered. One of the variants is a stochastic Markovian process with independent increments, constant and separable. The stochastic process $\bar{a}(t)$ can have one or several final steps of one and the same sign of the value h. The step of such a process has a Poisson distribution. The correlation function will be the following $R_a(\tau)=h^2$ and mutual correlation functions are approximated by the functions:

$$R_{fa}(\tau)=c_f^2 h e^{-|\tau|}$$
$$R_{ax_c}(\tau)=\frac{c_f^2 h\Omega^2}{(1+\Omega^2)\Omega^2-h^2}e^{-|\tau|} \tag{2.6}$$

The dependence of the dispersion of the earthquake response of the system out of the dispersion of the change of spring-constant has been built on the ground of /2, 4) by utilizing the property of the mutual correlation function. The diagram of the dependence of the function $\frac{R_x(0)}{C_f^2}$ from $\frac{R_a(0)}{q_o}$ taking into account $f(t)$ as white noise, is shown in Fig.1. As it can be seen from the diagram great changes of spring-constant lead to small changes of amplitude of the system response.

Let us compare the elastic response $R_{x_0}(0)\,q_0$ in initial system (without cutting-out of the ties) and in the system with cut-out of the ties $[(R_{x_{np}}(0) + \langle x \rangle)\,q_{np}]$ The diagram characterizing the decreasing of the spring response with the accumulation of brittle local damages, is given for the case $R_f(\tau) = C_f^2\,\delta(\tau)$ in Fig. 2-a, for the case $R_f(\tau) = e^{-\alpha|\tau|} \cdot \cos\Omega\tau$ in Fig. 2.b,c.

Thus the designing of the limit system, that is the system, left after the internal brittle ties being damaged, (for example the frames after the filling being damaged) can be analysed, taking into account the stresses, which are less, than the stresses acting upon the initial system under the same external forces.

B. The Energetic Evaluation of the Bearing Capacity of the Earthquake-proof Structures, Taking into Account brittle local Damages.

The way of evaluation of the systems with brittle damaging ties, given below, is based on the following hypothesis. The earthquake motion of the ground is Gaussian stationary process with the correlation function of $R_f(\tau) = e^{-\alpha|\tau|} \cdot \cos\Omega\tau$ /1/. At the first stage the earthquake response is determined and the usual analysis of an elastic system is done. At the second stage of the designing the summarized value of the outliers of a random process during the time of an earthquake is calculated. As a design criteria for characterizing the bearing capacity of the earthquake-proof structure the following condition is taken into account

$$\sum K = W \qquad (2.7)$$

where $\sum K$ - is a summarized value of a kinetic energy given to the system in the account of outliers above the level, determined on the first stage of the designing,

W - the energy, absorbing by the system during the accumulation of the brittle local damages.

The number of outliers during the time T_e of an earthquake is determined by the formula

$$N_a = 2T_e \frac{\sigma_{\ddot{x}}}{2\pi\sigma_{\dot{x}}}\, e^{\frac{(b-\langle x \rangle)^2}{2\sigma_{\dot{x}}^2}} \qquad (2.8)$$

In order to estimate the value W for different structures a programme of the experimental investigations was led. The programme of the experiments is going on at the present time.

Nomenklature

$\bar{a}(t)$ - stochastic process of the rigidity decreasing,

$f(t)$ - ground acceleration,

q_o - spring constant of the initial system,

q_{np} - spring constant of the limit system,

$\langle \ldots \rangle$ -expectation,

$x_c(t)$ - random component of the process,

$R_{ax_c}(\tau)$ - mutual correlation function of processes $\bar{a}(t)$ and $x_c(t)$

$R_{fa}(\tau)$ - mutual correlation function of processes $f(t)$ and $\bar{a}(t)$

