The use of seismic isolators for improving the seismic response of a concrete bridge structure

I.R. Răcănel, D.I. Crețu & T.Labiş-Crețu
Technical University of Civil Engineering, Bucharest, Romania

T.Ivănescu
S.C. I.P.T.A.N.A. S.A., Bucharest, Romania

ABSTRACT:
The earthquakes from the second half of XX-th century have produced severe damages through the collapse of some bridges in the U.S.A., Taiwan and Turkey.
Even if the earthquake from 4-th of March 1977 in Romania has not produced significant damages on the Romanian bridges, the problem of seismic protection has become very important.
The research done world wide regarding the rehabilitation and earthquake protection of structures has generated innovative solutions for the control of the structural response.
In this paper the behaviour of a reinforced concrete bridge structure equipped with seismic isolators is investigated. The numerical studies presented are performed for two types of seismic isolators placed on the piers: elastomeric isolators and friction penduli devices.
The obtained results show the effect of the reduction of inertial forces on the piers and foundation stresses as a measure of the improvement of structural performance in the case of seismic action.

Keywords:earthquake,bridge,piers,concrete,isolators

1. INTRODUCTION

The bridge analyzed in this paper is located on the national road DN1 at the km 8+900. The bridge superstructure consists, in cross section, in 20 precast concrete girders, having a height of 1.03 m and a distance between them of 1.22 m. Above the girders was poured a concrete slab of variable thickness, starting from 12 cm next to the footpath and reaching 35 cm in the bridge longitudinal axis. The total width of the superstructure is 25.80 m and cover 6 circulation lanes, 3 on each side of the bridge, 2 footpaths of 1.20 m each side and a central separation zone of 1.00 m.

In longitudinal direction, the bridge comprises 13 spans which are covered by four structures separated by expansion joints. The first structure has 3 spans (26.10 m+26.00 m+26.10 m) and a length of 78.20 m, the second structure has the same distribution and values for the spans, the third structure has four spans (26.10 m+32.00 m+32.00 m+26.10 m) and a length of 116.20 m and finally, the fourth structure was designed and built identically to the first two. The total length of the bridge, including also the opening of the expansion joints (10 cm) has resulted 350.80 m.

From static point of view, in longitudinal direction the precast concrete girders can be considered as simply supported, the continuity above the piers being ensured through the concrete slab. Over the bearings, on the piers, the girders are transversally connected through cross beams.

The bridge substructure consists in two abutments and twelve piers which have deep foundations on piles having the diameter of 1.08 m and a length of 20.00 m. The piers elevation has a depth of 2.00 m in bridge longitudinal direction and a variable width, starting with 3.00 m at the level of the foundation and reaching 5.70 m at the pier cap, below the cross beams.

The connection between super- and substructure is made through neoprene bearing devices which were installed below the cross beams, over the piers.

In this paper the behaviour of the third structure under seismic loads was investigated. A general layout of the whole bridge structure is shown in figure 1.

