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SEISMIC PERFORMANCE OF MR FRAMES PROTECTED BY VISCOUS OR HYSTERETIC DAMPERS

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ABSTRACT

This study concerns the behaviour of steel frames protected by different anti-seismic devices (dampers). Typical hysteretic and viscous dampers are arranged in three steel moment resisting frames having different dynamical features, but designed in order to accomplish determined performance objectives. The proposed devices are selected following an iterative procedure based on the use of a suitable damage functional, which has been applied in order to control the behaviour of the protected structures under a specific seismic record.

The outcomes obtained by implementing Incremental Dynamic Analyses (IDAs), carried out on the basis of seven historical records characterized by different features, allow to analyze the improvement of the structural performance due to the considered dampers and, therefore, to provide design information about their employment.

The comparison of results is carried out taking into account the dampers capacity to protect the structures from damage, the inter-storey drifts, the residual deformations and the possible amplification effects. In conclusion, the equivalent behaviour factors for each damper type are given, with the aim of providing useful design parameters for the implementation of simplified conventional linear analyses.

Keywords: Dampers, Passive Seismic Protection, Dual Steel Frames, Incremental Dynamic Analysis (IDA), Behaviour Factor.

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1 INTRODUCTION

The growing attention of designers and technicians about the use of dampers requires a continuous effort from the research side in providing design advices and indications finalized to their optimal exploitation for the protection of new and existing buildings against winds and earthquakes. Important aspects to be faced are related to the demanded performance goals under different type of inputs, in terms of maximum displacement, velocity, acceleration, ductility, vibration, failure mechanism, residual deformation, etc.

The large variety of devices proposed in the last decades (Symans et al, 2008), which can work according to several dissipative mechanisms (hysteretic, viscous, etc.) entailing different possibilities of interaction with the protected structure, pushed many researchers to investigate the dynamic response of entire buildings equipped with dampers, with the final goal of providing useful design indications and guidance rules to be adopted in codes and guidelines.

Whittaker et al. (1993) were probably the firsts to give robust provisions about the implementation of damping systems for enhancing the seismic response of building frames. They provided explicit procedures for directing the dissipation of earthquake-induced energy into the special damping devices and away from the other structural members, in order to reduce the repairing costs and the operational interruptions following even severe seismic shakings. These procedures were included in the SEAONC (Structural Engineers Association of Northern California) guidelines (1993) and enhanced in other following Documents such as the FEMA-273, the FEMA-274 (ATC 1997), in which the allowed methods of seismic analysis were also defined, and the 2000 NEHRP Provisions, in which useful indications about the implementation of both equivalent lateral forces and response-spectrum analyses for structures with supplemental damping were given (Ramirez et al., 2002; Whittaker et al., 2003).

Another crucial design issue that has been dealt with in the last years is related to the optimum arrangement of dampers into a framed structure. One of the first effective methods has been proposed by Takewaky (1997), who introduced a procedure able to retrieve a configuration of dampers able to minimize the sum of amplitudes of the transfer functions, evaluated at the un-damped fundamental natural frequency, without indefinite iterative operations. Levy and Lavan (2006) faced the problem by outlining a procedure based on the use of inter-storey performance indexes to be restricted to allowable values under realistic ground motions records. Genetic algorithms were used by Apostolakis and Dargush (2010), in order to evaluate the best solution to be adopted, in terms of strength, placement and size, for hysteretic dampers arranged in two moment resisting frames subjected to four natural ground motions. Whittle et al. (2012) compared the effectiveness of several viscous damper placement techniques for achieving determined performance objectives and evaluated the usability of the proposed methods.

In the above research field, the current paper aims to provide useful information and guidance provisions regarding the seismic behaviour of dual steel frames protected by hysteretic or viscous dampers. Several natural ground motion records, characterized by different features, are used in order to carry out incremental dynamic analyses for evaluating, on both protected and un-protected structures, the seismic responses in terms of demanded drifts, residual displacements, amplification phenomena, accelerations producing different levels of damage, ductility capacity, etc. In addition, the force reduction q-factors of the considered structures are evaluated as useful design parameters for conventional linear analyses. All the analysed frames are conceived by means of well known optimization procedures, in order to comply determined performances objectives under specific design earthquakes. This allowed to carry out a clear and critical comparison of the obtained results for the two considered energy dissipation systems, putting into evidence pros and cons of both types.

2 THE STUDIED FRAMES

2.1 General

Three steel Moment Resisting Frames (MRFs) characterized by a variable number of bays and/or storeys have been considered. These have been characterized by different dynamical features, depending on the geometry, the selected profiles and the imposed masses.

The first (see fig. 1a) has three 3.50 m length spans and four 3.00 m height storeys, with floor masses of 226 tons. The second frame (see fig. 1b) has eight storeys and the same number of bays, storey heights, bay widths and floor masses of the first structure. Both the former and the latter have been supposed to belong to buildings with perimeter moment resisting frames only in one direction, braced frames in the other, characterized by a rectangular plan of 420 m^2. The last analysed frame (see fig. 1c) has five bays and twelve storeys, with the same span dimensions of the other studied frames and floor masses of 372 tons. In this case, it has been supposed that the structure belong to a building with perimeter moment resisting frames only in one direction, braced frames in the other and a rectangular plan of 730 m^2.

It must be underlined that the choice of small span lengths is due to the fact that the proposed study intends to provide results which are not excessively influenced by the moment resisting frame deformability (P-Δ effects), which usually has a huge effect for large spans. Nevertheless, new numerical analyses on frames with more conventional span length (i.e. 7.5 m) are currently in progress in order to extend the reliability of these outcomes also to different frame configurations.

