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Building Materials and Structures

Analysis of the seismic response of an RC frame structure with lead rubber bearings

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A B S T R A C T

Base isolation of buildings is the most efficient way of designing seismically resistant structures. Application of seismic isolators allows mutually independent movements of the ground and the structure during earthquakes. The application of seismic isolators increases the natural period of vibrations, which reduces the seismic forces in the structure. The paper analyzes the influence of the application of lead rubber bearings on the response of the structure to the action of an earthquake. A reinforced concrete frame structure was analyzed both for the case of base isolation and rigid foundation. Based on the comparative analysis of the natural period of vibrations, base shear seismic forces, displacement of the top level of the structure and relative inter-storey drift, conclusions were drawn about the efficiency of application of this type of seismic isolator. The required amount of ductility of a structure, as well as damage to structural and non-structural elements, is greatly reduced by base isolation.

1   Introduction

The influences on the structure caused by an earthquake are quite often dominant in the design of structures in seismically active areas. During an earthquake, damage to the structural and non-structural elements can be caused, as well as the collapse of the structure. This leads to significant consequences, such as huge material costs or, potentially, to the loss of human lives, which are irretrievable. Therefore, the seismic protection of buildings became a very attractive field of research in the 20th and 21st centuries.

The beginning of the concept of the design of seismically resistant structures dates from the end of the 19th and the beginning of the 20th century, resulting in the registration of various patents within this field. The proposed solutions consisted of separating the structure and the foundation by the system of the balls in concave bearings [1], as well as by a layer of sand or talc [2]. Such a system provided relative ground movement with regard to the building, lowering seismic forces in the structure. Nowadays, this concept is known as base isolation.

The modern base isolation concept is based on the application of devices that are set in seismic dilatation. Seismic dilatation is constructed at the level of the foundation or above the stiff basement structure, so the structure is divided into isolated structures and substructures. Seismic isolation devices are stiff enough in the vertical direction to transfer the gravity load, but they are less stiff in the horizontal direction. As a result, the natural period of vibration of the isolated structure increases up to several times compared to the rigidly founded structure. By increasing the natural period of vibration of the structure, the values of mass acceleration are decreased as well as the intensity of the seismic forces in the structure. The application of these devices changes the response of the structure during an earthquake. A rigidly founded structure is dominantly deformed by bending and shearing due to an earthquake, while in the case of the base isolation, the structure is moving predominantly translational. That is why the damage to the structural elements of the seismically isolated structures is smaller compared to conventional structures. The intensity of the seismic forces in the structure also depends on the characteristics of the foundation ground. Due to the increasing natural period of vibration of the structure, the intensity of the seismic forces is decreased, while in the case of the soft ground seismic forces are increased. The seismic isolation of a structure has the greatest effect on the structure with the shortest natural period of vibration, as in the case of a good foundation in the ground.

According to the way the horizontal flexibility is provided, seismic isolators can be divided into elastomeric, sliding, and combined bearings [3]. Elastomeric bearings provide seismic isolation of structures with the flexibility of the material used for their manufacture. Natural or synthetic rubber is used in the production of elastomeric bearings. According to the level of damping, elastomeric bearings can be: low-damping rubber bearings, lead-rubber bearings, and high-damping rubber bearings [4-7].

A lead rubber bearing (LRB) has been developed in order to increase the damping of elastomeric bearings and to reduce the displacement of the structure. It consists of steel

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plates, rubber, steel shims, and a lead core in the central part (Figure 1). The lead core is dominantly deformed by shear, characterized by a relatively small yield strength (usually about 10 MPa). Due to the plastification of the lead core, seismic energy is absorbed, and the force-displacement dependence of this isolator can be idealized by a bilinear diagram [8, 9].
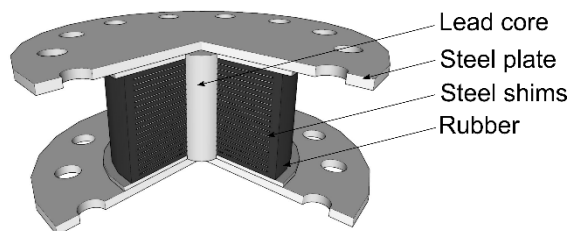

Figure 1. Characteristic section and basic parts of lead rubber bearing

Numerous experimental research has been conducted in order to determine the mechanical characteristics of this type of LRB. It has been confirmed that the lead core provides an adequate level of energy dissipation [4, 10-13]. The numerical models for the analysis of the properties of a LRB were also developed [12, 14-17]. The seismic analysis of base isolated structures confirmed that the use of LRB has a favourable effect on the structural response [18-20]. Similar results were confirmed in the analysis of the seismically isolated bridges [21].

