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METHOD OF CALCULATION OF RISK OF BEGINNING OF LIMITING STATES IN ROAD BRIDGES FERRO-CONCRETE SPANS AT EARTHQUAKES

Method of calculation of limiting states beginning probability in normal cross-section of main beams of articulated ferro-concrete spans of a road bridge on the base of models applied in risk theory is described. Suggested procedure orients on using of modern finite elemental complexes and programs of probability analysis. Example of calculation of span of standard construction with the use of suggested method is given.

Keywords: ferro-concrete spans, road bridge structures, seismic forces, risk, limiting state, probability, analysis procedure.

Ensuring of operational reliability of transport facilities on the roads in earthquake-prone areas around the world is an important problem, and its solution determines security and economic stability of the region. In this paper a probabilistic approach to assessing the risk of destruction of buildings, which can be used both for bridges on roads, and for others responsible buildings and structures of various purposes is offered.

According to statistics, during the earthquake the most likely failures are failure of beam, slab-beam and slab bridge spans made of reinforced concrete, because they are the most massive, and thus under seismic actions considerable forces of inertia are developed in them.

In accordance with the theory of risk [1] the probability of failure (risk) of span consisting of several united one-span beams $P_м$ is

$$P_м = P_F \cdot P_f\,, \tag{1}$$

where P_f is the probability of a limit state in the most intense beam of span, P_F is the probability of span failure at limiting state in the most intense beam.

Assuming in the first approximation $P_F = 1.0$, that meets the requirements of [2—4], the risk of collapse during the operation in earthquake-prone period is determined by the probability of a limit state in the most intense beam $P_м = P_f$.

As a criterion for assessing the possibility of a limit state in one of the beams assume bending moment in its normal cross-section near the middle of span. The reserve of strength *S* in the base of which we will make the calculation of risk is determined by formula

$$S = \mathrm{M}_{\mathrm{pred}} - M_{\mathrm{max}}\,, \tag{2}$$

where M_{max} is the maximum bending moment from the of joint effect of permanent and temporary loads and seismic forces determined in accordance with the expression

$$M_{\mathrm{max}} = \psi_1 \cdot M_{post} + \psi_2 \cdot M_{vrem} + \psi_3 \cdot M_{seysm}, \tag{3}$$

where $\mathrm{M}_{\mathrm{pred}}$ is limit bending moment on the strength condition or crack resistance of the normal section; M_{post}, M_{vrem}, M_{seysm} are the maximum bending moments in the middle section of the beam from the effect of permanent and temporary loads and seismic forces, respectively; ψ_1, ψ_2, ψ_3 are the coefficients of combinations for permanent, temporary and seismic loads. In accordance with Building Code (SNIP) 2.05.03-84 * [2] the following values are recommended to use.

Temporary loads are taken into account only for moving columns of class AK. Presumably, it is believed that in the earthquake-prone period of operation the movement of single heavy vehicles of class NC-80 will be limited. The present study evaluates the validity of excepted in existing regulatory standards calculation regulations.

Assuming that distribution of limit and maximum bending moments for the middle sections of the beams is normal, the risk of collapse of span is determined using the Laplace function

$$P_f = 0.5 - \frac{1}{\sqrt{2\pi}} \int_0^u \exp\left(-\frac{x^2}{2}\right) dx. \tag{4}$$

The following designations are worked in:

$$u = \frac{m_{Mpred} - m_{M_{\max}}}{\sqrt{\sigma^2_{Mpred} + \sigma^2_{M_{\max}}}}, \tag{5}$$

$m_{M_{pred}}, m_{M_{\max}}$ are the average values of the limit and the maximum bending moments; $\sigma_{M_{pred}}$, $\sigma_{M_{\max}}$ are the mean-square deviation of the same magnitudes.

Let us consider in detail the order of the calculations of risk of collapse of ferro-concrete spans of the most widely used structural forms with the use of modern finite-element computer complexes.

Let us compile a finite-element design model of span with the main beam, which are combined with transverse diaphragms (Fig. 1) [5] from presented in the current finite-element computational complexes core and shell finite elements (Fig. 2): slab of roadway is presented in design model by right-angled shell KE with 24 degrees of freedom, the longitudinal ribs (beams) and transverse ribs (diaphragms) are modeled by the core spatial KE with 12 degrees of freedom. For the pair of nodes of shell KE for slab and core KE for beams of span infinitely rigid cantilever spatial KE are used. This design model is used further in determination of the maximum bending moments for all loads, including forces, arising at earthquakes.

