Efficiency of Passive Control Devices in Rehabilitation of a Building in the Seismic Conditions of Romania

E. Tulei, D. Cretu, C. Ghindea & R. Cruciat
Technical University of Civil Engineering of Bucharest, Romania

ABSTRACT:
The possibility of implementing modern solutions for the seismic rehabilitation of buildings in Romania is analysed in the paper. The study is made on a 5 storey reinforced concrete framed structure located in Bucharest that needs seismic upgrading in accordance to the present Romanian codes. In order to draw conclusions regarding the influence of the site conditions on the structural response, the building is considered in different seismic zones from Romania, for which the response spectrum has the same PGA values, but different corner periods. The efficiency of two solutions based on passive control devices is analysed, namely base isolation with lead-rubber elastomeric bearings and in structure linear viscous dampers. The analyses results show that base isolation is the most efficient solution for ensuring the building seismic safety.

Keywords: passive control, viscous dampers, base isolation

1. EVALUATION OF THE EXISTING STRUCTURE SEISMIC SAFETY

The building with a 5 storey reinforced concrete framed structure was constructed in Bucharest, in the early seventies. The normalized elastic spectrum from the Romanian seismic code used at the designing of the building corresponded to Californian shallow earthquakes. The spectrum from the present seismic code corresponds to accelerations recorded at the site INCERC-Bucharest induced by earthquakes of medium depth from the source Vrancea. The severe earthquake from 1977 produced minor damages in the concrete structure, but the masonry partition walls were fractured. The analyses in accordance to the present Romanian codes show a high level of deformability of the structure and reduced resistance capacity of the structural members, which need strengthening interventions.

The analyses of the structure with passive control devices have been done by considering the building located in different seismic zones from Romania, for which the response spectrum has the same PGA values as in Bucharest, but different corner periods. Thus, conclusions regarding the influence of the site conditions on the structural response could be drawn. The effect of two types of passive control devices on the seismic behaviour of a building is analysed in the paper: base isolation devices and in structure viscous dampers.

The structure is regular in plan and elevation, with 9 longitudinal spans of 4.7 m and 3 transverse spans of 6.18, 3.4 and 7.18 m, respectively. The five storeys are 3.8 m high each. The columns have rectangular cross sections with different dimensions on the building's height. The slab is 12 cm thick on the lateral transverse spans and 10 cm thick on the middle span. Experimental investigations showed that the structure is made of concrete C12/15 with reinforcing steel Fe360. Longitudinal and transverse cross sections through the structure are shown in Fig. 1.1. Planar analyses of the transverse frame have been performed in order to evaluate the seismic safety of the building. The characteristic vertical loads in the combination of the seismic action are given by the dead loads and 40% of the live loads.

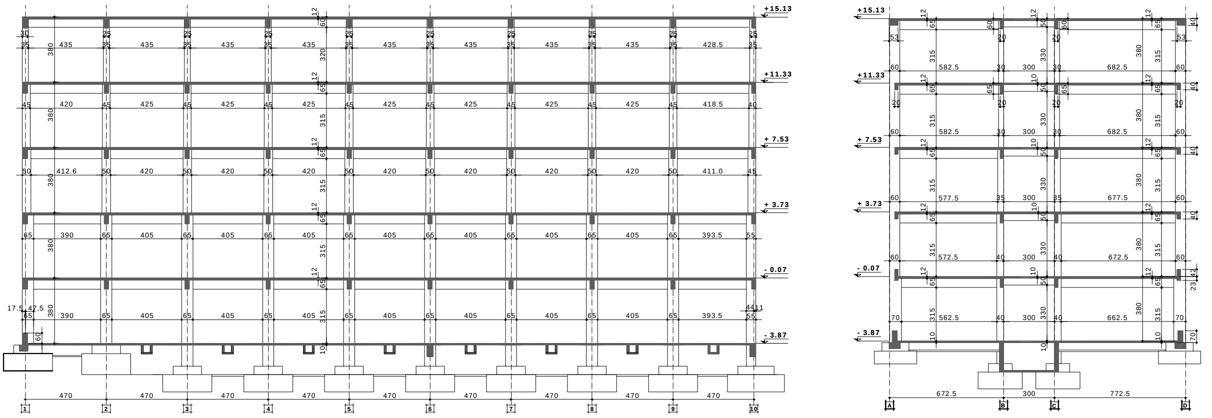


Figure 1.1. Longitudinal and transverse cross sections

Table 1.1. Modal characteristics of the existing structure

Mode n	T_n (s)	ε_n	m_n (t)
1	1.059	0.801	351.6
2	0.354	0.117	51.4
3	0.194	0.044	19.3

The natural periods, equivalence modal factors and effective modal masses calculated for the first three vibration modes are given in Table 1.1.

