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- Challenges and Solutions -

Spatial oscillations of a railway bridge under the impact of a real earthquake

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ABSTRACT: This paper deals with the specifics of calculating the earthquake resistance of reinforced concrete railway bridges using real earthquake records. The finite element and finite difference methods are used to discretize the problem taking into account the interaction of the bridge foundation with the soil using the Winkler-type model. Nonlinear deformation of the bridge abutments is also taken into account. As a calculation example, a 29.6 m long railway bridge in a span located on Navoi-Bukhara railway section is selected. The analysis of stress and strain state of bridge's elements on the basis of Gasli earthquake records has been carried out. The case of displacement with dry friction of span structure on ledgers in lateral direction is considered. The shifts and stresses of the considered railway bridge in calculations for an earthquake of more than 9 points have shown that their values correspond to the accepted norms.

KEYWORDS: railway bridge, seismic protection, span, support, real earthquake records.

1 INTRODUCTION

The construction of railway bridges mainly uses reinforced concrete elements. Reinforced concrete bridges account for 70% of the bridges on railways in Uzbekistan. The advantage of such bridges is their relatively low construction cost, low maintenance and durability.

Railways are of particular importance for livelihoods in areas of high seismic impact, especially in urbanised areas, due to its natural conditions and spatial location combined with its robust calculation of economic utility and safety (Kuznetsova, Uzdin, Dolgaya, Frese and Shulman 2012). Most urbanised areas around the world are located in areas of high seismicity. In this regard, great attention is paid to the quality of seismic resistance in the construction of transport facilities.

In recent years, in our country, large-scale comprehensive measures have been implemented to develop the areas of seismology, seismic stability of structures and seismic safety, as well as to radically improve the efficiency of industry organisations. Today, it is important to consistently continue reforms in these areas and to introduce new methods of ensuring seismic safety of the population.

It is known that the territory of Central Asia, especially Uzbekistan, is a seismically active zone. Therefore, high requirements are imposed on the design and construction of bridges, overpasses and flyovers, as their temporary failure or breach causes social and significant economic damage. Preventing this from happening requires the use of modern software packages to ensure earthquake resistance at the design stage. The software package SHARK (Stepwise Algorithms for Structural Calculation) makes it possible to calculate various complex spatial structures, in particular bridges and overpasses, based on existing seismogram records, for the effects of earthquakes (Rashidov, Kuznetsov, Mardonov and Mirzaev 2019, Rashidov, Kuznetsov, Mardonov and Mirzaev 2021). Earthquake resistance in transport structures mainly uses seismic isolation devices, i.e. in order not to transfer part of the seismic energy to the structure and to dampen its vibrations (Shermukhamedov 2020, Mirzaev, Shermukhamedov and Karimova 2021, Buckle, Constantinou, Dicleli and Ghasemi 2016).

Seismic isolation is currently one of the main means of ensuring seismic resistance of bridges, especially when the seismic intensity is of magnitude 8 or more. To increase the seismic resistance of bridges, i.e. between bridge spans and abutments, bias deformation or sliding seismic isolation devices are installed. Rubber, rubber-metal, flat or spherical seismic isolators are usually used for this purpose, which provide significant mutually safe displacement between the span and supports. As a result, large stresses in the spans and supports will be prevented. Although this method is an alternative solution and is used in almost all countries, it has mainly been used for road bridges (Sukonnikova 2016). In railway bridges (Kuznetsova, Uzdin, Dolgaya, Frese and Shulman 2012) a combined bearing part has been proposed which provides elastic tangential displacement with transition to dry friction state and damping of vertical vibrations. A method of scientific and technical justification for the effectiveness of seismic isolation of friction-damping support parts has also been proposed (Uzdin, Mazhiyev, Andreev and Andreeva 2015).

The current normative documents KMK 2.01.03-19 “Construction in seismic areas” for seismic design in Uzbekistan and ShNK 2.01.20-16 “Construction of transport structures in seismic areas” contain requirements for design of transport facilities. Also ShNK 2.05.03-12 “Bridges and pipes” normative document gives instructions for bridges in order to provide smooth movement of vehicles on them by limiting elastic bends of their spans and for corresponding forms of longitudinal displacement. Calculation of bridges and viaducts for earthquake effects with their existing records allows to analyse their stress-strain state (Shermukhamedov, Mirzaev, Karimova and Askarova 2022).