Mean values of maximum bending moments in all the main beams of span from permanent loads $m_{M_{post}}$ are estimated by spatial calculation on the normative loads from dead weight of beams, bridge carpet (sidewalks, fences, etc.) and all segments of the road surface. Standards of maximum bending moments in the main beams of span from permanent loads $\sigma_{M_{post}}$ are determined using the coefficients of reliability γ_{fi} for each components of bending moment on the well-known formula of A. R. Rzhanitsyn:

$$\sigma_{M_{post}} = \sqrt{\frac{\sum_i m^2_{M_i} \cdot \gamma_{fi} - 1^{\;2}}{1.64}}. \tag{6}$$

Components of mathematical expectations of maximum bending moments in the main beams of span from temporary loads $m_{M_{vrem}}$ in the form of columns of car loads A11 and single heavy vehicles NK-80 are calculated with the use of spatial calculation of the finite-element design model for temporary regulatory loads, increased taking into account the dynamic effect in accordance with the regulations of Building Code (SNIP) 2.05.03-84* [2] in (1+μ) times. Installation of temporary loads on the span is done in the most dangerous position in accordance with the recommendations of SNIP 2.05.03-84* [2] for revealing the most loaded beam of span.

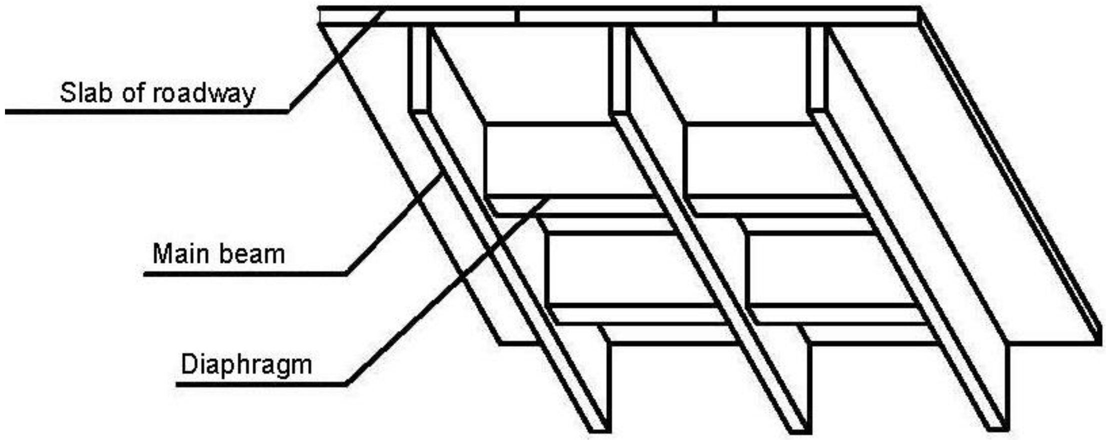

Fig. 1. The design of span

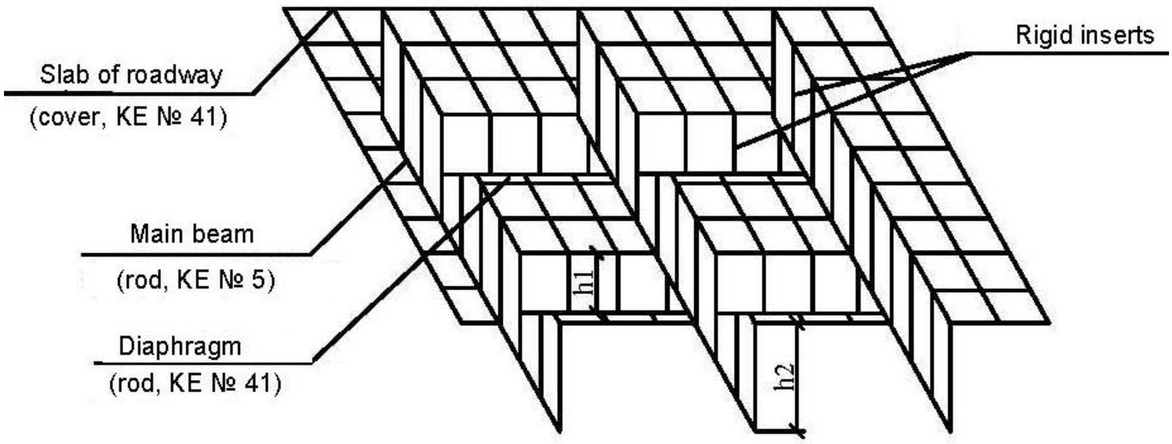

Fig. 2. Finite-element design model

Standards of maximum bending moments in the main beams of span from the effect of a single heavy vehicle are identified using standard reliability coefficient γ_f by the formula