According to the present Romanian seismic code, P100-2006, the base shear force is $F_b = \gamma_I\, S_d(T_1)m\lambda$, where $S_d(T_I) = a_g\beta\,(T_1)/q$ for $T_B \leq T_1 \leq T_C$. For Bucharest, the design ground acceleration is $a_g = 0.24$g and for $T_1 = 1.059$ s, the amplification factor is $\beta(T_1) = 2.75$. The behaviour factor q corresponds to a framed structure with ductility class M and overstrength ratio $\alpha_u/\alpha_1 = 1.35$. Therefore, $q = 3.5\alpha_u/\alpha_1 = 4.725$. The vertical load resultant for a longitudinal span is 4308 kN, the live load representing about 10% from it. The building's class of importance is II, so that $\gamma_I = 1.2$. The base shear force is $F_b = 1.2 \times 0.24 \times (2.75/4.725) \times 4308 \times 0.85 \cong 614$ kN.

The structure safety factor can be determined by the ratio $r = F_{Rd,k}/F_{Ed,k}$, where $F_{Rd,k}$ is the design storey shear force associated to the resistance of the critical sections from the storey k and $F_{Ed,k}$ is the design storey shear force computed as for a new structure. If the critical sections are considered only at the column ends, $F_{Rd,k}$ can be calculated with Eqn. 1.1,

$$F_{Rd,k} = \sum_{j=1}^{nc} \left(\frac{M_{Rd,j}^{top} + M_{Rd,j}^{bot}}{h_k} \right) \tag{1.1}$$

where nc is the number of columns that resist the seismic forces at the storey k, and $M_{Rd,j}^{top}$ and $M_{Rd,j}^{bot}$ are the design flexural resistances at the top and bottom ends of the column j, respectively. $M_{Rd,j}^{top}$ and $M_{Rd,j}^{bot}$ are determined by taking into account the presence of axial force from vertical loads and the indirect effect of the overturning moment at the storey under consideration, of height h_k. The design flexural resistance and the shear force capacity are determined based on the design values of the concrete and reinforcing steel strengths, f_{cd} and f_{yd}. The value of the safety factor must be at least 1 at each storey. Values smaller than 0.6 indicate the necessity of strengthening interventions.

The results obtained for the loading case with the seismic action in the positive direction are given in Table 1.2. The resistance condition is fulfilled only by the columns from the top storey. Storey mechanisms can occur at the 2nd and 3rd storey, where the safety factor is smaller than 0.6.

Table 1.2. Safety factor and interstorey drift values

Storey k	$F_{Rd,k}$ (kN)	$F_{Ed,k}$ (kN)	$r = F_{Rd,k}/F_{Ed,k}$	$d_{s,k}$ (m)	$d_{r,e} = d_{s,k} - d_{s,k-1}$ (m)	$d_r^{SLS} = \nu q\, d_{r,e}$ (m)	$d_r^{ULS} = cqd_{r,e}$ (m)
1	454	614	0.740	0.00941	0.00941	0.0178	0.0598
2	313	568	0.551	0.0257	0.0163	0.0308	0.1083
3	281	479	0.587	0.0431	0.0174	0.0329	0.1108
4	227	348	0.652	0.0586	0.015	0.0293	0.0996
5	186	174	1.069	0.0683	0.0097	0.0183	0.0644

Table 1.2 also gives the elastic lateral displacements $d_{s,k}$, elastic relative displacements $d_{r,e}$ and the values of the interstorey drift d_r, calculated for the serviceability limit state and ultimate limit state. For the SLS, the interstorey drift has been calculated with the formula $d_r^{SLS} = \nu\, q\, d_{r,e}$, where $\nu = 0.4$. The interstorey drift limit, $d_{ra}^{SLS} = 0.004\, h = 0.004 \times 3.8 = 0.0152$ m is exceeded at all stories. For the ULS, $d_r^{ULS} = cqd_{r,e}$, where $c = 3 - 2.5\ T_1/T_C$, and $d_{ra}^{ULS} = 0.025\ h = 0.025 \times 3.8 = 0.095$ m. The interstorey drift limit is exceeded at the stories $2 \div 4$.

