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Seismic assessment of existing under-designed reinforced concrete buildings retrofitted with friction devices

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ABSTRACT

The aim of this paper is to examine the efficiency of retrofitting with passive energy dissipative systems on the seismic performance of existing reinforced concrete buildings designed according to the old seismic Greek Codes. Specifically, is investigated the response of two 3-strorey frame structures. Both structural systems have tall first storey height, while the second one has irregularities in elevation also to the top floor levels. Friction devices supported by chevron brace assemblages implemented in the ground floor are studied. The intervention is aimed at increasing the overall seismic response of the structures by removing the softstorey weakness but without transferring the problem to the stories above, since over-strengthening the ground storey would transmute the top floors more vulnerable. Both braces and friction devices are designed for two different percentages of base shear capacity, in order to determine the effectiveness of each retrofitting level in reducing the seismic demand of the existing reinforced concrete cross sections of the entire structure. Subsequently, the frames are subjected to a set of seven code compatible accelerograms carrying out nonlinear time history analyses. Results in terms of plastic rotation demands, interstorey drifts, joint displacements and ductility requirements are presented. From the results obtained can be deduced that strengthening only the weak open ground storey can effectively reduce the vulnerability of the buildings by removing the inherent weakness without shifting the problem to the floors above, and this makes such buildings at least as strong as those without a weak first storey.

1. INTRODUCTION

Reinforced concrete frame buildings, designed according to old seismic codes and without adequate antiseismic provisions, constitute a significant number of structures all over the world. These buildings suffered most from heavy damage and collapses during strong earthquakes the last decades worldwide. As confirmed, their seismic

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response is undesirable (Thermou 2011). Various strategies based on structural strengthening have been proposed for the seismic retrofit of existing buildings (Thermou 2006). Among the alternative retrofitting methods the seismic structural response control techniques are currently applied.

The past few decades remarkable development has been achieved in earthquake engineering and in seismic structural response control (Soong 2002, Hanson 2001), which is classified into passive, active and semi-active control (Habibi 2012). Passive energy dissipation system is the most effective and reliable method for improving the seismic response of existing buildings. Installing energy dissipative devices in a structure a considerable percentage of the input seismic energy is absorbed and the total damage is limited (Martinez-Rueda 2002, Lu 2006, Lu 2004). Damping devices, using a dry-friction mechanism, represent one of the most common device categories. These types of devices are generally referred as friction dampers (Hanson 2001, Lu 2006). Examples of friction dampers include devices developed by Pall (1982), Roik (1988), Aiken (1990), Fitzgerald (1992) and Mualla (2002).

The purpose of introducing energy dissipating devices in a reinforced concrete structure is to increase its hysteric damping. This kind of energy constitutes one part of the total energy requirement in the system. The complete energy balance is given by the following equation:

$$\mathrm{E_I = E_S + E_K + E_D + E_H} \tag{1}$$

In the previous equation, at a given instant in time t, $\mathrm{E_I}$ represents the earthquake input energy, $\mathrm{E_S}$ the strain energy stored by the structure, $\mathrm{E_K}$ the kinetic energy of the moving mass, $\mathrm{E_D}$ the total viscous damping energy, and finally $\mathrm{E_H}$ the total hysteretic energy. At the end of earthquake phenomenon strain energy $\mathrm{E_S}$ is zero for elastic system and zero or approximately zero for inelastic system, kinetic energy $\mathrm{E_K}$ is zero, and hysteretic energy $\mathrm{E_H}$ is equal to the energy demand (Symans 2008, Uang 1988). The expected is to increase $\mathrm{E_H}$ so that for a known level of input energy, the strain energy in the structure is decreased. This implies that the structure will be submitted in smaller deformations, for given input energy, than without including energy absorbers. Alternatively, while increasing $\mathrm{E_H}$ permits $\mathrm{E_S}$ to be reduced for a raise of the input energy (Aiken 1992).

Friction dampers are categorized as displacement-dependent energy dissipation devices, because their damper force is independent from frequency and velocity (Hanson 2001). They have found large practical application in seismic retrofitting of existing structures due to their low cost and simple installation.

A typical friction damper consists of series of steel plates tighten together with high durability steel bolts in order to achieve more reliable friction through the interfaces. Damper's maximum friction force or slip force is constant according to Coulomb's friction theory, since the clamping force is definite and predetermined by the design engineer. The motions of friction damper switches between stick or slip state. Specifically, a friction damper dissipates seismic energy only if it is in slip state, which means that friction force exerted on its friction interface overdraws the maximum friction force. In case that there is no relative motion on the friction interface, the damper is said to be in stick state and subsequently the friction force is equal to the loading force (Hanson 2001, Lu 2006, Lu 2004).

A major importance issue in designing a friction damper is the appropriate choice of slip load. The limits of the force required to slip should be between a minimum level, so that the friction damper should not start to slip against minor forces, like wind or slight earthquake, and a maximum level so that it does not slip during powerful earthquakes. Several studies have shown that an optimum selection of the slip load gives a minimum response of the structure and also variation up to ±20% of the optimum slip load do not affects the response remarkably (Filliatrault 1987, Amiri 2012).