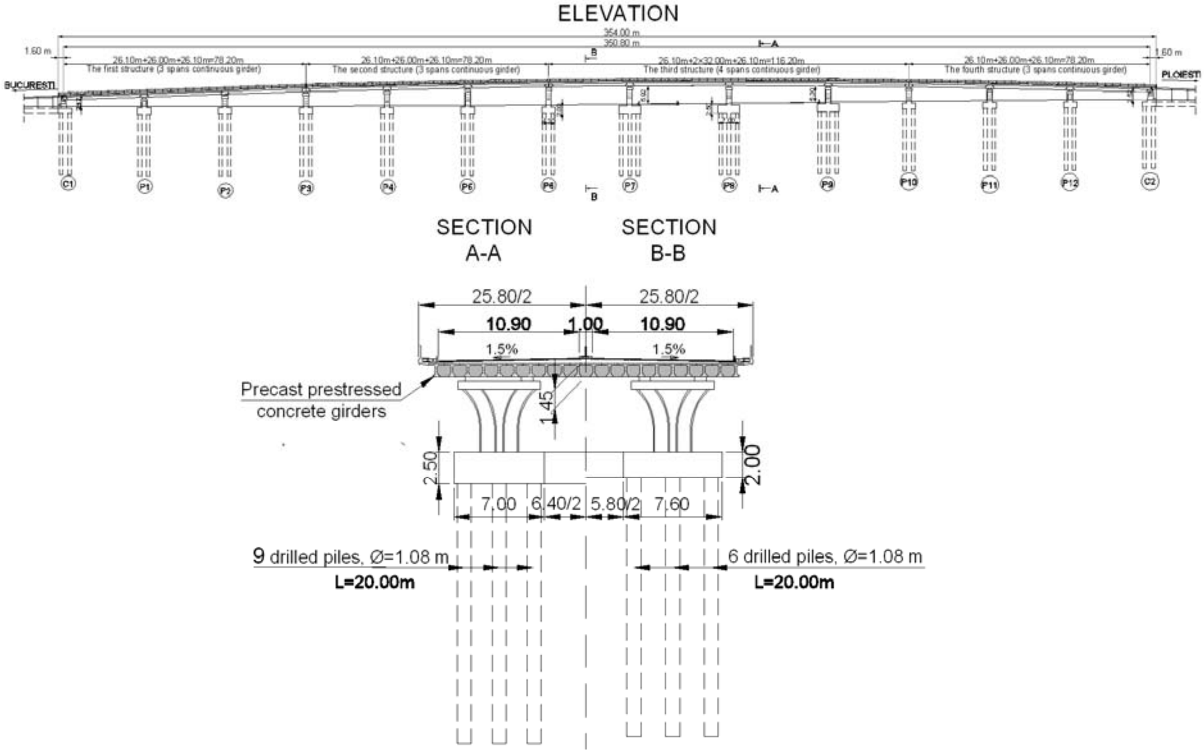


Figure 1. General layout of the analyzed bridge: elevation and cross sections

2. OBJECTIVES OF THE STUDY

Romania is a seismically active country and there are two major types of earthquakes which were registered during the time on Romanian territory:

- medium depth earthquakes, in this case the seismic source being placed 110-160 km under the earth surface in Vrancea Mountains. These are rare but severe earthquakes and they have an estimated return period of 40-50 years. Generally this kind of earthquakes affect an elliptical region of the Romanian territory, the major axis of the ellipse being oriented in NE-SW direction. The earthquake from the 4-th March 1977 had the epicentre in Vrancea and a 7.2 magnitude on Richter scale. This seismic event killed 1500 people and produced severe damages on a large number of structures.
- surface earthquakes which are directly related with intracrustale fractures and have the source at small depth, 10-20 km under the earth surface. These kind of earthquakes have a short return period and medium intensity. The major faults are situated in the West and South-West zone of the Romanian territory, in Transylvania and Banat.

In Romanian standard the seismic hazard is described through the peak value of the horizontal ground acceleration a_g established for a average recurrence time interval (IMR) of 100 years. The zoning of the territory with respect to a_g is presented in figure 2. The ground movement during an earthquake is described through elastic response spectra for absolute accelerations. The local site conditions are accounted for through the values of the corner period T_c of the response spectra. These values show synthetically the composition in frequencies of the earthquake movements. On this basis, a second map which includes the values of the corner period T_c is established and is presented in figure 3.

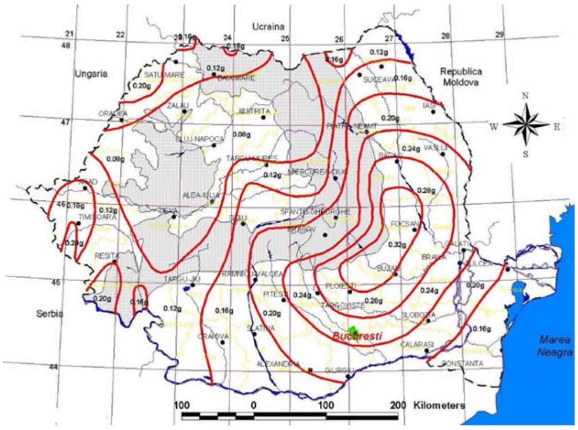


Figure 2. Zoning of Romanian territory according to the peak ground acceleration a_g values