2. THE SEISMIC RESPONSE OF THE STRUCTURE WITH CONTROL DEVICES

The maximum base shear force that the structure can resist is $F_b^* = rF_b$, where r is the minimum value of the safety factor (see Table 1.2). This means that the elastic response accelerations should be reduced by $c_r = q/r = 4.725/0.551 = 8.6$. This reduction can be obtained either by increasing the fundamental period of the structure and placing it in the spectrum area with reduced accelerations, either by increasing the structure damping.

Two solutions of passive control are analysed in the paper: base isolation and viscous dampers. In the first solution, base isolation devices are placed under a raft foundation of 60 cm thickness and 1260 kN weight (Fig. 2.1, *a*). In the second solution, viscous dampers are introduced in the structure. They are attached to steel braces placed on the diagonal of the panels from the lateral spans of the building in each transverse frame (Fig. 2.1, *b*).

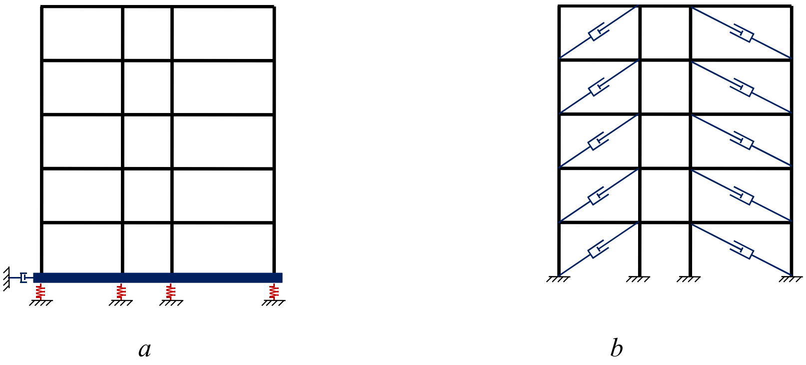


Figure 2.1. Passive control devices: (a) base isolation; (b) viscous dampers

In order to determine the efficiency of the passive control devices, three accelerograms have been used: the north-south component of the accelerogram recorded in Bucharest at the site INCERC, during the severe earthquake from 4th March 1977, with PGA = 0.211g, named INCERC 77; an artificial accelerogram compatible with the elastic response spectrum for Bucharest, with $a_g = 0.24$g and $T_C = 1.6$ s, named AccTc1.6; an artificial accelerogram compatible with the elastic response spectrum for different sites with the same a_g, but reduced corner period, $T_C = 1.0$ s (AccTc1.0). According to the seismic Romanian zonation maps, the city Galati corresponds to this case. The sites for the artificial accelerograms have been chosen such as the seismic base shear force for the horizontal transverse direction of the structure with fixed base is the same, that is $T_1 \leq T_C$ and $a_g = 0.24$g. The accelerograms considered in the analyses and the corresponding Fourier spectra are shown in Figs. 2.2.