$K(\theta)$ - impulsive transitive function,

$R_a(\theta)$ - autocorrelation function of the process $\bar{a}(t)$,

$R_{x_c}(\theta)$ - auto-correlation function of the process $x_c(t)$,

$R_{x_o}(0)$ - dispersion of the process $x_o(t)$

$x_o(t)$ - motion of the system with constant parameters,

C_f^2 - intensity of the earthquake acceleration represented as "white noise",

$\Omega = \sqrt{\omega_o^2 - \left(\frac{p}{2}\right)^2}$

3. The Dynamical Response of a n -Degree of Freedom System Under Non-Stationary Earthquake Random Excitation.

The majority of specialists in the field of earthquake -proof structures believe that at present time the best method of description of an earthquake vibration is the method of representing them as a non-stationary random process. Present stochastic theories of earthquake-proof structures /1, 2, 14, 16/ apply the representation of forces as a stationary random process or special cases of non-stationary processes, restricting the class of regarded structures. The method of calculation given below, using the generalization of spectral representation of the stationary random processes on non-stationary ones makes it possible, having rather general assumption about the character of the vibration of the ground, to obtain probabilistic characteristics of the random process, describing the behaviour of a structure. The class of structures under consideration embraces all the structures being under the investigation of other stochastic theories. The present stochastic theories of earthquake-proof structures use in general time representation of random processes. However, as far as the class of harmonized random processes /9, 18/ is concerned, it is also possible to apply the frequency of frequency-time representation of the non-stationary processes, which have in some cases the advantages in comparison with the time represented /17/.

The main characteristics of a random non-stationary process $a(t)$ within the correlation theory are the second moments:

$K(t,t')$ - the second moment of time representation,

$Q(\omega,\omega')$ - the second moment of frequency representation,

$S(\omega,t)$ - the second moment of frequency time representation.

Having no possibility to regard in detail the physical essence of the given conceptions,we shall present some equations, clearing out their sense.

$$K(t,t')=\langle a(t)\,a(t')\rangle;\quad Q(\omega,\omega')=\langle F(\omega)\bar{F}(\omega')\rangle;\quad S(\omega,t)=\langle F(\omega)a(t)\rangle,$$

where

$$F(\omega)=\int_0^\infty e^{j\omega t}\cdot a(t)\,dt$$

All the second moments are connected with each other by the equation, which permit, if it is necessary, to transfer from one type of representations of a random process to another one.

Let us suppose that the structure is represented as a linear n -degree of freedom system. As it is known, the vibration of any point k of a structure can be written as

$$y_k(t)=\sum_{\ell=1}^{n} q_\ell(t)\, X_\ell(x_k), \qquad (3,1)$$

where $X_\ell(x_k)$- the normal modes of vibration,

$q_\ell(t)$ - generalized coordinates.

For every generalized coordinate it is possible to obtain the following equation of vibration

$$\ddot{q}_\ell + 2h_\ell \dot{q}_\ell + \omega_\ell^2 q_\ell = f_\ell(t)\ , \quad \ell=1,2,\ldots,n \qquad (3.2)$$

here

$$f_\ell(t) = -\delta_\ell\, a(t) \qquad (3.3)$$

where $a(t)$ - is a random earthquake non-stationary process,

δ_ℓ - some constant.

If it is a time representation of a random earthquake process the autocorrelation function for a random process $y_k(t)$, describing the vibrations of any point k of a structure, is presented as

$$K_{y_k}(t_1,t_2)=\sum_i\sum_\ell \eta_{ik}\eta_{\ell k}\int_0^t\int_0^t w_i(t_1-\tau_1)\,w_\ell(t_2-\tau_2)\,K_a(\tau_1,\tau_2)\,d\tau_1\,d\tau_2\ , \qquad (3.4)$$

where $K_a(\tau_1,\tau_2)$- is an autocorrelation function for a random earthquake process.

$$\eta_{ik} = \frac{X_i(x_k)\sum_{\ell=1}^{n} m_\ell X_i(x_\ell)}{\sum_{\ell=1}^{n} m_\ell X_i^2(x_\ell)} \qquad (3.5)$$

m_ℓ - is mass in the point with the coordinate x_ℓ

$w_\ell(t-\tau)$- is the impulsive transitive function of the system being described by the equation (3.2).

Let us introduce a new function $W_K^*(t-\tau)$ with the help of the equation

$$W_K^*(t-\tau)=\sum_{i=1}^{n}\eta_{iK}w_i(t-\tau) \qquad (3.6)$$

Then according to the formula (3.4) we have

$$K_{y_K}(t_1,t_2)=\int_0^{t_1}\int_0^{t_2}W_K^*(t_1-\tau_1)W_K^*(t_2-\tau_2)K_a(\tau_1,\tau_2)\,d\tau_1 d\tau_2 \qquad (3.7)$$

In particular for a dispersion of the displacement of the point K of a structure we have

$$\langle y_K^2(t)\rangle=\sum_i\sum_\ell\eta_{iK}\eta_{\ell K}\int_0^t\int_0^t w_i(t_1-\tau_1)w_\ell(t-\tau_2)K_a(\tau_1,\tau_2)\,d\tau_1\cdot d\tau_2 \qquad (3.8)$$

or

$$\langle y_K^2(t)\rangle=\int_0^t\int_0^t W_K^*(t-\tau_1)W_K^*(t-\tau_2)K_a(\tau_1,\tau_2)\,d\tau_1\cdot d\tau_2 \qquad (3.9)$$

