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Seismic evaluation and retrofitting of reinforced concrete buildings with base isolation systems

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Abstract. A parametric study on the nonlinear seismic response of isolated reinforced concrete structural frame is presented. Three prototype frames designed according to the 1954 Hellenic seismic code, with number of floor ranging from 1 to 3 were considered. These low rise frames are representative of many existing reinforced concrete buildings in Greece. The efficacy of the implementation of both lead rubber bearings (LRB) and friction pendulum isolators (FPI) base isolation systems were examined. The selection of the isolation devices was made according to the ratio T_{is}/T_{fb}, where Tis is the period of the base isolation system and T_{bf} is the period of the fixed-base building. The main purpose of this comprehensive study is to investigate the effect of the isolation system period on the seismic response of inadequately designed low rise buildings. Thus, the implementation of isolation systems which correspond to the ratio T_{is}/T_{fb} that values from 3 to 5 is studied. Nonlinear time history analyses were performed to investigate the response of the isolated structures using a set of three natural seismic ground motions. The evaluation of each retrofitting case was made in terms of storey drift and storey shear force while in view of serviceability it was made in terms of storey acceleration. Finally, the maximum developed displacements and the residual displacements of the isolation systems are presented.

Keywords: reinforced concrete frame assessment; base isolation; nonlinear analysis; pushover analysis

1. Introduction

Reinforced concrete buildings constitute a significant number of structures all over the world. The majority of these were designed according to old seismic codes. During the earthquakes that have occurred up till now, significant damage has been reportedin these buildings. In the period referred to, the design philosophy was based on allowable stress design while mainly considering gravity loads, without adequate provision for seismic detailing (Thermou and Pantazopoulou 2011, Kunnath *et al*. 1995). Their main weaknesses are shortage of ductility, low strength, lack of damage hierarchyand small lateral stiffness. Owing to these factors the damage was caused in the columns of the structure and it was brittle, which render the earthquake response of these structures undesirable. Modern seismic codes suggest alternative strategies and methods for theretrofitting of existing buildings through which they aim to ensure an efficient energy

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absorption mechanism. Usually, traditional seismic retrofitting methods like concrete jacketingare applied in order to increase the cross sections' strength and the available ductility. There has been an examination of the upgrading of RC buildings with weak open ground stories by installing steel braces restricted to the open ground stories by Antonopoulos and Anagnostopoulos (2012),while Mistakidis *et al*. (2007) have investigated the implementation of low yield metal shear panels for the seismic upgrading of concrete structures.

Another option is to apply base isolation systems. Retrofitting an existing structure with base isolation devices is based on the isolation of the superstructure from the ground motion by reducing the seismic forces (Skinner *et al*. 1993, Naeim and Kelly 1999). With this technique structural damage can be minimized or even completely avoided. Storey displacements in the structure together with the accelerations will be reduced significantly, while the acceleration reduction protects the non-structural elements, the reduction in the storey displacements will allow both the structural and non-structural elements to survive the earthquake without any or with minimal damage. Parametric studies have been presented about the effectiveness of base isolation systems in reinforced concrete buildings (Providakis 2008, Cardone *et al*. 2013) and steel buildings (Varnava and Komodromos 2013). Throughout the world there are examples of application of base isolation systems for the retrofitting of existing historic buildings (Kelly 1998, Mokha *et al*. 1996). Also there are studies examining acceleration-sensitive contents in facilities that allocate museums, healthcare facilities and manufacturing facilities Konstantinidis and Makris 2006, Alhan and Gavin 2005).

The isolators are installed at a specific level (Kelly 2001). This level may be either the foundation or the ground floor. In existing buildings which have been designed according to old seismic codes it is not easy to avoid any damage in the structural elements of the superstructure, thus the fundamental period of the isolated building (T_{is}) may need to be between 5-6 s. These very long periods result in very large lateral displacements that are incompatible. Nevertheless, limited plastic deformation could be proposed and that will lead us to a shorter period of the isolated structure.

The behavior and the simulation of the most practical isolation systems is bilinear, however all design codes invariably ask the design engineer to work with avibration period that is the isolation system period. In view of this demand the concept of equivalentlinear parameters has become central in the analysis and mainly in design of seismic isolated structures and this has led to the wide acceptance of the effective period and the associated effective stiffness. The effectiveness of widely used methods to predict the modal characteristics of structures supported on bearing with linear but also with bilinear behavior, is studied by Kampas and Makris (2012).

While for the case of sphericalsliding bearings, the concept of the effective period, is abandoned, and the period of the isolation system, is derived from the second slope of the bilinear system, $T_2 = 2\pi\sqrt{G/g}$, for other isolation systems the quantities of effective period and effective stiffness is used for estimating through an iterativeprocedure peak inelastic displacements and the associated peak shear forcesaccording to most current design codes. (AASHTO 1991, FEMA 1998, Eurocode 2009) Nevertheless, recent studies (Makris and Kampas 2013) concluded that the period associated with the second slope of bilinear isolation system is a better approximation regardless the dimensionless strength $Q/(K_d \cdot D_y)=1/\alpha-1$, of the isolation system.

In this paper the results of a comprehensive study on the seismic response of underdesignedlow rise reinforced concrete buildings with seismic isolation are presented. A total of 54 non-linear time history analyses have been made to evaluate the effect of these systems. Specifically, three reinforced concrete building prototypes were analyzed using a set of three seismic ground motions.

These buildings were designed according to the Hellenic seismic code that was in operation until 1985 (RD 1959). Following the assessment of the structures with both static non-linear and dynamic non-linear methods, the seismic response of 6 different seismic isolation implemented systems have been studied using the same set of accelerograms. Both LRB and FPI isolation systems are examined while different values of isolator stiffness considered in order to investigate how the isolation system period affects the seismic response of the examined buildings. The selection of the isolation devices was made according to the ratio T_{is}/T_{fb} which ranges from 3 to 5. The period associated with the second slope of the bilinear isolation system T_2 was considered as the period of the isolation system T_{is}.

2. Isolation system properties

In this study we examined two types of isolation systems, lead rubber bearings (LRB) and friction pendulum isolators (FPI) (Kelly 1999). A significant amount of both past and recent research has focused on the use of elastomeric and friction bearings (Kelly 1997, Su *et al*. 1989, Constantinou *et al*. 1990). The isolation system is often represented as a simplifiedbilinear hysteretic representation. Fig. 1 depicts the idealized force-deformation relation. In that figure, Q symbolizes the characteristic strength, k_d the post-elastic stiffness and k_{ef} the effective stiffness. Furthermore F_y is the yield force and D_y is the yield displacement.