The paper analyses the dynamic response of a reinforced concrete frame structure with LRB under the action of the north-south component of the Imperial Valley (El Centro) earthquake. A comparative study of the dynamic response of the structure was conducted with regard to the case of a rigidly founded structure. The analysis was performed on the SAP2000 software package.

2 Setting up the analysed problem

The effects of the application of base isolation of buildings are analysed on the reinforced concrete frame structure G_r+13S_t, with a storey height of 3 m. Frames are set at an equal distance of 4 m. The structure has 6 bays in one and 4 bays in the other horizontal direction (Figure 2). The columns have a square cross-section of 60 cm, and the beams have a rectangular cross-section of b/h = 25/50 cm. The floor slabs are 20 cm thick. All structural elements are made of concrete, class C25/30. In both the lower and upper zones, the columns are reinforced with 12BØ20 and the beams with 5BØ16. The reinforcement was adopted so that the structure could bear both dead and live loads. The intensity of the dead load is 2.00 kN/m², while the live load is 3.00 kN/m². The loads are uniformly distributed over all floor slabs.

The analysis of the case when the structure is rigidly founded and supplied with base isolation with LRB is conducted. The LRB of the Dynamic Isolation System company is used. The maximum axial force in the columns under the serviceability load, which acts as a vertical force in the seismic isolator, is around 2300 kN. Based on the technical documentation, an isolator of 650 mm in diameter is adopted under each column, and its axial capacity is 2700 kN [22]. The isolator is composed of twenty layers of 12 mm thick rubber, and between each of them are 3 mm thick steel shims. In the central part of the isolator, a lead cylinder with a diameter of 150 mm is placed. The bilinear hysteretic behaviour of the isolator is described by elastic and post-elastic stiffness and yield strength. For the adopted isolator, the post-elastic stiffness is 505.3 kN/m and the yield strength is 100 kN. According to the manufacturer's recommendation, the horizontal elastic stiffness of the isolator is approximately 10 times larger than the post-elastic stiffness [22], and it is adopted in the paper. Analysis of the dynamic response of the structure under earthquake action also includes the influence of the vertical stiffness of the isolator, which is 700,000 kN/m [22].

3 Numerical analysis of the structure's seismic response

A numerical analysis of the structure´s response during the earthquake was conducted in the software package SAP2000. The program is suitable for modelling 1D, 2D and 3D problems, including material and geometric nonlinearities, and analysing dynamically loaded structures using direct integration of equations of motion in the time domain.
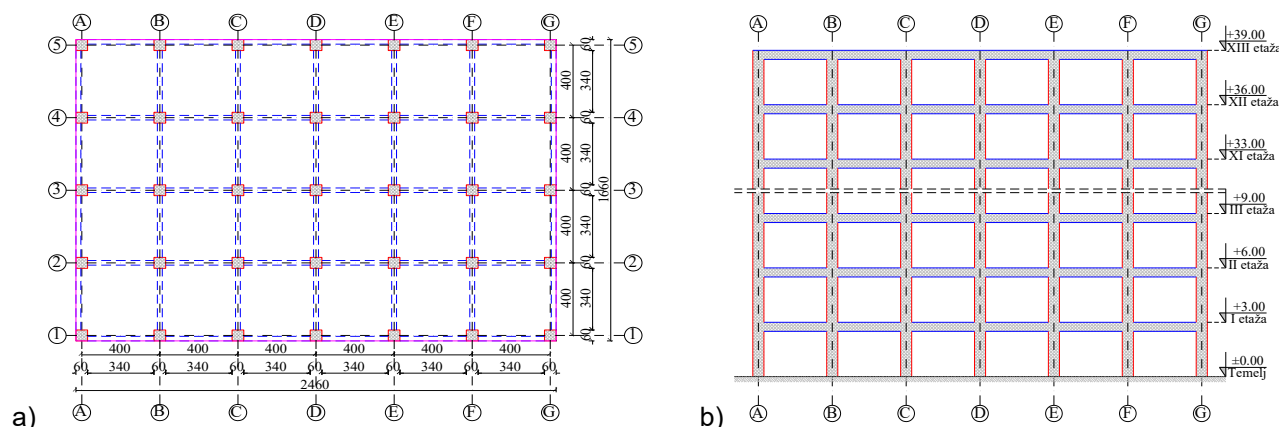

Figure 2. Disposition of the analysed structure; a) floor plan, b) longitudinal section