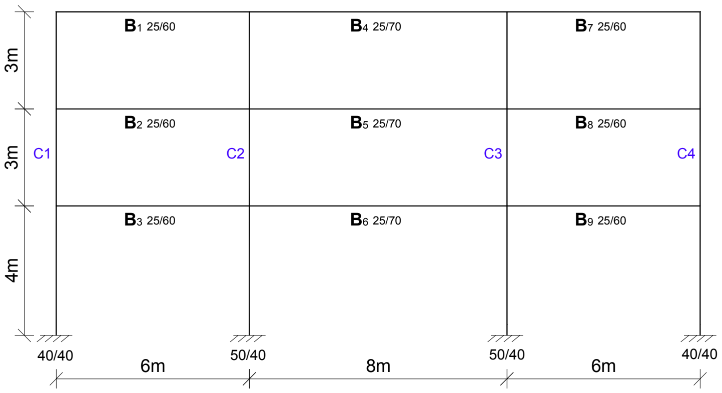
Friction dampers attribute large rectangular hysteresis loops, similar to an ideal elasto-plastic behavior and that's why considerable amount of energy can be dissipated per cycle of motion. The energy dissipation of a friction damper is the largest, compared to other damping devices, for a given force and displacement and as a result fewer friction dampers are required to provide a given amount of damping.

The most appropriate procedure for evaluating the behavior of systems that incorporate passive energy dissipation devices, like friction dampers, is nonlinear dynamic analysis. Exploiting the available software and academic programs these individual devices can be clearly modeled and analyzed.

2. FRAMES MORPHOLOGY

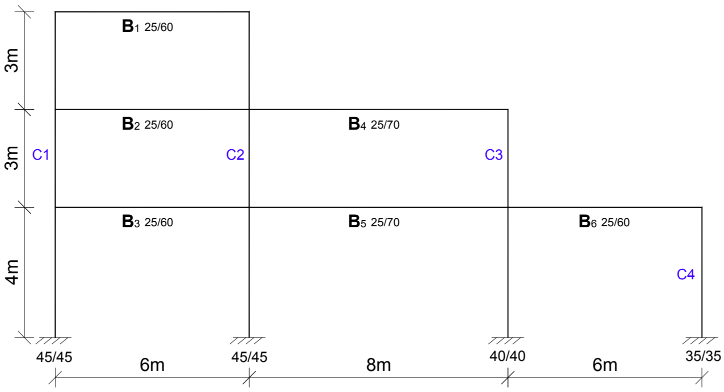
In this paper is investigated the response of two individual 3-strorey frame structures of existing reinforced concrete buildings designed according to the old seismic Greek Codes (RD 1959). Both structural systems have tall first storey height, while the second one has irregularities in elevation also to the top floor levels. Specifically, the ground floor has a height of 4.00m, whereas the other two floors have a height of 3.00m. Studies on the seismic response of reinforced concrete buildings with first floor irregularities such as tall ground floor height (Favvata 2013), have shown that the global capacity of the structures is decreased due to the considered first floor morphology irregularities in comparison to the capacities of the regular structure. Specifically, the increase of the demand for interstorey drift at the ground floor and softstorey mechanisms are noticed.

The materials used in both structures are concrete "B225" (f_{ck}=16MPa) and rebar steel "StIII" (f_{yk}=400MPa). All the columns sections have stable dimension in elevation. The beams of the structures are alternated between 25x60cm and 25x70cm. Both columns and beams cover is determined 30mm. All the plates of both structures are 20cm thick. In both structures all the columns are considered to be fully supported to the ground floor. In Fig. 1 and 2 the two examined frames and their columns' longitudinal reinforcement are presented.



Floor	Column	Longitudinal Reinforcement
1^{st}–2^{nd}	C1-C4	8Φ20
	C2-C3	8Φ18 + 2Φ16
3^{rd}	C1-C4	8Φ18
	C2-C3	8Φ18

Fig. 1 First examined frame



Floor	Column	Longitudinal Reinforcement
1^{st}	C1-C2	8Φ18
	C3	8Φ16
	C4	8Φ14
2^{nd}	C1-C2	8Φ18
	C3	8Φ16
3^{rd}	C1-C2	8Φ18

Fig. 2 Second examined frame

3. EARTHQUAKE INPUT

The seismic behavior of the buildings before and after strengthening with friction dampers is evaluated using nonlinear time history analyses, based on the Part-3 of Eurocode 8 (2005) and the Greek Retrofitting Code (KAN.EPE 2012).

In order the nonlinear dynamic analyses to be accomplished seven different accelerograms from intense earthquakes all over the world are taken into consideration. The earthquake data have been gathered from the Pacific Earthquake Engineering Research Center Strong Motion Database (PEER) of the University of Berkeley. The selected response spectrums are matched in order to conform to the design spectrum of Eurocode 8 (2005) for Seismic Zone II (α_{gR}=0.24g), Soil Class C, Importance Class II (ordinary buildings) and behavior factor q*=1.7. The rudimental characteristic of all the seismic excitations and the acceleration spectrums are presented in Table 1 and Fig. 3, respectively.

Table 1 Seismic excitations

Earthquake	Record date	Station	Magnitude (Ms)	Depth (km)	Epicentral Distance (km)
Kobe	16-01-1995	Nishi-Akashi	6.90	17.9	8.70
Kalamata	13-09-1986	Kalamata	6.20	5.0	9.97
Corinth	24-02-1981	Corinth	6.60	7.2	19.92
Loma Prieta	18-10-1989	Gilroy Array	6.93	17.5	28.64
Northridge	17-01-1994	LA - Chalon Rd	6.69	17.5	14.92
Imperial Valley	15-10-1979	Casa Flores	6.53	10.0	12.43
Manjil	20-06-1990	Abbar	7.37	19.0	40.43