A smooth bidirectional hysteretic model, which is based on the Bouc-Wen model (Wen 1976, Park *et al*. 1986) shown in Fig. 2, is often used for the simulation of elastomeric bearings in horizontal shear. The need of using smooth bilinear instead of sharp bilinear models to simulate LRBs is highlighted by Mavronicola and Komodromos (2014). The characteristic values that are needed to determine anisolator are given in the following relations (Eqs. (1)-(3)). Post yielding stiffness depends on the elastomeric shear modulus, the thickness of the layers and on the isolator area. However, effective stiffness and effective damping ratio depends on characteristic strength, yield and design displacement.

$$K_d = \frac{GA_r}{t_r} \tag{1}$$

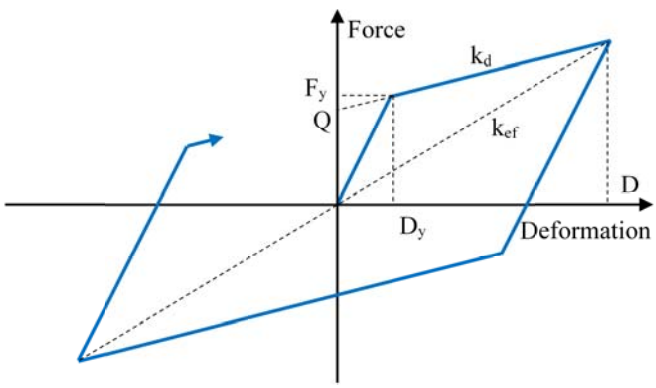

Fig. 1Bilinear force - deformation relation of an isolator

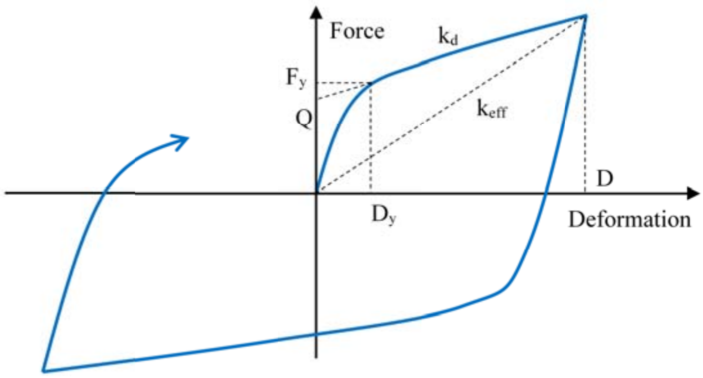

Fig. 2 Force - deformation relation of lead rubber bearings

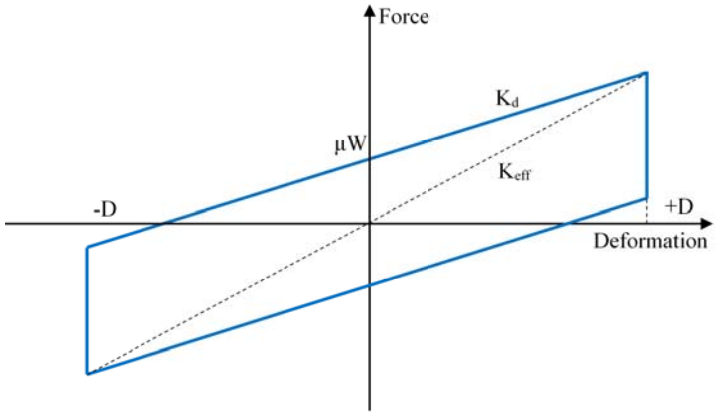

Fig. 3 Force - deformation relation of friction pendulum isolator

$$K_{eff} = \frac{Q}{D} + K_d \tag{2}$$

$$\beta_{eff} = \frac{4Q(D - D_y)}{2\pi K_{eff} D^2} \tag{3}$$

Fig. 3 shows FPI systems simulation whichis performed in accordance with the model recommended for base isolation analysis by Nagarajaiah *et al*. (1991). The determination of themajor parameters is given in the relations below (Eqs. (4)-(6)). The main featureswhich must be determined with accuracy are the coefficient factor μ and the radius of the curved surface R.

$$K_d = \frac{W}{R} \tag{4}$$

$$K_{eff} = \frac{W}{R} + \frac{\mu W}{D} \tag{5}$$

$$\beta_{eff} = \frac{2}{\pi} \frac{\mu}{\frac{D}{R} + \mu} \tag{6}$$

The selection of the alternative installed isolation systems has been calculated according to the ratio T_{is}/T_{fb}, where T_{fb} is the structure frame period with fixed base and T_{is} is the structure frame period with isolated base. The period associated with the second slope of the bilinear isolation system T_2 is considered as the isolation period T_{is}. T_2 is a better approximation of the "vibration period" than the effective period T_{eff} (Makris and Kampas 2013). The calculation of T_{is} is presented in the Eqs. (7)-(8), for LRB and FPI systems respectively. The effects of three ratios ranging approximately from 3 to 5 both for LRB and FPI systems were investigated. A total of 6 different cases for each frame have been examined and 54 analyses have been considered

$$T_{is} = 2\pi\sqrt{\frac{m}{K_d}} \tag{7}$$

$$T_{is} = 2\pi\sqrt{\frac{R}{g}} \tag{8}$$

3. Analyzed frames

A collection of three frames were studied. The studied frames are representative of low rise reinforced concrete buildings designed and constructed in Greece before 1985. The collection consists of a one storey with one opening frame, a two storey frame with two openings and a three storey frame with three openings. The corresponding 2-D representation of the structures is presented in Fig. 4. These frames were designed according to the Greek seismic code of 1959 (RD 1959) and their common characteristic is the strong beams in conjunction with weak columns

Table 1 Frames 1st mode period

Frame	Period (s)
1st Frame	0.262
2nd Frame	0.476
3rd Frame	0.610

Table 2 Columns cross section longitudinal reinforcement

Frame	Position	1st Storey	2nd Storey	3rd Storey
1st Frame	Exterior	8Φ18+4Φ16	-	-
2nd Frame	Exterior	8Φ20	8Φ20	-
	Interior	8Φ18+4Φ16	8Φ18+4Φ16	-
3rd Frame	Exterior	8Φ20	8Φ20	8Φ18
	Interior	8Φ18+4Φ16	8Φ18+4Φ16	8Φ18

because the main design action was the gravity loads and not the seismic lateral forces. Furthermore, the first level of the frames is 4 m high so they have the same response as apilotis building. The seismic performance of (RC) frame structures with irregularities leading to soft first floor was studied by Favvata *et al*. (2013).The assumed distributed dead load is 5.5 kN/m^2 which is consistent with a concrete slab of 160 mm thickness and the assumed live gravity loads is 2 kN/m^{2}which is in agreement with the modern codes for residential buildings.