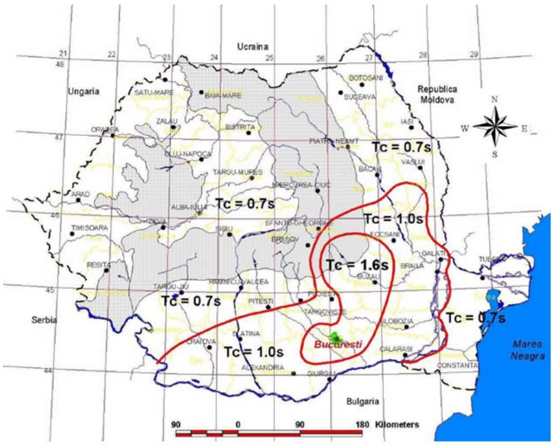


Figure 3. Zoning of Romanian territory according to the corner period T_c values

The behaviour of the bridge, situated near Bucharest town, under seismic action is investigated through numerical analyses.

3. STRUCTURE MODELING

In order to analyze the behaviour of the bridge under seismic action, because of model complexity and dimensions, a full 3D finite element model was built (figure 4).

The longitudinal and cross girders were modelled using two nodes straight frame finite elements having activated all six degrees of freedom at each node. The concrete slab was introduced in the model using four nodes finite elements with a membrane and shell behaviour. They have also activated all six degrees of freedom at each node. In order to respect the position of all elements in the model, the frame elements (longitudinal and cross girders) are eccentrically disposed with respect to the midplan of the concrete slab.

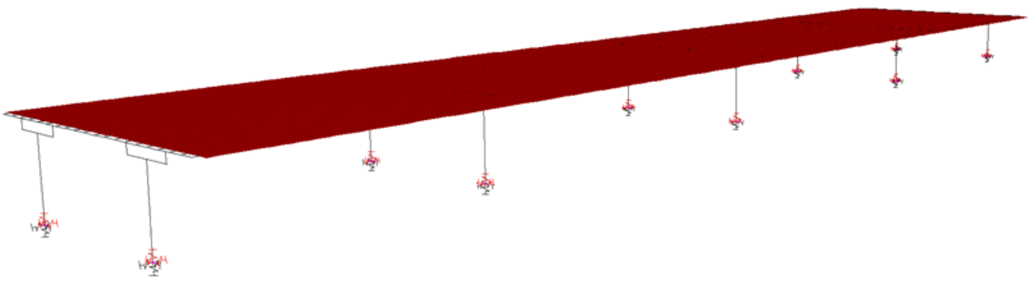


Figure 4. The 3D finite element model used in the analyses

The substructure elements, consisting in piers with variable dimensions in elevation and cross section, were modelled also using two nodes straight finite elements. The variable section of the pier elevation was modelled using a frame element with a nonprismatic cross section.

The connection between super- and substructure was modelled using vertical and horizontal frame elements, with very high axial, shear and bending stiffness. These rigid elements allow to place the bearing devices in the right position with respect to the bottom part of the longitudinal girders.
The bridge structure is equipped with two types of neoprene bearing devices: on the piers P6, P7, P9 and P10 were disposed bearing devices type 16 and on the pier 8 type 19. Generally the bearing devices used for this bridge look like in the scheme presented in figure 5 and have the main characteristics given in table 3.1. The difference between them consists in different dimensions in plan, but also in height and

these lead to different levels of mobilized axial and shear stiffnesses. These existing neoprene bearing devices are modelled through link elements with elastic behaviour. For the shear stiffnesses, in all models, two values were considered, for slow and fast loads respectively.