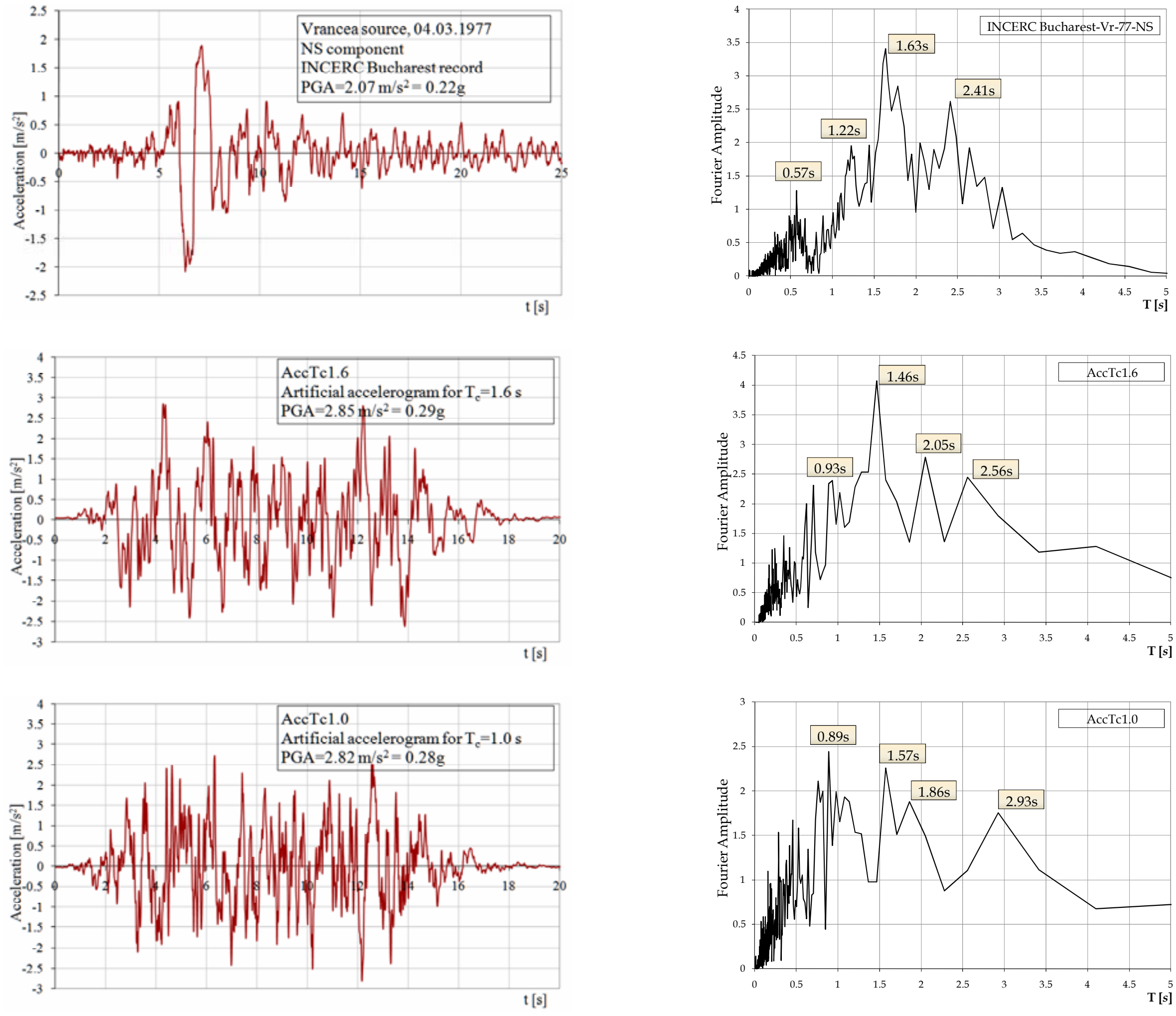


Figure 2.2. Recorded and artificial accelerograms and the corresponding Fourier spectra

2.1. Base isolation devices

The structural response has been determined for two devices with lead-rubber elastomeric bearings. The stiffness and dynamic characteristics of the devices are obtained in the hypothesis of a total isolation, meaning an elastic response of the structure. The necessary maximum absolute response acceleration is obtained from one of the relations given in Eqn. 2.1:

$$a_s = \frac{S_e(T)}{c_r} = \frac{a_g \beta(T)}{c_r} = \begin{cases} a_g \beta_0 \dfrac{T_C}{T} \text{ for } T_C < T \le T_D \\ a_g \beta_0 \dfrac{T_C T_D}{T^2} \text{ for } T_D < T \end{cases} \tag{2.1}$$

According to the normalized spectra for absolute horizontal accelerations, in Bucharest $T_C = 1.6$ s and $T_D = 2.0$ s, while in Galati $T_C = 1.0$ s and $T_D = 3.0$ s. If the second relation of Eqn. 2.1 is applied, the fundamental period of the structure with base isolation devices can be determined as $T_1 = \sqrt{c_r T_C T_D}$. This leads to $T_1 = \sqrt{8.6 \times 1.6 \times 2.0} = 5.25$ s for Bucharest and $T_1 = \sqrt{8.6 \times 1.0 \times 3.0} = 5.08$ s, both values being greater than T_D. For this area of the response spectra, the structural damping does not reduce significantly the absolute response accelerations. Considering the superstructure as a rigid body, T_1 determined from the Eqn. 2.1 is exactly the necessary period of the base isolation device, T_{bi}. The close values of the fundamental period obtained for Bucharest and Galati indicate that the same devices have to be used for different sites with the same a_g.

The device horizontal stiffness can be determined with the first formula from Eqn. 2.2 and the device damping constant is given by the second formula from Eqn. 2.2:

$$K_{bi} = \frac{1}{n_{bi}} \omega_{bi}^2 M = \left(\frac{2\pi}{T_{bi}} \right)^2 \frac{M}{n_{bi}}, \quad C_{bi} = \frac{2\omega_{bi}\xi_{bi}M}{n_{bi}} \tag{2.2}$$

where M is the total mass of the structure and raft foundation, ω_{bi} is the circular frequency of the base isolation device, n_{bi} is the number of devices corresponding to the transverse frame, and ξ_{bi} is the device fraction of critical damping.