3.1 Finite element model

The geometry of the model is done in accordance with the geometry of the building (Figure 2). The columns and beams are modelled by 1D beam finite elements (FE), while floor structures are modelled with shell elements, where the appropriate geometric characteristics are defined. The length of the 1D finite elements is 1 m, while the floor structures are meshed with square finite elements with an edge length of 1 m. Relatively large dimensions of finite elements are adopted because of the complexity of the nonlinear calculation of the dynamic response of the structure by the method of direct integration of equations of motion. It did not significantly affect the accuracy of the results because the floor structures act as rigid diaphragms. The geometry of a numerical model with a finite element mesh is shown in Figure 3.
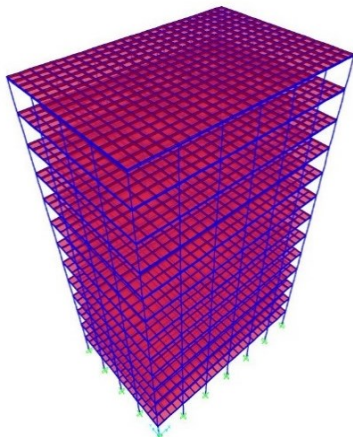

Figure 3. Geometry and FE mesh of the FEM model

Concrete is modelled as an isotropic material, whereby the modulus of elasticity is 31 GPa and the Poisson's ratio is 0.20. The Mander's stress-strain curve is adopted in order to describe the nonlinear behaviour of concrete, where the strain corresponding to the compressive strength of concrete is 2 ‰ and the ultimate strain is 5 ‰. Upon reaching the compressive strength, the material softens, and the slope of the stress-strain function is equal to 10 % of the modulus of elasticity (Figure 4a). In order to describe the hysteretic behavior of the concrete, the Pivot model is adopted, which is recommended by software documentation and other researchers [23, 24]. The Pivot hysteresis model is defined by five parameters, which determine the reduction of stiffness due to the cyclic load. The parameters $\alpha_1 = \alpha_2 = 1$, $\beta_1 = \beta_2 = 0.30$, $\eta = 10$ are adopted in the paper.

The stress-strain relationship (Figure 4b) is defined in order to describe the nonlinear behaviour of reinforced steel. The modulus of elasticity is 200 GPa and the yield strength is 500 MPa. After yielding, material hardening occurs up to the ultimate strain of 20 ‰, after which the material fails. In order to model the hysteretic behaviour of reinforcement under cyclic load, a kinematic model suitable for modelling ductile materials is adopted [23].

The characteristics of developing plastic hinges at the ends of beams and columns are defined with the purpose of covering the damage to the structure during an earthquake. Beams are dominantly loaded to bend about a horizontal central axis. Regarding that, the plastic hinges of beams are defined based on the bending moment. The moment-curvature relation in post-yielding behaviour depends on the cross-sectional dimensions, reinforcement ratio, and shear force. The yield curvature and post yield moment-curvature relation for beams are calculated based on the recommendations of FEMA-356 table 6-7 [25] for the adopted dimensions of cross-section and reinforcement of beams. Columns are dominantly loaded by axial force and bend about both horizontal central axes. Yield moment of columns depends on the intensity of axial force. The yield curvature and post yield moment-curvature relation for columns with interaction of axial force and bi-axial bending moments are defined in FEMA-356 table 6-8 [25]. Plastic deformations of column hinges are calculated according to these recommendations and adopted cross-sectional dimensions and reinforcement of columns.