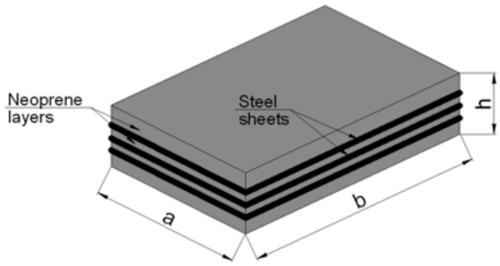


Figure 5. The scheme of a neoprene bearing device

Table 3.1. Main dimensions and capacities of the used bearing devices

Bearing device type	a [mm]	b [mm]	h [mm]	Bearing capacity [kN]
16	400	500	110	3000
19	500	600	52	4500

All substructure elements, abutments and piers have deep piles foundations, the piles diameter being 1.08 m and the length 20 m. The piles are modelled with two nodes straight frame elements and at the cap, appropriate joints constraints were introduced, in order to simulate the presence of the concrete piles cap. This is stiff enough, according to it dimensions, to be considered as a rigid body. The soil-structure interaction is considered in the model through elastic springs placed in plan in two orthogonal direction and also in vertical direction. The stiffness characteristics of these springs are established according to the variable soil properties on the depth. Because of the large number of piles, in the 3D model used for the dynamic analyses (figure 4), the group of piles are replaced by an equivalent group of six springs (three for translations and three for rotations) with coupled stiffnesses.

4. PERFORMED ANALYSES. RESULTS

In order to study the bridge behaviour under seismic loads some finite element model were built. The first model considered was the existing structure, having the neoprene bearing devices placed on the piers as was presented in the section above. On this model linear modal eigen vectors, response spectrum analyses and linear time-history analyses were performed. On this basis, the stress levels in the structural elements, but also the nodal displacements were obtained. For the response spectrum analyses, the used spectrum is that proposed by the Romanian norm P100-2006, for a corner period T_c of 1.6 seconds (figure 6). For linear time-history analyses, three ground motion functions were used. One of them was the ground acceleration record on direction N-S during the earthquake from 4-th of March 1977 (figure 7a), the other two (figure 7b and c) being generated accelerograms.

For all response spectrum analyses, the behaviour factor q was considered equal to 1.0 assuming that the pier cross sections behaviour will remain in linear elastic domain.

For the existing structure, the fundamental period is T_1=1.216 s and that leads to significant accelerations at the superstructure level according to the response spectrum curve in figure 6. It can be noticed that in this case, the piers are not uniformly loaded from point of view of bending moments and shear forces, the central pier where the bearing device type 19 is placed, taking the highest values of stresses. In table 4.1 are given, for comparison, the values of the bending moments and shear forces at the base of central pier, but also the displacements at the superstructure level, following a response spectrum and linear time-history analyses using as input function the record of ground acceleration from Vrancea earthquake in 1977.

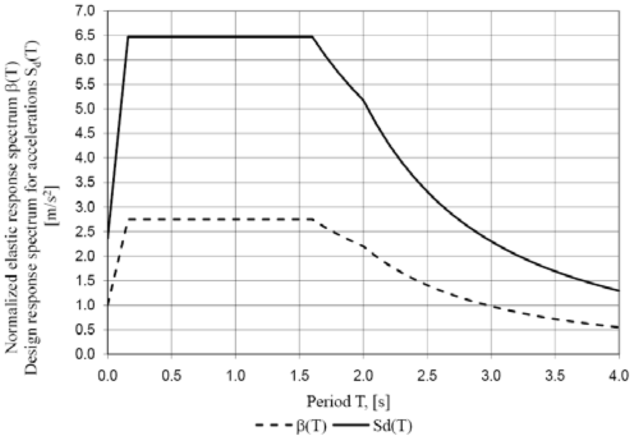


Figure 6. The design response spectrum

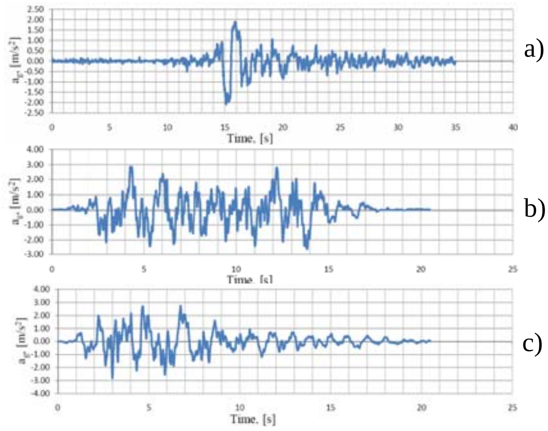


Figure 7. The accelerograms used in analyses

Table 4.1. Bending moments, shear forces and displacements

Analysis type	*M [kNm]*	*T [kN]*	u_x *[mm]*
Response spectrum	46383	8013	244
Linear time-history	42068	7686	231

The obtained values of the bending moments and shear forces at the base of the pier require high quantities of reinforcement on the pier cross section. For this reason, in the second stage, in order to obtain almost a uniform distribution of the stresses and displacements for all piers, it was assumed that the bridge is equipped on all piers with type 16 bearing devices.