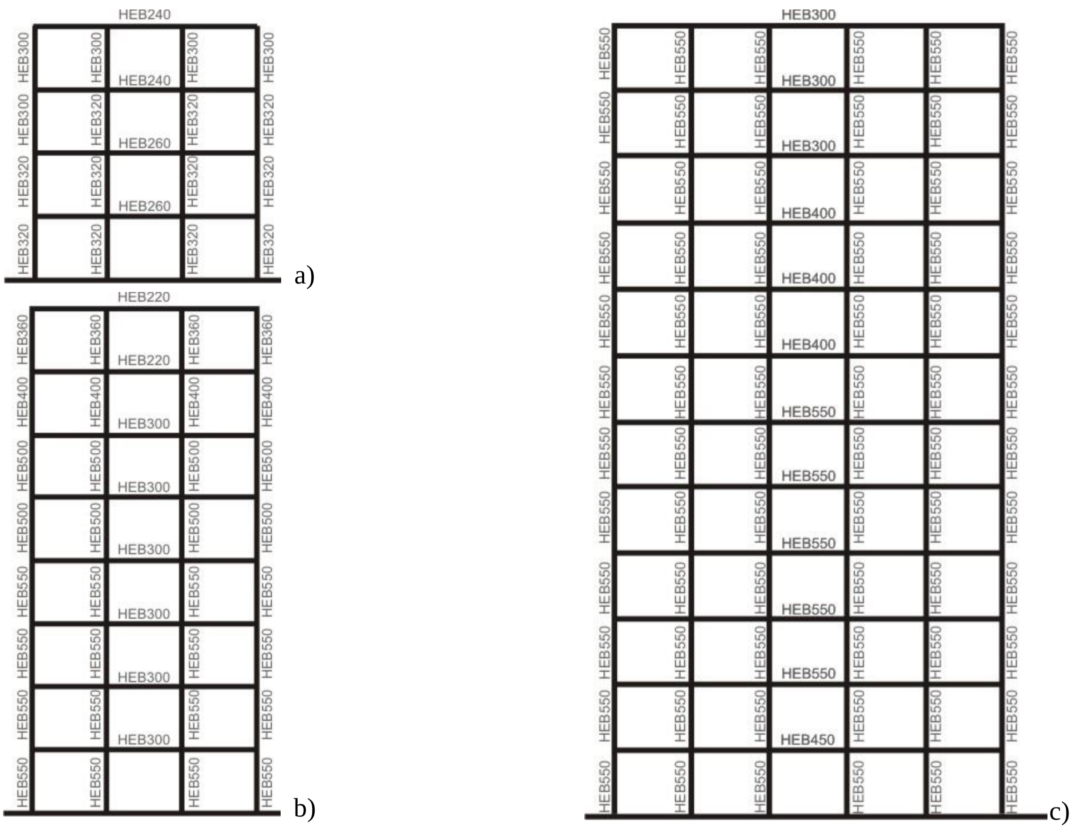


Figure 1. The 3 bays - 4 storeys a), 3 bays - 8 storeys b), 5 bays – 12 storeys c) studied frames.

Beams and columns (see fig. 1) have been sized so that, under selected design earthquakes, a maximum transient lateral drift of 2.5% is not exceeded, according to the limit proposed by the FEMA 356 (ASCE, 2000) for the life safety performance level that has to be assured for rehabilitated MRFs. In detail, the Reykjavik–Island (2000) earthquake with a maximum PGA (Peak Ground Acceleration) of 0.5g has been considered for the 3 bays–4 storeys frame, while the Hachinohe–Japan (1979) record, with the same PGA, has been applied on both the 3 bays–8 storeys and the 3 bays–12 storeys frame (see chapter 4). In addition, structural elements have been chosen in order to get a first modal participation mass ratio larger than 85%, so that their first vibration mode, characterized by an almost linear shape, prevails on the higher ones.

The above frames have been equipped in one case with viscous dampers and, in the other case, with dissipative braces, according to the design criteria that will be provided in the Section 3 of the current paper. The added elements have been selected in order that the whole structure is able to satisfy, under the same design earthquakes used for the bare MRFs, the requirements provided by FEMA 356 for braced frames, namely a maximum transient lateral drift of 1.5%. The arrangement of the proposed devices is shown in figure 2.

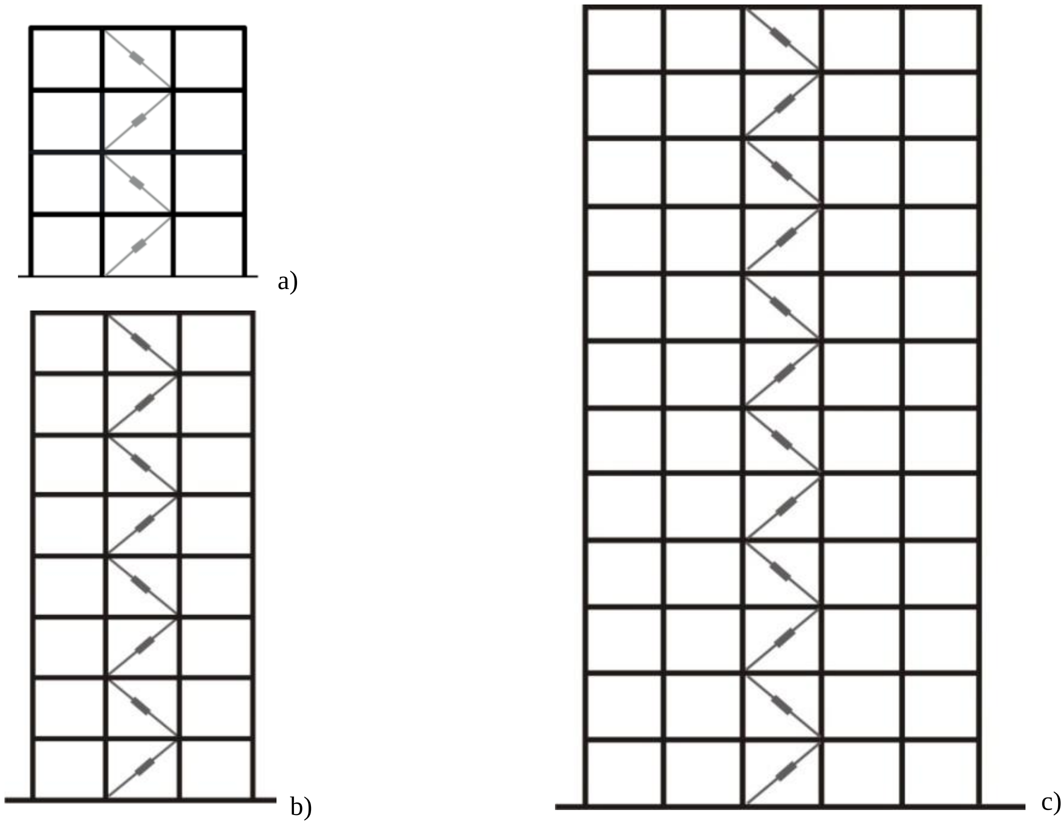


Figure 2. Dampers arrangement in the 3 bays-4 storeys a), 3 bays-8 storeys b), 5 bays–12 storeys c)frames.

Indeed, the drift limit of 1.5% has been assumed in a conventional and conservative way, due to the fact that FEMA 356 guidelines do not provide information for MR frames with dampers. A more correct value of this limit should be assessed on the basis of the expected ductility demand of the dampers, as well as on the basis of the structural demands to the other parts of the analyzed frames.

2.2 FEM modelling and analysis implementation

The proposed structures have been modelled through the Midas GEN 2010 non linear software (MIDAS, 2010). All the degree of freedoms at the base of the frames have been restrained and full strength-rigid beam-to-column joints have been assumed. Beam finite elements have been used for modelling both girders and columns and lumped inelastic hinges have been placed at both ends of the members, so to model the non linear flexural behaviour of their sections. For these plastic hinges, elatic-perfectly plastic moment-curvature relationships have been adopted and the axial force-bending moment interaction has been considered according to the AISC Provisions. This type of interaction accounts for both the in-plane and the out-of plane buckling, which have been evaluated according to the real buckling length in the frame plane, whereas in the out of plane direction these have been evaluated considering that the frame nodes are restrained with respect to horizontal translation.

