Seismic Protection of Railway Bridges in Sochi

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SUMMARY:
The development of the city of Sochi as the place of the Olympic Games in 2014 required the construction of new railway lines. These lines include more than 50 bridges up to 500 m long and with piers up to 25 m high. The region is characterized by high seismicity. the most part of bridges were designed using seismic protection devices. The designing process was difficult due to the rigorous conditions put forward by the railway owner. Vertical displacements of span bearings under the railway load are forbidden and horizontal displacements under railway loads are limited. These conditions predetermined the new technical solutions of bridges seismic protection, which were worked out by the Joint-Stock Company "Transmost" with the scientific support of St-Petersburg Railway Transport University (PGUPS). Three limit states and three input seismic levels were used to design the bridges under consideration. Accordingly three levels of earthquake protection were proposed. The protection system includes a flexible steel bearings, viscous dampers and friction-movable joint with high-strength bolts.

Keywords: railway bridges, seismic protection, performance based designing

1. INTRODUCTION

The development of the city of Sochi as the place of the Olympic Games in 2014 required the construction of new railway lines. These lines include more than 50 bridges up to 500 m long and with piers up to 20 m high. The region is characterized by high seismicity. Earthquakes of intensity 8 on the MSK scale have the repeatability of more than one time per 500 years and those of intensity 9 have the repeatability of more than one time per 1000 years. In some building sites there can occur earthquakes with intensity 10 on the MSK scale. For example in the resort area Krasnaya Poliana they have the repeatability of one time per 5000 years. Taking into account these conditions the most part of bridges were designed using seismic protection devices. The designing process was difficult due to the rigorous conditions put forward by the railway owner. Vertical displacements of span bearings under the railway load are forbidden and horizontal displacements are limited by the value U_{lim}

$$U_{lim} < 0.5\sqrt{L}, \qquad (1.1)$$

where L is the span length, taken in m, and U_{lim} is taken in cm.

These conditions predetermined the technical solutions of bridge seismic protection. These solutions were worked out by the Joint-Stock Company "Transmost" with the scientific support of St-Petersburg Railway Transport University (PGUPS).

A preliminary analysis of the influence of seismic isolation stiffness of the bridge behavior under operational loads, which include transverse impact of the rolling stock and the longitudinal breaking force, was carried out. Bridge calculations for the abovementioned loads were made for three types of installing bearings, namely:

1. Conventional type, including ordinary bearings with the rigid transversal joint between the pier and the span.
2. Simple seismic isolation using isolating bearings instead of fixed ones, with linear movable bearings remaining the same.
3. Joining seismic isolation. In this case all bearings are overall flexible

Besides, the analysis was made both for ballast decks, and for decks without ballast (DWB). The results of the transversal load analysis of the two-span scheme 2x33.6 m are given in Table 1.1 and Fig.1

Table 1.1 Maximum Stresses For The R65 Rail Under The Transverse Impact Of The Rolling Stock

Types ob bearing installation, ballast stiffness (t/m)	Maximum displacement of the rail track, mm	M_{rail} maximum, tm	Q_{rail} maximum, t	Rail stresses, kg/sm^2
№1, 7500	0,15	0,02	0,23	27
№1, 1000	0,52	0,05	0,24	67
№2, 7500	3,5	0,46	1,78	613
№2, 1000	3,8	0,23	0,68	307
№3, 7500	137	1,64	7	2186
№3, 1000	137	1,82	6	2427

