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Calculation of the stress-strain state of monolithic bridges on the action of real seismic impacts

Ulugbek Shermukhamedov*, *Ibrakhim* Mirzaev, *Anora* Karimova, and *Dilbarkhon* Askarova

Tashkent State Transport University, Tashkent, Uzbekistan

Abstract. To improve the transport infrastructure of the Republic of Uzbekistan, there is a tendency to build monolithic structures of bridges and overpasses. The article presents the calculation of a monolithic bridge on the 1083rd km of the M-39 highway passing through the city of Samarkand. The results of the monolithic bridge calculation on dynamic loads are presented, based on the records of two real seismograms of two dangerous earthquakes registered for a system with a spectral composition: the Gazli (Uzbekistan) and Manjil (Iran) earthquakes. The results of calculations of longitudinal and vertical displacements, and normal stress in the upper and lower parts of the span along the length of the bridge were analyzed. The calculations performed, show that the span structure and bridge supports have an overestimated margin of safety for an 8-point earthquake according to MSK-64. To ensure the guaranteed seismic safety of bridge structures, it is required to conduct design calculations based on a set of records of earthquake events that are close in dominant frequencies to the characteristics of the construction site. This provision must be included in the relevant regulatory documents.

1 Introduction

To date, at an active pace, new pages are being opened in the Uzbek bridge building. The design and construction of monolithic bridges and overpasses are considered important aspects. Time dictates new requirements in the approach to the construction of transport facilities, so the construction technology may also change. In this regard, designers and builders are thinking about the creation of non-traditional engineering solutions. It should be noted that due to high architectural qualities, cost-effectiveness and strength, monolithic bridges with continuous spans are recommended to be used in cities.

As is well-known, a significant part of the construction in Uzbekistan falls on seismically hazardous areas. Protection of structures from seismic impacts is an important task of engineering. A study of world experience in the field of seismic protection of bridges shows that the use of the principle of seismic isolation is widely used. With this method, flexible or sliding bearing parts are installed between the supports and spans,

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*Corresponding author: ulugbekjuve@mail.ru which work as a seismic isolation element.

At present more than twenty monographs were published on the issues of seismic isolation of buildings and bridge structures. The classification of seismic isolation devices is given by scientists and specialists V.A. Verkholin [1], A.M. Uzdin and T.A. Sandovich [2], G.S. Shestoperov [3], R. Skinner, W. Robinson and G. Mtsverri [4] and other authors). In [5], the issues of seismic isolation and seismic damping of bridges, considering the features of seismic vibrations of bridges, assessing seismic loads, as well as setting the design impact and coefficients of seismic and moving loads combinations are considered.

The non-linear response of continuous girder bridges with insulating lead rubber bearings (LRB) under bidirectional horizontal earthquake excitation is presented in [6]. That article presents the results of laboratory tests on a vibration stand of a model of isolated bridges with continuous spans with the LRB under appropriate loads, and the comparison of analytical results with experimental ones. There is a significant underestimation of bearing displacements if the LRB restoring force is idealized independently in the longitudinal and transverse directions. Therefore, for the seismic isolation of bridges, it is necessary to take into account the bidirectional interaction of the LRB.

In [7-9], the influence of various types of insulating bearings on the bridge behavior was studied, and the parameters of the lead-rubber bearings were developed. The issues of the influence of the duration of ground motion on the seismic response of bridges with an isolated base are considered. The results of the analysis show that the LRB modeling approach has a significant impact on the seismic response of a bridge with an isolated base and requires careful selection of modeling parameters.

Conventional lead-rubber bearings can have problems with instability and unrecovered deformation under severe ground motion. As a way to solve the problem, a new concept of insulating devices was proposed in [10-13], in which alloy metal sheets are embedded into elastomeric support. Seismic isolation based on Shape Memory Alloy (SMA) can successfully reduce the peak and residual displacements of bridges during a major earthquake.

Today, in the world practice of earthquake-resistant construction, the most important issue is the dynamic calculations of structures based on seismograms of earthquakes. It should be noted that such calculations are especially important when designing seismic damping and seismic isolation systems, when calculating large multi-span bridges, unique buildings and other critical objects, when assessing the damage of structures, etc. At present, there are two opposing approaches in modeling the calculated accelerograms: modeling the impact for the construction site and modeling the impact for the structure. In [14], the kinematic, spectral, and energy properties of the impact were identified. The values of the energy characteristics I_A, CAV, SED, as well as the values of the harmonicity ratio κ and the accelerations of the base PGA should be set in such a way that they correspond to the values of the same characteristics at the construction site with a given probability. Assessment of impact hazard for a structure is performed using spectra of kinematic quantities and spectra of work of plastic strain forces.