It is a frequency time representation of random processes the connection between the second moments of external random processes and the vibration of a structure is

$$S_{y_K}(\omega,t)=P_{nK}^{-1}(j\omega)\sum_{\ell=1}^{n}\int_0^t\eta_{\ell K}w_\ell(t-\tau)S_a(\omega,\tau)\,d\tau\,, \qquad (3.10)$$

where

$$P_{nK}^{-1}(j\omega)=\sum_{i=1}^{n}\eta_{iK}P_{2i}^{-1}(j\omega)\,, \qquad (3.11)$$

$$P_{2i}(j\omega)=(j\omega)^2+2h_i j\omega+\omega_i^2 \qquad (3.12)$$

Taking into consideration formula (3.6) from the equation (3.10) we have

$$S_{y_K}(\omega,t)=P_{nK}^{-1}(j\omega)\int_0^t W_K^*(t-\tau)S_a(\omega,\tau)\,d\tau\,. \qquad (3.13)$$

If the random processes are given in a frequency representation than the connection between the second moments of the random earthquake excitation and the vibration of the structure is the simplest

$$Q_{y_K}(\omega,\omega')=P_{nK}^{-1}(j\omega)P_{nK}^{-1}(-j\omega')Q_a(\omega,\omega') \qquad (3.14)$$

The initial conditions were supposed to be deterministic. The analogous equation in the case of random initial conditions is not given here for lack of space.

Given in the present report equations which connect the second moments taken in the time, frequency or time-frequency representations of the random processes give the possibility to obtain the probabilistic characteristics of a random process, describing the behaviour of a structure according to the probabilistic characteristics of the random process of the vibration of ground. This gives the possibility to estimate the reliability of earthquake-proof structures.

4. The Designing of Tanks and Structures with Tanks upon the Earthquake Forces.

The hypothesis of a stationarity of an earthquake process with a normal distribution is accepted. It is supposed that the acceleration mean value is equal to zero. The vibration of the structure is regarded in the transient regime.

A. The Earthquake Analysis of the Tank.

The main suppositions. The hydro dynamic designing on the base of which one can determine the earthquake leads of the liquids, which is in the tank, is the main factor for the earthquake analysis of ground tanks. It was taken into account that the mass filling in the tank is considerably larger than the mass of floors and the walls of the tank. Therefore, the loads which are created by the weight of liquid on the account of its mobility are the main factors on the designing of the walls of the tank and its bottom. It is supposed, that the tank is absolutely rigid. The viscosity of liquid is taken into account according to Raley's theory.

On designing the tanks in the earthquake regions it will be useful to establish the necessary chink between the surface of liquid and the floor of the tank. In the presence of such a chink a wave will not be strucked by the floor.

The solution of the hydrodynamic task for a circular cylindrical tank which is partially filled is given below.

Having calculated all the values connected with the probabilistic analysis and the dispersion of the design values and a number of simplicities we shall have the following design formulas:

1) for a chink between the surface of liquid and a floor

$$A_s = 0{,}836\, a K_c \xi_\xi \sqrt{1 - e^{-2\hat{\nu}_1 t}}$$

where $$\xi_\varepsilon = \sqrt{G_{H\ddot{x}_o}(\tilde{\omega}_1)(\tilde{\nu}_1 + \frac{\tilde{\omega}_1}{4\tilde{\nu}_1})}$$

$G_{H\ddot{x}_o}$ - is a normed spectral density of an earthquake acceleration

$$G_{H\ddot{x}_o}(\omega) = 2\alpha \frac{\omega^2 + m^2}{\omega^4 + 2a\omega^2 + m^4} \tag{4.1}$$

$m = \alpha^2 \div \beta^2$; $a = \alpha^2 - \beta^2$; α = 6÷8 (1/sec.) β =14-20 (1/sec.):

K_c - the coefficient of an earthquake $K_c = \frac{K_{\ddot{x}_o}\sqrt{B(0)}}{g}$

$\sqrt{B(0)}$ - the standard of an earthquake acceleration;

t_1 - a design duration of the earthquake

$$P(y) = \gamma_{o\delta}\, a\, K_c\, \xi_p \sqrt{1 - e^{-2\tilde{\nu}_1 t_1}}$$

where $\gamma_{o\delta}$ - a volume weight of liquid

ξ_p - is defined according to Fig. 3 in dependence on $\tilde{\nu}_1$, E_1, $\tilde{\omega}_1$

$$E_1 = 0{,}418\left[1 - \frac{ch(1{,}84\, h_o\, y/h}{ch(1{,}84\, h_o)}\right]$$

2) a summarized force of the hydrodynamic pressure

$$X_r = P_ж K_c \xi_p \sqrt{1 - e^{-2\tilde{\nu}_1 t_1}}$$

ξ_p - is defined according to Fig. 3 in dependence on $\tilde{\nu}_1$, E_1, $\tilde{\omega}_1$

