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INNOVATIVE METHOD FOR BUILDINGS PROTECTION TO SEISMIC ACTIONS

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Abstract: *The base isolation method is increasingly used to the seismic protection of buildings due to its advantages over the conventional design. The paper presents a comparative study between two seismic isolation systems. The first system is composed of high damping rubber bearings and of linear fluid viscous dampers (HDRB+LFVD) and the second system is composed of triple-pendulum friction bearings (TPFB). The study shows that the displacements and base shear forces of the isolated structure, with the two isolation systems, have close values. With the isolation system composed of high damping rubber bearings and of linear fluid viscous dampers are recorded the minimum accelerations, both at the level of the isolation plane and at each level of the building.*

Key words: *friction bearing, rubber bearing, time-history analysis, viscous damper.*

1. Introduction

The first seismic isolation system was proposed in 1909 by the English doctor Johannes Avetican Calanteriants. He suggested the separation of the structure from the foundation through a layer of talc. [5]. Later in 1969, the natural rubber bearings were used for seismic protection of Pestalozzi school in Skopje, Macedonia. The bearings were made of rubber blocks without metal plates, which have been deformed about 25% under the weight of the building. The vertical stiffness of the bearings was only several times larger than the horizontal stiffness and the rubber does not possess damping [5].

The study make a comparison between different seismic isolation systems, considering a reinforced concrete building subjected to seven pairs of seismic actions. The main objective of the study is represented by the comparison - in terms of displacements (of the isolation systems and of the structure), base shear forces, energy dissipation and accelerations - between two seismic isolation systems.

2. Description of the Analysed Structure and of the Isolation Systems

The analysed structure is a dual structure, with reinforced concrete shear walls and frames on both directions of the building. The location of the building was considered Bucharest city due to the long predominant periods of the seismic actions and due to the difficulties given by this long predominant periods to the isolated buildings design.

The height regime consists of ground floor and eight storeys, with storey height of 2.8 m. The building has three spans - the central one of 3 m and the marginal ones of 7 m - and four bays of 8 m.

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The resistance to the lateral forces is provided by the reinforced concrete shear walls, placed on both directions of the building and reinforced concrete frames. The wall thickness on the x direction is 35 cm and on the y direction is 30 cm, being constant on the entire height of the building. The columns are made of square section of 70 cm x 70 cm, without cross-section reduction with height. The longitudinal beams are made of T cross-section with the web thickness of 35 cm, the height of 70 cm, the flange thickness of 16 cm and flange width of 100 cm. The transversal beams are also made of T cross-section with the web thickness of 30 cm, the height of 60 cm, the flange thickness of 16 cm and the flange width of 100 cm. The thickness of the reinforced concrete slabs was taken of 16 cm [2].

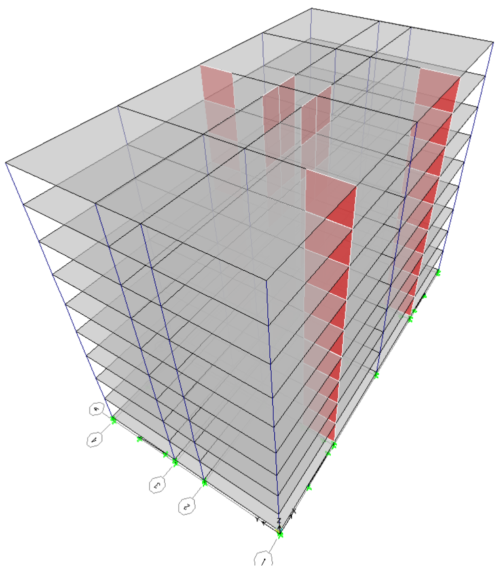


Fig. 1. *The analysed building*

At the level of the isolation plane, a reinforced concrete slab of 16 cm thickness was considered, having the main role of distributing the horizontal forces to the isolation system. The reinforced concrete slab is supported on the longitudinal and on the transversal beams of the T cross-section with the web thickness of 100 cm, the height of 60 cm, the flange thickness of 16 cm and the flange width of 170 cm. This beam has greater cross-sections in order to avoid the plastic hinges occurrence and to ensure a good connection with the isolation devices [2].