Under seismic vibrations, the length of structures is one of the important factors affecting their seismic resistance. Research in this sphere given in [15], led to the development of a methodology for calculating multi-support structures, which in turn led to the need to change the existing methodology for calculating seismic resistance, considering the asynchrony of excitations of the structure supports. The calculation results obtained, show that an account for the asynchronous excitation of the bearing points of an extended system significantly reduces the inertial seismic loads on its elements.

In [16–18], studies were conducted in detail on the method of calculating damped systems; the effectiveness of using the mass damper setting to reduce damage after strong earthquakes was obtained.

Recently, seismograms of earthquake records have been taken as a seismic impact [19, 20]. In this regard, it is of interest to develop methods and software for calculating bridges and overpasses for earthquake impacts based on the available seismic records.

2 Methods

Bridge structures are usually modeled in the form of beam-cut, beam-continuous and beam-cantilever schemes when performing calculations for static and seismic effects. Bridge structures are made up of many elements, the most important of which are piers and bearings. The support and the bearing are the most vulnerable elements of the bridge structure, therefore, seismic isolating devices, in particular, rubber-metal ones, are used for the bearings [22]. The bearing is a seismic isolation device and allows the span to move in the longitudinal direction within the range from 0.1 m to 0.35 m, depending on the models used, due to low shear rigidity. A convenient method for carrying out calculations is the finite element method. The finite element simulates axial tension-compression, bending about axes perpendicular to the longitudinal axis of the bridge, and torsion about the longitudinal axis. In this regard, the calculations are conducted by the finite element method for bridge structures, the Newmark method is used for the time variable. The impact is set in the form of a series of records of three-component seismograms with amplitude correction for different intensities. The equation of motion of the structure after applying the discretization by the finite element method is reduced to the following form:

$$[M]\{\ddot{u}\}+\eta[C]\{\dot{u}\}+[K]\{u\}=\{P\} \tag{1}$$

with initial conditions from the static solution of the problem

$$\{u(t)\}_{t=0}=[u(0)],\ \{\dot{u}(t)\}_{t=0}=\ \{\dot{u}(0)\} \tag{2}$$

where $\{u(t)\}$ is the vector of absolute displacements of the nodal points of the finite element model of the structure, for nonlinear problems the matrices $[M]$, $[C]$, $[K]$ depend on the absolute displacement vector, $\{P(t)\}$ includes the specified ground motion and acting forces. Ground motion is specified in the form of seismogram records [21].

3 Results and discussions

Consider a three-span reinforced concrete monolithic bridge, 110 m long and 10.5 m wide, with variable thickness along the bridge. The span structure of the bridge is made by a continuous monolithic reinforced concrete design scheme 33m + 42m + 33m of individual design. On the facade, the superstructure is made of a slab of variable height - 1.3 m in the span and 2.3 m above the support. The vertical stiffness of the bearing is $5.908\text{x}10^9$ N/m, the horizontal stiffness is $4.72\text{x}10^6$ N/m. The bearing is modeled as a finite element operating in tension-compression, shear in two directions, and torsion. Intermediate supports have dimensions: height 5.85 m, width along the facade - 2 m, in the lateral direction it has a variable size in height from 5 m to 8.4 m. In this calculation, we assume that their movement is equal to the movement of the base during an earthquake. The material of all structures is concrete of class B35 in strength, with specific gravity γ = 25000 N/m^3, elastic modulus E=35200 MPa, Poisson's ratio ν = 0.2.

The seismicity of the territory of Samarkand, according to the map of seismic microzoning plotted by the Institute of Seismology in 1980, is estimated at 9 and 8 points. The site of the projected construction is located in the 8-point zone.

In accordance with Table 1.1 of KMK 2.01.03-96 within the site in the upper 10-meter thickness, counting from the base footing, soils of category II in terms of seismic properties occur – (loams with a porosity coefficient of e<0.8, pebble soil). With this in mind, the seismicity of the projected construction site is recommended as 8 points. The scheme of the monolithic bridge is shown in Fig. 1.