The superstructure was designed so that both the longitudinal reinforcement and the stirrup yield strength (f_{yk}) is 400 MPa and the concrete compressive strength (f_{ck}) is 16 Mpa. These materials have been selected to be in agreement with the typical mechanic properties of the materials of pre-1985 reinforced concrete buildings in Greece. Table 1 lists the 1st mode period of each frame. Additionally in Table 2 the longitudinal reinforcement of the columns is presented whereas Table 3 shows the longitudinal reinforcement of the beams. The stirrups at columns and beams are 8 mm at 200 mm.

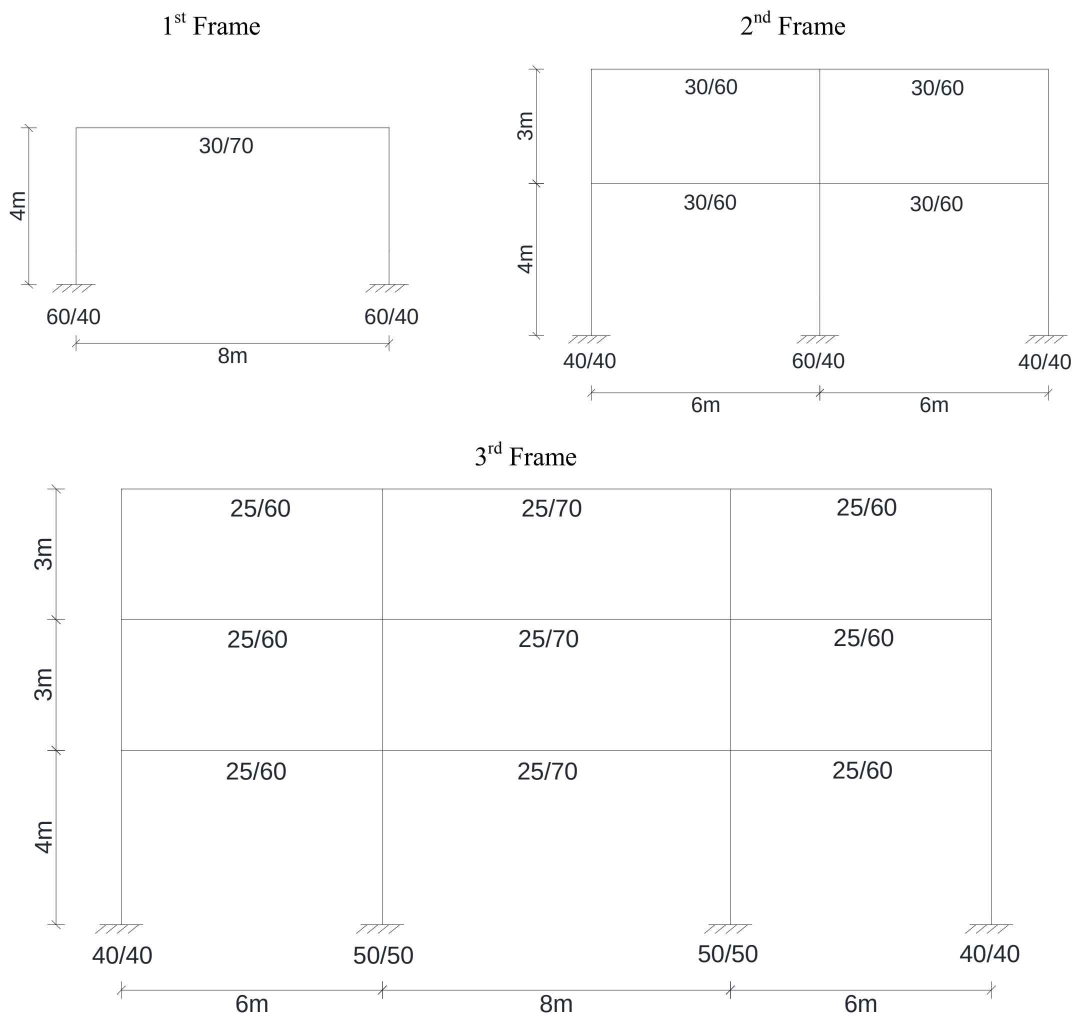

Fig. 4 Analyzed frames

Table 3 Beams cross section longitudinal reinforcement

Frame	Position		1st Storey	2nd Storey	3rd Storey
1st Frame		end i& end j	3Φ18 Top 5Φ20 Bottom	-	-
2nd Frame		end i	3Φ16 Top 5Φ14 Bottom	1Φ14 Top 4Φ14 Bottom	-
		end j	8Φ16 Top 5Φ14 Bottom	5Φ16 Top 4Φ14 Bottom	-
3rd Frame	Interior	end i& end j	7Φ18 Top 6Φ16 Bottom	8Φ16 Top 6Φ16 Bottom	5Φ16 Top 4Φ18 Bottom
	Exterior	end i	5Φ16 Top 5Φ14 Bottom	5Φ14 Top 5Φ14 Bottom	4Φ14 Top 4Φ14 Bottom
		end j	7Φ17 Top 5Φ14 Bottom	8Φ16 Top 5Φ14 Bottom	5Φ16 Top 4Φ14 Bottom

4. Non - Linear pushover analysis

Lateral Force - Displacement diagrams have been exported by Pushover analyses in order to determine the performance point of the frames and to estimate the initial collapse mechanism. A concentrated plasticity model has been adopted for the beam simulation whereas for the columnsimulationa fiber cross section model was applied for accurate axial moment interaction in order to obtain the most accurate results. For the concentrated plasticity model the yield and the ultimate bending moment have been calculated from a moment-curvature analysis while the hinge rotation capacities derived from the Greek Retrofitting Code (KAN.EPE 2012), which is similar to those given by EC8 Part-3 (Eurocode 8 2006). The performance levels of the cross sections and the whole structure were determined in accordance with the Greek Retrofitting Code.

