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RISK ASSESSMENT FOR BRIDGES

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SUMMARY

Two ways of risk assessment for bridges are considered.

The first way is based on statistical data of the previous earthquakes. This data was obtained in Turkmenistan after earthquakes in Ashkhabad, Gazli and Kum-Dag. In 1985 it were included in guideline for the estimation of the earthquake stability of bridges in operation.

Using these data the approximation function of damage D depending on earthquake intensity and structure bearing capacity was obtained. The earthquake intensity was characterized by I value on MSK scale or by the earthquake peak acceleration A_I. In this approach the structure bearing capacity can be evaluated by the value of earthquake intensity (K_S) on MSK scale or by the earthquake peak acceleration (A_K) and the dispersion of damage function can be evaluated by differentiation of $D(I,K_S)$ function. So, the risk function was considered as a random one.

The second way bases on the theoretical investigation of bridges behavior under earthquake loads. To carry out this investigation some typical bridges were chosen. For each type of bridge the correlation between the piers displacements and their damages was estimated. Each time-history bridge computation was considered as a realization of a random process. The use of 50 time-history computations allows to estimate:

- the average value;
- the dispersion of displacements and the corresponding pier damage.

Important part of theoretical risk analysis is to set earthquake input level. This problem is also observed in the paper. The sum of three damped sinusoids was assumed as a seismic input model.

INTRODUCTION

Problem of seismic resistance of bridges and other structures is of great importance as a part of the general problem of ensuring life support systems in seismic prone areas. The peculiarities of bridges seismic oscillations single out the problem of their seismic resistance as a separate section of the general theory. This problem has been elucidated in a great number of monographs and other publications, a lot of

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investigations have been carried out. However, up to now, the problem of calculating, designing and strengthening the bridges still involves a lot of difficulties.

The level of a bridge antiseismic strengthening is determined by the value of design acceleration and the adopted ultimate state. The risk analysis is one of efficient methods to solve this problem. Seismic risk is the average of distribution of negative profit caused by earthquakes during the life time of the structure [1]. The risk value R can be calculated by the following formula

$$R = f(\kappa, T) \cdot \sum_{I=I_{min}}^{I_{max}} D(K_s, I) \cdot L(I), \tag{1}$$

where

$f(\kappa, T)$ - long-term costs factor;

$D(K_s,I)$ – damage to the structure, calculated for the design input intensity K_s from the earthquake with intensity I on MSK scale;

$L(I)$ – the frequency of earthquakes with intensity I;

T – life time of the structure;

$\kappa = \dfrac{d + d^*}{1 + d}$;

d – average annual income from capital investments;

d^* - the value of annual depreciation of the structure.

Risk estimation can be used to set the level of strengthening the structure. The level of structure strengthening is determined by K_s value for the single-level designing. The minimum of $R(K_s)$ should be provided and $R(K_s)$ value is to be less than the admissible value in this case. For the multi-level designing the level of structure strengthening is determined by $D(K_s,I)$ function.

Two ways of risk assessment for bridges are considered.

EMPIRICAL CONCEPT OF RISK ASSESSMENT

The first way is based on the statistical data of the previous earthquakes. This data was obtained in Turkmenistan after the earthquakes in Ashkhabad (1948), Gazli () and Kum-Dag(). It was included in the guidelines for estimation of the earthquake stability of bridges in operation (1985). Using this data the approximation function of damage D depending on earthquake intensity I and structure bearing capacity K_s was obtained. The earthquake intensity was characterized by its value I on MSK scale or by the earthquake peak acceleration A_I. The structure bearing capacity was determined as intensity of the weakest earthquake which can cause the structure ultimate state. In this approach the structure bearing capacity can be characterized by the value of the earthquake intensity (K_S) on MSK scale ore by earthquake pick acceleration (A_K). To approximate dependence $D(I,K_S)$, the quadratic polynomial was used.

$$D(K_s, I) = a_{0,1}K_s + a_{1,0}I + a_{1,1}K_sI + a_{0,2}K_s^2 + a_{2,0}I^2 \tag{2}$$

$D(I,K_S)$ dependence is presented on Fig.1.