As far as the damper models are concerned, "force type" uni-axial general link elements, available in the MIDAS library, have been used. These have been characterized by proper analytical relationships which have been calibrated on the basis of specific performance goals that the whole structure is imposed to respect under specific seismic demands (see Chapter 3).
For the hysteretic damper, schematically described in figure 3a, a typical Bouc-Wen (Bouc, 1963 and Wen, 1976) relationship has been adopted according to eq. (1):

$$F = r \cdot k \cdot d + (1-r) \cdot f_y \cdot z \qquad (1)$$

where F is the force developed by the damper when a deformation d arises between the two connected nodes N_1 and N_2, k is the initial stiffness, f_y is the yield strength of the damper, r is the post-yield stiffness reduction, conventionally imposed at a value of 0.02, and z is the dimensionless internal variable determining the hysteretic behaviour, whose expression is given by the following derivate (eq. 2):

$$\dot{z} = \frac{k}{F} \cdot \left\{ 1 - |z|^n \cdot \left[\alpha \cdot \mathrm{sgn}(\dot{d} \cdot z) + \beta \right] \right\} \cdot \dot{d} \qquad (2)$$

In the above equation α and β are the two parameters determining the shape of hysteretic curve, conventionally imposed equal to 0.9 and 0.1, respectively, n is the parameter controlling the sharpness of the smooth transition from the elastic to the inelastic region of the hysteresis, which has been considered equal to 2 in the current study, and $\dot{d}$ the rate of change in deformation between the nodes N_1 and N_2.
It has to be highlighted that the above model approximates the real behaviour of a hysteretic damper. As discussed by Karavasilis et al. (2012), it does not account for the isotropic hardening, which, indeed, could be of fundamental importance for some metallic devices characterized by low yield strength materials (Brando et al., 2011, 2013 and 2014; De Matteis et al., 2012). At this stage of the research, this approximation has been accepted, but it definitely represents a challenging issue to be faced in the future. In addition, further studies should regard the sensibility of the proposed models to the parameters that have been here fixed conventionally for analysis purposes.
For the viscous dampers, whose dissipative capacity is usually due to the friction generated by a piston moving inside a viscous liquid, the velocity-dependent behaviour has been described by the Maxell model (Bird et al., 1987), also provided in the MIDAS library, once that its component related to the force rate is neglected. As stated by Symans and Constantinou (1998) this last assumption is acceptable for common earthquakes, as the seismic input frequencies are usually below a cut-off limit of 4 Hz, upon which the dampers exhibit a sort of fluid viscoelastic behaviour. The structural

behaviour of the damper is therefore represented by eq. (3), whereas in figure 3.b, the cyclic damper behaviour is schematically shown.

$$F = c_d \cdot \dot{d}_d^{\alpha} \tag{3}$$

In the above equation $\dot{d}_d$ is the velocity, c_d is the viscous damping coefficient, representing the capacity of the damper to dissipate energy through viscous damping, α is the parameter which defines the shape of the dissipative cycle, rectangular or elliptical. This has been considered equal to 0.2 for the current study.

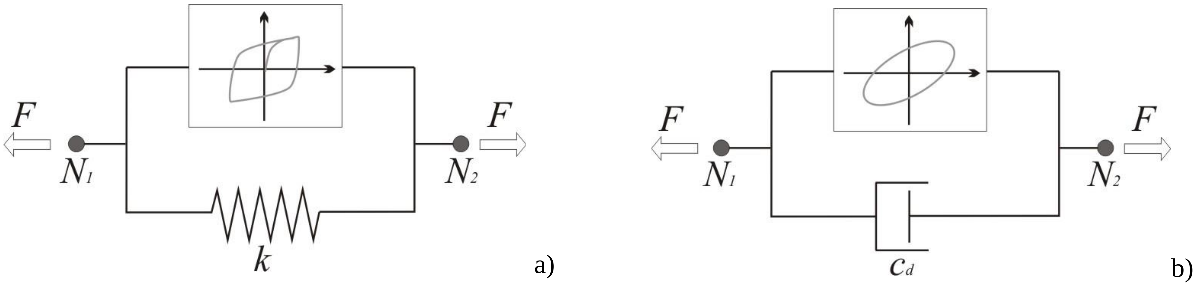
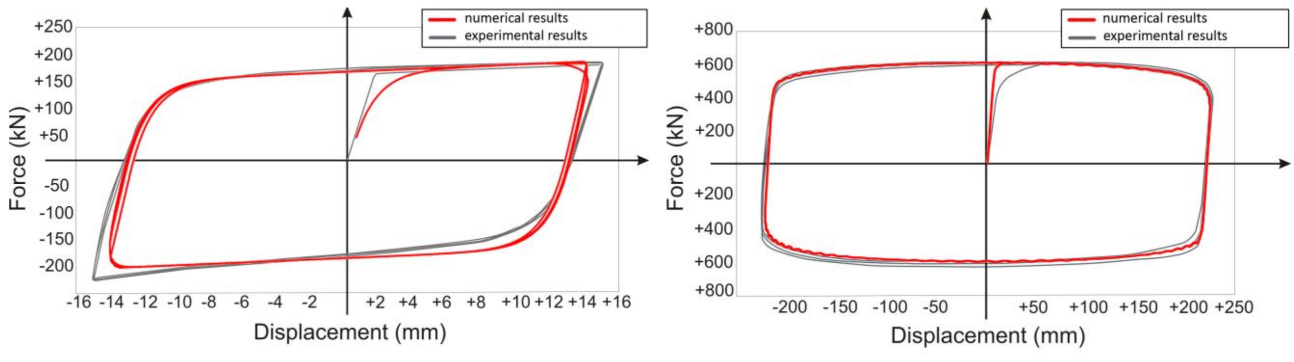


Figure 3. Schematic representation of the modeled a) Hysteretic and b) Viscous dampers.

Both the two modelling approaches described above for hysteretic and viscous dampers have been validated by reproducing experimental tests retrieved by literature. As far as the hysteretic dampers are of concern, the experimental test carried out by the company FIP-Industriale on a BRAD (Buckling Restrained Axial Dampers) system characterized by a stiffness *k* of 130 kN/mm, a yielding force of 180 kN, a post-yielding stiffness ratio *r* of 0.01, under a sinusoidal cyclic displacement of ±15 mm imposed with a frequency of 0.6 Hz., has been implemented (Castellano et al., 2009). On the other hand, the experimental test always carried out by the company FIP-Industriale on a viscous damper with a damping coefficient of 160 kN sec/mm, a damping exponent of 0.2, under a sinusoidal cyclic displacement characterized by an amplitude of ±220 mm, imposed with a frequency of 0.5 Hz., has been performed as well (Castellano et al., 2004). The comparison between the experimental and the numerical results prove the reliability of the adopted modelling technique for both hysteretic (fig. 4.a) and viscous (fig. 4.b) dampers.

The non linear dynamic behaviour of the described frames has been analysed by implementing time history analyses, following the imposition of the permanent vertical loads, according to the direct integration procedure. P-Delta effects have been also accounted for. The Newmark method has been used for the numerical solution. Iterative analyses by the Newton-Raphson method have been carried out in each time step in the process of obtaining the displacement increment until that the unbalance between the members and the external forces is minimized.



a) b)

Figure 4.Validation of the modeling techniques on experimental tests carried out by Catellano et al. (FIP Industriale, 2009 and 2004), on a Hysteretic (a) and a viscous damper (b).