The analysed building was seismically isolated using two different seismic isolation systems. The first system (HDRB+LFVD) is composed of twenty-five high damping rubber bearings (one bearing under each column and two bearings under each reinforced concrete shear wall) and twelve linear fluid viscous dampers (six on the longitudinal direction and six on the transversal direction of the building). The second system (TPFB) is composed of twenty-five triple-pendulum friction bearings (one bearing under each column and two bearings under each reinforced concrete shear wall).

3. Preliminary Design of the Isolated Building

The linear static analysis was performed using the ETABS v9.2.0 [6] computer program, considering the stiffness of the elements reduced with fifty percent due to concrete cracking, according to P100-1/2006 [10] seismic code.

The horizontal seismic forces, f_i, applied to the each level of the analysed structure, were determined using Eq. 5 [9], [11]:

$$f_i = m_i S_a(T_{ef}, \xi_{ef}) = m_i \frac{S_e(T_{ef}, \xi_{ef})}{q}, \quad (1)$$

where m_i is the mass of each storey; $S_a(T_{ef}, \xi_{ef})$ is the design spectral acceleration corresponding to the effective period of vibration, T_{ef}, is the fundamental mode of vibration of the analysed structure, and to the effective damping, ξ_{ef}; $S_e(T_{ef}, \xi_{ef})$ is the elastic spectral acceleration corresponding to the effective period of vibration, T_{ef}, in the fundamental mode of vibration of the analysed structure, and to the effective damping, ξ_{ef}; q is the behavior factor considered with the value of 1.5, as recommended by [9] and [11].

The preliminary design of the isolation systems was performed, considering the analysed structure a system with one dynamic degree of freedom. The structure was isolated at a vibration period, T_{is} = 3.5 s, taking into account a damping ratio of the isolation systems, ξ_{ef} = 28%.

The displacement demand of the isolation systems, d_{dc}, to the design earthquake was determined using Eq. (2) [9], [11]:

$$d_{dc} = \left(\frac{T_{is}}{2\pi}\right)^2 \cdot a_g^d \cdot \beta(T_{is}) \cdot \eta = \left(\frac{3.5}{2\pi}\right)^2 \cdot 0.24 \cdot 9.81 \cdot 0.718 \cdot 0.55 = 0.289\,\text{m}, \quad (2)$$

where a_g^d is the ground acceleration corresponding to the design earthquake; $\beta(T_{is})$ is the normalised spectral ordinate, corresponding to the vibration period, T_{is} and η is the damping correction factor.

The damping constant, C_{lvd}, of the linear fluid viscous dampers was determined using Eq. (3) [1].

$$C_{lvd} = \frac{4\pi \cdot G_{SC} \cdot \xi_{lvd}}{n_{lvd} \cdot T_{is} \cdot g} = \frac{4\pi \cdot 65668.16 \cdot 13\%}{6 \cdot 3.5 \cdot 9.81} = 520.9\,\text{kN}\frac{\text{s}}{\text{m}}, \quad (3)$$

where G_{SC} is the total weight of the building in the special combination of loads; ξ_{lvd} is the damping ratio of the linear fluid viscous dampers; n_{lvd} is the number of the linear fluid viscous dampers on x and y direction, respectively and g is the ground acceleration.

The effective horizontal stiffness, k_{ef}^{hdrb}, of one high damping rubber bearing was determined using Eq. (4) [5]:

$$k_{ef}^{hdrb} = \left(\frac{2\pi}{T_{is}}\right)^2 \cdot \frac{G_{SC}}{n_{hdrb} \cdot g} = \left(\frac{2\pi}{3.5}\right)^2 \cdot \frac{65668.16}{25 \cdot 9.81} = 862.9\frac{\text{kN}}{\text{m}}, \quad (4)$$

where n_{hdrb} is the number of high damping rubber bearings.