The target displacements for the base isolation devices are given by Eqn. 2.3:

$$d_{\max} = \gamma_I \frac{S_e(T_{bi})}{\omega_{bi}^2} \eta = \begin{cases} \gamma_I \beta_0 \frac{T_C T_D}{4\pi^2} a_g \eta \text{ for } T_{bi} \geq T_D, \text{ where } S_D\ (T) = const. \\ \gamma_I \beta_0 \frac{T_C T_{bi}}{4\pi^2} a_g \eta \text{ for } T_C \leq T_{bi} \leq T_D \end{cases} \tag{2.3}$$

where $\eta = \sqrt{10/(5+\xi)}$ is the correction factor applied to the response spectrum for fractions of critical damping different from 5%.

The total mass $M = M_s + M_{bi} = 439.3 + 126 = 563.3$ t is supported by 4 devices with the fraction of critical damping $\xi_{bi} = 10\%$, for which the correction factor η is 0.817. The elastic and dynamic characteristics of the base isolation devices and the corresponding target displacements are given in the Table 2.1.

Table 2.1. Characteristics and target displacements of the base isolation devices

Device	T_{bi} (s)	K_{bi} (kN/m)	C_{bi} (kNs/m)	$d_{\max}$ (m)
IS3.0	3.0	620.0	59.2	0.515
IS4.8	4.8	242.2	37.0	0.515
IS2.0	2.0	1394.8	88.8	0.322

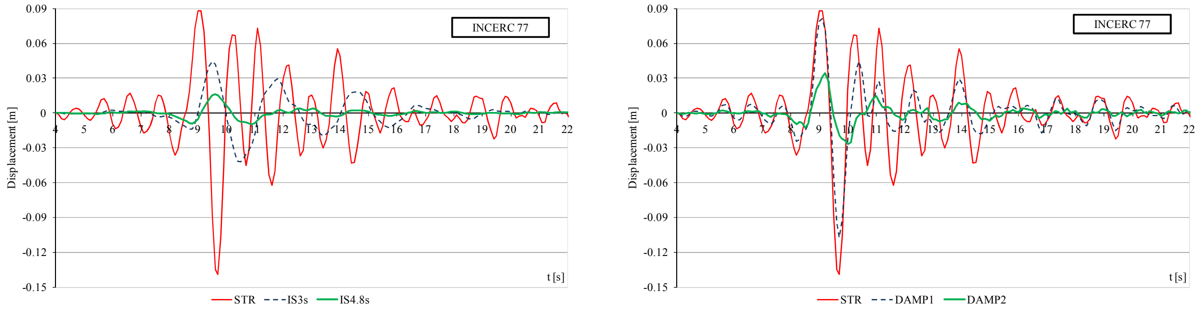
2.2. In structure viscous dampers

The force developed in a viscous damper is $F_d = c\,|\mathring{u}|^{\alpha}\,\mathrm{sgn}(\mathring{u})$. For linear viscous dampers, the force is proportional with the deformation velocity ($\alpha = 1$). The structure behaviour has been analysed for two linear viscous dampers that are attached to steel braces placed on the transverse lateral spans of the building. The damping constant of the devices is $C_{vd} = 10$ kNs/cm for the viscous damper named DAMP1 and $C_{vd} = 100$ kNs/cm for the viscous damper named DAMP2. The stiffness of the steel braces is $k_1 = 3240$ kN/cm in the long lateral span of the transverse frame and $k_2 = 2900$ kN/cm in the short lateral span.