Two different "families" of natural records have been selected. The former consists in the seven time histories, normalized to the relative peak ground acceleration, depicted from figure 5.a to figure 5.g. They have been applied on the 3 bays-4 storeys frames. The second set of records is also made of seven time histories, which are shown from figure 5.f to figure 5.n. They have been applied on the higher frames (3 bays-8 storeys and 5 bays-12 storeys), due to the fact that they present higher spectral accelerations for lower frequencies with respect to the records belonging to the first group, as it can be observed by the analysis of the 5% damped spectral shapes depicted in figure 6.

3 PROCEDURE FOR DEVICE SELECTION

The devices used for increasing the performance of the proposed frames are strongly different among each other, as their behaviour and activation modes are originated by different resisting mechanisms. For this reason, in order to define comparable solutions, common target performances of the protected frame have been imposed for their design. In particular, both the viscous and the metallic dampers features have been chosen in such a way that the storeys where they are arranged, when subjected to the time history used for the design of the bare frames, provide the same prefixed value of the following Damage Index D_{PA} given in eq. (4), i.e. the Park & Ang functional (1985):

$$D_{PA} = \frac{x_{\max}}{x_{u,mon}} + \beta \frac{E_h}{F_y x_{u,mon}} \tag{4}$$

where:

x_{max} : is the sum of both maximum and minimum inelastic displacements of the frames;

$x_{u,mon}$: is the maximum allowable value of x_{max}; it can be get according to a certain imposed performance objective of the whole structure when this undergoes one hysteretic cycle only;

it depends on the target displacement, which, according to FEMA 356, for braced structure is 1.5% ($x_{u,max}$=3%) in its transient aliquot;

β : is a damage parameter (for steel frames β=0.025);

E_h : is the dissipated energy;

F_y : is the maximum force that the structure can resist.

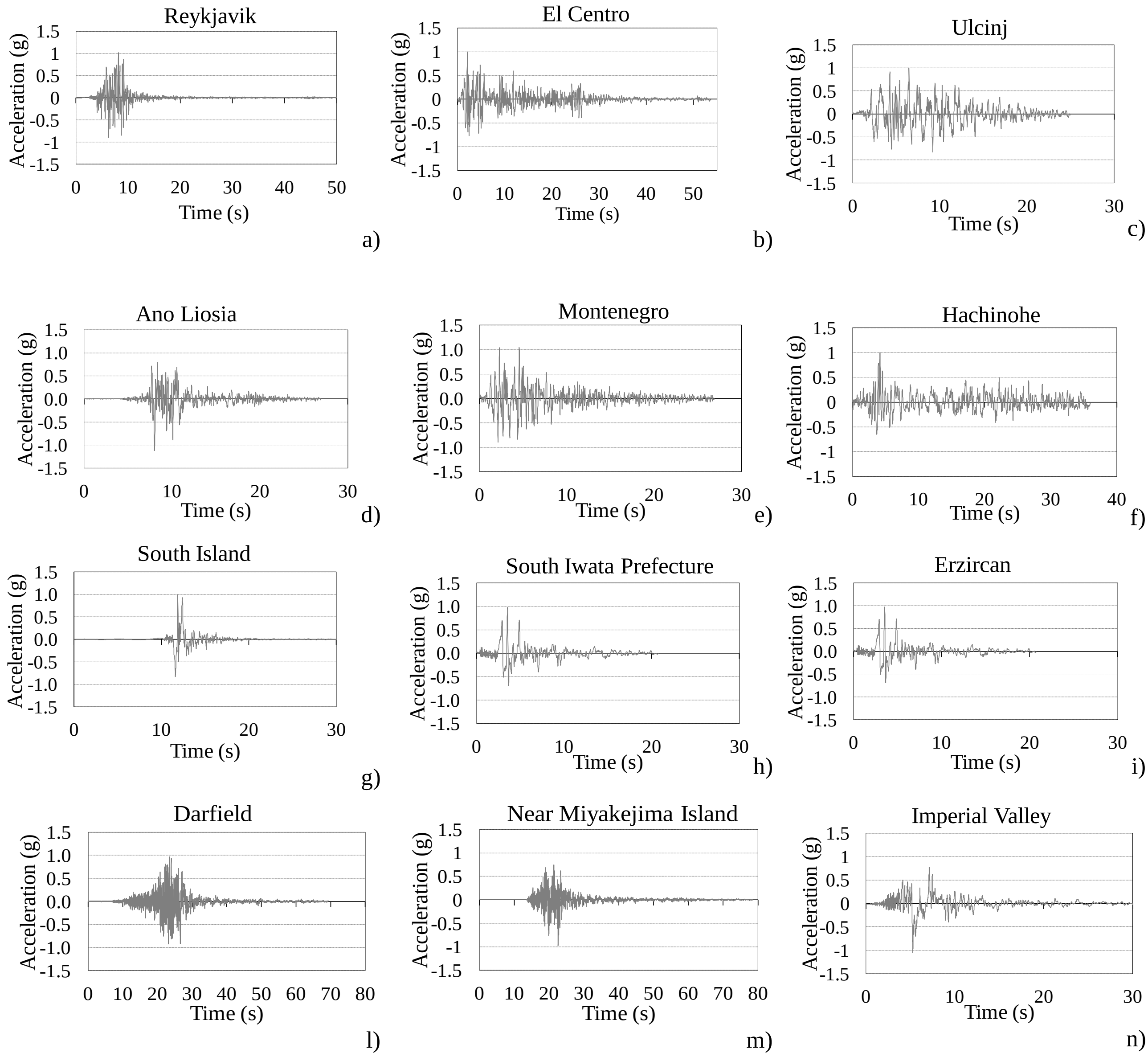


Figure 5. The normalized natural records applied to the 3 bays-4 storeys frames (from a to g) and to the 3 bays-8 storeys and 5 bays-12 storeys steel frames (from f to n)

The D_{PA} index is usually defined to describe the damage level of a single structural member, beam or column, taking into account both the dissipated energy and the maximum deformation undergone. In particular, this damage functional assumes the value 0 in absence of plastic phenomena and the value 1 at the achieving of the prefixed performance objective in the non linear field (Cosenza and Manfredi, 1998).