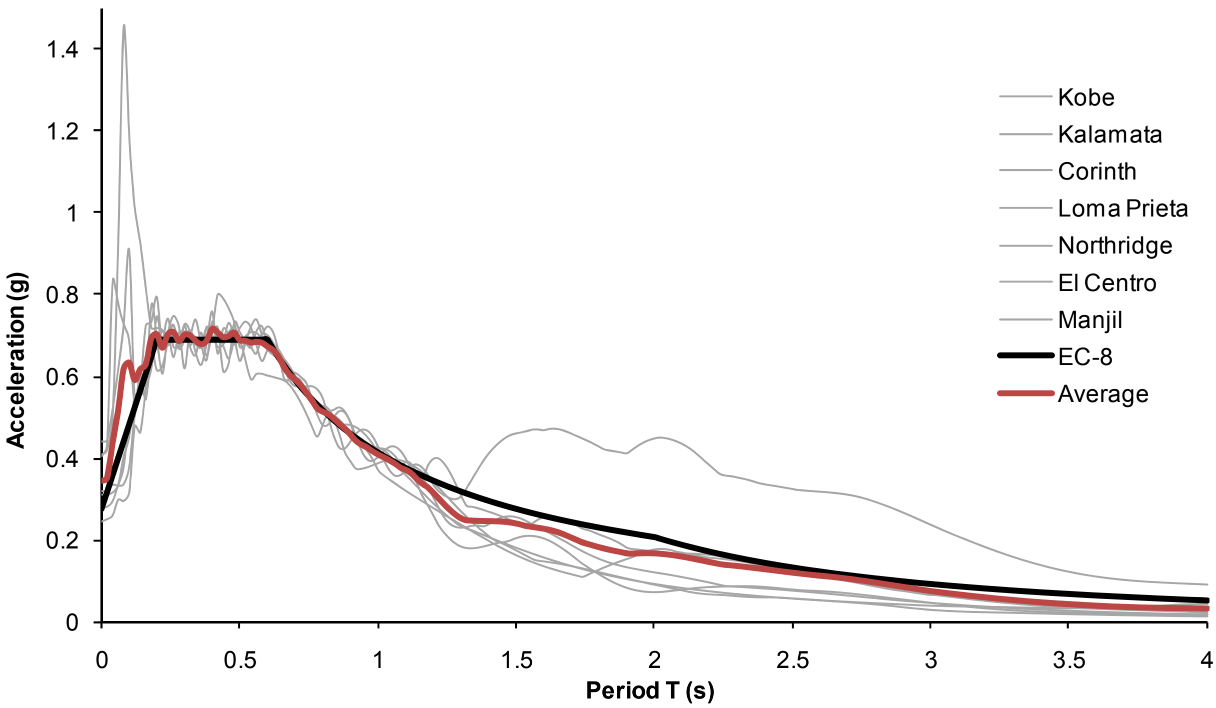


Fig. 3 Acceleration spectrums

4. MODELING OF STRUCTURAL MEMBERS

All the structural members, beams and columns, contribute to the durability and stability of the buildings under the effect of the seismic loads. These horizontal and vertical members are modeled using individual frame elements with three degrees of freedom. For the nonlinear analyses the reinforcement of each member is taken into consideration. The influence of infill walls is ignored in the structural models.

Subjected to seismic excitations the RC members subsists yielding and nonlinear deformations. Elements which exhibit nonlinear response are identified at the two ends of structural members. Specifically, the nonlinear behavior of the beams is simulated with link elements. The hysteresis of the model is defined according to Pivot hysteresis rule (Dowell 1998). The interaction of axial force and moment of columns is determined by using fiber model, which define the plasticity of column section (Spacone 1996).

5. MODELING OF RETROFITTED BUILDINGS

For improving the seismic response of the existing structures passive energy dissipation devices are installed in the ground floor. Partial strengthening of RC buildings with steel braces restricted to the open ground storey have investigated by Antonopoulos (2012). In this paper the implementation of friction damper devices supported by chevron brace assemblages are studied. Consequently, a considerable percentage of the input energy is absorbed and the total damage is limited.

The sections of the chevron braces are chosen according to the resistance in compression and flexural buckling and as a result they can endure the developed axial force, as defined in the Eurocode 3 (2005). So, the chosen section for the symmetric frame is HE120A and for the second is HE100A. These braces are pinned to the frame nodes and have been placed symmetrically. Moreover, the material of the chevron braces is steel S355 (f_y=355 N/mm^2). In Fig. 4 the two retrofitted frames are presented.

Both braces and friction devices are designed for two values of slip load. The values of slip load are defined as a percentage of base shear capacity and specifically 5% and 10%. Carrying out a repeated procedure the above percentages are chosen in order not to transfer the damage to the upper floors. In the following table (Table 2) the examined cases of the friction force of the dampers are presented.

Table 2 Dampers friction force

		First Frame	Second Frame
Base Shear V_o (kN)		1215.90	857.70
Friction Force (kN)	5%V_o	60.80	42.89
	10%V_o	121.60	85.77

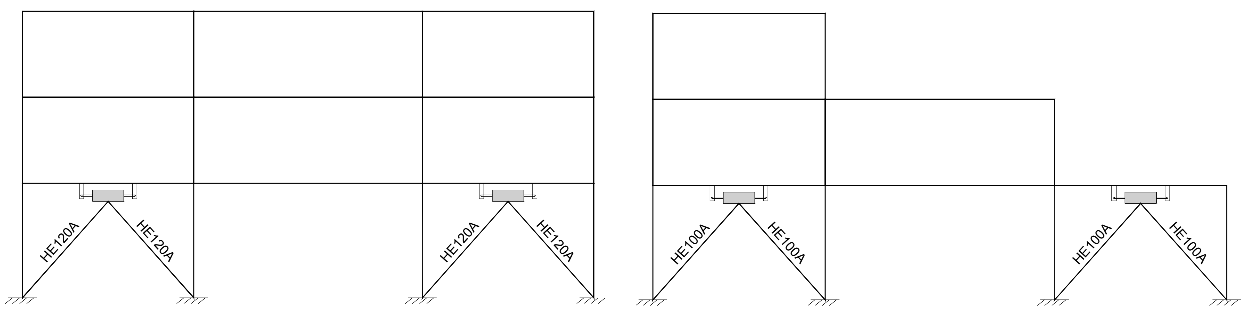


Fig. 4 Retrofitted frames

6. RESULTS

Modal analysis is an initial estimation of the buildings behavior. The periods of the two first modes are taken into consideration for the nonlinear time history analysis. The results of the modal analysis of the initial structures and after retrofitting with passive energy dissipative systems are represented in the following table (Table 3).