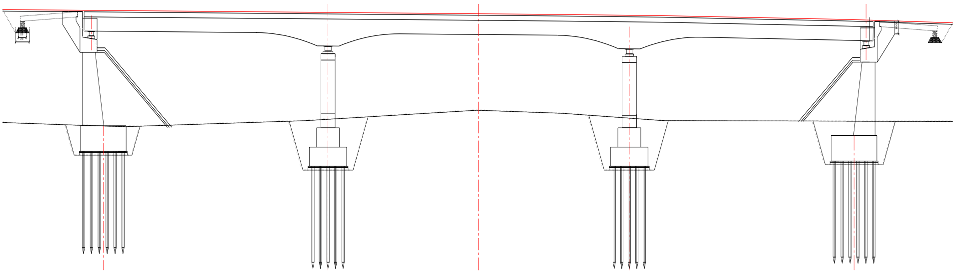

Fig. 1. Scheme of a monolithic bridge passing the 1083rd km of the M-39 highway

The seismic impact is transmitted to the structure at four points through the supports in the form of equal displacements of the supports and the base surface. The beginning of the bridge - the left end of the span is rigidly connected to the abutment, and the end of the bridge - the right end is connected to the abutment with movable bearings.

The results of calculations of a monolithic bridge for a dynamic load are presented, based on the records of two real seismograms of strong earthquakes: in Gazli (Uzbekistan) and in Manjil (Iran).

Gazli (Uzbekistan) earthquake dated May 17, 1976, was more than 9 points on the MSK-64 scale, maximum acceleration, velocity and displacement in the direction of seismic wave propagation were: 7.22 m/s^2; 0.62 m/s; 0.18 m. Vertical acceleration was 14 m/s^2.

Manjil (Iran) earthquake of June 20, 1990, was 8 points on the MSK-64 scale, maximum acceleration, velocity and displacement in the direction of seismic wave propagation were 1.93 m/s^2; 0.21 m/s; 0.064 m.

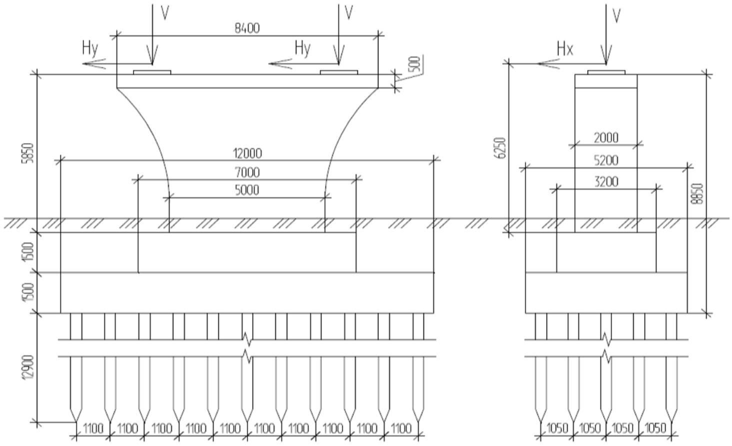

Fig. 2. General view of the intermediate support

For discretization, the bridge was partitioned into 240 finite elements, taking into account the operation of each type of finite element. The calculations were conducted using an implicit scheme with a time step of 0.005 s. The energy loss was taken in the Rayleigh form.

Earthquake records were taken from the European Strong Motion Database [21]. Figs. 3 and 4 show the longitudinal movement of the right end of the bridge (a black line), which is free from longitudinal stress, and the movement of the corresponding abutment from the active pressure of soil (a red line) during the Gazli earthquake.

As seen from the graph, the maximum displacement in a seismic wave propagating along the longitudinal axis of the bridge is 0.18 m. The seismic wave propagation velocity is 500 m/s. In this case, the maximum difference between displacements is 0.07 m; at a seismic wave propagation velocity of 250 m/s, this displacement difference will be 0.14 m. This is due to the difference in wave propagation velocities in the monolithic bridge structure and in soil, which leads to a delay in the wave arrival in soil. It follows that the bearing of the right end of the bridge does not allow the span to fall from the abutment due to the fact that the maximum displacement of the rubber-metal bearing is 0.2 m.