The load pattern based on the 1st mode shape was taken into account although multimode pushover analysis (Manoukas *et al*. 2011) or even energy based pushover (Manoukas *et al*. 2012) were presented. The performance point was estimated through the capacity spectrum method (ATC-40) which is recommended by the EC8 Part-3 and also by the Greek Retrofitting Code. In Fig. 5 thepushover curves of the three framesis depicted along with the performance point as well as the performance levels. Base shear is expressed in the diagram as a fraction of the weight of the structure. The performance points are also listed in Table 4.

The first developed plastic hingesoccurred in the 1ststorey columns in both frames and as such these haveundergone a soft - storey mechanism. The demands for plastic rotation in the ground floor columns for all the frame structures overcame the available capacities. The structural performance levels of Life Safety and Collapse Prevention are defined by the rotational criteria. The one storey and the threestorey frames reached the Life Safety performance level. While, the second frame exceeds the Collapse Prevention performance level. Thus, it is imperative for these structures to be retrofitted. It can be observed that the one storey structure demonstrates the higher

Table 4 Performance Point of the frame structures

Frame	Performance Point	
	Displacement (m)	Shear Force (kN)
1st Frame	0.060	321.34
2nd Frame	0.117	311.17
3rd Frame	0.130	654.79

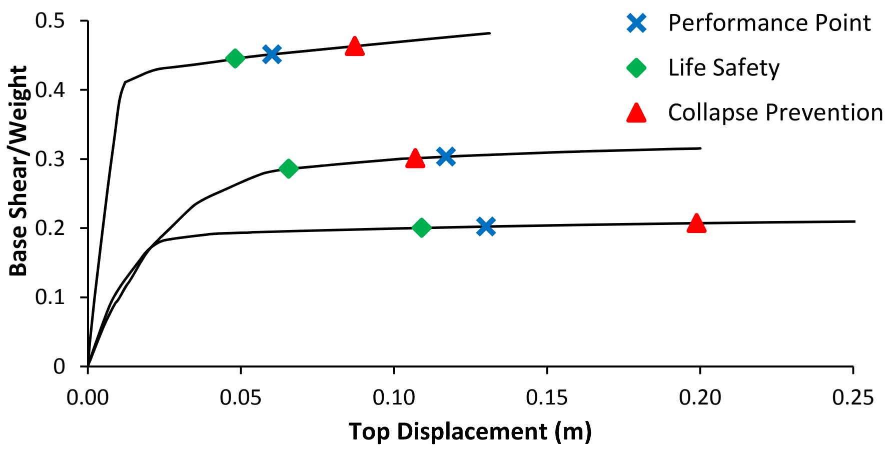

Fig. 5 Capacity curves of the frame structures

capacity in shear force as a fraction of structures' weight while in the two storey structure the most critical demand for plastic rotation is developed which is mainly concentrated at the ground level.

5. Ground motions

A total of three accelerograms were used for the time history non-linear analyses. The accelerograms used (Fig. 6) are from Greek natural ground motions which have caused devastating damage in structures like those have studied. Their main characteristics are listed in Table 5. These characteristics include the moment magnitude Mw, the distance to the fault ruptured, the peak ground acceleration (PGA). The magnitudes of the motions are between 5.9 and 6.4 and the distances to the fault rupture are between 10 and 22 km. The elaboration of the accelerograms was made in accordance with the EC8 provisions. Fig. 7 compares the target EC8 spectra with the matched spectra components of the natural ground motions used for the two storey frame.

Table 5 Characteristics of ground motions

Earthquake	Year	Station	Magnitude (Mw)	Distance (km)	PGA (g)
Kalamata	1986	KAL1	6.2	10.0	0.25
Aegion	1995	AIGA	6.4	21.6	0.48
Athens	1999	ATH3	5.9	15.3	0.26