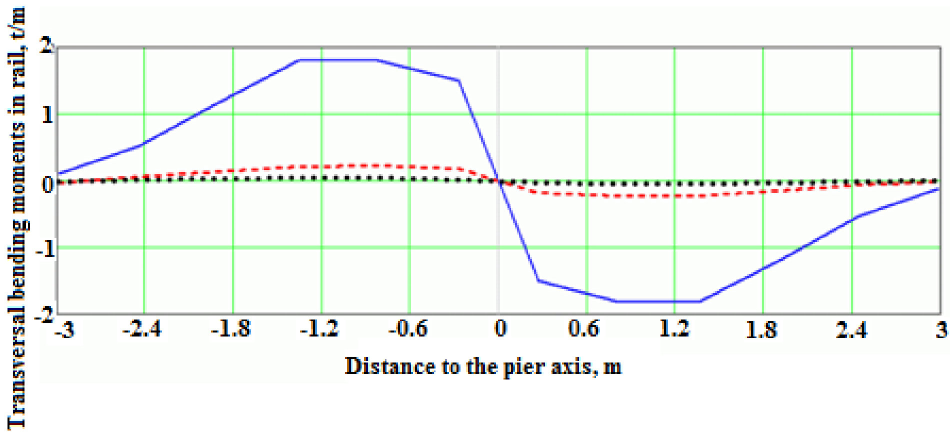

Figure.1. Transversal bending moments in the rail on the ballast deck above the pier
Dotted line is built for traditional bearings, slash line is built for simple isolation; solid line is built for joining isolation.

It should be stressed that in the given research the restrictions (1.1) for the pier flexibility limits was not observed. The displacements of the span relatively to the pier were approximately two times bigger than it is allowed by the Guidelines (7 cm according to the analysis, 2.7 cm – according to the Guidelines), but the forces and stresses in the rail were acceptable. The results of the research show that in all cases it is preferable to have a ballast deck for the seismic isolated bridge.

For the stress-strain state analysis of the rail under the breaking force three types of bearing installation on a bridge with deck ballast were also studied. The analysis of the three-span system was made in dynamic statement and its results are presented in Table 1.2.

The results of the research show that it is possible to decrease the Guidelines restriction (1.1) for the pier flexibility, but it is necessary to calculate rail stresses and displacements under the transverse impact and the train breaking. The most effective way is to use joining seismic isolation in the longitudinal direction with the longitudinally-flexible bearings installed on each pier and the fixation of every second bearing in the transversal direction. Besides, the thermo-strengthened rails should be used, with the permissible stress equal to 5800kg/sm^2. Following these solutions it is possible to obtain the partial frequency of the seismic isolated bridge in the interval of 0.9-1.2 s and to decrease the loads by 50-70%. The dependence of the relative span displacement caused by the breaking force on the partial period of seismic isolation is shown in the Fig.2.

Table 1.2. Span displacements under the train breaking along the whole bridge length

The bearing type	Type of displacement	Span displacements, sm		
		№1	№2	№3
Ordinary bearings	Displacements	3.52	3.55	3.56
	Mutual displacements	0.029	0.013	
Simple isolation	Displacements	27.316	27.320	27.321
	Mutual displacements	0.039	0.0123	
Joining isolation	Displacements	15.250	15.254	15.250
	Mutual displacements	0.033	0.033	

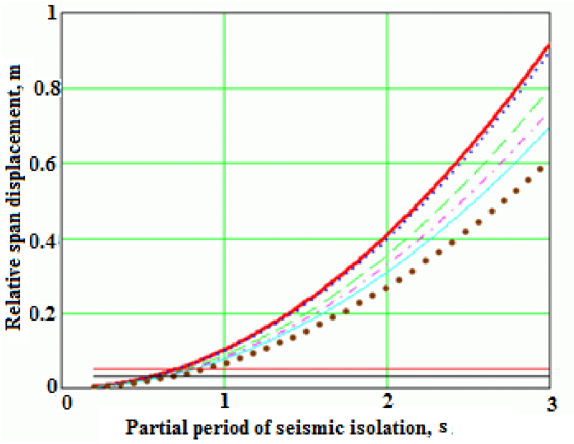

Figure.2. The dependence of the relative span displacement caused by the breaking force on the partial period of seismic isolation

Solid line is for L=44 m, dotted line is for L=55 m, slash line is for L=55 m, slash-dotted line is for L=66 m, thick dotted line is for L=110 m; horizontal lines show the range of the acceptable displacements.

Besides, it is necessary to alternate the seismic isolating bearings with longitudinally-movable or longitudinally-flexible ones, excluding the possibility of the transversal span movement as a rigid body.