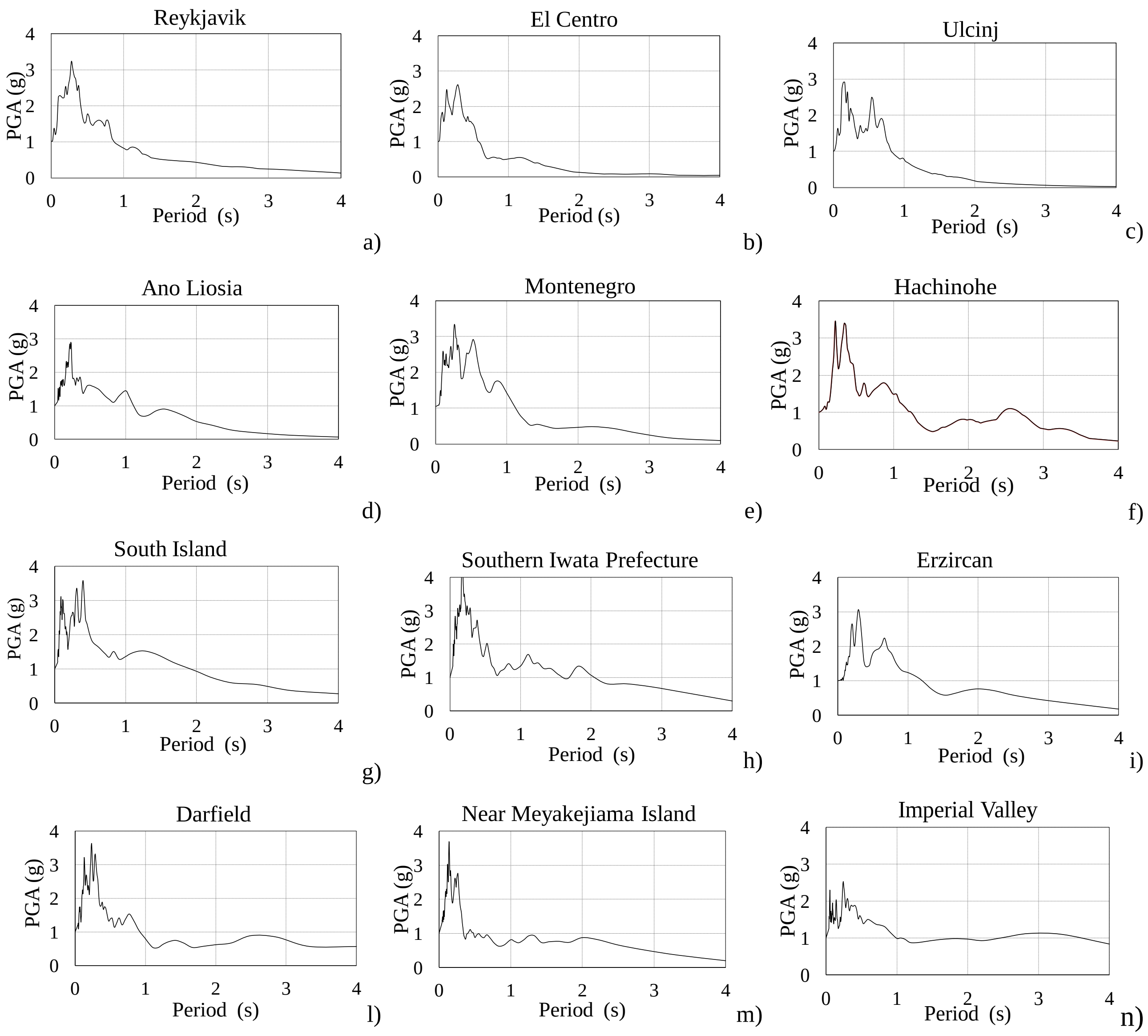


Figure 6. The 5% damped spectral shapes of the natural records applied to the 3 bays-4 storeys frames (from a to g) and to the 3 bays-8 storeys and 5 bays-12 storeys steel frames (from f to n)

In the case being, as the studied frame is a multi degree of freedom system, the Ang & Park functional has been considered in order to describe the behaviour of each storey of the frame. In this way the features of each damper have been optimally set so to retrieve back a damage index equal to 1. In order to achieve this result, an iterative procedure has been applied. It is based on the rational variation of a representative parameter P of the behaviour of the used damper and, therefore, on the consequent variation of the D_{PA} index.

For the hysteretic devices, the parameter P is the stiffness, whereas it is the damping coefficient for viscous dampers. Therefore, at the generic storey, a starting value of P is chosen consistently with certain design criteria. Once the corresponding D_{PA} index is found by a non linear time history analysis based on the design earthquake, if it is different by 1, a new value of P is chosen by multiplying the previous one by the corresponding performance index raised to a convergence factor

ζ; then the analysis is re-run and the procedure repeated up to the attainment of $D_{PA}=1$ at each storey. As proposed by Levy and Levan (2006) for non linear analyses, the convergence factor ζ has been imposed equal to 5 and this allowed to lead to a rapid convergence of the design procedure, which is schematically synthesized in figure 7.

As far as viscous dampers are concerned, the adopted design procedure for the choice of the initial value of the damping coefficients ($c_{di(1)}$) at the generic storey *i*, is the one proposed by Levy and Levan (2006). Regarding the hysteretic dampers, the procedure proposed by Mazza and Vulcano (2008) has been applied to select the initial value of damper stiffness.

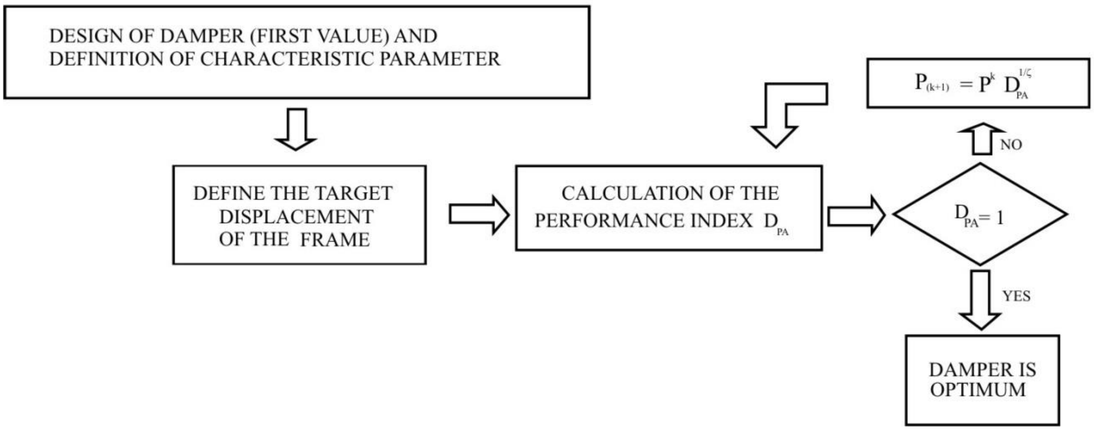


Figure 7. The applied optimization procedure.

The obtained values of the damper features are listed in tables 1, 2 and 3, for the 3 bays–4 storeys frames, 3 bays–8 storeys frames and 5 bays–12 storeys frames, respectively. It must be highlighted that some preventive analyses carried out on the three protected structures, under sinusoidal acceleration characterized by the same frequencies of the protected structures, allowed to state that the designed devices produce an increase of the equivalent viscous damping coefficient of the whole structures ranging from 5% to about 50%.

Table. 1. Optimized dampers features for the 3 bays- 4 storeys frame.

	Hysteretic Dampers		Viscous dampers
STOREY	Stiffness (kN/m)	Yielding Force (kN)	Damping coefficient (kN s/m)
1	438977	2701	12288
2	423471	2605	9005
3	372371	2290	6488
4	177335	1091	500

Table. 2. Optimized dampers features for the 3 bays- 8 storeys frame.

STOREY	Hysteretic Dampers		Viscous dampers
	Stiffness (kN/m)	Yielding Force (kN)	Damping coefficient (kN s/m)
1	28337	174	136
2	75022	462	10772
3	142884	879	23861
4	115874	713	22658
5	80150	493	10676
6	38612	238	2582
7	46042	283	2801
8	44620	275	748

Table. 3. Optimized dampers features for the 5 bays- 12 storeys frame.