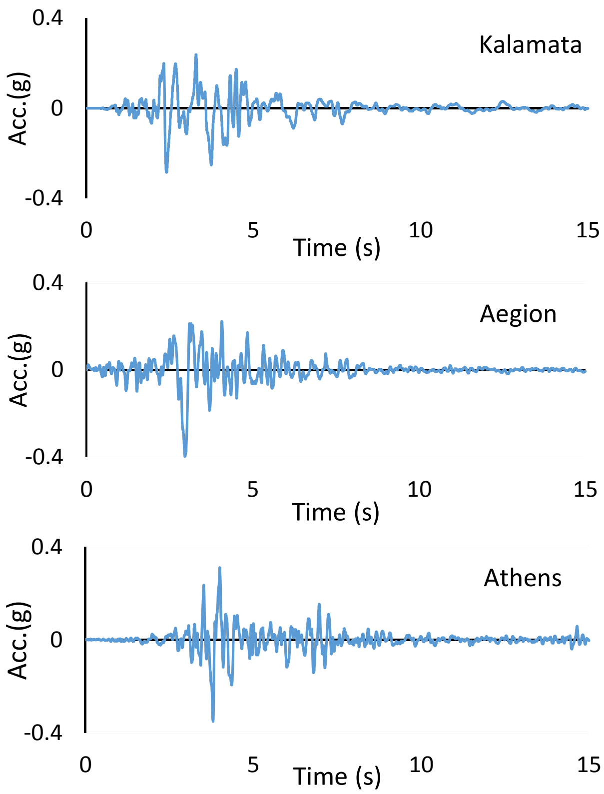

Fig. 6 Accelerograms used in the analysis

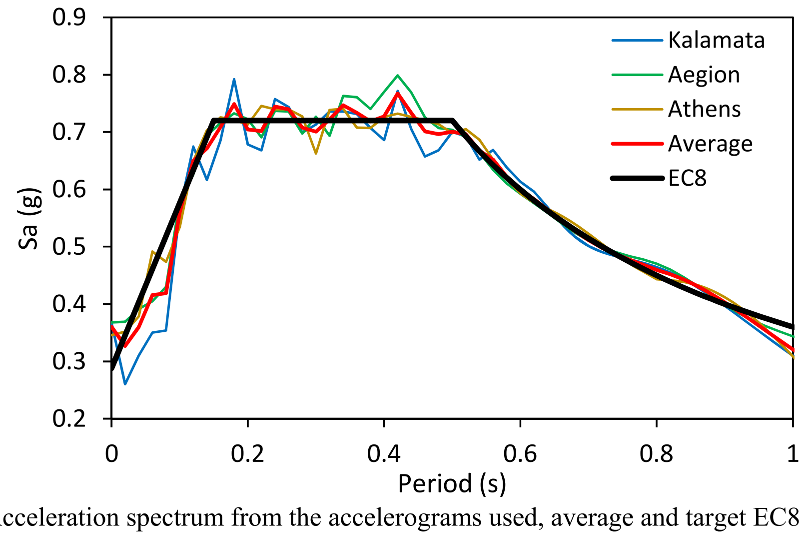

Fig. 7 Acceleration spectrum from the accelerograms used, average and target EC8 spectrum

6. Results

As indicated above we studied the implementation of base isolation systems which are designed so as to correspond to the three ratios of T_{is}/T_{fb} with values from 3 - 5. Three different LRB systems and three FPI systems were applied with the purpose of investigating the effectiveness of these systems to the seismic response of low rise reinforced concrete buildings. The design of the isolation systems was made pursuant to the ratio T_{is}/T_{fb}. Thus the stiffness of the second slope of the bilinear model (K_d) is roughly the same between the two different applied isolation systems in order to achieve the target isolated period T_{is}.

The effectiveness of the investigated isolation systems occurred by evaluating the comparative results between the different retrofitting strategies. The assessment of the alternatives was made mainly in terms of storey drift, storey shear force and followed by the storey accelerations. Finally, the maximum and the residual developed displacements of the isolation systems are listed.

6.1 Maximum storey drift demands

The maximum storey drifts of the examined frame structures without considering base isolation systems are displayed and compared with the corresponding responses of the isolated structures. Fig. 8 demonstrates the storey drift of the 1st frame, while Figs. 9-10 presented the storey drifts of the 2nd and the 3rd frame respectively.

It can be observed that the FPI systems are less effective than the LRB isolation systems to the 1st one storey structure. In order to achieve an isolated period T_{is}=0.8 s, the FPI system with limited radius (R=0.16 m) has a minor effect on the seismic response of the structure. On the other hand, the isolation LRB system corresponding to the same T_{is} is noticeably more efficient and this is achieved largely due to the effective damping which is much higher than that of the FPI systems. The decline of the developed storey drift on average takes a value of 54%. For higher values of isolated periods the installed systems are quite efficient. Generally the LRB systems seem to have greater impact on the one floor structure concerning the storey drift demands. The maximum reduction noticed is about 79%.

As expected for the 2nd and the 3rd structural frame the developed storey drift are critically increased at the ground floor. This fact arises also from the pushover analysis in which the cross sections of the 1st floor columns were damaged first and demanded the greater plastic rotation. However on the top floor of these structures the developed drifts are very small. The reduction of the ground floor storey drift is the main objective in order to prevent the creation of a soft storey mechanism. Is obvious the remarkable reduction of the demands for storey drift especially to the vulnerable ground floor.

Regarding the two floor structure the reduction on average takes a value of 70%. Both the most flexible FPI and LRB systems obtain the smaller values of ground floor storey drift. The improvement of the isolated structure response in terms of storey drift (Fig. 13) is more intense for the FPI installed systems. From 62% reduction in the storey drift for the case where the ratio of T_{is}/T_{fb}=3, it reaches a reduction of 82% for the isolated structure which corresponds to T_{is}/T_{fb}=5.For the LRB applied systems the reduction was between 57% - 74%. In absolute numbers in the case of FPI isolation system corresponding to T_{is}/T_{fb}=5 the drift took a value of 0.18% while the initial value was 1.05%.

For the three floor structure the implementation of the isolation systems is the most effective compared to the other frame structures. The decrease of the developed storey drift on average takes a value of 78%. The reduction of the first floor storey drift ratio ranges from 71% to 87% for the FPI systems while from 67% to 87% for those of the LRB. The most important alleviation of the ground floor occurred. In the case of LRB system corresponding to T_{is}/T_{fb}=5 the drift took a value of 0.18% while the initial value was 1.40%.

Comparing the results in terms of storey drift between all the examined frames for every case of isolation systems corresponding to the same ratio T_{is}/T_{fb} it is remarked that as the isolated period takes higher values in absolute numbers the storey drifts decreased. Therefore, the effect of isolation systems are more considerable on the third three storey frame.