2. THE MAIN PRINCIPLES OF THE RAILWAY BRIDGES SEISMIC PROTECTION IN SOCHI

The principles of performance based designing (PBD) were used in working out the seismic protection. Three limit states and three input seismic levels were used to design the bridges under consideration. Accordingly three levels of earthquake protection were proposed.

The three limit states are:

- the failure of the bridge serviceability which does not result in a long break in traffic;
- the failure of the bridge serviceability resulting in some hours traffic break but without damages to bridge load-bearing members
- Limited damages to the bridge load-bearing members without the piers destructions and a span fall

The probability of the abovementioned limit states in 100 years was taken equal to 0.1, 0.04 and 0.001 accordingly. The input level was calculated taking into account these values of limit states probability and the probability of earthquakes of different intensity. In the region of Sochi there are three main areas with different seismicity (intensity).

The main part of the region is characterized by seismic intensity 8,9 and 9 in accordance with seismic zoning maps A,B and C. It means that earthquakes with intensity 8 on the MSK scale have the frequency of one time per 500 years and earthquakes with intensity 9 on the MSK scale have the frequency of one time per 1000 years. Earthquakes with intensity 10 on the MSK scale are impossible.

The resort area of Krasnaya Poliana has seismic intensity 8,9 and 10 in accordance with seismic zoning maps A,B and C. It means that earthquakes with intensity 8 on the MSK scale have the frequency of one time per 500 years and earthquakes with intensity 9 on the MSK scale have the frequency of one time per 1000 years and with intensity 10 on the MSK scale occur once per 5000 years. The area of Imiritinskaya valley has seismic intensity 9,10 and 10 on the MSK scale. The frequency of earthquakes with intensity 10 on the MSK scale is one time per 1000 years.

In accordance with seismic safety and probability of limit states for each structure were generated three levels of the seismic input. The input was generated taking into account the dependence of peak accelerations on the predominant earthquake period (Dolgaya, 1996; Uzdin, 2005; Guidelines, 1996). This dependence is shown in fig.3. Finally we generated three earthquake inputs for each bridge for areas with different seismic danger: design earthquake with probability of 0.1 in 100 years, moderate earthquake with the probability 0.04 in 100 years and maximum earthquake with the probability of 0.001 in 100 years. To generate earthquake input, only one seismological characteristic, Areas intensity I_A was used. Spectral distribution of the input was selected as the most dangerous for the structure. We called this approach "the generating input for structure" in contrast to "the generating input for building site".

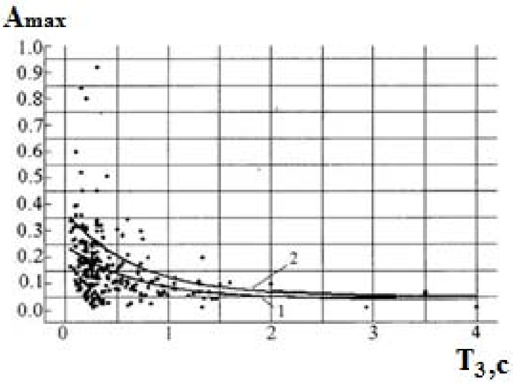

Figure.3. Dependence of peak accelerations A on the predominant earthquake period
1 – the average value of A; 2 – the average value plus standard deviation

To provide the abovementioned limit states under earthquakes with different repeatability, three levels of seismic protection were presented and realized. They are shown in fig 4.

The device of the first protection level is a flexible bearing with stiffness C. The bearing flexibility was set to provide the limit value of displacement under the train breaking load T_{br} equal to U_{lim} in accordance with formula (1.1). If seismic loads S are smaller, than the bearing capacity H, the flexible bearing ammortizes seismic impact. Otherwise the second seismic protection level comes to work. The second and the third levels of protection include a bolted joint with oval holes for bolts. The difference between the second and the third protection levels is in the value of bolt friction force. The friction force of the second level protection is smaller than the elastic limit for the bearing and the pear. If seismic load turns out to be greater than the above mention the bolt friction force, bolts of the second level connection begin to slide along the oval holes and cut the peaks of structure accelerations.