The effective horizontal stiffness, k_{ef}^{tpfb}, of one triple-pendulum friction bearing was determined using Eq. (5) [3]:

$$k_4 = \frac{G_{GS}}{R_1 - h_1 + R_4 - h_4} = \frac{65668.16\,\text{kN}}{3.3 - 0.175 + 3.3 - 0.175} = 10506.9\frac{\text{kN}}{\text{m}},$$

$$D^* = (\mu_{1f} - \mu_{2f}) \cdot (R_2 - h_2) + (\mu_{1f} - \mu_{3f}) \cdot (R_3 - h_3) = (0.035 - 0.03) \cdot (0.6 - 0.1) + (0.035 - 0.03) \cdot (0.6 - 0.1) = 0.005\,\text{m},$$

$$D^{**} = D^* + (\mu_{4f} - \mu_{1f}) \cdot (R_1 - h_1 + R_3 - h_3) = 0.005 + (0.08 - 0.035) \cdot (3.3 - 0.175 + 0.6 - 0.1) = 0.168\,\text{m},$$

$$k_{ef}^{tpfb} = \frac{F_{f4} + k_4 (D_{cp} - D^{**})}{n_{tpfb} \cdot D_{cp}} = \frac{5253.4 + 10506.9 \cdot (0.289 - 0.168)}{25 \cdot 0.289} = 903.1\frac{\text{kN}}{\text{m}}, \quad (5)$$

where k_4 is the horizontal stiffness of the triple-pendulum friction bearings after sliding onset on the surfaces 1 and 4, R_1 is the radius of curvature of the sliding surface 1, R_2 is the radius of curvature of the sliding surface 2, R_3 is the radius of curvature of the sliding surface 3, R_4 is the radius of curvature of the sliding surface 4, h_1 is the distance from the pivot point of the articulated slider to the sliding surface 1, h_1 is the distance from the center of the bearing to the sliding surface 1, h_2 is the distance from the center of the bearing to the sliding surface 2, h_3 is the distance from the center of the bearing to the sliding surface 3, h_4 is the distance from the center of the bearing to the sliding surface 4, D^* is the displacement corresponding to the sliding initiation on the surface 1, D^{**} is the displacement corresponding to the sliding initiation on the surface 4, μ_{1f} is the friction coefficient of the sliding surface 1, μ_{2f} is the friction coefficient of the sliding surface 2, μ_{3f} is the friction coefficient of the sliding surface 3, μ_{4f} is the friction coefficient of the sliding surface 4, F_{f4} is the friction force corresponding to the sliding initiation on the surface 4 and n_{tpfb} is the number of triple-pendulum friction bearings.

In the Figure 2 is presented a schematical configuration of a triple pendulum friction bearing with the indicated notations, used in Eq. (5). Figure 2 was made using information from [3].

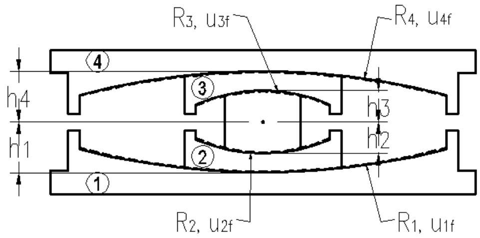


Fig. 2. *Schematic configuration of a triple pendulum friction bearing*

4. The Seismic Action

The seismic action is described by six artificial accelerograms compatible with the design spectrum for Bucharest and one recorded seismic motion on INCERC-Bucharest site, corresponding the March 4, 1977 earthquake [2].

The recorded seismic motion was scaled, to the maximum ground acceleration of 0.24 g, corresponding to the design ground acceleration for Bucharest, having the mean reocurrence interval of 100 years. The artificial accelerograms were generated by means of the SeismoArtif [8] computer program [2]. In Figure 3 is given an artificial accelerogram used in the nonlinear time-history analysis.