Table 3 First mode period

Periods (sec)			
	Initial frame	First case of retrofitting ($5\%V_o$)	Second case of retrofitting ($10\%V_o$)
First frame	0.426	0.409	0.387
Second frame	0.376	0.361	0.337

The seismic behavior of the investigated buildings is evaluated using nonlinear time history analyses. Seven different accelerograms from intense earthquakes all over the world are taken into consideration. The local limit states of Immediate Occupancy (IO), Life Safety (LS) and Near Collapse (NC) in terms of plastic rotation are defined based on the requirements of Greek Retrofitting Code (KAN.EPE 2012).

According to the exported results from the initial frames, it can be observed that the majority of the critical sections of the columns of the ground floor responded inelastically, exceeding the Life Safety (LS) limit state and as a result softstorey is created. The majority of the critical sections of the structural members in the upper floors exhibit inelastic behavior whereas indicate damage of minor importance. In order to improve the seismic response of the structures, friction dampers supported by chevron braces are implemented in the ground floor. Both braces and friction devices are designed for two different values of slip load and specifically 5% and 10% of the base shear capacity.

In Fig. 5 the average values of plastic hinge rotations θ_{pl} of the seven examined seismic excitations for the two investigated frames and the corresponding limit states are depicted. The acceptable rotational limits of the critical sections of each structural member that are given according to the Greek Retrofitting Code (KAN.EPE 2012) are compared to the plastic rotational demands at every deformation step of the analyses. In order to define the limit state the values of yield rotation θ_y and ultimate rotation θ_u for the beams and columns are required. The plastic rotation θ_{pl} is defined as the difference of θ_u minus θ_y. The results of the initial frames and the retrofitted frames for both values of the slip load of friction devices are also depicted in Fig. 5.