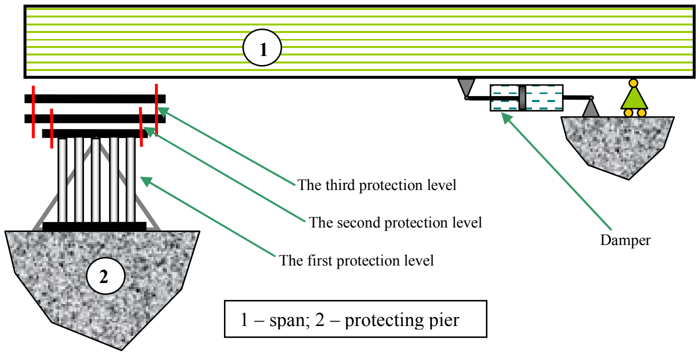

Figure 4. Three levels of bridge seismic protection

If in the process of sliding the displacement becomes equal to the size of the oval holes, the process of sliding begins in the bolt connection of the third protection level. The friction force of this connection is greater than elastic limit of the pier but it is not enough to result in progressive collapse or low-cycle fatigue of the bearing.

3. SETTING PARAMETERS OF THE FIRST PROTECTION LEVEL

The first protection level is defined by two main parameters – stiffness and damping of isolating bearings. Setting the bearing stiffness C depends on two system parameters. They are the relative mass of span and the span displacement under the train breaking load.

The relative mass of span $\nu = \frac{m_{span}}{m_{pier}}$ is the ratio of the span mass m_{span} to the pier mass m_{pier}, reduced to the top of the pier. If $\nu<2$, there is the optimal value of C_{opt}, minimizing the displacement U of the pier top and the moment in the foundation bed. If $\nu>2$, the smaller the value of C, the smaller the value of U. This fact is illustrated in fig. 5.

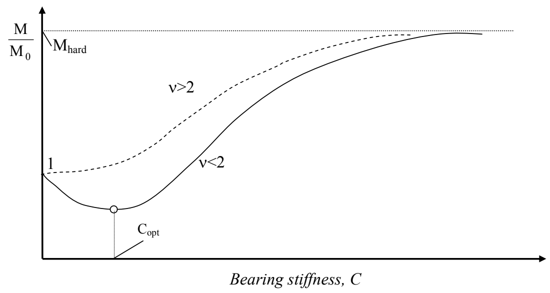

Figure 5. Dependencies of the relative bending moment in the foundation bed on the bearing stiffness
M_0 is the value of the bending moment for the pier without a span;
M_{hard} is the value of the bending moment for the pier with hard bearing

If $\nu>2$, the stiffness value is to be set equal to the minimum limit stiffness of C_{lim} which provides the limit displacement value U_{lim} according to inequality (1.1). If $\nu<2$, the stiffness value is to be chosen as the maximum between C_{opt} and C_{lim}. The stiffness limit in accordance with inequality (1.1) guarantees the absence of serviceability limit state under working loads. But on the other hand, it decreases the efficiency of seismic isolation. According to the results of our investigations limit (1.1) can be diminished, using the following inequality:

$$U_{lim}<\sqrt{L}, \tag{3.1}$$

The two-fold increase of the limit displacement according to inequality (3.1) in comparison with inequality (1.1) does not result in dangerous rail stresses and makes it possible to increase the efficiency of seismic isolation. Unfortunately, there was not enough time to coordinate inequality (3.1) with the existing Guidelines, so all bridges in Sochi were designed in accordance with inequality (1.1).

To estimate the structure behavior under earthquake loads, both the response spectra method (RSM) and the time-history analysis were used. The peculiarity of RSM used lies in building the damping spectrum alongside with the frequency spectrum. The damping spectrum $\Gamma = \lceil \gamma_1, \gamma_2, \ldots \gamma_n \rfloor$ is the a of mode inelasticity resistance coefficient. $\gamma_j = 2\zeta_j$, where ζ_j is the damping in portion of the critical value. In accordance with the value of γ_j the correction $K_\Psi(\gamma_j)$ to seismic forces for each mode was introduced.

$$K_\Psi(\gamma_j) = \sqrt{\frac{\gamma_r}{\gamma_j}} \tag{3.2}$$

where γ_r is the reference value of γ.