$\tilde{\omega}_1$ - in this case is calculated according to the formula

$$E_1 = 0{,}418 - 0{,}237\,\frac{th(1{,}84\, h_o)}{h_o}$$

B. The Designing of the Structures, Carrying Partially Filled Tanks on the Action of Earthquake Forces.

The main equations for the investigation of one-mass system in the presence of an earthquake vibration of the ground are the following

$$\begin{aligned} &\ddot{x}_1 + (u + iv)\Omega^2 x_1 + \sum_{K=1}^{\infty} m_K \ddot{\xi}_K = -\ddot{x}_o \\ &\ddot{\xi}_K + 2\tilde{\nu}_K \dot{\xi}_K + \tilde{\omega}^2 \xi_K + \ddot{x}_1 = -\ddot{x}_o \end{aligned} \tag{4.2}$$

where $\ddot{x}_0$ - the acceleration of the vibration of the ground

$$\Omega^2 = \frac{k}{m + m_ж}$$

k - the coefficient of the rigidity of a system,

$m_ж$ - the mass of liquid,

m - the mass of structural elements

$$m_к = \frac{B \cdot B_к}{m + m_ж}; \quad u = \frac{4 - \gamma^2}{4 + \gamma^2}; \quad v = \frac{4\gamma}{4 + \gamma^2}; \quad \gamma = \frac{\delta}{\pi}$$

δ - a logarithmic decrement of a damping.

In order to determine the complex transmission in a transient regime it is necessary to solve an equation (4,2) when $\ddot{x}_0 = e^{i\omega t}$ and we have zero initial conditions.

Solving the equation (4.2) by the operational method we shall have

$$x_1(i\omega t) = \sum_{n=1}^{\infty} \frac{F_1(P_n)}{P_n F_2'(P_н)} e^{P_n t}$$

where

$$F_1(p) = -\bar{c}\frac{P}{P - i\omega} + \sum_{k=1}^{\infty} m_к \frac{(\tilde{\omega}^2 + \tilde{\nu}_к^2)}{2i\tilde{\omega}_к}\left\{\frac{P e^{-2i\varphi_к}}{(P - i\omega)[P - (-\tilde{\nu}_к + i\omega)]} - \right.$$

$$\left. - \frac{P \cdot e^{2i\varphi_к}}{(P - i\omega)[P - (-\tilde{\nu}_к - i\tilde{\omega}_к)]}\right.$$

$$F_2(p = \bar{c}p^2 + (u + iv)\Omega^2 - \sum_{к=1}^{\infty} \tilde{m}_к \frac{(\tilde{\omega}_к^2 + \tilde{\nu}_к^2)}{2}\left[\frac{P^2 e^{-2i\varphi_к}}{P - (-\tilde{\nu}_к + i\tilde{\omega}_к)} - \frac{P^2 e^{2i\varphi_к}}{P - (-\tilde{\nu}_к - i\tilde{\omega}_к)}\right]$$

P_n - the root of the equation $F_2(p) = 0$

As we have the value of a complex transmission in a transient regime we can calculate the dispersion of a displacement

$$\langle x_1^2 \rangle = \frac{B(0)}{2\pi}\int_{-\infty}^{\infty} |x_1(i\omega t)|\, G_{н\ddot{x}_0}(\omega)\, d\omega$$

The dispersion of the dynamical coefficient $\xi(t)$ which is the ratio of $\langle x_1^2(t)\rangle$ K $\langle x_{ст}^2\rangle$ $\langle x_{ст}^2\rangle = \frac{B(0)}{\Omega}$ is the dispersion of a statical displacement

$$\langle \xi^2(t)\rangle = \langle x_1^2(t)\rangle \Omega^4 B^{-1}(0)$$

Let us assume that formula (4.1) is for a normed spectral density of acceleration. In Fig. 4 there are diagrams $\xi(t)$ built with the help of a computer.

Analysing the diagrams given in Figs. 3, 4 one can come to the following conclusions.

1. The maximal value of the standard of a dynamical coefficient of the system with liquid masses in a transient regime is approximately in 1.3-1.4 times larger than the corresponding standard of a dynamical coefficient in a stationary regime.

2. The duration of the transient regime in the system with liquid masses is considerably larger than that of the transient system in which the liquid can be considered as a solid body.