2.3. Analysis results

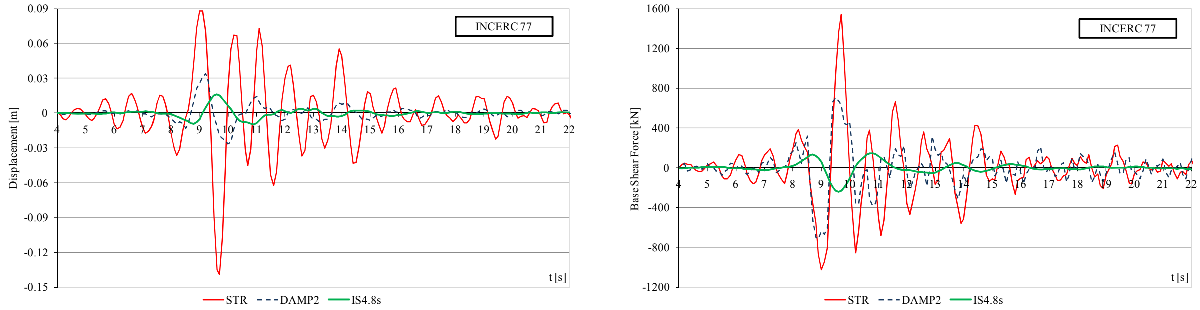
Time history analyses of the existing structure and of the structure with base isolation devices and linear viscous dampers have been performed, for each site and accelerogram, respectively. The effect of the passive control devices on the structural behaviour has been evaluated by considering the following response parameters: the roof lateral displacements and the base shear force. The analysis results showed that the base isolation device IS4.8 and the viscous damper of type DAMP2 are the most efficient in reducing the lateral displacements and the base shear force.

In the following graphical representations, the roof lateral displacements do not include the displacement of rigid body corresponding to the ground movement and the base isolation device deflection.



a *b*

Figure 2.3. Roof displacement for Bucharest, recorded accelerogram



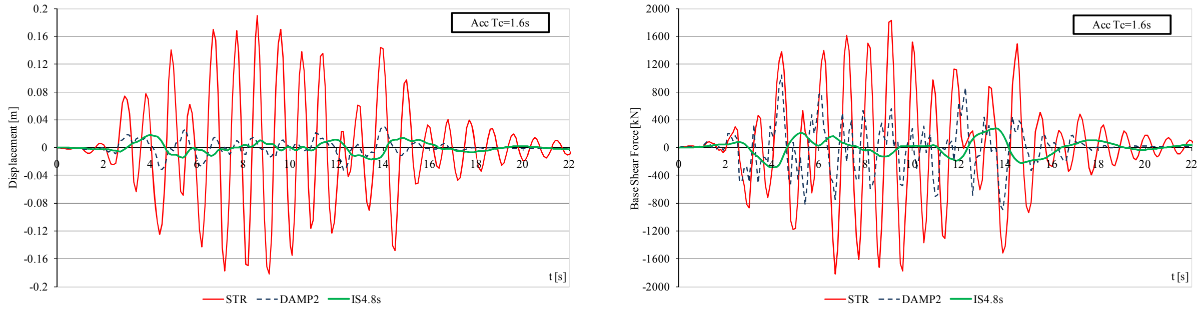
a *b*

Figure 2.4. Roof displacement (a) and base shear force (b) for Bucharest, recorded accelerogram

The time variation of the roof displacements for the structure placed in Bucharest and subjected to the recorded accelerogram INCERC 77 is shown in Fig. 2.3, *a* for the initial structure (STR) and the base isolated structure with the devices IS3.0 and IS4.8, and in Fig. 2.3, *b* for the initial structure and the structure with viscous dampers DAMP1 and DAMP2. The time variation of the roof displacements and base shear force is shown in Fig. 2.4, *a* and *b*, respectively, for the initial structure, the structure isolated with the devices IS4.8 and the structure with viscous dampers of type DAMP2, in the same site and for the same accelerogram.

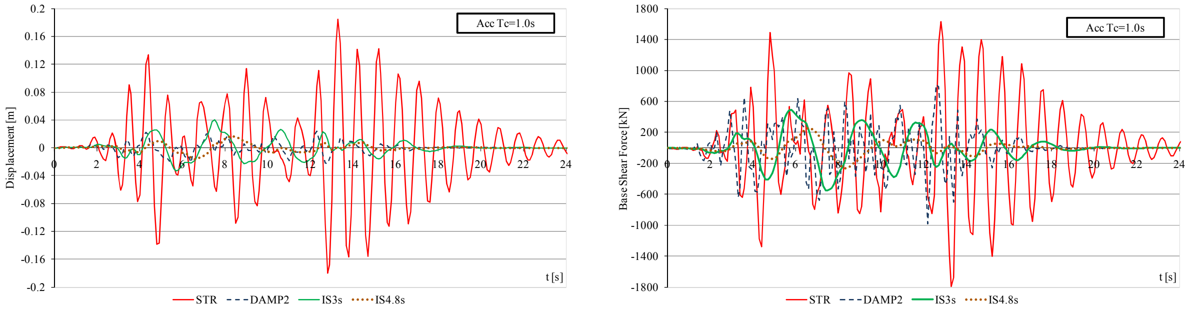
For the structure placed in Bucharest and subjected to the artificial accelerogram ACCTc1.6, Fig. 2.5, *a* and *b* show the time variation of the roof displacements and base shear force, respectively, for the initial structure, the structure isolated with the devices IS4.8 and the structure with viscous dampers of type DAMP2. The same results are shown in Fig. 2.6, *a* and *b* for the structure placed in Galati and subjected to the artificial accelerogram ACCTc1.0.