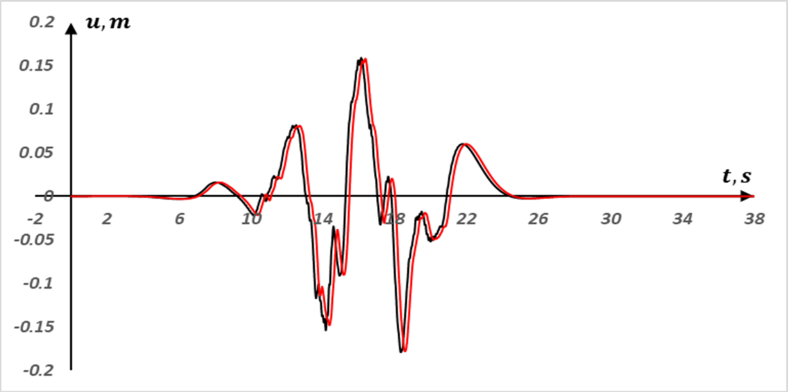

Fig. 3. Change in time of the longitudinal movement of the right end of the bridge span and base at a wave propagation velocity in soil 500 m/s (Gazli earthquake)

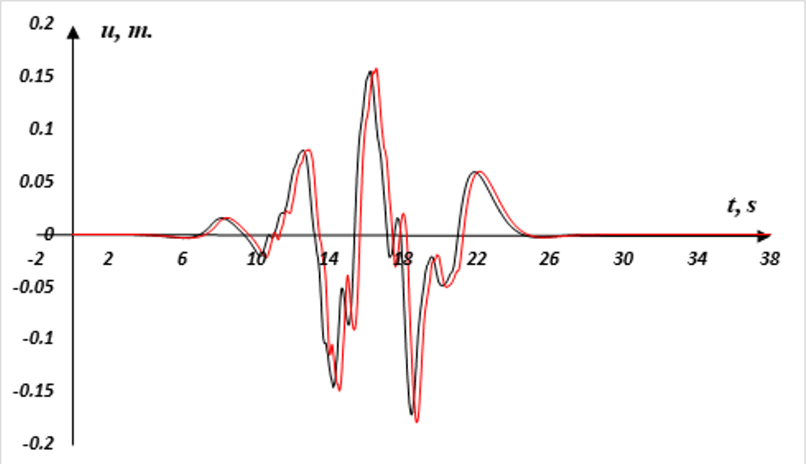

Fig. 4. Change in time of the longitudinal movement of the right end of the bridge span and base at a wave propagation velocity in soil 250 m/s (Gazli earthquake)

Fig. 5 shows graphs of changes in the normal stress along the length of the bridge in the upper (a, b) and lower (c, d) parts of the span at times t=0 s (a, c) and t=16.4 s (b, d) (Manjil earthquake). It is seen from the graph that tensile stresses in the upper part of the span occur above the supports, while their values do not exceed 0.30 MPa, which is less than the allowable value according to the design standards of 2.73 MPa. For the lower part, tensile stresses in the middle part of each span do not exceed 0.12 MPa.

Fig. 6 and 7 show the graphs of the change in time of the normal stress in the upper (a) and lower (b) parts in the middle of the second span and the left span of the bridge (Manjil earthquake). Due to the impact of a seismic wave, the stress increases more than 2 times.

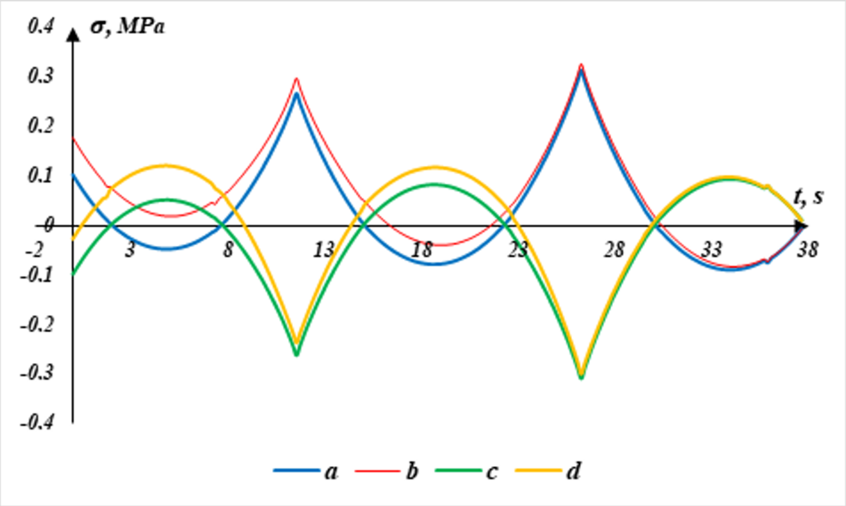

Fig. 5. Change along the length of the bridge of the normal stress in the upper (a, b) and lower (c, d) parts of the span at times t=0 s (a, c) and t=16.4 s (b, d) (Manjil earthquake)