$$\sigma_{M_{HK-80}} = \frac{m_{M_{HK-80}} \cdot \left(\gamma_f - 1\right)^2}{1.64}. \quad (7)$$

For automotive load A11, consisting of two components in the form of the trolley and the band load, the standard $\sigma_{M_{A11}}$ should be determined by summing the bending moments by formula similar to (6).

Statistical characteristics of maximum bending moments of seismic loads (mathematical expectation $m_{M_{seysm}}$ and standard $\sigma_{M_{seysm}}$) are to be determined in accordance with the theory of random functions. As accelerogram of accelerations acting on the ground of facilities, represent centered non-stationary random processes, mathematical expectation $m_{M_{seysm}}$ is zero. Therefore, the estimated value of the maximum bending moment from seismic loads is entirely determined by mean-square deviation $\sigma_{M_{seysm}}$.

Majority of used in the practice of bridge design modern finite-element computer complexes (SCAD, LIRA, ProfetSTARK, MicroFe and others) is equipped with two alternate blocks for seismic calculations. It uses the following algorithms:

- Quasi-static calculation by methodology of Building Code (SNIP) II-7-81 [3];
- Dynamic calculation of span for the kinematic perturbation of the support devices in the form of a given accelerogram.

A detailed description of the methodology and results of application of the SCAD program for seismic calculation of the two methods mentioned above for reinforced concrete diaphragm spans, performed on a standard design of Soyuzdorproekt, Iss. 56, is given in [5].

This study shows that at a deterministic seismic design dynamic design span dynamic design of a span for the kinematic perturbation of the support devices in the form of set accelerogram provides values more adequate to actual deflected mode.

In applications to program complexes known earthquake accelerograms are given, and users are offered to use scale factors for the calculations on earthquake of different intensity based on the assumption of non-dependence of the spectral density of the accelerogram from numericality of seismic impact. Such approach cannot be used when calculating the probability of risk, since it does not allow to take into account the fluctuations of spectrum of frequencies and amplitudes of seismic effects.

To take into account the random nature of the seismic impacts we propose to use numerical algorithms of generation of seismic impact accelerograms on specified correlation function. In presenting the accelerogram by stationary random process for description of the correlation function the following analytical expression can be used [5].

$$K_a\ (\tau) = D_a \cdot e^{-\alpha|\tau|} \cdot Cos\beta\tau\ . \tag{8}$$

It is the most convenient to calculate on a computer discrete realization of a random process $\xi(t)$ by a linear transformation of a stationary sequence $\nu(n)$ of independent normally distributed random numbers with parameters <ν> = 0, < ν^2> = 1 in the sequence $\xi(n)$, correlated to a specific law

$$<\xi\ \ n\ \ ,\xi\ \ n+k\ \ >\ \ = K\ \ k\cdot\Delta t\ \ \ , \tag{9}$$

where $K\ \ \tau\ \ = K\ \ k\cdot\Delta t$ is the correlation function of the modeled process.

With a constant step of the discretization operator of a linear transformation is written in the form of a moving summation with some weight $c_\kappa = c(\kappa)$:

$$\xi(h) = \sum_{\kappa=1}^{n} c_\kappa \nu \;\; h - \kappa \;\; . \tag{10}$$

In accordance with the described algorithm the program GESAZ-1 of generation by set-up parameters of correlation function of suitable for use in computational complex SCADfile of discrete ordinates of accelerogram was developed. With the help of this program shown in Fig. 3 stationary realization of accelerogram of vertical accelerations of ground surface at earthquake with intensity of 8 points, which was generated by the correlation function (8) with the parameters $D_a = 1881$ m^2/s^4, $\beta = 141$/s; $\alpha = 61$/s is obtained. Full duration of implementation is 40 s, step of discretization of ordinates — 0.01 s.