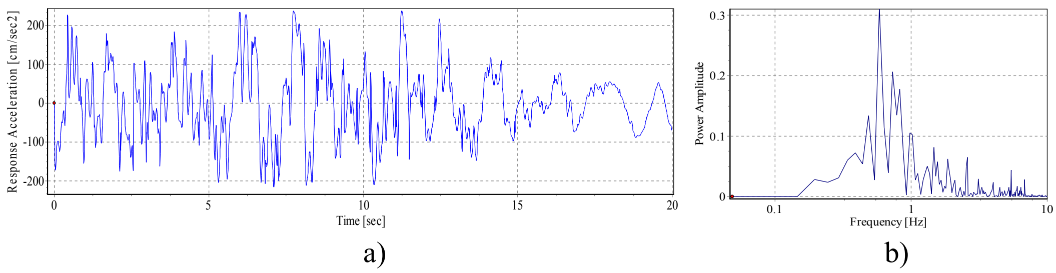


Fig. 3. *Artificial accelerogram (a) and power spectral density of the accelerogram (b)*

5. The Nonlinear Time-History Analysis

The nonlinear time-history analysis of the isolated structure was performed using the SAP2000 v15.1.0 [7] computer program, considering the structural elements and the isolation systems with nonlinear behavior [2].

The nonlinear behavior of beams and columns was modelled with plastic hinges at the elements ends (concentrated plasticity model) of *M3* type and *PM2M3*, respectively [2]. The shear walls were modelled with *shell layered-nonlinear* elements, with nonlinear behavior in both bending with axial force and shear force. For the concrete from the boundary elements of the shear wall was used a model with constant confinement - Mander 1988 model [4] - and for reinforcement was used the model automatically generated by the program with yielding plateau and post-elastic hardening. The strength of the materials was considered with mean values [2].

The devices which form the isolation system HDRB+LFVD were modelled in the following manner: the high damping rubber bearings were modelled using the link type element *Rubber Isolator*, which was put in parallel with a *Gap* element to take into account the different behavior in tension and compression and the linear fluid viscous dampers were modelled using *Damper* element. In Table 1 are given the parameters of the devices which compose the HDRB+LFVD isolation system, used in the nonlinear time-history analysis.

Table 1

Parameters of HDRB+LFVD isolation system used in nonlinear dynamic analysis

Direction	**Rubber Isolator**			**Gap**	**Damper**		
	k_e	f_y	k_p/k_e	k_e	k_e	C	α
	[kN/m]	[kN]	[-]	[kN/m]	[kN/m]	[kNs/m]	[-]
U1	182900	-	-	2561100	243600	520.92	1.0
U2	2728	85.93	0.2327	-	-	-	-
U3	2728	85.93	0.2327	-	-	-	-
* k_e is the elastic stiffness; k_p is the post-elastic stiffness; f_y is the yielding strength; C is the damping coefficient and α is the velocity exponent.							

Since the behavior of TPFB is not adequately captured in SAP2000 computer program, modeling of this device was made using two *Rubber Isolator* link type elements and a *Gap* element, which were put in parallel. Modeling the triple-pendulum friction bearing, using two *Rubber Isolator* elements, was considered to simulate, in the first stage, the initiation of sliding in the first pendulum and then continuation of sliding in the third pendulum.

In Table 2 the parameters of the devices which compose the TPFB isolation system, used in the nonlinear dynamic analysis are given.

Parameters of TPFB isolation system used in nonlinear dynamic analysis Table 2

Direction	**Rubber Isolator 1**			**Rubber Isolator 2**			**Gap**
	k_e	f_y	k_p/k_e	k_e	f_y	k_p/k_e	k_e
	[kN/m]	[kN]	[-]	[kN/m]	[kN]	[-]	[kN/m]
U1	0	-	-	0	-	-	3000000
U2	800000	78.8	0	724.6	131.34	0.58	-
U3	800000	78.8	0	724.6	131.34	0.58	-
* k_e is the elastic stiffness; k_p is the post-elastic stiffness; f_y is the yielding strength.							

The seismic action was considered simultaneously in the three directions of the building (two horizontal directions and one vertical direction), complying with the provisions of paragraph 4.5.3.6.2 (4) from the P100-1/2006 seismic code.

The elastic damping was taken into account by using Rayleigh damping, considering the damping ratio of 3% for the vibration modes between 0.5 T_1 and 1.25 T_1 (T_1 is the period of vibration in the fundamental mode).

The response of the isolated building is highlighted for each seismic action described in the Section 4 and for each horizontal direction of the building.