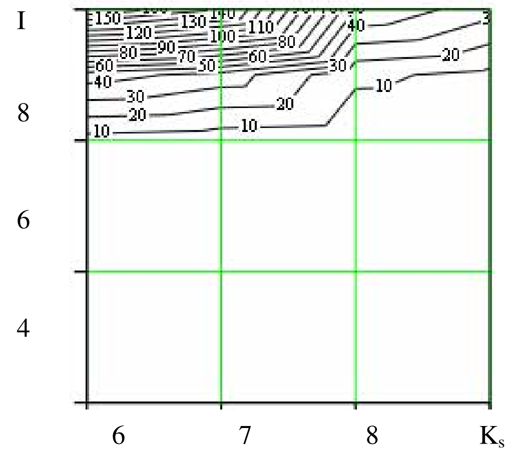

Fig 1. D(I,K_S) dependence

In this case the dispersion of damage function can be evaluated by differentiating D(I,K_S) function.

$$\sigma \approx \frac{\partial \langle D \rangle}{\partial I} = a_{10} + a_{11}K_s + 2a_{20}I \tag{3}$$

So, the risk function was regarded as a random one.

THEORETICAL CONCEPT OF RISK ASSESSMENT

The second way is based on theoretical investigations of bridges behavior under earthquake loads. To carry out these investigations some typical bridges were chosen. For each type of bridge the correlation between the piers displacements and their damages was estimated. Each time-history bridge computation was regarded as a realization of a random process. The use of 50 time-history computations allowed us to estimate the average value and the dispersion of displacements and the corresponding pier damage.

To describe the nonlinear behavior of the bridge pier, we used a model with stiffness which decreases linearly as the maximum pier displacement during the loading history increases [2]. At the same time the structure damping γ increases linearly.

Reaction at piers can be described by the following formula

$$R(y) = \frac{r(u)y}{1+\kappa(u)y^2}, \tag{4}$$

where
y - the pier displacement;
u - the maximum displacement during the loading history;
κ - the parameter of nonlinearity.

The parameter of nonlinearity decreases linearly as the maximum pier displacement during the loading history increases. The decrease of r and κ values, as well as the increase of damping begins when pier displacement exceeds its limit of elasticity.

Examples of dependencies r(u), γ(u), κ(u) and R(y) are shown on Fig. 2-4.

There are 3 points of pier condition:

1. the conventional displacement (U_{conv}) for which $R(U_{conv})=0$.
2. the displacement of pier damage U_{damage} for which the reaction R is maximum.
3. if $U<U_{elastic}$ the pier behavior is linear.

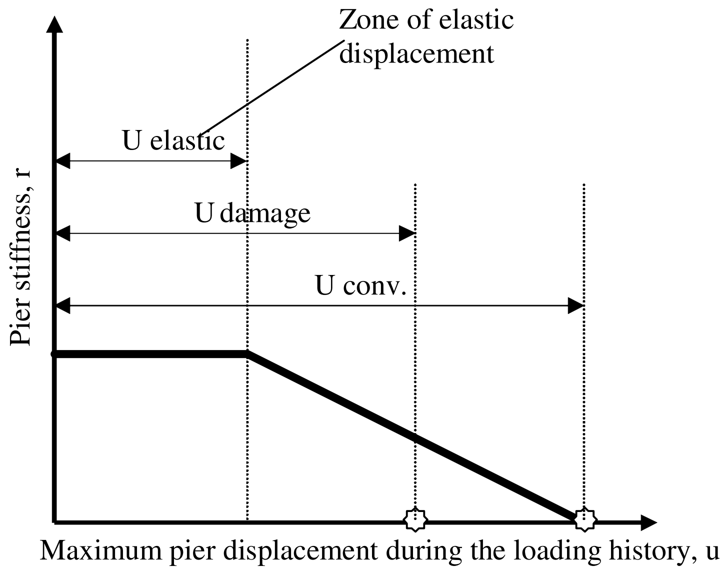

Fig. 2. Dependence of pier stiffness on maximum of pier displacement during the loading history.