The histograms of the maximum values of the roof lateral displacement and base shear force from Fig. 2.7, *a* and *b* show that the base isolation system is the most efficient passive control device.



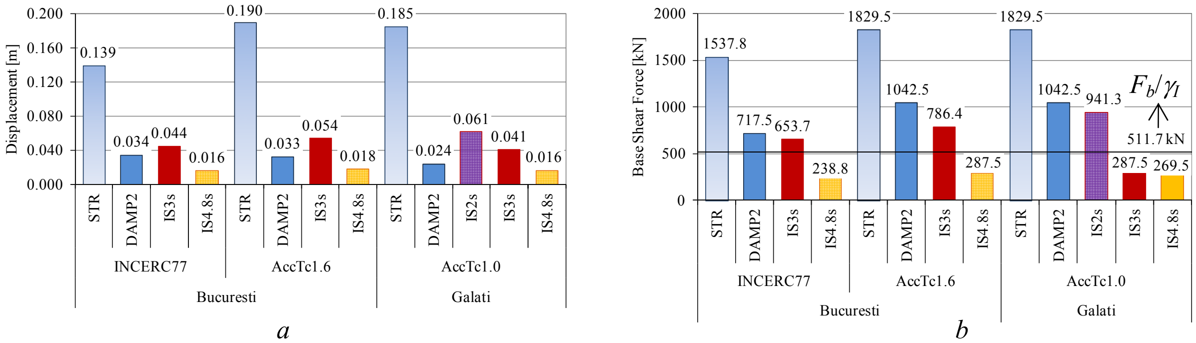
a *b*

Figure 2.5. Roof displacement (a) and base shear force (b) for Bucharest, first artificial accelerogram



a *b*

Figure 2.6. Roof displacement (a) and base shear force (b) for Galati, second artificial accelerogram

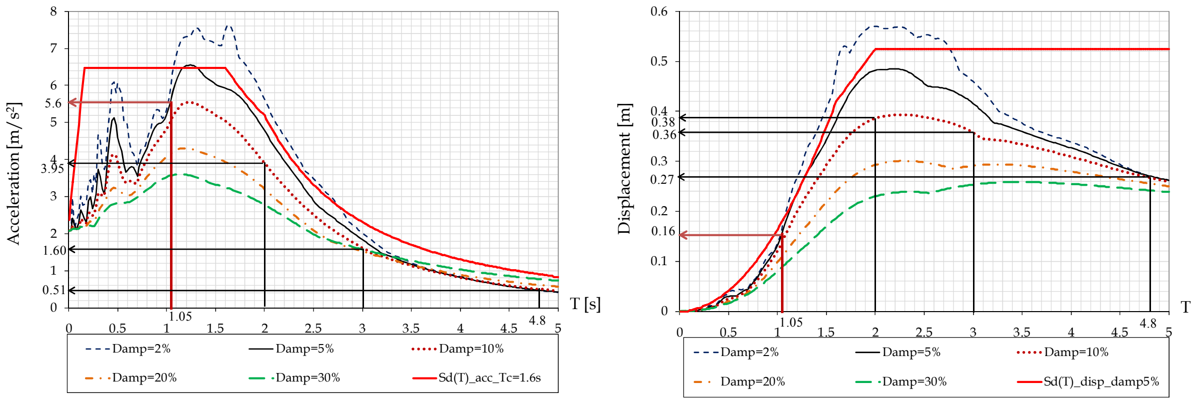


a *b*

Figure 2.7. Maximum values of the roof lateral displacement (a) and base shear force (b)