STOREY	Hysteretic Dampers		Viscous dampers
	Stiffness (kN/m)	Yielding Force (kN)	Damping coefficient (kN s/m)
1	117745	724	2157
2	232984	1434	26652
3	277086	1705	35313
4	249360	1534	20567
5	290184	1785	16686
6	389977	2399	29828
7	728200	4480	53343
8	516329	3177	40494
9	473079	2911	18068
10	285132	1754	2184
11	258806	1592	62
12	73476	452	1

4 OBTAINED RESULTS

4.1 General

IDAs (Incremental Dynamic Analyses) have been carried out on both the protected and the un-protected frames. To this purpose all the records shown in Chapter 3 have been scaled considering ground accelerations ranging from 0.1g to 1.0g. Based on such results, the variation of the main structural response parameters is analysed in the following.

4.2 Damage distribution

Figure 8 shows the first plastic hinges produced by the imposed records on the members of both the protected and un-protected 3 bays-4 storeys frames (on the left), as well as the damage distribution corresponding to the observed incipient collapse mechanisms (on the right). For each situation, the corresponding ground accelerations are also provided.

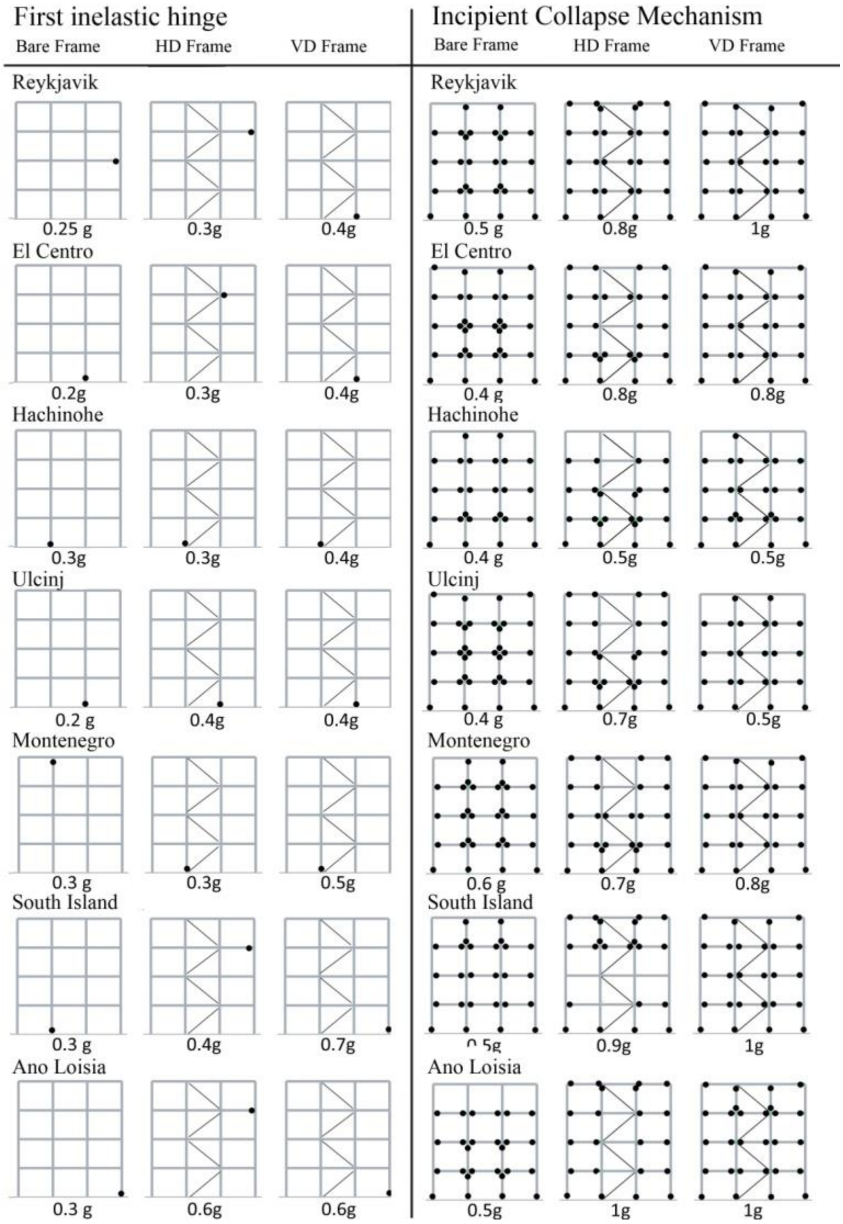


Figure 8. Damage distribution (on the left) and collapse mechanisms (on the right) on the 3 bays-4 storeys frames.

As an average, the design procedure described in Chapter 3 allowed to retrieve dampers features able to increase almost twice the values of the ground accelerations of the bare structures provoking the arising of the first damage. Then, the considered dampers allowed to fully satisfy one of the main requirements for which they are usually applied, i.e. postponing the damage of beams and columns. Only some exceptions have been registered, because of the higher structural demand produced by some specific earthquakes (Hachinoe and Montenegro earthquakes) with higher amplifications in the period range where the protected structure with hysteretic dampers fails due to the increased stiffness. In these cases, the higher axial forces on the columns produced an anticipated damage. On the contrary, the application of viscous dampers always entailed a better performance with a large amount of energy dissipated.

Similar considerations can be outlined also with reference to the PGA provoking incipient collapse mechanisms of the structure. However, it must be observed that the lack of a global collapse mechanism in case of some protected structures leaves to suppose that a design process explicitly accounting for the capacity design criteria of the protected frame could produce even better results.

The above results have been also observed for both the 3 bay-8 storey and the 5 bay-12 storey frame. As a matter of example, in figure 9 the results related to the Imperial Valley and the Hachinohe earthquakes are represented.