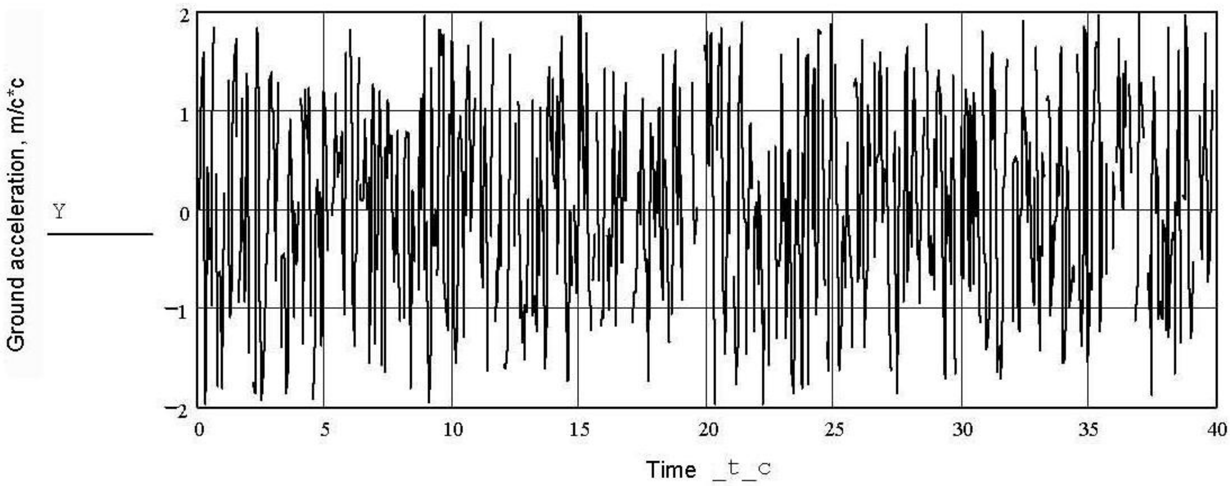

Fig. 3. Reconstructed stationary realization of accelerogram of ground accelerations

Application of simpler in terms of calculation stationary realizations of accelerograms for seismic design of bridges should be limited, as in fact fluctuations in the soil during the earthquake are not-stationary. First, the amplitudes of acceleration increase, reaching extreme values within 5—8 s, and then the amplitudes of acceleration decrease. To convert a stationary random process in the not-stationary we used proposed by V. V. Bolotin method of modulating of the generated stationary accelerogram $a(t)$, deterministic function of the following form:

$$A(t) = A_0 \cdot (e^{c_1 \cdot t} - b \cdot e^{c_2 t}), \tag{11}$$

where A_0, b, c_1, c_2 are the constant parameters chosen for reasons of approximation of the trend of development of fluctuations of soil during the earthquake to real conditions.

Described approach is included in the mentioned above program GESAZ-1 of generation by set-up parameters correlation functions of suitable for use in computational complex SCAD file of discrete ordinates of accelerogram (modules GESAZ-1M). Using this program shown in Fig. 4 non-stationary realization of accelerogram of vertical accelerations of ground at earthquake with intensity of 8 points was obtained. It was obtained on the base of simulation of shown in Fig. 3 accelerogram with the help of $a(t)$ determined by a function with parameters $A_0 = 1.5; b = 0.6; c_1 = 0.069/c;$ $c_2 = 0.622/c$.