The designing value of an earthquake load is

$$S = \xi(t)\, m\, j_{расч} ,$$

where m - is the mass of all the structural elements and liquid,

$j_{расч} = K_{\ddot{x}_0}\sqrt{B(0)}$ - is the designing value of the earthquake acceleration for a given region, which is expressed through the coefficient of an earthquake K_c

The designing value of an earthquake acceleration is

$$j_{расч} = K_{\ddot{x}_0}\sqrt{B(0)}\, g^{-1}\cdot g = K_c\, g$$

Let us assume that the maximal value of the standard of a dynamical coefficient in a transitional regime is in 1.4 times larger than that of a stationary regime. Then the earthquake force for one-mass system will be calculated according to the formula

$$S = 1{,}4\, Q\cdot K_c\, \xi$$

where Q - is the weight of all the structural elements and liquid

ξ - is the standard of a dynamical coefficient in a ststionary regime.

The material given in this paper is taken out of Nickolaenko's book /12/ in which the methods of the designing of elastic linear, non-linear and parametric systems with tanks containing liquid are under the influence of earthquake, windy and transport loads.

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The investigations, the results of which are given in this report, have been carried out in the Central Laboratory of Earthquake-Proof Building Structures (TSNIISC, Gosstroy USSR) in accordance with the programme on research of the earthquake-proof structures.

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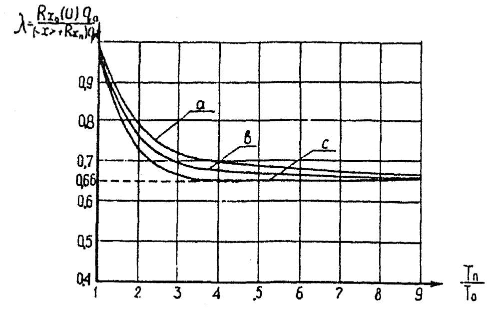
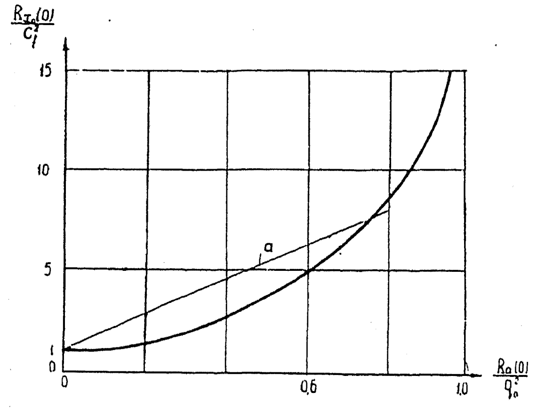


Fig 2



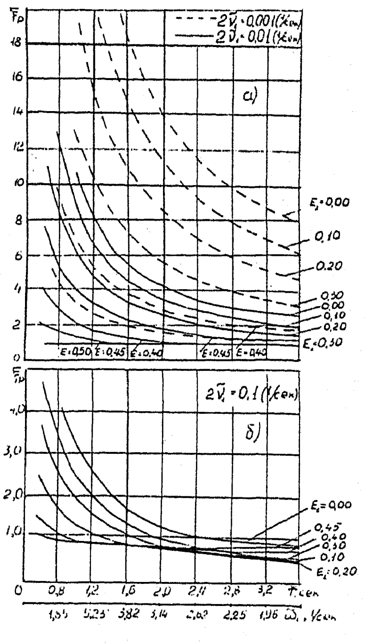
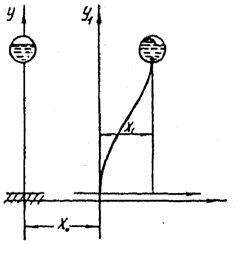


Fig 3. The ξ-T diagram.



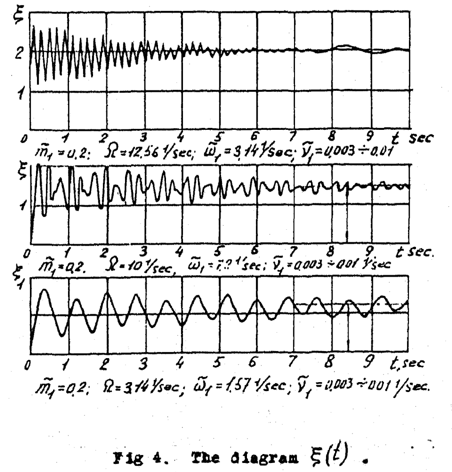


Fig 4. The diagram $\xi(t)$.