The value of γ_r was set, in order to keep on seismic loads for mass building facilities. Using the Russian guidelines we got γ_r equal to 0.11; 0.15 and 0.22 according to soils of the first, second and third category.

In order to use the proposed variant of RSM, the linearization of damping forces is necessary. A more accurate approach is based on the time-history analysis of the oscillation process. The calculations carried out on the basis of both RSM and time-history analysis show that seismic isolation can decrease seismic loads under design earthquake 1.4-1.7 times. The relative efficiency of isolation is not very large. Usually seismic loads can be decreased 2-2.5 times. It is caused by condition (1.1), which limits the isolation stiffness. But the obtained loads decrease turned out to be very important for the bridges under consideration, because the cost of piers was decreased by about of 20%.

The first protection level provides the elastic behavior of all structure members during the design earthquake, while the second and the third protection levels stay idle at the structure oscillations.

4. SETTING PARAMETERS OF THE SECOND PROTECTION LEVEL.

Under the moderate earthquake, seismic bearing loads approach the bearing capacity. To stave off the bearing and the pier collapse, steel sheets connected by the friction movable bolt joint (FMJ) begin to slide. In other words, the second protection level begins to work (Fig.6b).

The FMJ (Uzdin, differs from the usual bolted joint by oval holes for the bolts and a special covering of connected steel sheets, providing fluidity of the sliding process. When the oval holes under the bolts are closed, the bolts abut against the sheets and transfer the seismic load to the third protection level. This kind of protection is similar to the second level protection and represents the FMJ, but the friction force of the third level protection is greater than that of the second level. This force causes plastic deformation in the bearings and in the reinforcement of piers as well as crack formation in the body of the piers. But the level of plastic deformation excludes the possibility of pier progressive collapse and low-cycle plastic fatigue.

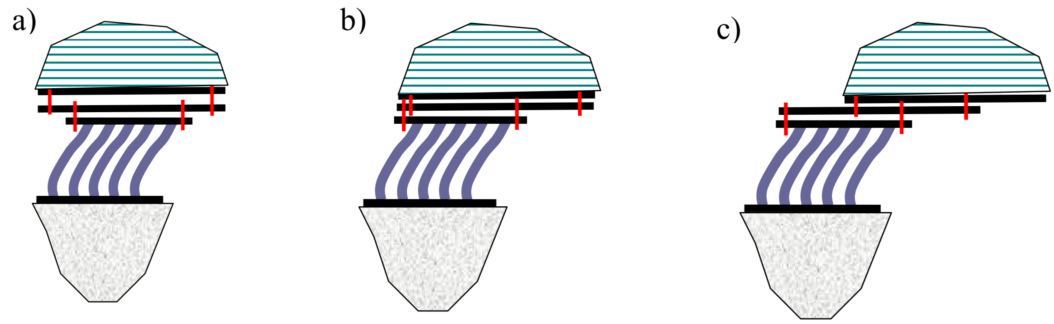

Figure 6. The behavior of protection levels
a) The first level; b) the second level; c) the third level

The friction force of the FMJ is given in accordance with the above mentioned demands. The design parameters of the FMJ are the sizes of the oval holes. The FMJ is to provide smooth mutual motion of connected members with the given friction force. The estimating of the size of oval was carried out on the basis of the time-history analysis. For this analysis two design diagrams of bridges were used.

The first simplified diagram has 2-5 degrees of freedom and includes viscous dampers and dry friction dampers. An example of such system is shown in fig.7. In this case the FMJ is modeled as a dry friction damper (3.1) and the real hydraulic damper is modeled as a viscous damper. Each dry friction damper of the design diagram can be in one of the two states - open or closed. The state of dry friction dampers defines the state of the system, which is an on-off system. The software for calculating such systems was worked out by the authors.

The system shown in fig. 7 in the first approximation was considered as three independent systems: two independent intermediate piers and the system including extreme piers connected with seismic protection devices by the span. On the left a flexible bearing with FMJ is installed and on the right a hydraulic damper is installed. Friction forces in usual movable bearings are ignored. The hydraulic damper is modeled by a viscous damper.