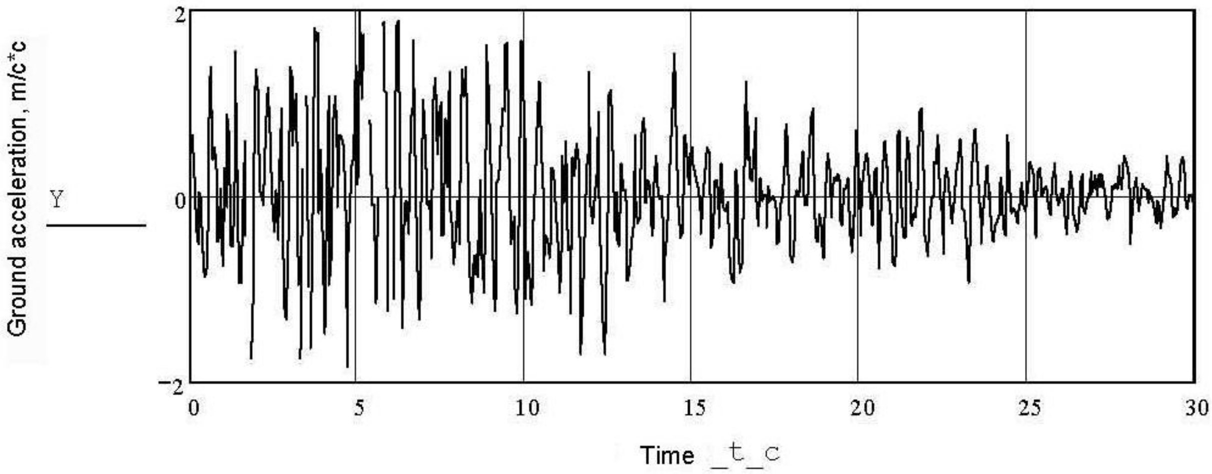

Fig. 4. Restored non-stationary realization of accelerogram of ground accelerations

Note that performed with the help of SCAD numerous seismic calculations for typical reinforced concrete beam bridges with span from 8 to 21 m showed that efforts, obtained with the use of stationary models of histograms, at 7…10 % lower than at application of stationary models of histograms. Therefore, in order to simplify calculations simplified stationary models can be used that give slightly inflated results.

Determination of statistical characteristics of the limit bending moment $\sigma_{M_{pred}}$ and $m_{M_{pred}}$ for reinforced concrete beams of span with the usual reinforcement is based on the following assumptions:

- cross-sectional size, diameters and location of the rods of working scaling multiplier in the compressed and extended zone are taken as deterministic in accordance with design data;
- strength characteristics of concrete and reinforcement have the variance by the normal low of distribution with prescribed average values and standards.

Using well-known from the theory of prismatic rods with double usual reinforcement calculation dependency for the limit bending moment by the first limiting state (the condition of strength) the calculation $\sigma_{M_{pred}}$ $m_{M_{pred}}$ is carried out using method of statistical tests in the following sequence:

1) by given average values and standards of durability of concrete and reinforcement two sets of random numbers, which represent normally distributed random variables of concrete and reinforcement strength are generated;
2) choosing one number from the two arrays, we get one of the for random variants of assignment of strength parameters of cross section, for which, on the base of consideration of the limit state of section calculate limit bending moment. However, in accordance with existing regulatory standards nonlinear diagrams of deformation of concrete and reinforcement are taken into account: two-line diagram of Prandtl for reinforcement and three-line for the concrete. As criterion of the evaluation of the limit bending moment restrictions of achievement of limit deformations by reinforcement or concrete and also condition of achievement of the extremum of load capacity are used;
3) repeating a sufficient number of times the calculations on paragraph 2, we accumulate statistical series of random values of a limit bending moments;
4) further, using an array of M_{pred} calculate statistical characteristics $\sigma_{\text{Мпред}}$ and m_{Mpred}. An important condition for further use of the data is a statistical calculation for testing the normality of the calculated allocations of limit bending moments M_{pred}.

The above calculations can be performed using well-known statistical complex STADIA and developed at the Department of Building Mechanics of VGASU program for the nonlinear phase calculation of normal cross-sections of reinforced concrete beams ETAP.

Using described in this article methodology probabilistic calculations of risk of the limit states in the main beams of reinforced concrete span of highway bridges during the earthquake were implemented.

These calculations were carried out both with the use of the standard ratios of combinations of loads, and without them. The calculation takes into account not considered in Building Code (SNIP) 2.05.03-84* the possibility of driving over the bridge in the seismic-prone period of a single wheel load NK-80.

Fig. 5 and 6 show graphs of dependency of risk of limit state in the main beams, respectively, when single standard workload NK-80 two columns A11 drive during the earthquake with intensity of 8 points.