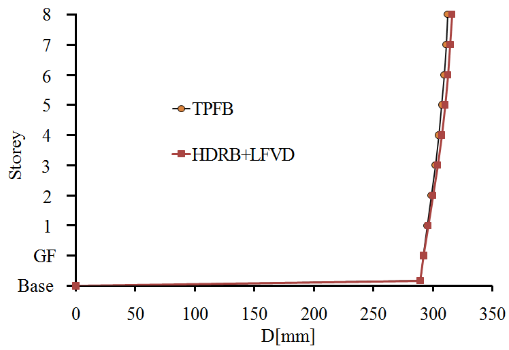
The mean relative displacements of the two isolation systems are given in the Figure 4. For the *x* direction of the building, the two isolation systems experience almost the same displacements. For the y direction of the building, the minimum displacements are obtained with the TPFB isolation system.

The percentage difference between the two isolation systems, at the level of the isolation plane, is 0.1% for the *x* direction and 5.1% for the *y* direction.

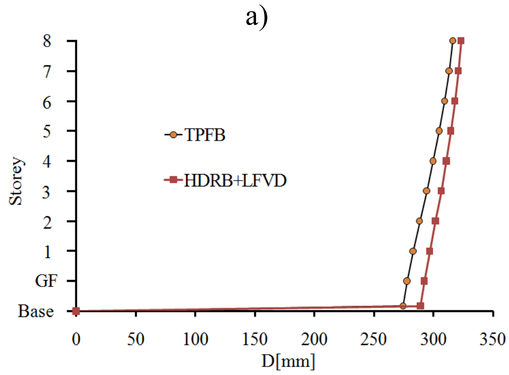
In some design situations it is necessary to limit the accelerations in the structure to protect a certain valuable content. Thus, were made comparisons in terms of accelerations both at the level of the isolation plane and at each floor level of the structure.

Figure 5 presents the mean absolute accelerations of the two isolation systems**.** In both horizontal directions of the structure, minimum accelerations are obtained with HDRB+LFVD isolation system. The percentage difference between the two isolation systems, at the level of the isolation plane, is 45.5% for the *x* direction of the building and 44.4% for the *y* direction.

In the conventional design the energy induced by an earthquake is dissipated through post-elastic deformations of the structural elements. Through base isolation the dynamic properties of the structure are

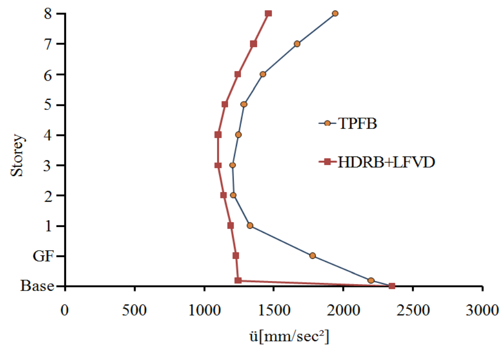


a)

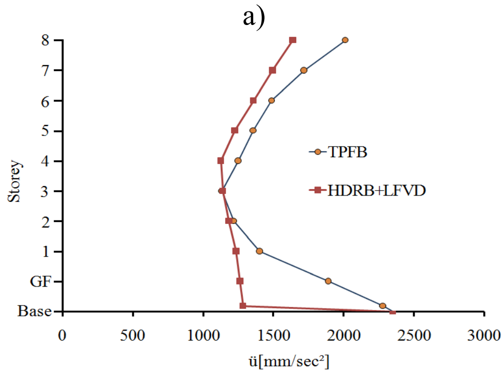


b)

Fig. 4. *The mean relative displacements of the isolated structure, with HDRB+LFVD system and with TPFB system:*
a) x direction; b) y direction



a)



b)

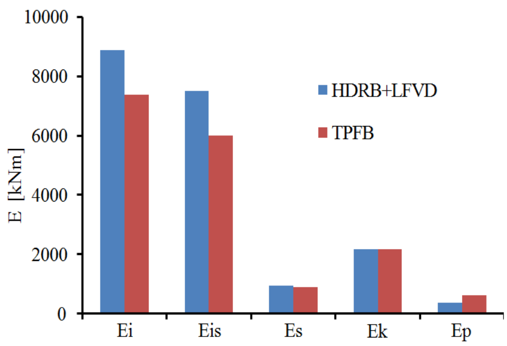
Fig. 5. *The mean absolute accelerations of the isolated structure, with HDRB+LFVD system and with TPFB system:*
a) x direction; b) y direction

changed, so that the energy induced by an earthquake is greatly diminished and is dissipated, for the most part, by the isolation system.