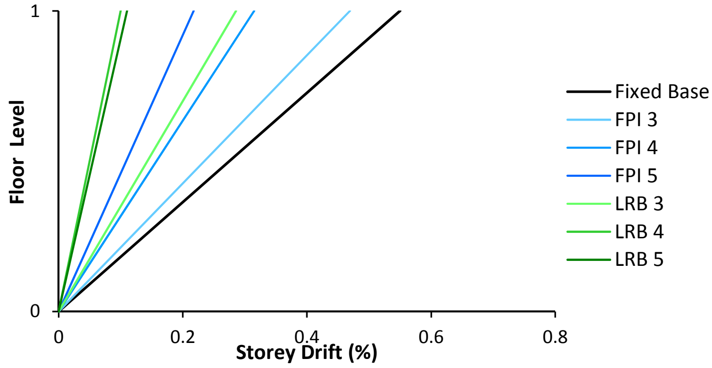

Fig. 8 1st frame storey drift

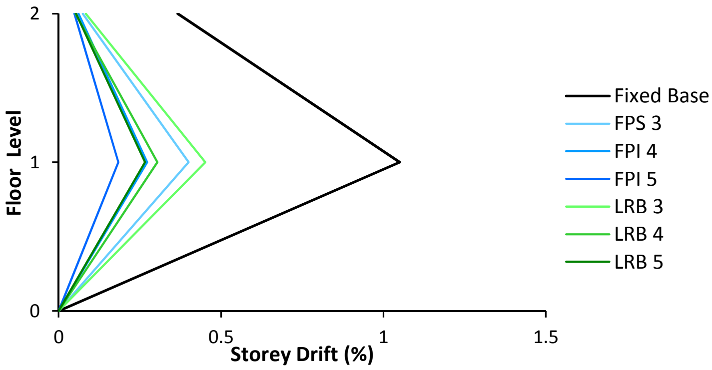

Fig. 9 2nd frame storey drift

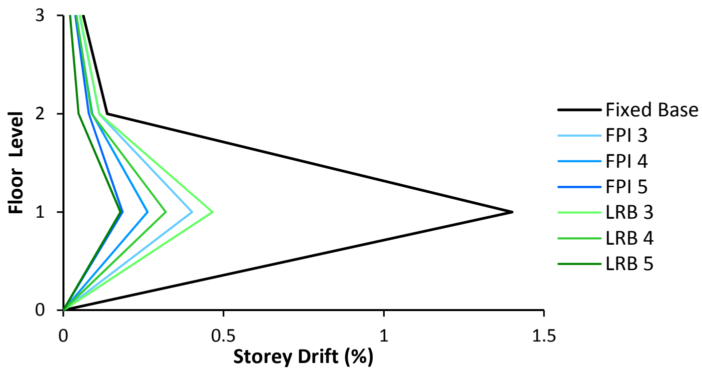

Fig. 10 3rd frame storey drift

6.2 Maximum storey shear demands

The main weakness of reinforced concrete structures designed mainly considering gravity loads, without adequate provision for seismic detailing is the shortage of ductility and the creation of brittle damage of the cross sections which are caused by the shear forces. Thus the storey shear forces are a useful index which depict the vulnerability of the seismic response of these structures. Figs. 11-13 present comparatively for each examined frame the developed storey shear forces.

It is apparent that the developed shear forces are in accordance with the developed storey drifts. In the case of FPS isolation systems corresponding to the target ratio T_{is}/T_{fb}=3 the shear force is vaguely reduced. For the rest of the isolation systems the shear forces are considerably decreased. The LRB isolation systems is noticeably more efficient as the reduction of the forces for all the cases examined ranges from 75% to 82% in comparison with FPI systems where a reduction between 11% and 59% occurred.

The vulnerability of the ground floor vertical structural members of the 2nd and the 3rd structural frame is clearly remarked in Figs. 12-13. The developed shear forces are greater in the ground floor than in the top floors. Thus the seismic response of these frames are obviously similar to a pilot is building seismic response.

The effect of the implemented base isolation systems in the seismic response of these structures is considerable for all the examined alternatives. In particular, the ground floor developing shear forces are significantly restricted. Therefore, the main weakness of the seismic response of these structural frame did not occur. Reduction of the shear forces are presented also in the upper floors but to a lesser degree.

It can be remarked that the less lateral stiffness the isolation device has, the more effective on the seismic response of the structure could be. Regarding the two floor frame, the reduction of the ground floor in the case of LRB systems takes values between 63%-84%, while for the FPI systems it was between 59%-76%. As a result, the most effective applied system is the FPI.

The effect of the isolation systems to the seismic response of the 3rd three floor structural frame is quite similar with those of the 2nd frame. The ratio of the alleviation in terms of ground floor shear forces ranges for the FPI systems between 59%-82% and for the LRB systems from 52%-82%. Thus, both the FPI and the LRB isolation systems substantially affectthe seismic response of the structure.

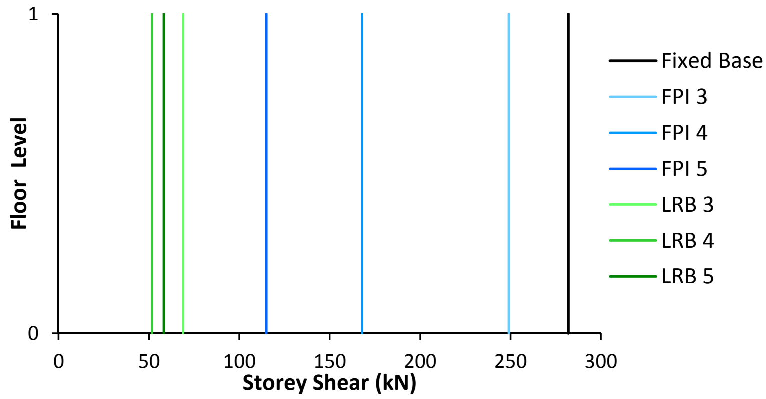

Fig. 11 1st frame storey shear force

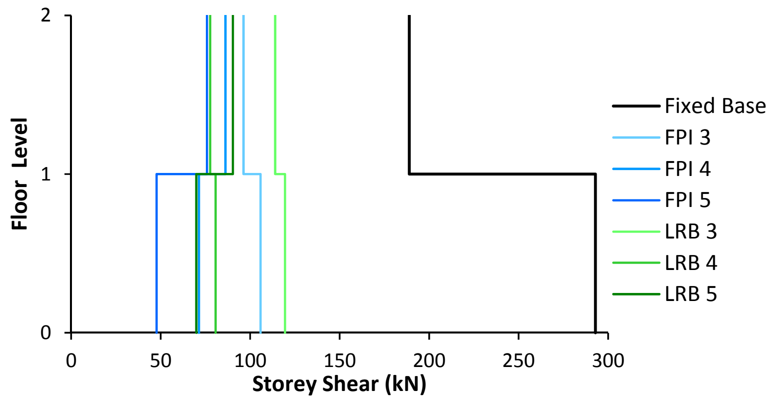

Fig. 12 2nd frame shear force

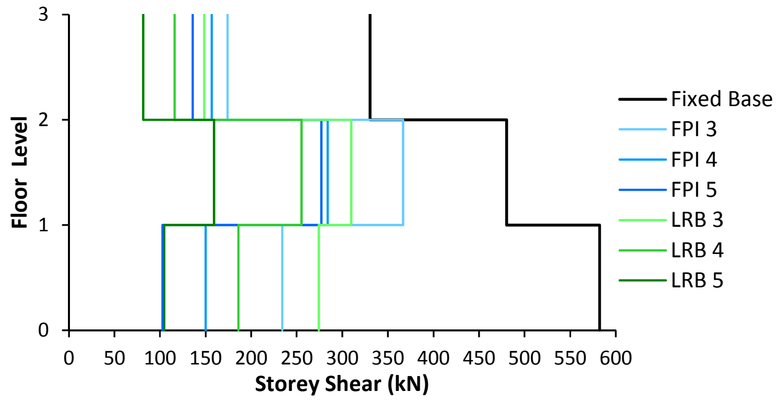

Fig. 13 3rd frame storey shear force

Comparing the results in terms of storey drifts and in terms of the developed storey shear forces between all the examined structural frames (Figs. 8 and 11) the less contribution of the FPI systems in the seismic response when the isolated period is restricted to very low values can be remarked. It is clear that the efficiency of the FPI and LRB isolation systems are similar when the isolation period takes higher values as in the 2nd and the 3rd examined frame.