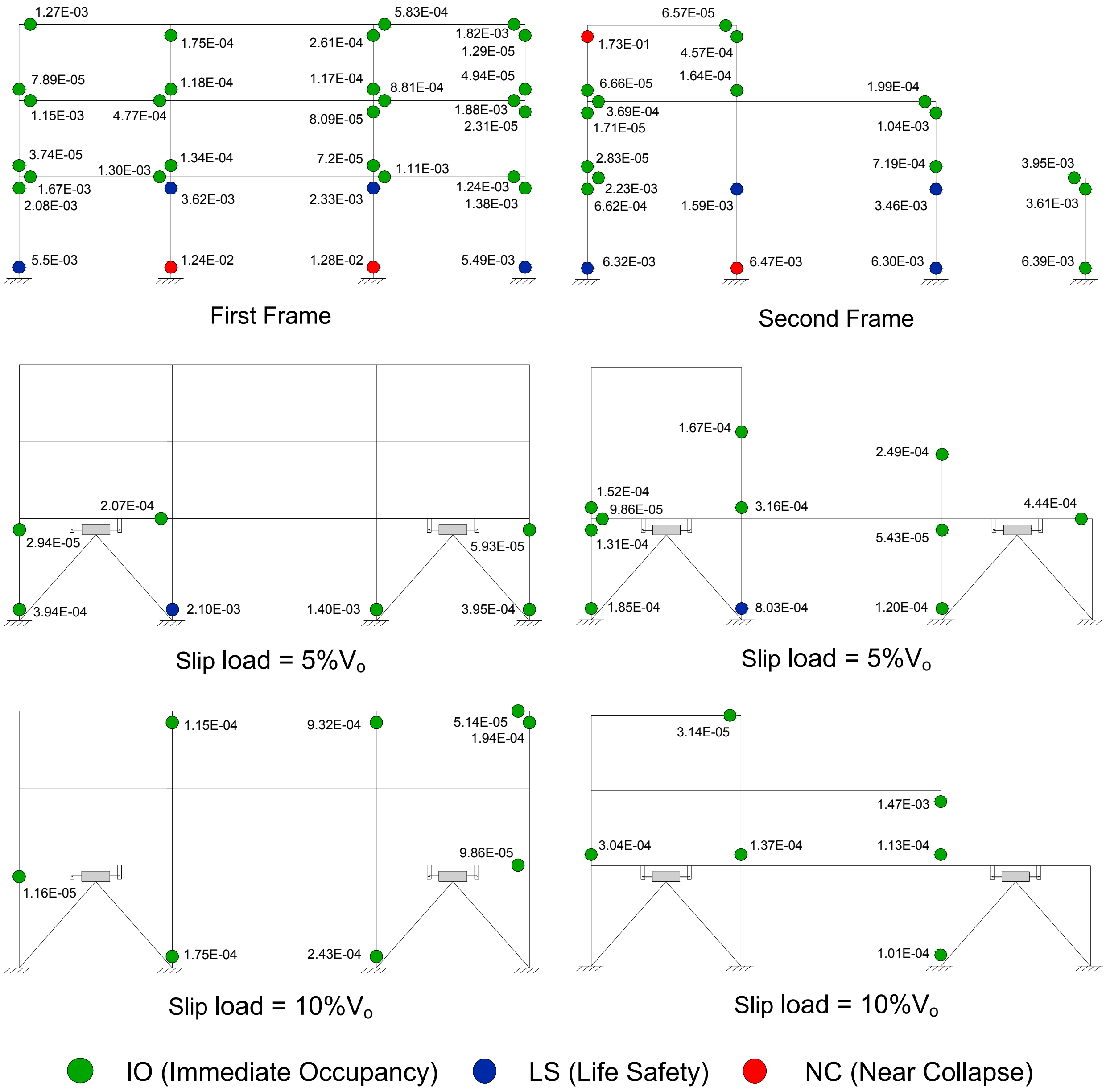


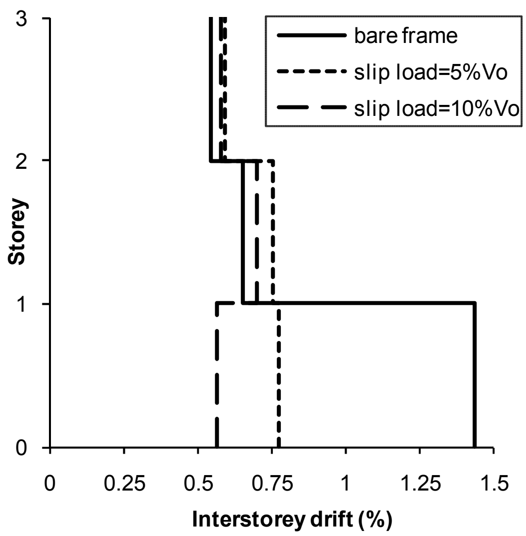
Fig. 5 Average values of plastic hinge rotations θ_{pl} for both structures

From both investigated frames it can be observed that the majority of plastic hinges created in the critical section of the columns instead of beams. The requirements for inelastic response of columns are mainly concentrated at the ground level and as a result softstorey is created. Specifically, the critical sections of the base of the internal columns are more vulnerable to the seismic excitation, as they exhibit inelastic behavior and therefore they exceed the Near Collapse (NC) limit state. Also, it can be observed that the 50% of the columns critical areas in the ground floor exceed the Life Safety (LS) limit state and the rest of them reach the Immediate Occupancy (IO) limit state for both frames, proportionally. The critical sections which exhibit inelastic behavior in the upper floors of both structures reach the Immediate Occupancy (IO) limit state, whereas one developed plastic hinge in a column of the second frame in the third floor exceeds the Near Collapse (NC) limit state. Regarding the critical sections of the beams, those which display inelastic response are observed in the outwards openings and reach the Immediate Occupancy (IO) limit state. Almost all critical sections of the beams of the internal openings remain in elastic range.

The effectiveness of the investigated retrofitted structures occurs by evaluating the comparative results between the two different percentages of the slip load given to the friction devices. It is noted that the improvement of the structures seismic response is more intense in the second case of retrofitting by using as slip load the 10% of the base shear capacity. Specifically, almost all the beams from both structures behave elastically, whereas them which exhibit inelastic characteristics reach the Immediate Occupancy (IO) limit state. More plastic hinges are created in the columns of the ground floor which reach Immediate Occupancy (IO) limit state or exceed Life Safety (LS) limit sate in case where the slip load is defined as the 5% of the base shear than 10%. In case that the slip load is defined as the 10% of the base shear the majority of columns critical sections in the ground floor exhibit elastic behavior, while the created plastic hinges reached the Immediate Occupancy (IO) limit state.

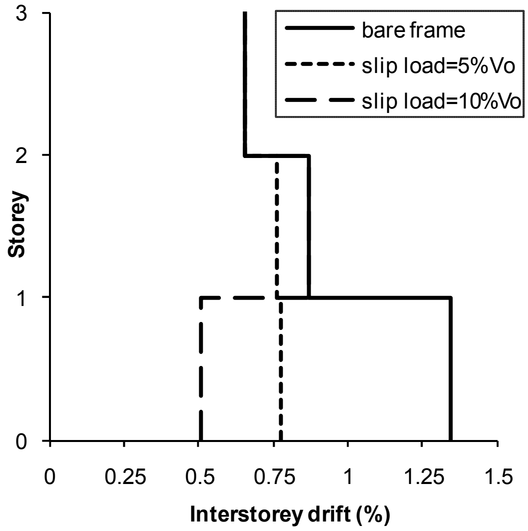
The investigated intervention increases the overall seismic response of the structures by removing the soft storey weakness but without transferring the problem to the stories above, since over-strengthening the ground storey would transmute the top floors more vulnerable. As a result, by applying this kind of retrofitting method such buildings convert at least as strong as those without a weak first storey.

In Figs. 6, 7 the average values of the interstorey drifts of the seven examined seismic excitations for the two investigated frames before and after retrofitting with friction devices are presented. It can be observed that the relative displacement of the upper floor is considerably less than the displacement of the previous floors. The maximum demand for interstorey drift at the ground floor is noted for both examine frames. It is obvious the remarkable reduction of the demands for interstorey drift especially to the vulnerable ground floor. The maximum interstorey drifts before and after retrofitting are also listed below.



Average Interstorey Drifts (%)			
Storey	Bare Frame	Slip Load 5%V_o	Slip Load 10%V_o
1st	1.44	0.78	0.56
2nd	0.65	0.76	0.70
3rd	0.55	0.60	0.58

Fig. 6 Average values of interstorey drifts of the first frame



Average Interstorey Drifts (%)			
Storey	Bare Frame	Slip Load 5%V_o	Slip Load 10%V_o
1^{st}	1.34	0.78	0.50
2^{nd}	0.87	0.76	0.87
3^{rd}	0.65	0.66	0.66

Fig. 7 Average values of interstorey drifts of the second frame

In Fig. 8 comparative average results of the curvature ductility requirements of all the columns of each floor separately, for both examined frames are presented. The results clearly demonstrate that the demands of the columns of the ground floors are critically increased in contrast to the upper floors. By using passive energy dissipative devices the total demand of all the floors is reduced. The reduction is more intense especially in the ground floor. In case that the slip load of the devices is determined as the 10% of the base shear greater reduction is observed.