Span of road bridge with gage travel G7 with six reinforced concrete diaphragm beams with full length of 16.76 m, relevant standard project of Soyuzdorproekt, vol. 56 was considered. In connection with the receipt of very low values of risk all factors of combinations were taken equal to 1. For the convenience of presentation of numerical results logarithmic rate of risk was used, which was calculated on a close to zero probability P_f values of the limit state from expression

$$\rho_f = \log (1/ P_f). \tag{12}$$

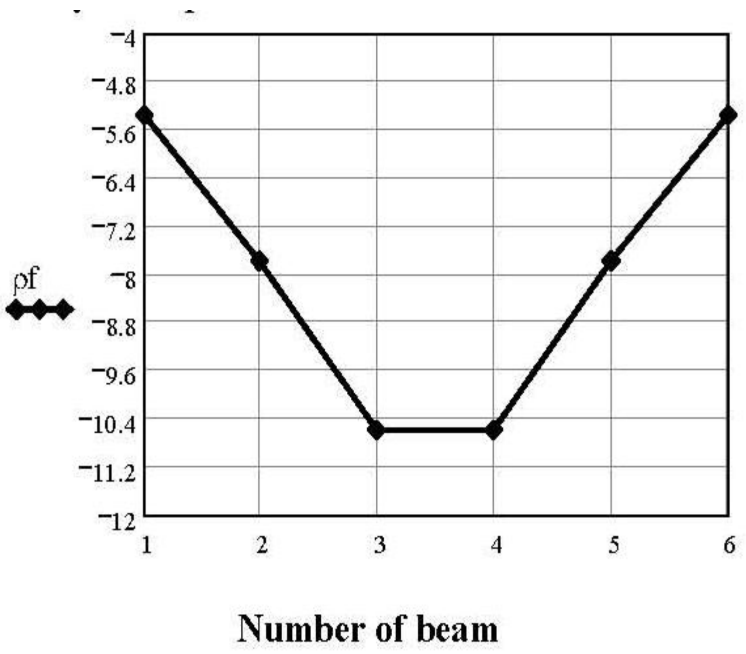

Fig. 5. Graphs of dependency of risk of limit state in the main beams when single standard workload NK-80 drives under seismic impact

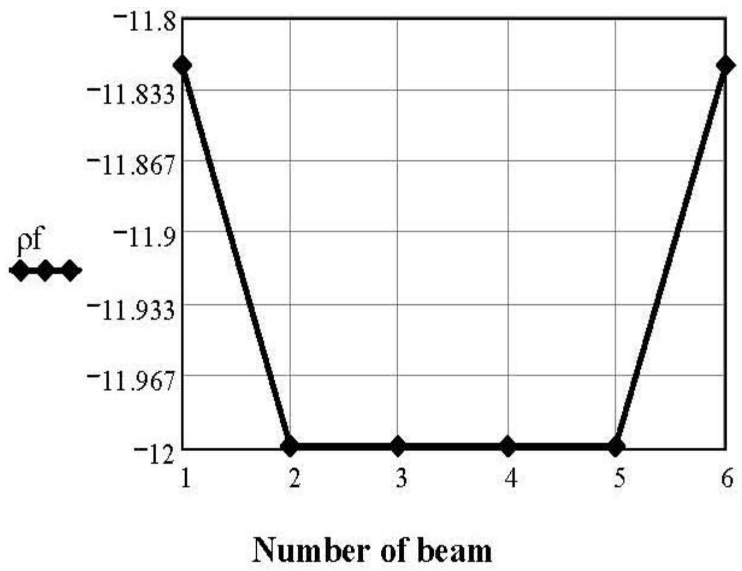

Fig. 6. Graphs of dependency of risk of limit state in the main beams when two columns of the regulatory load A11 drive under seismic impact

Note the feature of logarithmic rate at the probability $P_f \le 10—12$.

Further refinement is senseless since in engineering calculations $P_f = 0$ can be taken.

Therefore, when $P_f \le 10—12$ the logarithmic rate is taken as constant and equal $\rho_f = 12$.

By the results of performed statistical calculations of risk for bridges with ferroconcrete spans of the six diaphragm beams (standard model of Soyuzdorproekt, vol. 56) and five diaphragm-free beams (standard model of Soyuzdorproekt, vol. 56D) with spans from 8 to 16.3 m and a clearance G7 the following conclusions were made:

- The distribution of limit bending moments at the normal variability of strength characteristics of concrete and reinforcement is also close to the Gaussian law;
- The method proposed in this article allows to designate regulatory ratios of combinations of load for a group of loads, taking into account the seismic impacts, or to evaluate the opportunity of the operation of transport facilities in earthquake-prone periods of operation;
- The application of the described methods is also possible in the presence of defects and damages in the span, which reduce the carrying capacity of the main elements.

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