In the case of the rigidly founded structure, the boundary conditions on the columns at ground-level are defined to prevent all translations and rotations. In the case of a base isolated structure, the modelling of seismic isolators is done using link elements with the definition of appropriate mechanical characteristics in three orthogonal directions. The isolator has linear elastic characteristics in the vertical direction and nonlinear bilinear behavior in the horizontal directions, where the corresponding characteristics are defined in accordance with the mechanical characteristics of the selected LRB (Section 2).
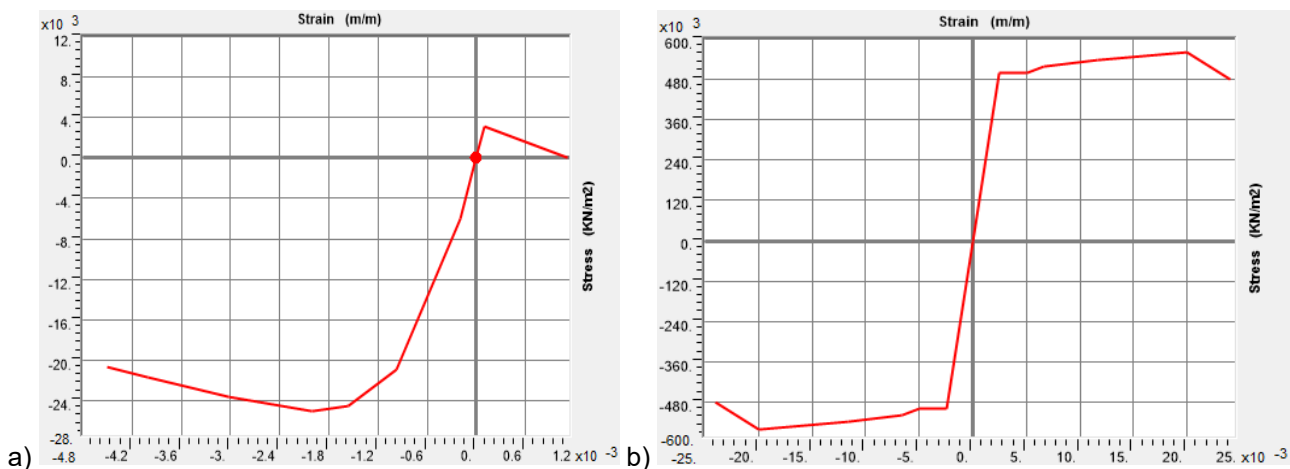

Figure 4. Stress-strain diagrams: a) concrete; b) reinforcement

In the dynamic analysis of models, the mass of the structure was defined as the sum of the dead and live load and the self-weight of the structure. A nonlinear static analysis of the structure for the combination of dead and live load is conducted. The nonlinear analysis includes material nonlinearity, while the P-Δ method models the geometric nonlinearity. P-Δ procedure is adequate in analyses in which the vertical load does not vary significantly. The stiffness matrix in the P-Δ procedure is constant during the calculation. This is considered as an advantage over the calculation methods that include large displacements where the stiffness matrix is calculated in each iteration step, because it requires less time for calculation.

The dynamic response of the structure to the action of the north-south component of the Imperial Valley earthquake is analysed. In the analysis, the earthquake load is defined as the acceleration of the supports in the x direction of the global coordinate system, using the earthquake accelerogram. The values of ground acceleration are defined in equal time intervals of 0.02 s, and the duration of the earthquake is 53.74 s (Figure 5).

The seismic response of the structure is calculated using direct integration of equations of motion in the time domain, which includes material and geometric nonlinearity via the P-Δ method. The dynamic calculation is conducted after the nonlinear static calculation for the combination of dead and live loads.

3.2 Model analysis parameters

Nonlinear dynamic analysis is conducted through 2687 sub-steps, with a time increment of 0.02 s, which corresponds to the discrete values of applied accelerogram of the Imperial Valley earthquake. The calculation includes the damping of 5 % defined by the Rayleigh model. The integration of the dynamic equations is performed by the implicit Hilber-Hughes-Taylor method, where the integration parameters are $\alpha = 0$, $\beta = 0.25$ and $\gamma = 0.5$.

The values of the natural periods of vibration, shear forces at the base, displacements of the top level of the building and relative inter storey drifts are considered in the comparative analysis of the dynamic responses of the structure with and without LRB. Also, based on the plastic hinges propagation, a conclusion about the degree of structural damage is made. The economic aspect of the application of the seismic isolators is shown based on the required area of reinforcement in the columns in the case with and without LRB, designed for the action of dead and live load and the action of the Imperial Valley earthquake.

3.3 The results of the analysis and discussions

The basic dynamic parameter of each structure is the natural period of vibration. Therefore, it is set as a starting point for comparison of the structural response with and without LRB. Figure 6 and Figure 7 show the first three natural modes with the values of natural periods for a rigidly founded structure and a base isolated structure, respectively. It is noticed that with the application of LRB there is an increase in the natural period of vibration of the structure, i.e., to frequency reduction, which is one of the goals of the application of the seismic isolators. The increase in the period of oscillation occurs due to the deformability of the seismic isolators in the horizontal direction, so, unlike the rigidly supported structure, there are displacements of the supporting nodes. In comparison to the structure without seismic isolators, the natural periods of vibration of the structure with LRB are increased by 45 %. As the natural period increases, the acceleration of masses decreases along with the intensity of inertial forces during the earthquake.