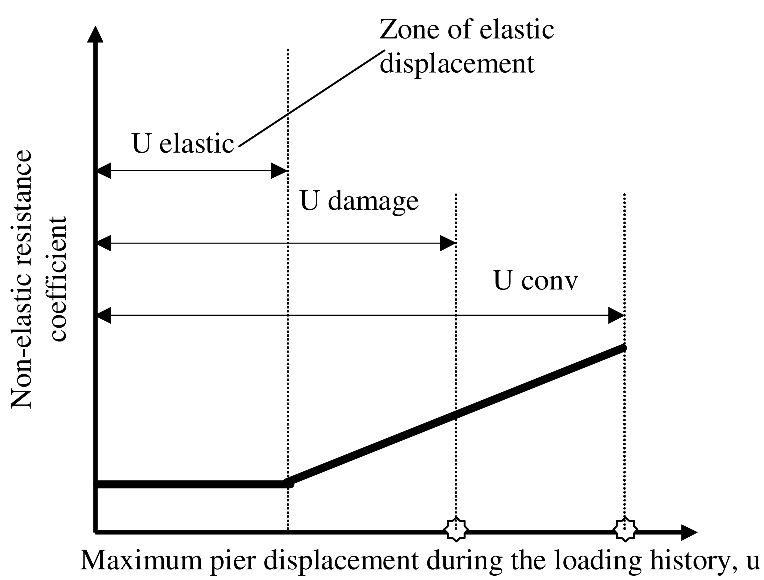

Fig. 3. Dependence of the pier non-elastic resistance coefficient on maximum pier displacement during the loading history

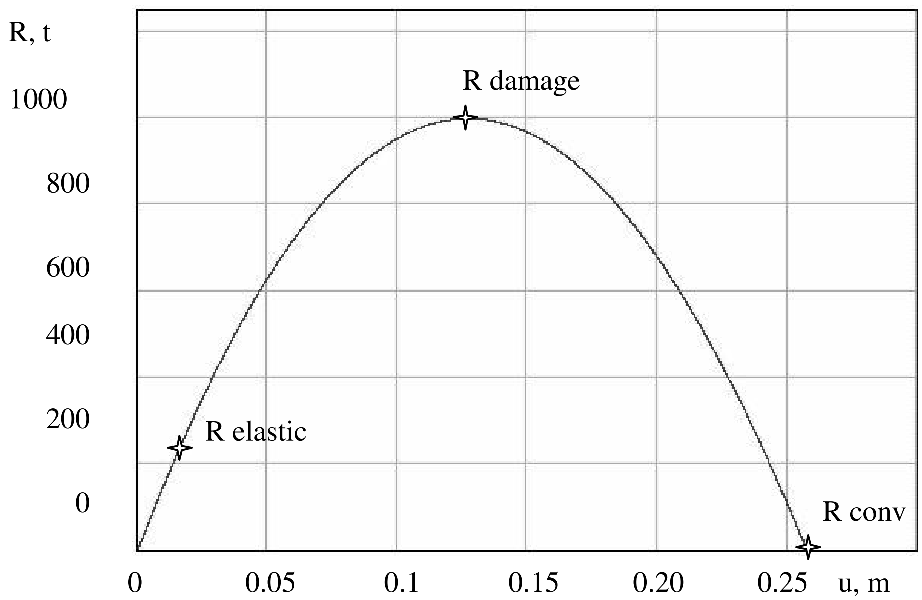

Fig. 4. Pattern diagram of the pier deformation

To define the dependence of damage D on ϰ(u) parameter, the representative objects of bridge piers were calculated. It makes possible to use the dependence similar to (2) on the basis of ϰ(u) estimating.

THE SETTING OF EARTHQUAKE INPUT.