For a bridge located in an area with a seismic intensity of 9, it was shown that the seismic resistance of the bridge can be ensured by reducing the maximum seismic load on the piers with the damper in comparison to the case where no damper is installed (Shermukhamedov, Shaumarov and Uzdin 2022).

A three-dimensional digital ground – foundation – construction model of a two-span reinforced concrete bridge of a highway located in California, USA, has been investigated. The application of geotechnical seismic isolation has shown positive results in reducing seismic impacts. The presence of geotechnical seismic isolators under the supports has reduced the impact of earthquakes. This has also reduced the cost of using the device during operation (Forcellini and Alzabeebee 2022).

Horizontal barriers in the form of metasurfaces made of granular metamaterials with the properties of broadband phononic crystals (Goldstein and others 2015, Norris and Johnson 1997), which impede the propagation of major types of seismic evanescent, Rayleigh – Lamb head and surface waves, Stonley interface waves, and bulk S-waves, are proposed to reduce the impact of earthquakes on objects. Granulated metamaterials are used as seismic isolation devices for both aboveground (Morozov and others 2021, Sen 2008, Dudchenko 2021) and underground structures (Qiu 2014). It should also be noted that seismic cushions of granular metamaterials are also used to protect the supporting structures of high-span bridges (Teyssandier 2000). An area seismic protection system based on granulated metamaterial has been developed for the construction of the Rion – Antririon bridge over the Corinthian Gulf (Greece).

The metamaterial is placed between the foundation and the abutment, which causes the abutment to slide on the foundation during an earthquake, thereby reducing the stress level in the abutment body (Borja and Lee 1990, Rashidov, Kuznetsov, Mardonov and Mirzaev 2021).

The article (Bedon and Morassi 2014) presents the results of the dynamic performance of an isolated Dogna bridge obtained by harmonic vibration testing and finite element analysis.

Soil condition is important for structural damage during earthquakes (Boozarjmehr and Emami 2008). An attempt was made to assess the influence of non-linear soil-structure interaction (SSI) on the behaviour of seismically isolated overhanging bridges, which are the most advanced type. It was found that a linear ground model does not accurately predict base shear and non-linear ground modelling is a prerequisite for accurately reflecting the behaviour of the ground-pile system.

The Bolu Viaduct, located in central Turkey, consists of two parallel bridges. During the Duzce earthquake in November 1999, this bridge was structurally complete but was not open to traffic. At the time of the earthquake none of the spans collapsed, but the seismic protection devices were severely damaged. Several spans were offset from their axis (Figure 1).

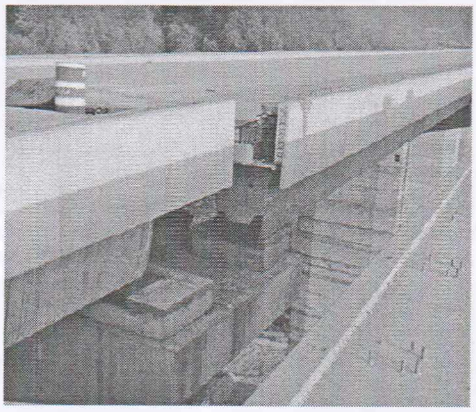

Figure 1. Damage to the Bolu viaduct during the 1999 Duzce earthquake.

Studies of earthquake effects have shown that stresses generated in the abutments have led to the failure of seismic protection devices, resulting in excessive displacements of some spans in transverse direction relative to the supports and adjacent spans (Buckle, Constantinou, Dicleli and Ghasemi 2016). This type of failure points to the need to consider issues related to the earthquake resistance of bridges, taking into account the friction between the superstructure and the supports or rigels.

When determining the stress-strain state of a bridge during an earthquake, the direction and magnitude of the friction force as well as the start-stop time of the slip will not be known. This leads to a mathematically non-linear problem, so this problem is often avoided when calculating seismic stability of bridges.