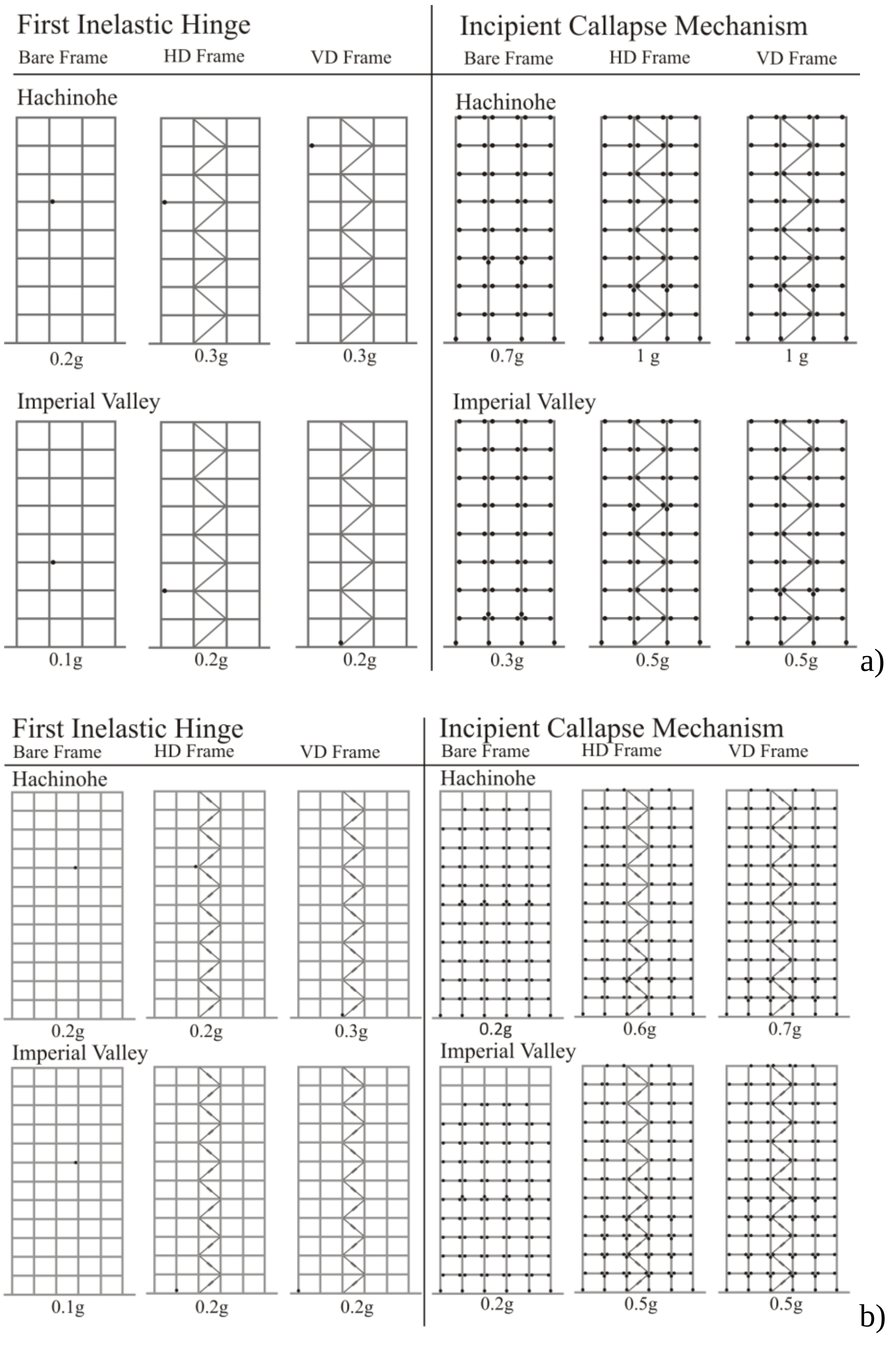


Figure 8. Damage distribution (on the left) and collapse mechanisms (on the right) on the a) 3 bays-8 storeys and b) 5 bays-12 storeys frames. Effects produced by the Hachinoe and the Imperial Valley earthquakes..

In these cases it has been observed that, although no capacity design criteria were explicitly applied in the design phase, the application of the selected dampers allowed to get collapse mechanisms conveniently involving also the members of the higher storeys, which, on the contrary, did not result damaged at the collapse of the bare frame.

4.3 Storey drifts

Storey drifts represent the most important parameter to be analyzed, as they are strictly connected to the damage suffered by both structural and non-structural elements. Thus, the possibility of reducing the lateral drift at each storey of a building entails minor costs for rehabilitating its functionality after a strong seismic shaking.

In figure 10, 11 and 12, the average of the measured storey drifts, for peak ground accelerations ranging between 0.1g and 0.7g, are given. Beyond this range of accelerations, global instabilities or detrimental effects on the ductility capacity of some plastic hinges have been evidenced for some analysed bare frames, this making un-useless the comparisons between the revealed performances. On the contrary, the frames equipped with dampers provided a stable structural behaviour.

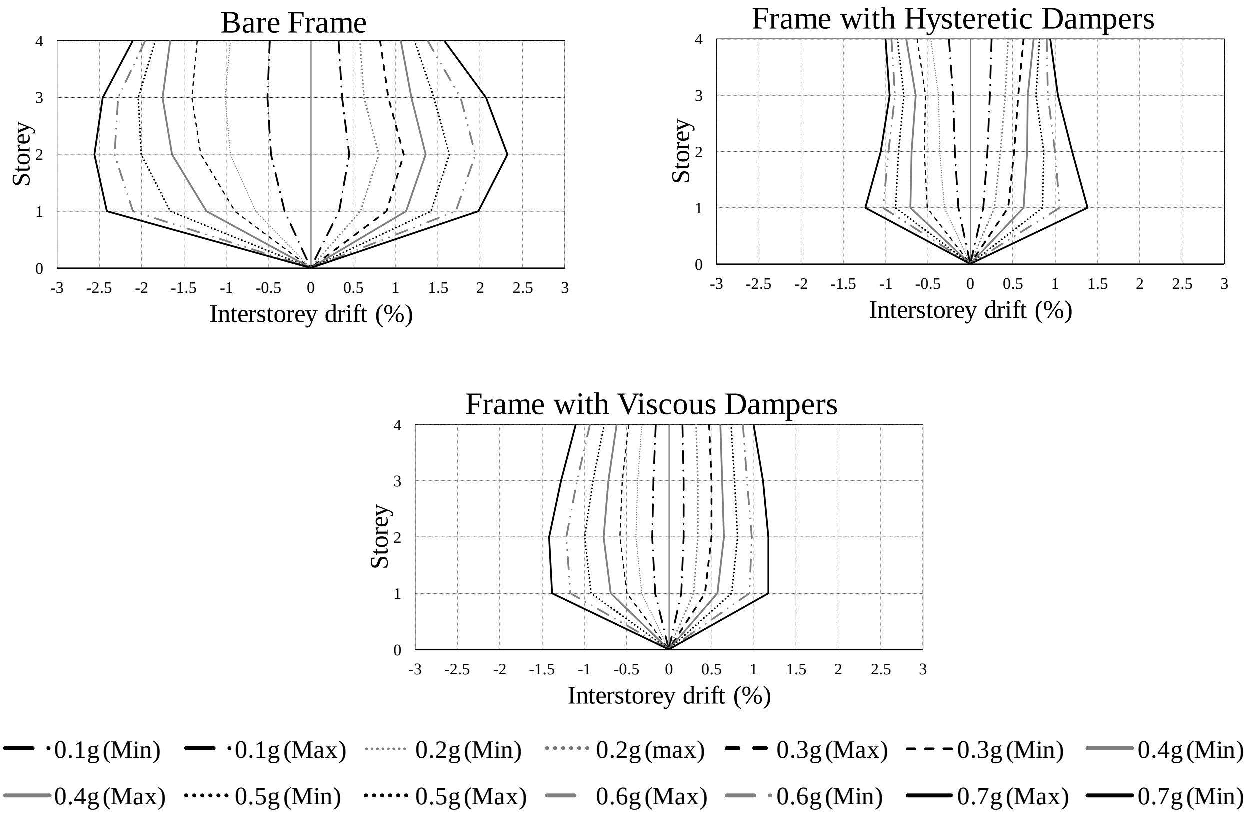


Figure 10. Average of the storey drifts on the 3 bays- 4 storeys frames.