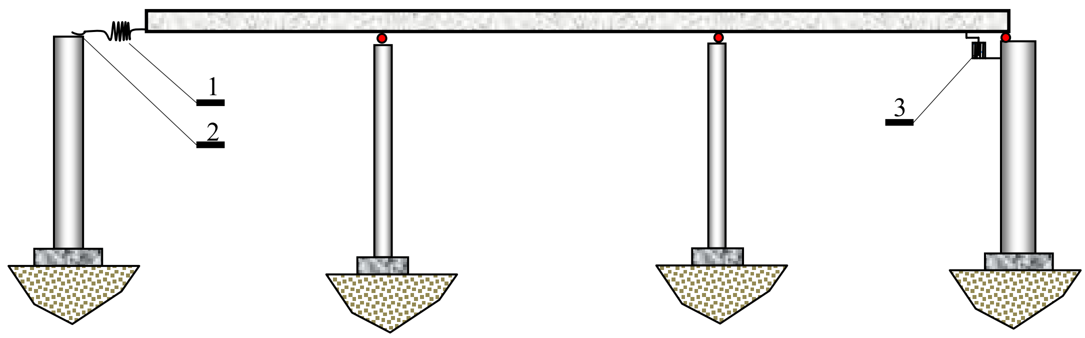

Figure 7. The simplified diagram for the analysis of FMJ behavior
1 – flexible bearing; 2 – FMJ; 3 – viscous damper

The bridge under consideration has a very heavy reinforced concrete span and displacements in the FMJ during the design earthquake with repeatability once per 500 years could not be excluded. These displacements for the most problematic bridge of the project come up to 4 cm. The time-history process of the structure oscillations are shown in fig.8

Table 4 presents the scenario of damage accumulation. All damages caused by the shift in the FMJ. For the design earthquake the shift value is 2.2 cm. For the moderate earthquake it is equal to 5.6 cm and for the maximum earthquake it is 9.5 cm. The protection system excludes other types of damages.