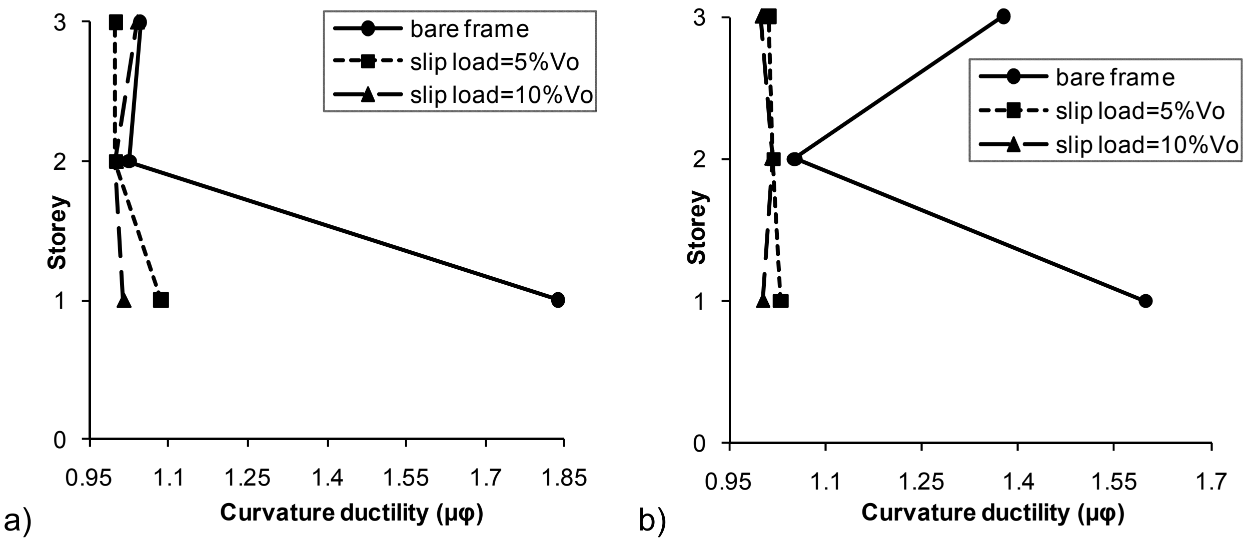


Fig. 8 Average curvature ductility of the a) first frame and b) second frame

Subsequently, individual results of one of the seven investigated seismic excitations for both structures are depicted. Specifically, hysteresis behavior, top displacements and interstorey drifts for both values of slip load of the friction devices for the El Centro earthquake are presented. Firstly, the results of the symmetric frame are presented.

In Fig. 9 the hysteresis behavior of friction devices for both percentages of base shears for the seismic excitation of El Centro are shown. From the exported results it can clearly be observed that in case that the slip load is defined as the 5% of the base shear the device displays greater values of deformations and more hysteresis loops are performed.

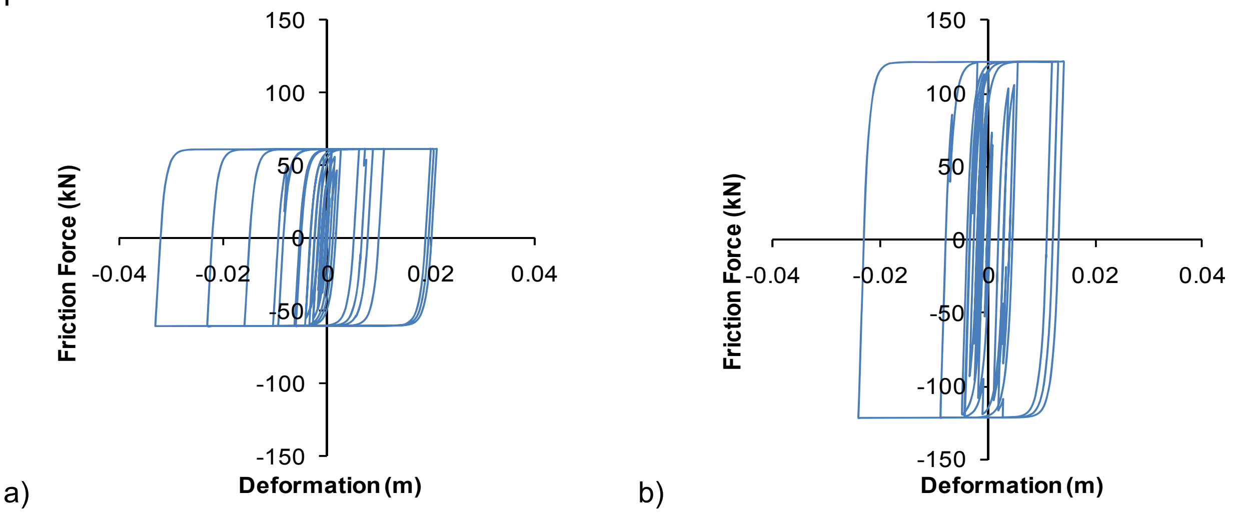


Fig. 9 Hysteresis behavior of friction damper for slip load a) 5%V_o and b) 10%V_o

In Fig. 10 the top displacement of the symmetric frame per time is shown. The maximum top displacement of the bare frame is 0.10042m. At the earthquake end the residual displacement is 0.00987m. After strengthening with friction dampers implemented in the ground floor the top displacement is decreased comparing to the initial case. Specifically, in the first case of retrofitting the maximum top displacement has reduced to 0.078m, while in the second case to 0.069m.