The change in base shear force as a function of time is shown in Figure 8. With the application of LRB, the base shear force is reduced by about 50 %. In addition to the significant reduction in shear force due to the earthquake in the case of the base isolated structure, it should be noted that the change in shear force over time is more uniform than in the case of the rigidly founded structure. In the latter case, a sudden change in the ground acceleration is followed by the sudden oscillations of the shear force. The maximum value of the base shear force with and without LRB does not occur at the same time (Figure 8), which is a consequence of the longer natural period of the structure with LRB.
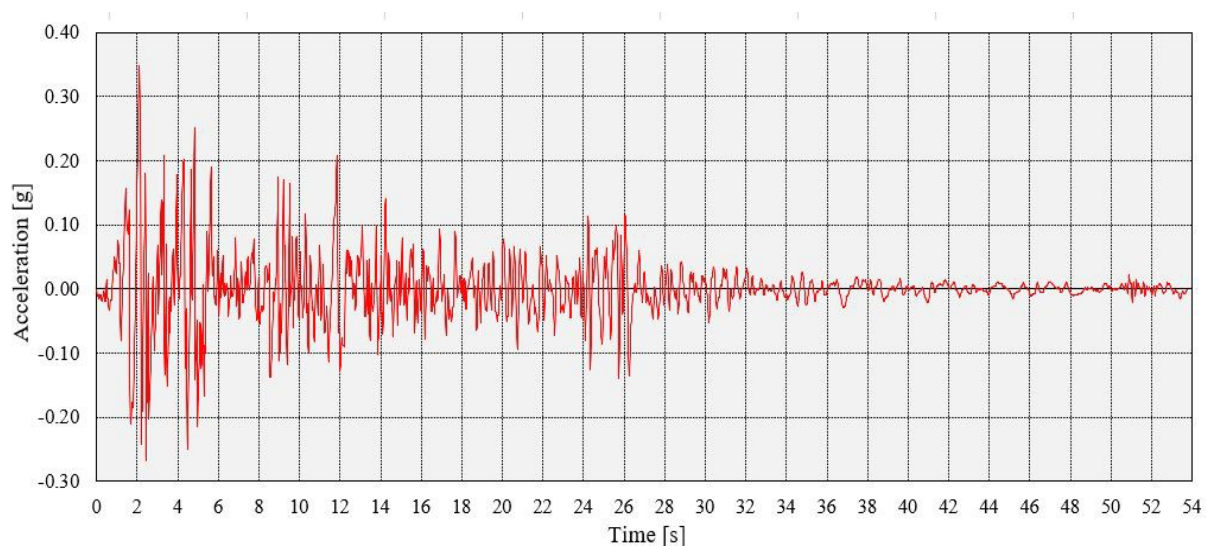

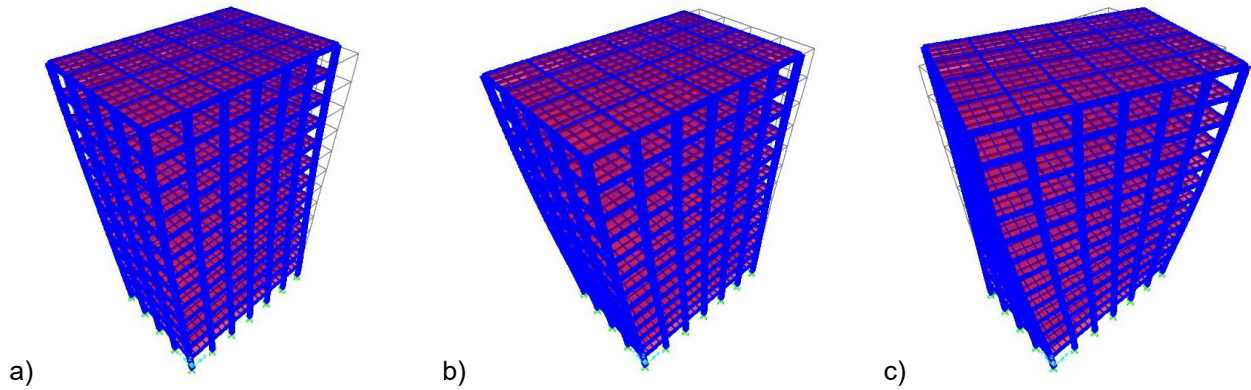*Figure 5. Accelerogram of Imperial Valley earthquake, component north-south*