An important part of theoretical risk analysis is to set an earthquake input. The sum of three damped sinusoids was taken as a seismic input model [3]. The input pick acceleration was regarded as a variate with Veybul law distribution. Input frequencies were set in accordance with the seismological data and structure eigen-frequencies. The amplitude of each sinusoid and their damping were computed to bring the Areas input intensity of the model close to the real one.

Using the risk function in accordance with (1, 2) dependences, the functional dependences of risk value on the structure seismic class $R(K_s)$ were obtained. The form of these dependencies presentation has some peculiarities in Russia. These peculiarities are determined by the Russian maps of the general seismic risk zoning.

In 1998 Russian Institute of The Earth put out three maps of general seismic risk zoning for Russia. "A" map has the exceeding probability of one time per 200 years; "B" map has exceeding probability of one time per 500 years; "C" map - one time per 1000 years. According to these maps each region is characterized by three values of seismic design intensity.

For example, formerly the Kamchatka region had been characterized by design seismic intensity I=9 on MSK scale. Now the seismic hazard of this region is characterized by three values of intensity I_A=9; I_B=9; I_C=10. In Russia such presentation of earthquake hazard is called as situation seismicity. The idea of the situation seismicity makes possible to use (1,2) dependences for designing. So, the risk for regions with equal design seismicity but with different situation seismicity is different. To illustrate this condition, $R(K_s)$ dependencies for Kamchatka and Irkutsk regions are presented in fig. 5. For Irkutsk region the situation seismicity is I_A=8; I_B=9; I_C=9. To provide risk R=0.07 for Kamchatka region, the structure has to be strengthened up to K_s=9 and for Irkutsk – up to K_s=6.5.

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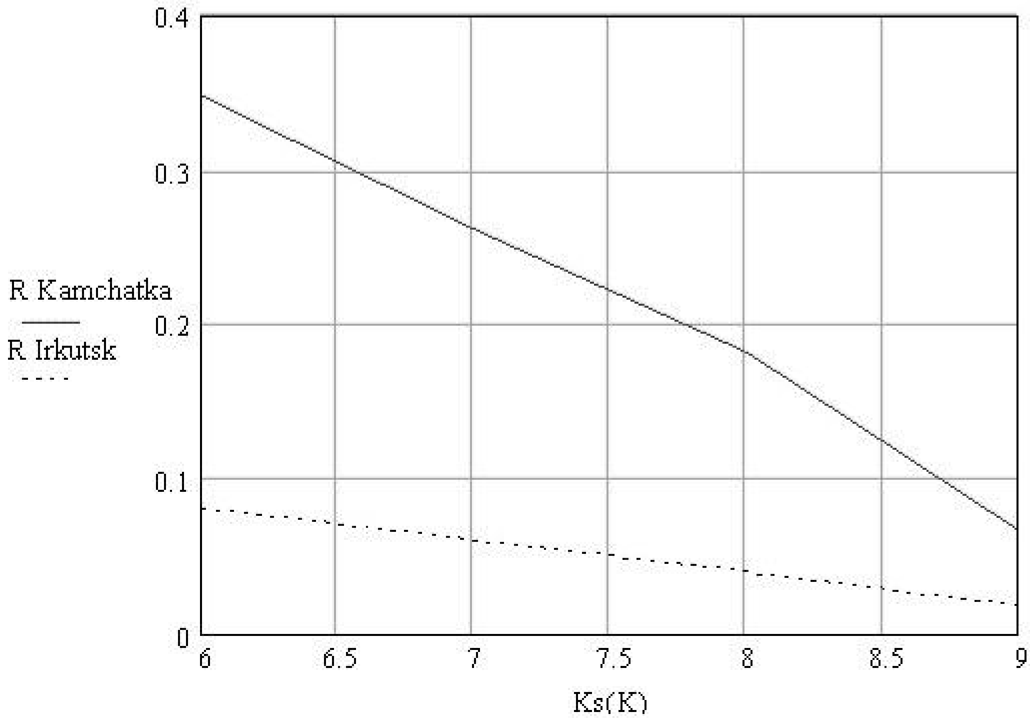

Fig. 5. R(K_s) dependencies for Kamchatka and Irkutsk.

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