For the structure placed in Bucharest and acted by the recorded accelerogram INCERC 77, the base isolation device IS4.8 reduces the base shear force with 84.5 % with respect to the maximum base shear force of the initial structure (with fixed base and considered perfectly elastic) and with 46.7% with respect to the design base shear force. The maximum roof displacements are reduced with 88.4% with respect to the maximum roof displacements obtained by linear dynamic analysis and with 76.6% with respect to the roof displacement obtained by linear static analysis ($d_{s,\max} = 0.0683$ m in Table 1.2). The isolation obtained with the device IS4.8 is total, the response of the structure being in the elastic domain. When the base isolation device IS3.0 is used, the maximum base shear force is reduced with 57.5% with respect to the initial elastic structure, but the reduced value is greater than the design base shear force. In this case, the necessary behaviour factor can be estimated as $q = F_{bi}/(rF_b) = 653.7/(511.7 \text{ x } 0.551) = 2.32$. When the viscous damper DAMP2 is used, a reduction of the elastic base shear force with only 53.3% is obtained and the necessary behaviour factor can be estimated as $q = 717.5/(511.7 \text{ x } 0.551) = 2.54 < 4.725$. The inequality shows that the structure can ensure this behaviour factor, but inelastic deformations occur at the design earthquake. The viscous damper DAMP2 reduces the maximum roof displacements with about 75% with respect to the maximum roof displacements obtained by linear dynamic analysis and with 50% with respect to the roof displacement obtained by linear static analysis. The numerical results for the linear viscous dampers are in concordance with the experimental results found in the technical literature.

The elastic spectra of the absolute accelerations and relative displacements are given in Fig. 2.8, for the recorded accelerogram INCERC 77 and fractions of critical damping of 2, 5, 10, 20 and 30%, respectively. An increase of the fraction of critical damping from 5% to 30% can produce the decrease of absolute accelerations and relative displacements with about 50% (see Fig. 2.8, b). That is why viscous dampers are more efficient in limiting the interstorey drift, while base isolation devices are more efficient in reducing the base shear force. By shifting the fundamental period of the structure in the spectrum area of relative constant displacements, $T_1 \geq T_D$, the base isolation system can reduce the base shear force with 84-86%. The isolation of the structure is thus almost totally ensured.



a *b*

Figure 2.8. Absolute acceleration spectra (a) and relative displacement spectra (b) for INCERC 77 accelerogram and different damping ratios

3. CONCLUSIONS

In order to obtain significant reductions of the response accelerations and inertia forces, the elastic and damping characteristics of the base isolation devices must be chosen such as the structural response is as much as possible moved from the area of resonance with the ground predominant oscillations. The Fourier spectra for Romanian soils with deep sedimentary deposits and with the bearing stratum at great depths indicate that the ground predominant period can be higher than T_C (see Figs. 2.2). For this reason, the authors consider that a better isolation of the structure is obtained for $T_{bi} \geq 3T_C$. In order to obtain an elastic dynamic structural response, the elastic seismic forces must be reduced at least by the coefficient $c_r = q/r$, where q is the behaviour factor and r is the seismic safety factor (for new structures, $r = 1$). This conclusion is verified through the results shown in Fig. 2.7. If stiffer devices are used, with $T_{bi} \cong 2T_C$, the structure will undergo moderate inelastic deformations. Since the base isolation devices have great lateral displacements, their choice in order to reduce the base shear force depends on the existence of neighbouring buildings. For $T_{bi} = 3T_C$, base isolation devices with lead-rubber elastomeric bearings have maximum lateral displacements of about 30 cm, while for $T_{bi} = 2T_C$, the maximum lateral displacement is around 40 cm (Fig. 2.8, *b*). A fraction of critical damping of 20% is recommended for the base isolation devices, in order to place the structural response in the area of the spectrum with constant relative displacements for $T_{bi} \geq T_D$. The use of in structure viscous dampers eliminates the risk of great lateral displacements. Viscous dampers significantly reduce the relative lateral displacements, but lead to lesser reductions of the base shear forces, which implies some incursions of the existing structure in the post elastic domain. Supplementary strengthening measures or other types of passive control devices might be necessary if the resistance capacity of the structural members is exceeded. The artificial accelerograms prove to be too restrictive, as a natural consequence of their compatibility with the design spectrum, which is an envelope of the actual response spectra.

REFERENCES

Soong T.T. and Dargush G.F. (1997). Passive Energy Dissipation Systems in Structural Engineering, John Willey&Sons

Bozorgnia Y. and Bertero V. (2004), Earthquake Engineering: from Engineering Seismology to Performance-Based Engineering, CRC Press LLC

Chopra A. K. (2007), Dynamics of Structures, Theory and Applications to Earthquake Engineering, Pearson Prentice Hall International

*** (2006) P 100-1/2006. Romanian Seismic Design Code - Part I: Design Provisions for Buildings

*** (1993) Isolation, Energy Dissipation and Control of Vibrations of Structures. *Proceedings of the Post-SMIRT Conference Seminar*, Capri, Italy