In this last case, the fundamental period of the structure increased to T_1=1.458 s, the stress level was corresponding decreased (more than 50%), but in the same time the horizontal displacement of the superstructure increased. In table 4.2 are presented, for comparison, the values of the stresses at the base of central pier and the superstructure displacement in both situations: for the existing structure and for the structure equipped on all piers with bearing devices type 16 following response spectrum analyses.

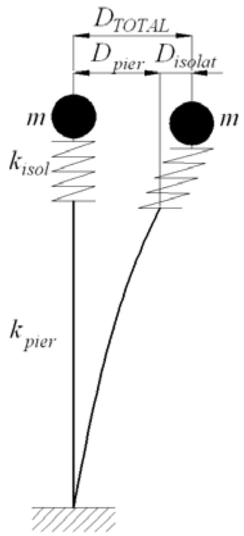
Table 4.2. Bending moments, shear forces and displacements for the central pier

Model type	*M [kNm]*	*T [kN]*	u_x *[mm]*
Existing structure	46383	8013	244
Structure with all bearings type 16	21350	3659	351

Analyzing these values it was concluded that, although the stress level on the pier cross section was strongly reduced, mounting on all piers the same bearing device type 16 as only measure will not solve the problem, because the fundamental period will place the structure in the range of high values of the absolute acceleration spectrum, on the horizontal line in figure 6.

It was clear that, in order to obtain a better behaviour of the bridge on horizontal accelerations induced by the earthquakes, it is necessary to shift the fundamental period of the structure to higher values and in the same time to introduce higher values for damping coefficients. This objective could only be achieved by using special bearing devices. For this study two types of bearings were chosen: elastomeric isolators and friction isolators.

In the case of using elastomeric isolators, the global response of the structure in terms of accelerations, displacements and stresses depends on isolators and piers stiffness and damping constants. Two finite element models of the structure equipped on all piers with elastomeric isolators were analyzed. The isolator characteristics were established assuming two target values for the fundamental period of the structure: $T_{1,1}$=3.0 s and $T_{1,2}$=4.8 s. These values are approximately two times, respectively three times grater than the corner period T_c=1.6 s on the site. Starting from these assumptions and considering the scheme presented in figure 8 the main characteristics of the isolators (stiffness and damping constant) were established. In the figure 8, m is the mass of the superstructure acting on a pier, D_{TOTAL}, D_{pier}, Di_{sol} are total, pier and isolator displacement respectively, k_{pier} and k_{isol} are pier and isolator stiffnesses.



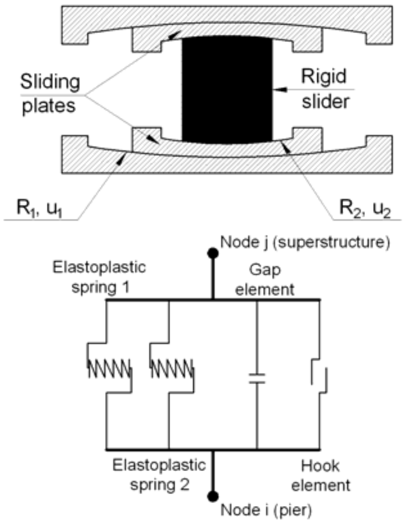


Figure 8. Simplified model for the pier with isolators

Figure 9. The scheme and model of a friction isolator

The equations used for establishing the isolators characteristics are :

$$T_{eff} = 2\pi\sqrt{\frac{m}{k_{eff}}}\ ;\ \ k_{isol} = \frac{1}{n}\cdot\frac{k_{pier}\cdot k_{eff}}{k_{pier}-k_{eff}} \tag{4.1}$$

$$\omega_{isol} = \sqrt{\frac{1}{n}\cdot\frac{m}{k_{isol}}}\ ;\ \ C_{isol} = \frac{2\cdot\omega_{isol}\cdot\xi\cdot m}{n} \tag{4.2}$$

Using the above presented equations the elastomeric isolators characteristics are determined and their values are presented in table 4.3. In these equations, n represents the number of isolators on each pier, T_{eff} is the target period of the structure isolator-pier , ω_{isol} is the circular frequency of the isolator, C_{isol} is the isolator damping constant and ξ is the fraction of the critical damping (in this case 15%).