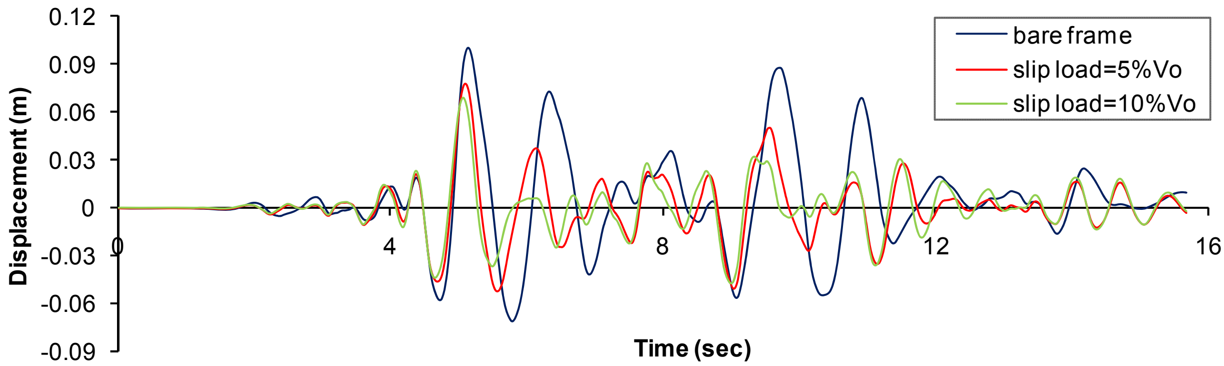
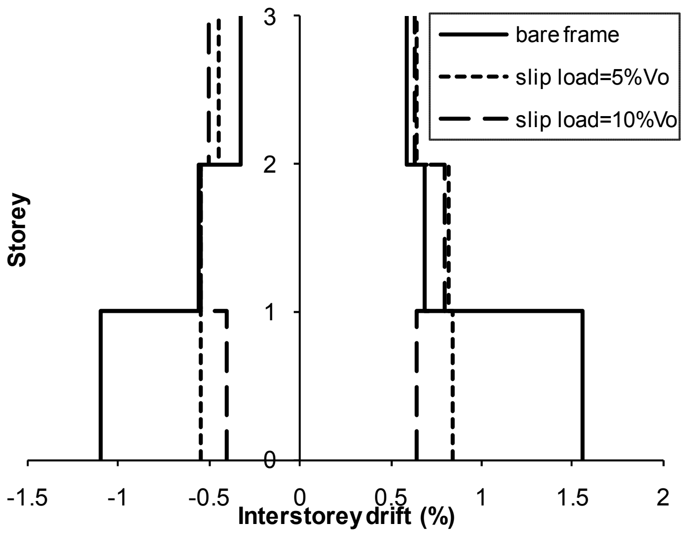


Fig. 10 Top displacement of the first frame

In addition, in Fig. 11 the interstorey drift of the first investigated frame in its initial condition and after strengthening is presented. It can be observed that the relative displacement of the ground floor is considerably larger than the displacement of the upper floors. The interstorey drift is significantly decreased with the use of friction dampers in the ground floor. The maximum interstorey drifts before and after retrofitting are also listed in the Fig. 11.



Maximum Interstorey Drifts (%)			
Storey	Bare Frame	Slip Load 5%V_o	Slip Load 10%V_o
1st	1.56	0.84	0.65
2nd	0.69	0.82	0.79
3rd	0.58	0.65	0.63

Fig. 11 Interstorey drifts of the first frame

Secondly, the results of the frame with the irregularities in elevation are presented. Specifically, the hysteresis behaviors of friction devices for both retrofitted cases for the seismic excitation of El Centro are shown in Fig. 12. From the exported results it can clearly be observed that in case that the slip load is defined as the 10% of the base shear the device displays decreased deformations comparing to the case that the slip load is defined with a less value and consequently less hysteresis loops are performed.

In Fig. 13 the top displacement of the second investigated frame per time is shown. The maximum top displacement of the bare frame is 0.1112m. At the earthquake end the residual displacement is 0.0071m which means that the vertical members of the structure have suffered from perceptible damage. At the retrofitted structures the top displacement is decreased comparing to the initial case. Specifically, in the first case of retrofitting the maximum top displacement has reduced to 0.082m, while in the second case to 0.07m.

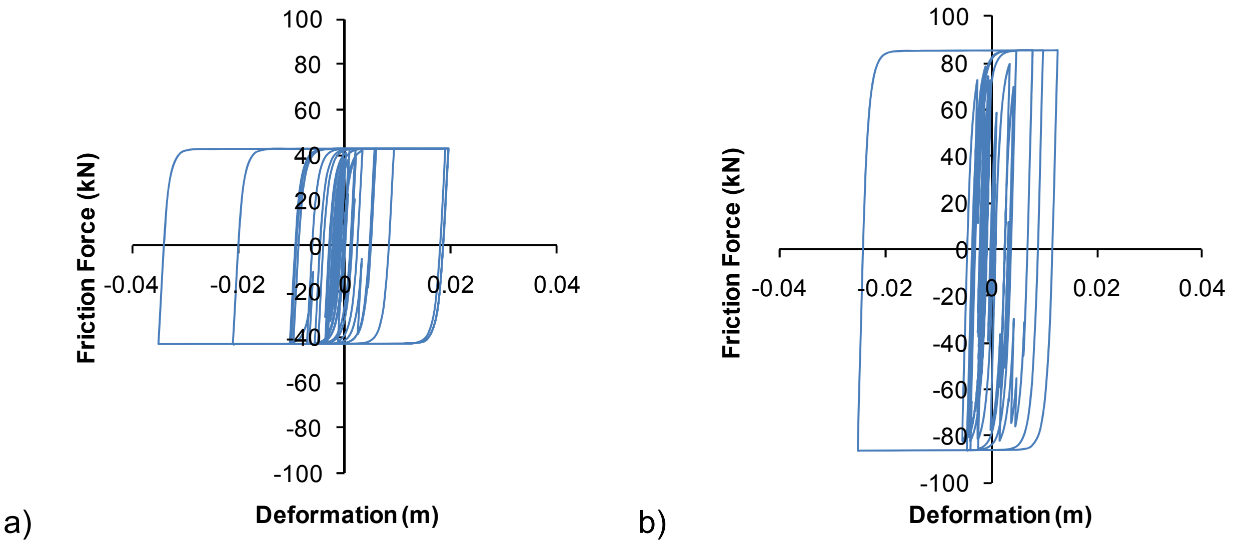


Fig. 12 Hysteresis behavior of friction damper for slip load a) 5%V_o and b) 10%V_o