6.3 Maximum storey accelerations

For the protection of sensitive internal equipment and non-structural elements, the reduction of the floor accelerations is of high importance. The floor accelerations for each structural frame are illustrated in Figs. 14-16.

Indicated above in terms of storey drift demands and developed storey shear forces, the implementation improves the seismic response of the superstructures. However regarding the floor accelerations this improvement is not so clear.

The accelerations of the top floors are decreased for all the examined isolation systems applied to the 2nd and the 3rd structural frame. Nevertheless, the accelerations of the ground floor are not reduced similarly.

Namely both LRB and FPI devices with the higher stiffness increases the acceleration of the 1st frame. The increase but also the decrease of the accelerations were more intense in the cases of the FPI systems. The LRB systems affect to a lesser degree the acceleration response.

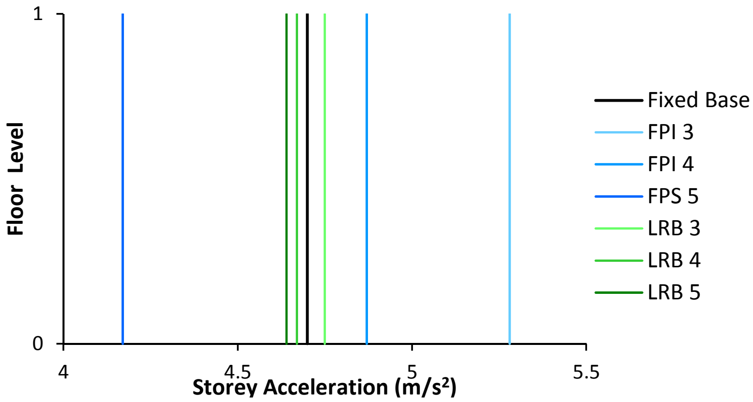

Fig. 14 1st frame floor acceleration

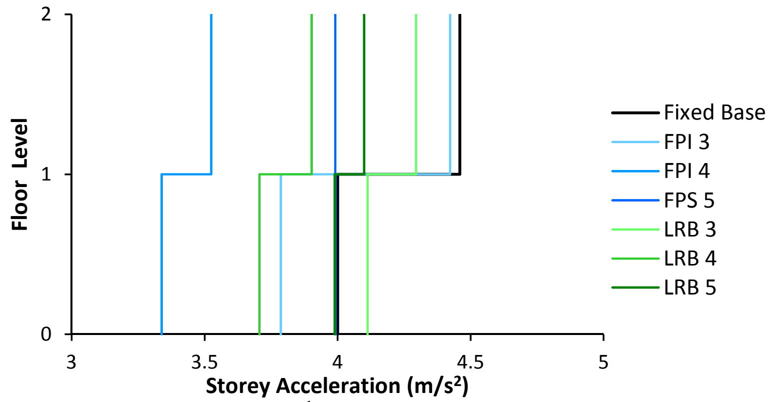

Fig. 15 2nd frame floor acceleration

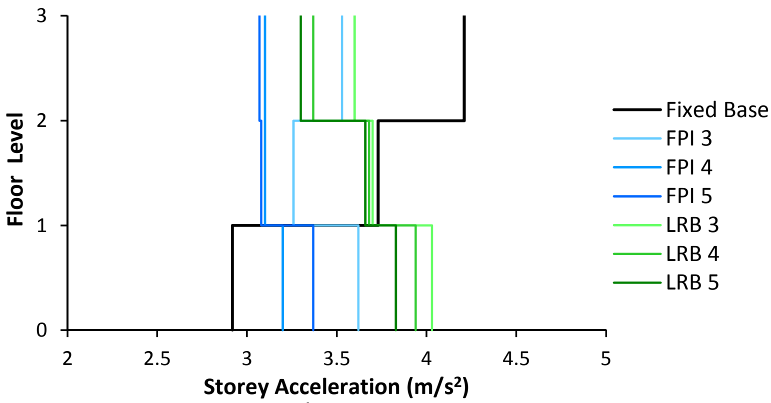

Fig. 16 3rd frame floor acceleration

The acceleration of the 2nd frame ground floor are reduced except for the LRB isolation system which corresponds to a ratio T_{is}/T_{fb}=3.The higher reduction of the ground acceleration performed in the case of FPI corresponding to a ratio of T_{is}/T_{fb}=4.

Concerning the 3rd examined structural frame the grounds floor accelerations in all cases, are increased. However, in the upper floors the accelerations are decreased. The FPI system caused less increase of the ground floor acceleration, while it reduces at a greater rate the accelerations in the top floors. It is obvious that the increase of the isolation system period for all the cases reduces the storey accelerations. After all, the most efficient isolation mechanism resulted in terms of floor accelerations is the FPI systems.

6.4 Isolation systems results

Another important issue is the functionality of the base isolation devices. Notable remarks are resulted from the inelastic bilinear cycles of the isolation systems. Figs. 17-18 indicatively present the hysteretic response from a time history non-linear analysis of a FPI and a LRB isolation system respectively.