Table 4.3. Elastomeric isolators characteristics

T_{eff} [s]	k_{isol} [kN/m]	C_{isol}	ξ [%]
3.0	1720	5.48	15
4.8	669	8.81	15

The values of k_{isol} and C_{isol} were used as effective stiffness and damping for the link elements which are modelling the elastomeric isolators in the linear time-history analyses.

In figure 9 are presented the scheme of a friction isolator device and the model used in the finite element analyses. The friction isolator was modelled with the aid of four link elements coupled in parallel: two elastoplastic links which introduce the hysteretic element that simulates the shear stiffness and energy dissipation, a gap element which simulates the axial vertical stiffness and resistance to uplift the bridge superstructure and a Hook element which simulates the stopper of the device when reaching the limit estimated displacement. The modelling parameters for elastoplastic link finite elements were determined starting from a target period of the isolator T=3.0 s. Knowing the period of the isolator, the radius of the concave surface and subsequently the effective stiffness (K_{eff}) and damping (β_{eff}) of the elastoplastic spring 2 are given by the equations:

$$T = 2\pi\sqrt{\frac{R}{g}}\ ;\ K_{eff} = \frac{V}{R} + \frac{u \cdot V}{D}\ ;\ \beta_{eff} = \frac{2}{\pi} \cdot \frac{u}{u + (D/R)} \tag{4.3}$$

In the above relations V is the vertical load acting on the isolator, u is the friction coefficient, D is the target displacement of the isolator and g is the gravity constant.

Starting from some isolators characteristics which are presented in [Zekioglu et.al], but taking into account the higher values of the vertical reactions produced by the analyzed bridge superstructure, the characteristics for both elastoplastic springs were adjusted in order to maintain the target period of the structure. For the elastoplastic spring 1 these characteristic are:

$k_1 = 1000000\,;F_{y,1} = u_1 \cdot V$ the initial stiffness and the yiled force respectivelly (4.4a)

$\alpha_1 = 1 \cdot 10^{-5}$, the post yield stiffness ratio (4.4b)

For the elastoplastic spring 2 the characteristic are given by:

$$k_1 = \frac{V}{R_1}\ ;\ k_2 = \frac{V}{R_2}\ ;\ F_{y,2} = (u_2 - u_1) \cdot V\ ;\ \ \alpha_2 = \frac{k_2}{k_1} \tag{4.5}$$

For the analyzed friction isolator the following values were used: $V=3800$ *kN*, $R_1=0.5842$ *m*, $R_2=2.236$ *m*, $u_1=2.95$ *%* and $u_3=3.2$ *%*. The target displacement imposed to the friction isolator was 0.35 m and introduced in the model for the Hook element.

The nonlinear behaviour of the friction isolator was considered in the time-history nonlinear analyses. The analyses were performed for all three accelerograms presented in figure 7.

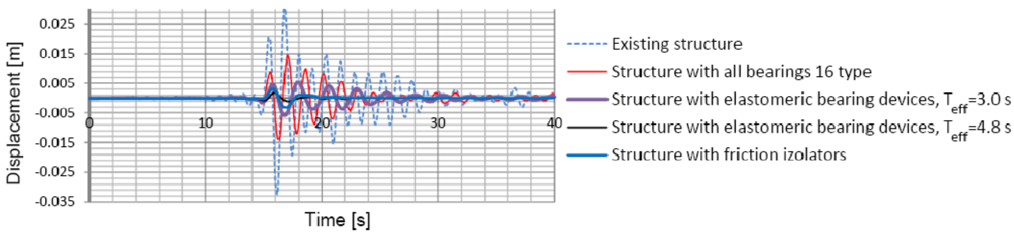


Figure 10. Time-history of the displacement at the top of central pier. Accelerogram: Vrancea, N-S, 1977