Figure 6. The first three mode shapes and natural periods of the rigidly founded structure:
a) I period - y direction, $T_1 = 1.395$ s; b) II period - x direction, $T_2 = 1.343$ s; c) III period - torsion, $T_3 = 1.203$ s
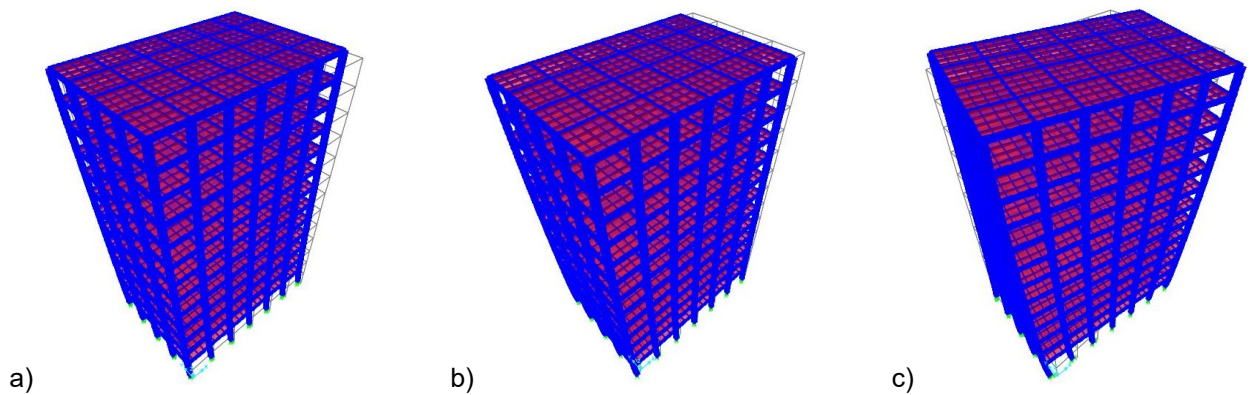

Figure 7. The first three mode shapes and natural periods of the base isolated structure:
a) I period - y direction, $T_1 = 2.044$ s; b) II period - x direction, $T_2 = 1.981$ s; c) III period - torsion, $T_3 = 1.759$ s
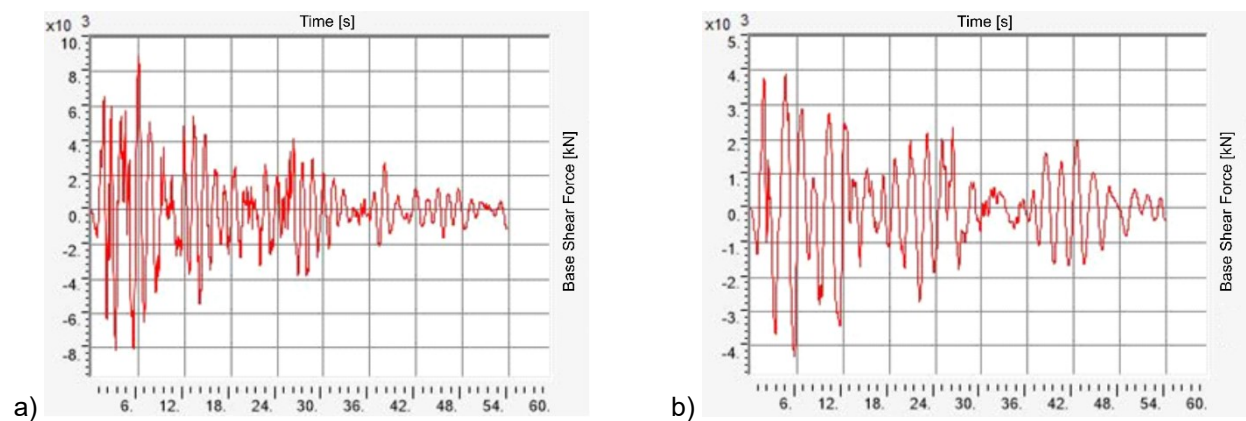

Figure 8. Base shear force vs. time, x direction: a) Rigidly founded structure, max 8909 kN at t = 6.00 s;
b) Base isolated structure, max 4303 kN at t = 5.68 s