Accounting for friction between foundations and structures during earthquakes using a rigid plastic model with the development of a unique algorithm for solving the nonlinear problem (Mirzaev, Yuvmitov, Turdiev and Shomurodov 2021, Mirzaev and Turdiev 2022) showed the effectiveness of seismic isolation based on sliding friction.

Mathematical models and algorithms for investigation of complex seismodynamic processes taking into account friction between spans and supports of railway bridges under the influence of seismic wave have been created on the basis of the abovementioned tasks. Using the effect of dry friction between the span and abutment allows by selection of dry friction coefficient to increase seismic resistance of the bridge due to reduction of stresses in its elements. At the same time, cost savings will be achieved by making the necessary changes in the design of the railway bridge.

2 MATERIAL AND METHODS

Seismic waves consist of a vertical and two horizontal displacements, bridges also have three dimensions. The spans of reinforced concrete railway bridges often consist of differently shaped cross-section beams. Their remaining elements have different deformation properties and are connected with each other by eccentricity. Since the problem is mathematically complex, numerical methods are used to solve it.

Structural elements of the bridge experience simultaneous compression – tension, bending, torsion or compression – tension, shear, torsion or massless (masses of which can be disregarded) compression – tension, shear, torsion. Concentrated masses may be located at certain points. We replace each type of structural element with a corresponding finite element model. We discretise the elements of the bridge with the finite element method and obtain the following system of simple differential equations (Rashidov, Kuznetsov, Mardonov and Mirzaev 2021).

$$[M]\{\ddot{u}\} + \eta[C]\{\dot{u}\} + [K]\{u\} = \{P\}. \quad (1)$$

The initial conditions are obtained from the solution to the static problem

$$\{u(t)\}_{t=0} = \{u(0)\}, \quad \{\dot{u}(t)\}_{t=0} = \{0\}, \quad (2)$$

where $\{u(t)\}$ – is the vector of absolute displacements of nodal points of the finite element model of the structure. For nonlinear problems, matrices $[M]$, $[C]$, $[K]$ depend on the vector of absolute displacement, $\{P(t)\}$ includes the given ground motion and acting forces. The ground motion is specified in the form of seismogram records (Ambraseys, Smit, Douglas, Margaris, Sigbjörnsson, Ólafsson, Suhadolc and Costa 2004).

To solve the above system of equations (1) under conditions (2), the implicit Newmark finite difference method is used (Chopra 2012). In nonlinear problems an additional Newton – Ruffson iteration is used to refine the solution (Rashidov, Kuznetsov, Mardonov and Mirzaev 2021). The Newmark method uses velocity and acceleration values, including their values in a given seismic wave. Direct specification of digitized seismograms when calculating velocities and accelerations leads to gross errors. Therefore, the seismic wave is specified in accelerations and then the velocities and accelerations are computed by Newmark's formulas. The Hermite spline is used to ensure that the calculations are performed in a smaller time step than the digitization step.

As an example, in order to account for the earthquake resistance of a 29.6-metre long railway bridge on the "Navoi – Bukhara" section of the high-speed electrified railway, a SHARK calculation was carried out based on actual earthquake records.

The reinforced concrete railway bridge uses 4 standard girders, each 11.5 metres long with a cross-section in the shape of a T-bar. The bridge structure consists of many elements such as piers, abutments, transoms, spans etc. The total number of intermediate supports is 6, the dimensions: height – 2 m, cross-section – 0.35 m×0.35 m. The abutments are made of steel at the beginning and end of the bridge, and the abutments in the middle of the bridge are made of dense rubber, allowing for sliding. The structure is constructed mainly of reinforced concrete material – concrete of strength class B25, modulus of elasticity E = 30000 MPa, Poisson's ratio ν = 0.2.

The calculations take into account the suppleness of the foundation by means of reduced stiffness and damping. The stiffness and damping coefficients are calculated considering the size of the contact surface for each foundation separately. Soil type is hard loam, modulus of elasticity E = 1800 MPa, Poisson's ratio ν = 0.37. For the foundations at the beginning and at the end of the bridge, the coefficients of slackness in the three directions are taken as $4.446 \cdot 10^9$ N. For the foundations under the intermediate supports these coefficients are $19.5624 \cdot 10^9$ N.