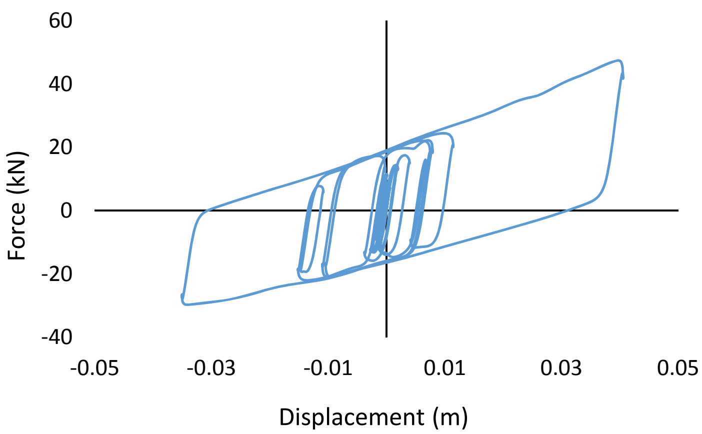

Fig. 17 Hysteretic behavior of FPI system

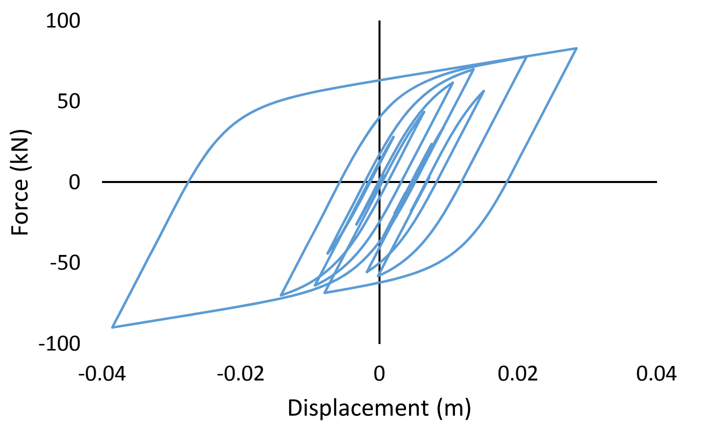

Fig. 18 Hysteretic behavior of LRB system

Table 6 Maximum and residual isolation systemsdisplacements

Frame	Isolation System	Ratio T_{is}/T_{fb}	Maximum Displacements (cm)	Residual Displacements (cm)
1st Frame	FPS	T_{is}/T_{fb}=3	5.38	0.072
		T_{is}/T_{fb}=4	6.45	0.049
		T_{is}/T_{fb}=5	6.95	0.049
	LRB	T_{is}/T_{fb}=3	0.92	0.190
		T_{is}/T_{fb}=4	1.17	0.204
		T_{is}/T_{fb}=5	2.08	0.444
2nd Frame	FPS	T_{is}/T_{fb}=3	5.23	0.105
		T_{is}/T_{fb}=4	6.28	0.129
		T_{is}/T_{fb}=5	6.55	0.308
	LRB	T_{is}/T_{fb}=3	4.25	0.436
		T_{is}/T_{fb}=4	5.00	0.495
		T_{is}/T_{fb}=5	5.88	0.885
3rd Frame	FPS	T_{is}/T_{fb}=3	6.02	0.168
		T_{is}/T_{fb}=4	6.86	0.497
		T_{is}/T_{fb}=5	7.34	0.636
	LRB	T_{is}/T_{fb}=3	5.12	0.369
		T_{is}/T_{fb}=4	5.7	0.779
		T_{is}/T_{fb}=5	5.13	0.575

The number of inelastic cycles experienced by the isolation systems tends to increase in number while decreasing the isolation ratio (T_{is}/T_{fb}). Base isolation systems with a short isolation period experiences several small inelastic cycles, which do not result considerable improvement on the seismic response of the superstructure. On the contrary, increasing the isolation period, the hysteretic response of the isolation systems is characterized by a few but large inelastic cycles, which affects significantly the response of the superstructure.

Table 6 lists the maximum and the permanent developed displacements. As expected increasing the isolation period provokes larger maximum displacements. In majority larger residual displacements are occurred when the developed maximum displacements are increased. Comparing the FPI with the LRB isolation systems is obvious that the FPI provided more sufficient restoring forces. While the LRB systems developed lesser maximum displacements, the permanent displacements are larger than those of the FPI systems. In both cases the restoring level are adequate and prevented permanent displacements to accumulate tounacceptable levels.

7. Conclusions

This paper presents an investigation on the dynamic performance of base isolation systems implemented in low rise reinforcement concrete structural frames. Both lead rubber bearings (LRB) and friction pendulum isolators (FPI) base isolation devices were examined. Three prototype frames ranging from one floor to three floor have been studied. These frames are representative of low rise buildings designed and constructed before 1985 in Greece. The selection of the features of the base isolation devices was based on the period of the isolated structure. The implementation of the base isolation systems corresponds to the ratio T_{is}/T_{fb} that values from 3 to 5 was studied. The isolation period associated with the second slope of the bilinear isolation system is considered as the isolation period. The main purpose of this comprehensive study is to investigate the effect of the base isolation system period on the seismic response of inadequately designed low rise buildings.

Considering the results from the nonlinear pushover and time history analyses is clear that the most vulnerable structural members of the examined structures are the grounds' floor columns. Is obvious especially to the most vulnerable, ground floor, the drastically reduction of the developed storey drifts and storey shear forces, for all the base isolation systems proposed.

The results indicate that the less lateral stiffness the isolation system has, the more effective on the seismic response of the structure is. The storey drifts along with the storey shear forces generated by the earthquake are reduced notably as the ratio T_{is}/T_{fb} is increased. Moreover as the isolated period takes higher values in absolute numbers the storey drifts and the storey shear forces are decreased progressively. Therefore, the effect of the isolation systems are more considerable on the second and even more on the third three storey frame.

Comparing the two types of isolation systems corresponding to the same isolation period, both substantially affect the seismic response of the two and the three storey structures. However regarding the one storey structure the FPI system corresponding to the ratio T_{is}/T_{fb}=3 has a minor effect on the seismic response of the superstructure while the LRB system corresponding to the same isolated period was noticeably more efficient. The efficacy of the FPI and the LRB isolation systems corresponding to the same isolation period are similar, when the isolation period takes higher values as in the 2nd and the 3rd examined frame.

Regarding the floor accelerations the effect of the base isolation systems is not so clear. The accelerations of the top floors are decreased significantly for all the examined isolation systems. However, the accelerations of the ground floor are not reduced similarly. In particular concerning the 3rd examined frame the ground floor accelerations in all cases are increased.

Considering the bilinear cycles of the isolation systems, the fewer but larger inelastic cycles which experienced when the isolation period takes higher values results more considerable improvement on the seismic response of the superstructure.

Definitely, more studies are needed to evaluate the applicability of a retrofitting strategy for RC frame buildings based on the use of seismic isolation while accepting the occurrence of plastic hinges in the superstructure. The extension of this study to different structural and base isolation systems, taking into account degrading cyclic effects and considering the shear resistance of the structural members should be performed. Furthermore, the assets of the structure response in terms of structural members' ductility could also lead to important results.

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