In Figure 9, the displacement of the top level of the structure with and without LRB as a function of time is presented. The displacement of the top level in the case with LRB is higher by about 35 % compared to the case without seismic isolators. Figure 10 shows the maximum horizontal displacements of the floors of the analysed models. Although the absolute displacement of the top of the structure with LRB is larger than in the case without isolators, the fact that the supports displace too should be pointed out. The relative displacement of the top level of the structure with LRB in relation to the supports is 0.0663 m, which is about 35% less than the displacement of the top level of the rigidly founded structure.
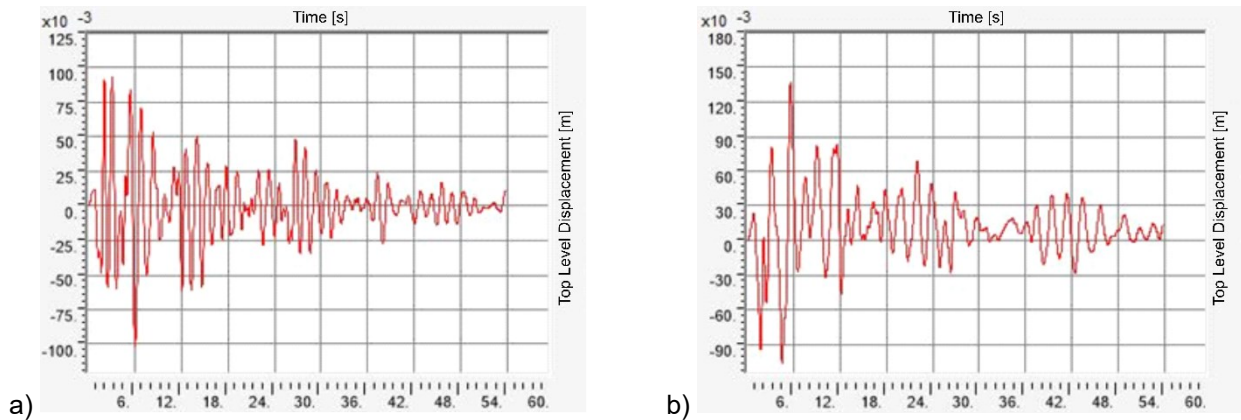
Figure 9. Horizontal displacement of top level of structure vs. time, x direction: a) Rigidly founded structure, max 0.1016 m at t = 6.04 s; b) Base isolated structure, max 0.1373 m at t = 5.60 s
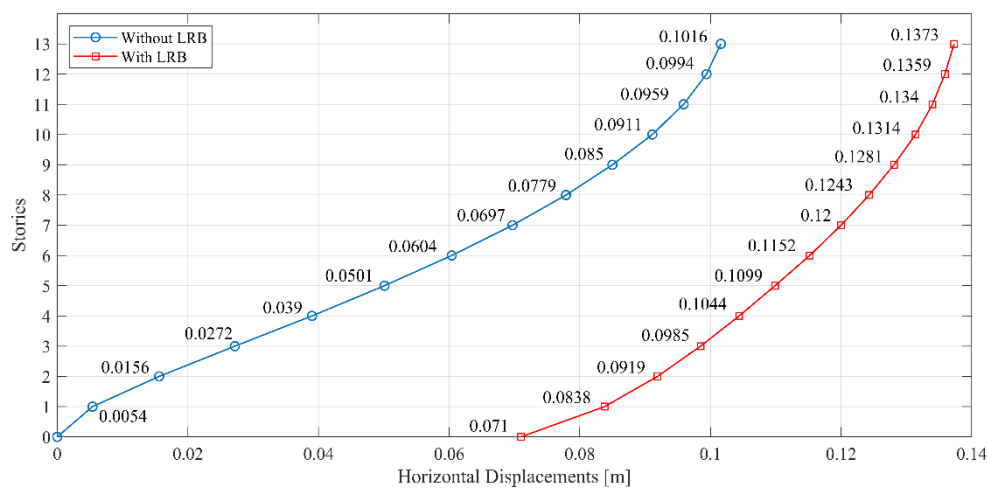

Figure 10. Maximum horizontal displacements of the floors in the earthquake directi

An important indicator of the response of the structure to the action of the earthquake is the relative interstorey drift. It is defined as the quotient of the divergence between the displacement of two adjacent floors and the storey height. Figure 11 shows the relative interstorey drifts of the structure with and without LRB when the maximum displacement of the top level of the building is reached. The relative interstorey drifts in the case of the structure with LRB are smaller in relation to the case of rigidly supported structure for the 2nd floor and higher, while in the 1st floor it is larger.
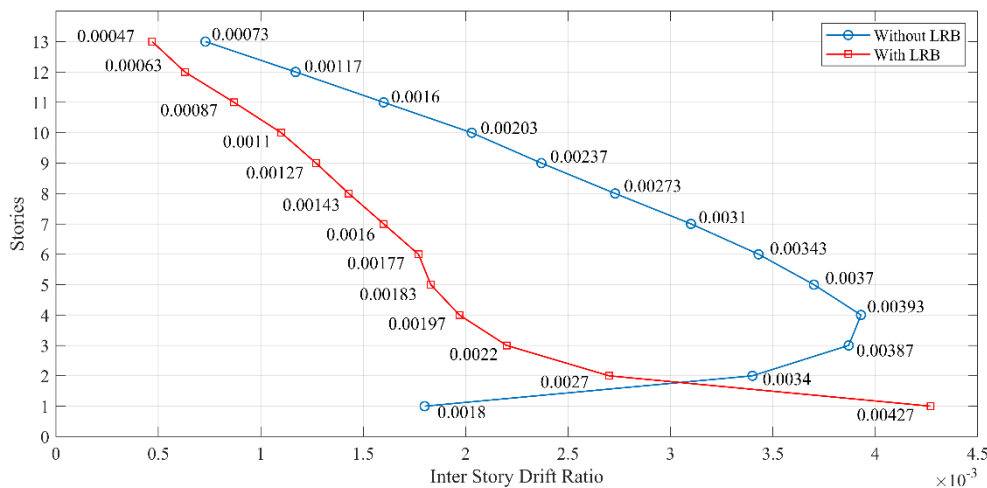