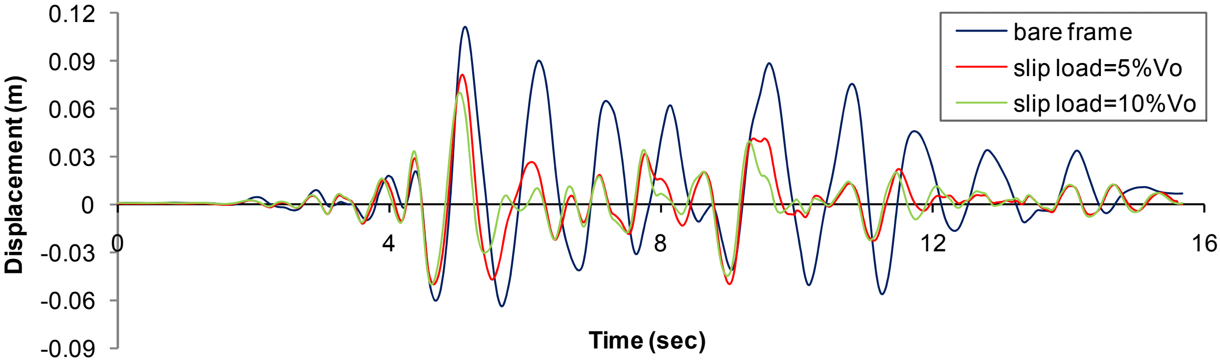
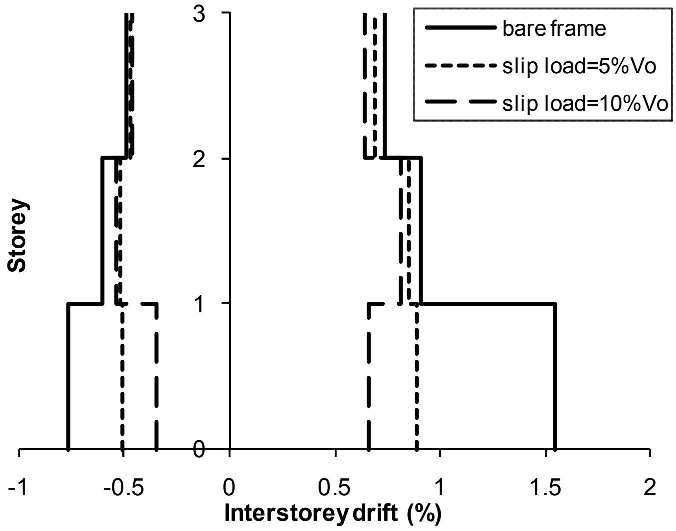


Fig. 13 Top displacement of the second frame

In addition, in Fig. 14 the interstorey drift of the second investigated frame in its initial condition and after strengthening is presented. It can be observed that the relative displacement of the upper floor is considerably less than the displacement of the previous floors. The interstorey drift is significantly decreased with the use of friction dampers in the ground floor. The maximum interstorey drifts before and after retrofitting are also listed in the Fig. 14.



Maximum Interstorey Drift (%)			
Storey	Bare Frame	Slip Load $5\%V_o$	Slip Load $10\%V_o$
1st	1.54	0.89	0.66
2nd	0.91	0.85	0.81
3rd	0.74	0.69	0.64

Fig. 14 Interstorey drifts of the second frame

6. CONCLUSIONS

In this paper the seismic assessment of two 3-strorey frame structures, one symmetric and one with irregularities in elevation, designed according to the old seismic Greek Codes (RD 1959) is performed. The seismic behavior of the buildings is evaluated using nonlinear dynamic analyses, based on the specifications of the Greek Retrofitting Code (KAN.EPE 2012). Moreover, the retrofitting of both examined frames with friction devices is investigated.

From the exported results of both structures it can be noticed that the ground floors display maximum relative displacements. Also, it is demonstrated that the ductility demands of the columns of the ground floor is intensively increased comparing to the upper floors, especially in the symmetric structure. As it is expected the majority of critical sections of the columns of the ground floor responded inelastically exceeding the Life Safety (LS) limit state and displaying serious damage. As a result, a softstorey mechanism is created in the ground floor. Also, the majority of the critical sections of the columns in the upper floors exhibit inelastic behavior whereas indicate damage of minor importance.

In order to improve the seismic response of the structures, friction dampers supported by chevron braces are installed in the two external openings of their ground floor. As a result, the stiffness of the buildings is increased and consequently the period is reduced. The interstorey drift of the ground floor is remarkable restricted for both examined frames. Moreover, a considerable percentage of the input energy is absorbed and the total damage is limited. It can be noted that the improvement of the structures' seismic response is more intense in the second case of retrofitting. In this occasion the majority of structural members' critical sections exhibit elastic behavior. The most noteworthy difference comparing to the initial frames is the reduction of the developed plastic rotation on the columns of the ground floor.

The intervention targets to improve the overall seismic response of the structures by eliminating the softstorey weakness but without transmitting the problem to the upper floors. Apparently, the vulnerability of the examined RC frames is eliminated by removing the inherent weakness for both intervention proposals.

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