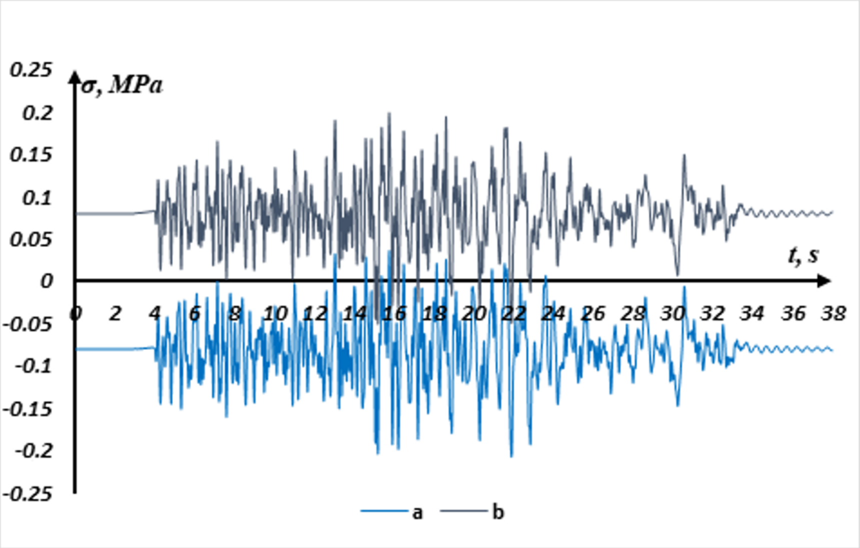

Fig. 6. Change in time of the normal stress in the upper (a) and lower (b) parts in the middle of the second span and bridge (Manjil earthquake)

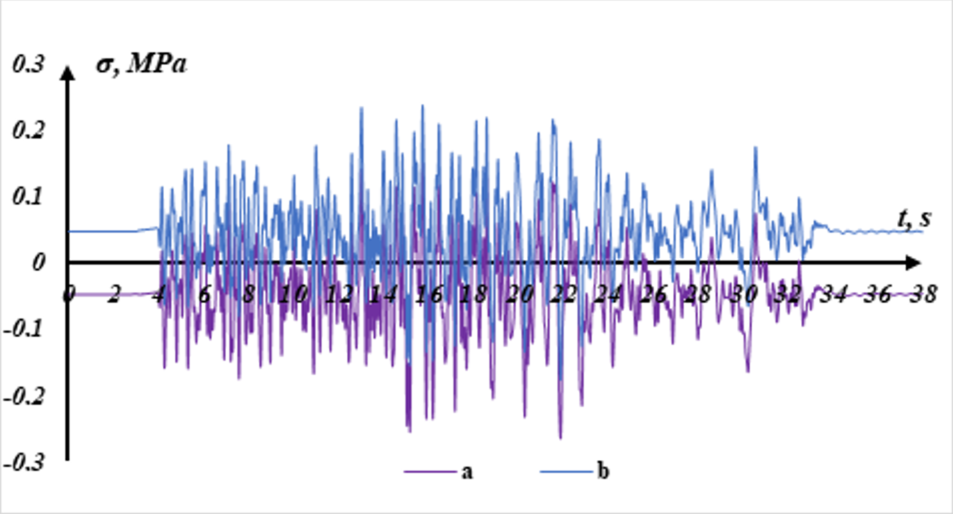

Fig. 7. Change in time of the normal stress in the upper (a) and lower (b) parts in the middle of the left span of the bridge (Manjil earthquake)

Comparison of the displacements of the lower and upper points of support along the bridge showed that the intermediate supports have an overestimated rigidity and, because of this, the displacements of these points coincide.

4 Conclusions

The current regulatory documents of the Republic of Uzbekistan on the earthquake-resistant construction of transport facilities do not reflect the issues of seismic resistance of structures, considering multi-level design. Unfortunately, in transport construction, domestic design institutes do not use records of real earthquakes in the calculations of bridges for seismic resistance and are limited to linear-spectral calculations due to the lack of information about the situational seismicity of the territory of Uzbekistan. In this regard, to ensure the guaranteed seismic safety of bridge structures, it is required to perform design calculations based on sets of records of past earthquakes that are close in dominant frequencies to the characteristics of the construction site. The results of calculations of longitudinal and vertical displacements, as well as normal stress in the upper and lower parts of the span along the length of the bridge, are analyzed. Numerical calculations have shown that the bearing of the right end of the bridge does not allow the span to fall from the abutment due to the fact that the maximum displacement of the rubber-metal bearing is 0.2 m. The performed calculations show that the span structure and bridge supports have an overestimated margin of safety for an 8-point earthquake according to MSK-64.

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