3 THE DISCUSSION OF THE RESULTS

The calculation results for the railway bridge are derived from the real record of Gazli (Uzbekistan) earthquake of 17.05.1976, intensity over 9 on MSK-64 scale. Maximum acceleration, velocity and displacement in the direction of seismic wave propagation: 7.22 m/s^2; 0.62 m/s; 0.18 m and in transverse direction: 5.9345 m/s^2; 0.4818 m/s; 0.141676 m. Vertical acceleration 13.163 m/s^2, vertical speed 0.57 m/s, vertical displacement 0.216664 m (Ambraseys, Smit, Douglas, Margaris, Sigbjörnsson, Ólafsson, Suhadolc and Costa 2004).

Real records of strong earthquakes are obtained from the European database (Ambraseys, Smit, Douglas, Margaris, Sigbjörnsson, Ólafsson, Suhadolc and Costa 2004).

For discretisation the railway bridge was broken down into 116 finite elements taking into account the operation of each finite element type, the number of node points being 83. The calculations were carried out using an implicit scheme with a time step of 0.005 s and 0.001 s, the energy loss being accounted for in Rayleigh form. The characteristics of the finite elements of 10 different types were given through the corresponding ordinal numbers.

A small distance of 0.05 m is left between the girders along the main axis of the bridge and this gap is closed with a expansion joint. It is known that an important factor in the construction of all traffic structures is temperature effects, which are related to temperature differences. The expansion joint prevents the detrimental effects of temperature differences. With a temperature gradient of 40°C, the elongation of the span is 0.00644 m so that the expansion joint prevents the unhealthy effects of temperature gradients.

Now let's check whether installing the spans sequentially along the axis of the bridge with a 0.05 m expansion joint will cause the railway bridge to collapse during an earthquake.

Based on the results of the calculations, it is concluded that the maximum mutual approach and removal of the neighbouring beam ends along the bridge axis under the action of seismic waves, with no traffic on the railway bridge, is 0.0066 m at t=15.275 sec, -0.0068 m at t=17.945 sec (Figure 2). It can be seen that the 0.05 m expansion joint ensures that the ends of the neighbouring beams are prevented from colliding even during a very strong earthquake. This is because the bridge has a short overall length, causing the seismic wave to be involved very quickly almost simultaneously. The wave propagation lag in the ground is not significant compared to the propagation of disturbances along the spans.

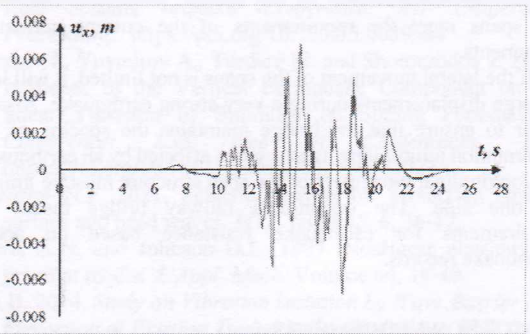

Figure 2. Mutual convergence of neighbouring faces by a seismic wave (Gazli earthquake).

Below are the results of the calculations, taking into account the mass of the locomotive on the bridge. The characteristics of the UZTE16M diesel locomotive were used to take the locomotive's mass into account:

- Total mass – 14271 kg;
- Length – 16.969 m.

The calculations show that the absolute Oz axis displacement in the middle of the span without a locomotive on the railway bridge was: the maximum value 0.217 m at t=15.725 s; the minimum value -0.119 m at t=18.445 s. Considering the mass of the locomotive: the maximum value is 0.215 m at t=15.73 s; the minimum value is -0.119 m at t=18.445 s (Figure 3). The maximum vertical displacement in a seismic wave is 0.216664 m. In static condition, the maximum deflection of the span is 0.00207 m including the mass of the locomotive, 0.00174 m without the locomotive. During oscillations, the maximum absolute displacements change due to the mass of the locomotive and there is also a phase shift. Therefore, the value of the dynamic deflection of the span in the presence of a locomotive is lower compared to the absence of a locomotive.