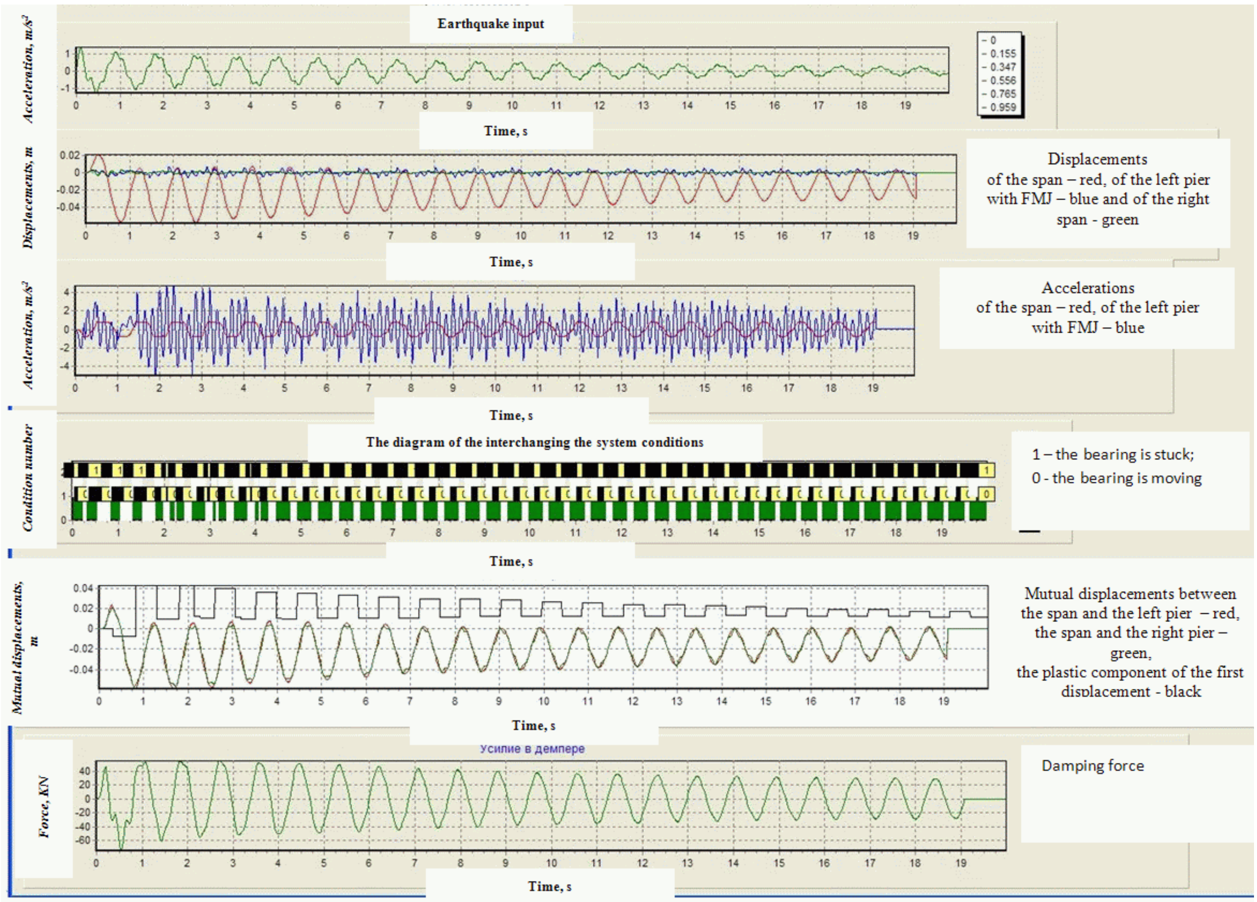

Figure 8. Time-history process of the structure oscillations for the DE7 and the structure life equal to 100 years
From the top-down
1) The original accelerogram;
2) mass displacements : red – for the span, blue – for the left pier, green – for the right pier
3) mass accelerations: : red – for the span, blue – for the left pier with FMJ,
4) the diagram of the system states change
5) mutual displacements: red – between the span and right pier, green - between the span and left pier, black – shift in the FMJ
6) damper strength/load

To make more detailed analysis, the second finite – element diagram and calculations using the software "MIDAS" were used.

The efficiency of the proposed technical solutions turned out to be very high, but it was limited by inequality (1.1). Using inequality (3.1) instead of (1.1) makes it possible to increase the efficiency twofold.

Table 4.1. Scenario of damages accumulation

Input	Lifetime T_{life}, years	Number of shifts	Maximum shift, sm	Residual sift in the FMJ, mm	Bearing travel, sm	Peak acceleration of the pier top, m/s^2	Peak acceleration of the span, m/s^2	Dynamic factor,β	Damper load, kN
MP3	100	49	9.5	21.7	12	10	1.9	β_{pier}=3.257 β_{span}=0.618	80
П3-8	100	42	5.6	16.6	8.5	7.2	1.8	β_{pier}=3.64; β_{span}=0.91	56
П3-7	100	42	2.2	1.19	4	6	1.7	β_{pier}=4.32; β_{span}=1.22	40

5. EXAMPLES OF THE APPLYING THE PROPOSED PROTECTION SYSTEM

The proposed protection system was used for more than 15 railway bridges in Sochi. All these bridges have reinforced concrete continuous heavy beam spans. To sustain seismic loads from the span, only one pier is used. It is impossible without seismic protection devices.

The view of one of the protected bridges is shown in fig.9, as an example. The bridge has 7-spans highstructure notably 23+33.6+4×44+22 m with the total weight of 8000 tons. Some piers with movable bearings are supplied with dampers of "Vibroseism" firm. These dampers are described by Dr.Kostarev (2011). The height of used dampers is about 55 cm.

The view of flexible fixed bearing, more accurately a flexible table without sustaining vertical loads on it, is shown in figure 10. It includes 198 flexible bars made of high-strength steel with the design resistance equal to 880000 kPa/m^2. The height of the used flexible table is about 90 cm.

Figure 9. The view of one of bridges with proposed protection system

Figure 10. The view of flexible table

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