Figure 11. Interstorey drifts in the moment of maximum displacement of the top level of structure
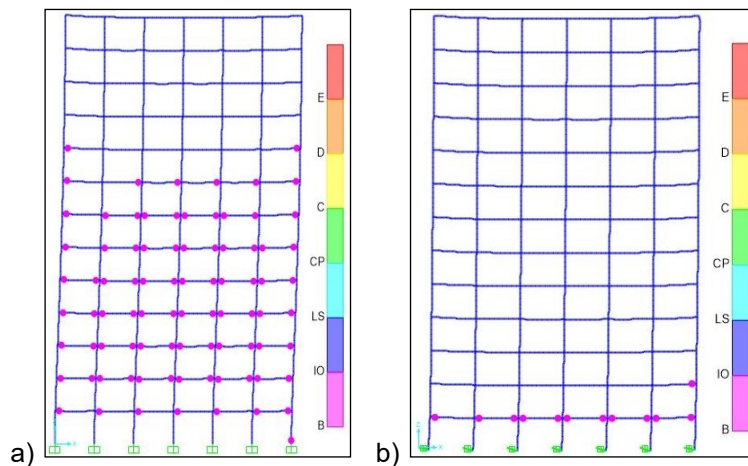
Figure 12. Plastic hinges in frame in the axis 3-3; a) rigidly founded structure; b) base isolated structure

According to the analysis of plastic deformations of the beam and column joints, it can be observed that the beam joint plastification occurs both in the model without and with LRB (Figure 12). In the case of the building without LRB the ends of all beams up to the 8th floor are plastificated, as well as the joints of individual columns, while in the case of a building with seismic isolators, the ends of all beams up to the first floor are plastificated. This is considered to be the consequence of lower seismic force in the case of the application of LRB, and of dissipation of the part of the seismic energy by plastic deformations of the lead core. The maximum plastic rotation in a model without LRB is 0.002559 rad, while by the application of LRB, maximum plastic rotation is decreased to 0.001318 rad. The application of lead rubber bearings significantly reduces the plastic deformations of the structure, which results in less damage to the structure during the earthquake compared to damage of the non-isolated structure.

The design of all columns for the effects of dead and live load and seismic action was conducted in accordance with Eurocodes 2 and 8 [26, 27]. The design of the columns was conducted in order to compare the required area of reinforcement in the columns in the model without and with LRB. In the case of the model without LRB, the maximum required area of reinforcement is 70,5 cm^2, while in the model with LRB it is 44,9 cm^2. The reduction of the seismic forces in the structure with seismic isolators is reflected in the reduction of the required area of reinforcement in the columns by 36%.

4 Conclusion

Based on the results of the numerical analyses conducted in this work, the following conclusions can be drawn:

• the application of the lead rubber bearings increases the natural periods of vibration of the analysed structure by approximately 45%,

• the application of lead rubber bearings significantly improves the dynamic response of the structure, which is reflected in a significant reduction of seismic forces, displacement of the top level of the structure and relative inter storey drifts,

• the application of lead rubber bearings reduces the development of plastic hinges in beams and columns compared to the rigidly founded structure, which is an especially important advantage because the occurrence of plastic hinges in columns can lead to the loss of bearing capacity and stability of the entire structure,

• lead rubber bearings contribute to the reduction of the influences in the structural elements, which results in the reduction of the required area of reinforcement in the RC columns.

Further research in this area should be focused on parametric analysis of the dynamic response of structures during earthquakes, varying the stiffness of the LRB. The structure analysed in this paper is regular in plan and elevation, so further research can be directed towards analysing the effects of LRB application in structures with irregularities.

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