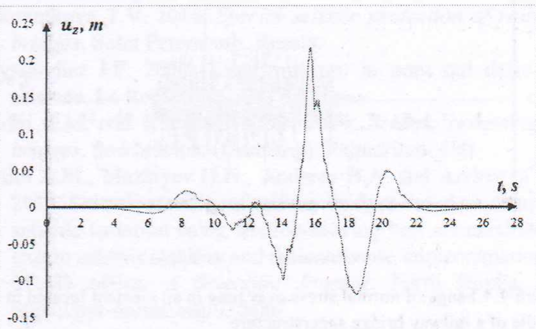

Figure 3. Time-varying displacement of the centre span of a railway bridge on the Oz axis (Gazli earthquake).

The maximum difference in displacement of the centre point of the span without and including the locomotive mass was 0.007 m.

The vertical elastic deflections of the spans calculated under moving temporary vertical loads must not exceed values in metres:

- for railway bridges – determined according to the formula $l/(800-1.25l)$, but no more $l/600$.

The oscillation of the superstructures along the axis of the bridge in the presence of rubber spacers at both ends during the earthquake leads to the mutual approximation of the beam ends in the middle of the bridge by 0.03 m at t=26.895 s and to the maximum mutual separation of 0.033 m at t=27.95 s. It follows that in this case a 0.05 m expansion joint is sufficient to prevent the collision of the girder ends. The small mutual separation and proximity of the bridge ends during a very large earthquake is due to the short length of the bridge. In this case, the seismic effects are almost equally transmitted along the length of the bridge.

The maximum and minimum values of displacement in the middle of beam for three values of damping parameter (β=0.01; β=0.05; β=0.1) were obtained and assumed equal for all bridge elements. The results are given in Table 1.

Table 1. Moving the centre point of the span along the axis of the bridge.

Attenuation parameters	*β=0.01*	*β=0.05*	*β=0.1*
Maximal	t=16.164 s 0.157747	t=16.161 s 0.157686	t=16.159 s 0.157649
Mininimal	t=18.342 s -0.179369	t=18.352 s -0.179215	t=18.354 s -0.179133

The time-varying stresses at the lower and upper points in the middle of the span were also obtained (Figure 4 and Figure 5). The weight of the locomotive was taken as the temporary load.

The results of bending moment in the centre of the superstructure in absence of temporary load: the maximum value was 0.37 MN·m at t=14.065 s, the minimum value was -0.38 MN·m at t=17.45 s. Given the mass of the locomotive: the maximum value was 0.48 MN·m at t=16.14 s, and the minimum value was -0.49 MN·m at t=15.905 s.

The following main analysis determined the maximum and minimum stress values at the top and bottom of the span.

Excluding the mass of the locomotive, the maximum value of normal stress in the upper central part of the span is 1.56 MPa at t=14.65 s, the minimum value is -1.58 MPa at t=15.895 s and the maximum value at the bottom is 2.9 MPa at t=15.895 s, the minimum stress value is -2.8 MPa at t=14.065 s (Figure 4).

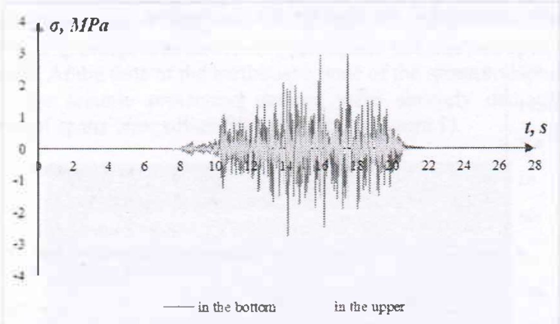

Figure 4. Change of normal stress over time in an element located in the middle of a railway bridge superstructure.

Considering the mass of the locomotive, the maximum value of the normal stress in the upper central part of the span is 2.1 MPa at t=14.07 s, the minimum value -2.0 MPa at t=16.24 s and the maximum value at the bottom was 3.78 MPa at t=15.905 s, the minimum value was -3.73 MPa at t=16.14 s (Figure 5).