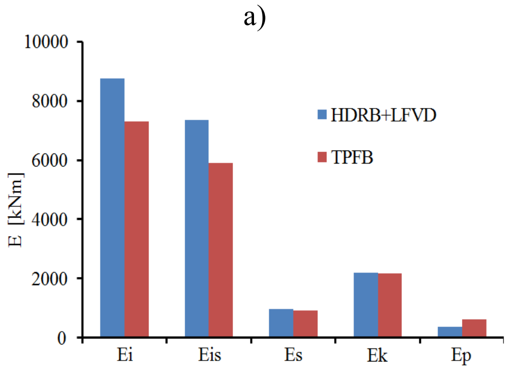
In Figure 6 is presented the mean energy induced by the seismic actions and dissipated through various mechanisms. There ware used the following notations:

- E_i - the energy induced by the seismic actions;
- E_{is} - the energy dissipated by the isolation system;
- E_s - the energy dissipated by the structure through post-elastic deformations and elastic damping;
- E_k - the kinetic energy;
- E_p - the potential energy.

In order to have a fair indicator of the energy dissipated by the isolation systems and by the structure, this must be reported to the energy induced by seismic actions.



a)



b)

Fig. 6. *The mean energies of the two isolation systems: a) x direction; b) y direction*

Thus, for the x direction of the building, the structure isolated with HDRB+LFVD system, dissipates 84.4% of the energy induced by the seismic actions through isolation system and 10.6% through post-elastic deformations and elastic damping. The structure isolated with TPFB system, dissipates 81.3% of the energy induced by the seismic actions through isolation system and 12.1% through post-elastic deformations and elastic damping.

For the y direction of the building, the structure isolated with the HDRB+LFVD system, dissipates 83.9% of the energy induced by the seismic actions through isolation system and 11% through post-elastic deformations and elastic damping. The structure isolated with the TPFB system, dissipates 80.7% of the energy induced by the seismic actions through isolation system and 12.6% through post-elastic deformations and elastic damping.

The base shear force is a key parameter in characterizing the seismic response of structures and is used to design them. In the Figure 7 is presented the mean base shear forces for the two isolation systems. For the x direction of the building, the two isolation systems have almost the same base shear force. For the y direction of the building, the minimum base shear force is obtained with the TPFB isolation system. The percentage difference between the two isolation systems is 0.4% for the x direction and 2.7% for the y direction.

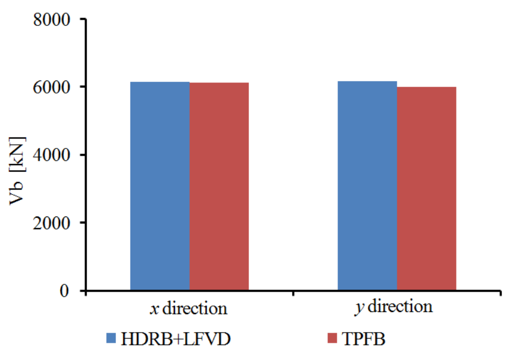


Fig. 7. *The mean base shear forces*

6. Conclusions

The performed study examines the seismic performance of two different base isolation systems, considering a vibration period of the isolated structure of 3.5s and a damping ratio of 28%. It was analysed the response in terms of relative displacements, absolute accelerations, dissipated energies and base shear forces of base isolated structure to recorded and generated earthquake ground motions.

The mean relative displacements of the structure with two isolation systems have close values regarding the relative displacements and base shear forces. The minimum absolute accelerations are obtained with the HDRB+LFVD isolation system. Regarding the dissipated energy, the HDRB+LFVD isolation system dissipates more energy than the TPFB isolation system.

Depending on the design requirements a system or another can be used; for example, if the limitation of the storey accelerations is required the HDRB+LFVD isolation system is more suitable.

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