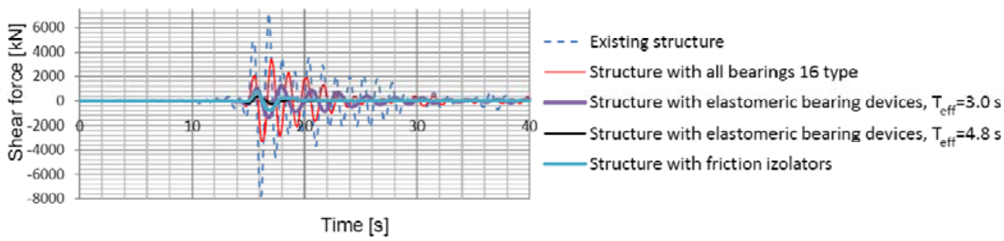


Figure 11. Time-history of the shear force at the base of central pier. Accelerogram: Vrancea, N-S, 1977

In figures 10 and 11 are presented the evolutions in time of the displacement at the pier top and of the shear force at the base of the pier, for all models considered. In figures 12 and 13 is presented the evolution of the horizontal displacement in both elastoplastic springs as a function of the shear force in each of them.

In table 4.4 are given, for comparison, the values for the stresses in the central pier and superstructure displacements resulted following the linear and nonlinear time-history analyses, on all considered finite element models, for the recorded accelerogram during the earthquake in Vrancea, 1977.

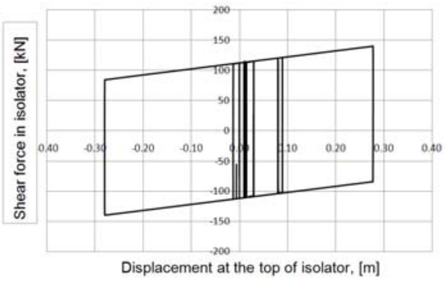


Figure 12. Displacement in elastoplastic spring 1

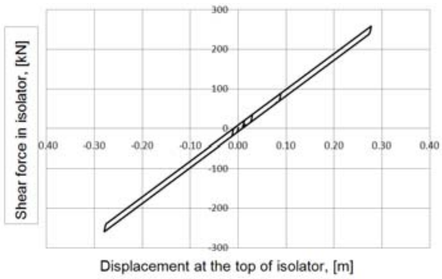


Figure 13. Displacement in elastoplastic spring 2

Table 4.4. Stresses in the central pier and superstructure displacements

Finite element model	Bending moment [kN]	Shear force [kN]	Displacement [m]
Existing structure	42068	7687	0.232
Structure with all bearings 16 type	19846	3421	0.322
Struct. with elastomeric isol. and T_{eff}=3.0 s	7264	1368	0.403
Struct. with elastomeric isol. andT_{eff}=4.8 s	2263	386	0.290
Struct. with friction isol. and T_{eff}=3.0 s	4246	786	0.285

5. CONCLUSIONS

In this paper a concrete bridge is analyzed from the point of view of reducing the effects by the seismic action on the behaviour of the structural elements. For this purpose five types of finite element models were built and analyzed through linear response spectrum and time-history analyses, but also through nonlinear time-history analyses. As input data for these analyses, response spectrum, recorded and generated accelerograms were used. Analyzing the obtained results it can be concluded that the approach of isolating the bridge superstructure from the substructure will bring real benefits from stresses level point of view but also, with some measures, from displacements point of view. Through a correct calibration of the isolation systems, the stress level in substructure elements, piers and abutments, can be reduced several times (more then 10 times) and through this, reduction of necessary concrete and reinforcement quantities will lead to real costs economy of the final project. Concerning the displacement values, it can be seen, from the table 4.4, that the structure displacements are below the limit of the isolation systems. Although, these values are bigger than those accepted in the current practice for Romanian bridges. This problem can be easy solved by designing special expansion joint devices which can be used together with the designed bearing isolators.

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