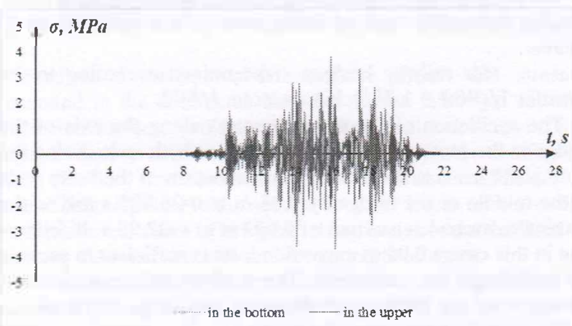

Figure 5. The change in normal stresses over time in an element located in the middle of the girder of a railway bridge, under the action of the mass of the locomotive.

The lateral movement of the bridge (Oy axis) during an earthquake was investigated. It was assumed that the superstructure was placed on ledgers without deformable connection. To facilitate calculations, a bilinear model of the interaction between the girder and the points of the span structure in contact with the girder has been adopted. This model approximates the dry friction process of the superstructure and the ledger between them in a dynamic process, since the relative displacement due to the linear connection is a very small fraction of the relative displacement as a whole. The force limit value was determined by Coulomb's formula based on the static pressure of the superstructure on the girder, the change in this pressure over time was not taken into account. The dry friction coefficient values used in the calculations are: f=0.1; f=0.2; f=0.4 (Figure 6).

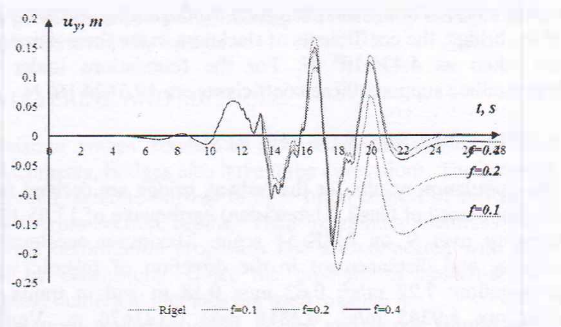

Figure 6. Time variations of transverse displacement of span and ledger at different dry friction coefficients.

Maximum difference of displacements of span and ledger at f=0.1 is 0.1666092 m, at f=0.2 is 0.0714271 m, and at f=0.4 is 0.01336 m (Table 2). Hence, the safe value of the dry friction coefficient between the superstructure and the waler of a reinforced concrete railway bridge during very strong earthquakes must be greater than or equal to 0.4, if no lateral motion limiters of the superstructure are available. The residual displacement after the seismic wave was: at f=0.1 it is 0.138143 m, at f=0.2 it is 0.0681626 m and at f=0.4 it is 0.0120574 m.

Table 2. Maximum displacement difference between span and rigel.

Dry friction coefficient	*f=0.1*	*f=0.2*	*f=0.4*
Difference	0.1666092	0.0714271	0.01336
Time	13.743 s	17.388 s	17.307 s
Residual shift	0.138143	0.0681626	0.0120574

The above results were compared with the value of the stresses allowed for the concrete classes according to normative document ShNK 2.05.03-12. For a typical 11.5 m long girder of a railway bridge, concrete grade B25 with a permissible compressive stress of 13 MPa and tensile stress of 3.0 – 3.6 MPa has been used. According to our results, a railway bridge can be damaged during earthquakes with a magnitude greater than 9.

In the case of the Bolu Viaduct, the supports are of large height and therefore during an earthquake the supports will undergo corresponding movements due to bending deformations. As a consequence, there are large forces in the supporting parts which can cause them to collapse and become in a frictional state.

4 CONCLUSIONS

According to the results of calculations obtained on the basis of real earthquake records of the 29.6 m long railway bridge located on the "Navoi – Bukhara" railway section, the normal stresses in the spans meet the requirements of the current normative documents.

If the lateral movement of the spans is not limited, it will lead to large displacements during a very strong earthquake. Also in order to ensure that the bridge maintains the spacing of the deformation joints along its axis when affected by an earthquake, the longitudinal movement of the span structure must be limited on one side. The operational railway bridge meets the requirements for earthquake resistance based on actual earthquake records.

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