It is evident that the application of the proposed dampers allows to get remarkable reductions (about 50%) of demanded drifts at each storey, consistently with the required performances established during the design phase.

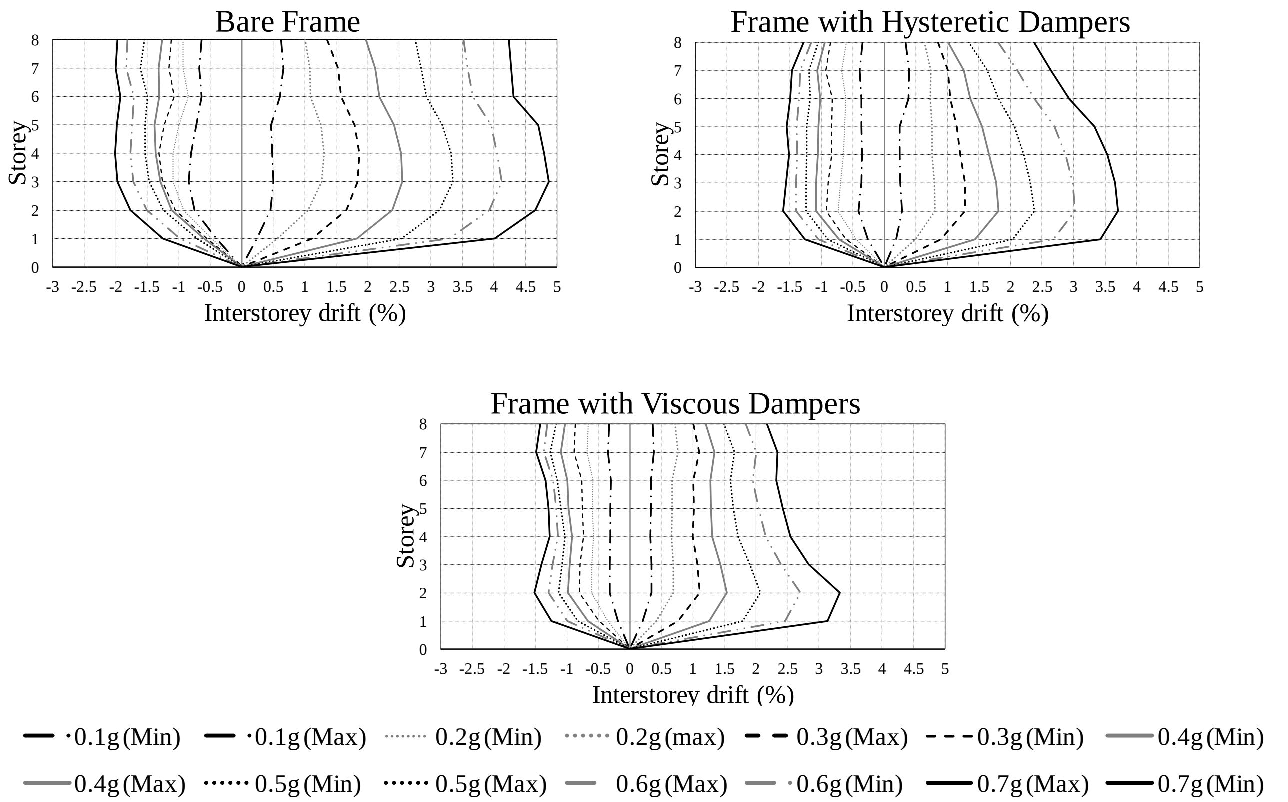


Figure 11. Average of the storey drifts on the 3 bays- 8 storeys frames.

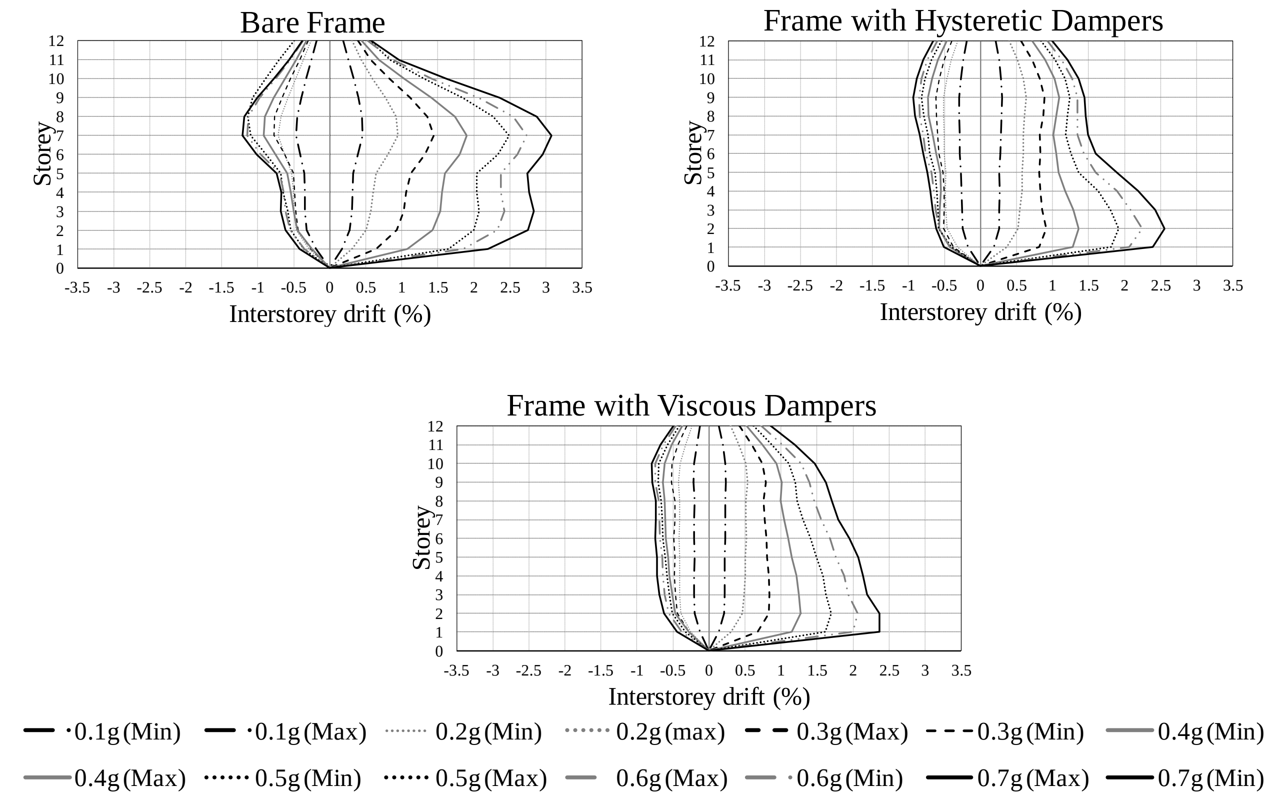


Figure 12. Average of the storey drifts on the 5 bays- 12 storeys frames.

Moreover, the proposed results put in evidence that, at least for reduced ground accelerations, as an average, the protected structures are able to develop a uniform drift distribution along the height, as well as to provide a symmetrical response, correcting the behaviour exhibited by the bare frames.

4.4 Residual drifts

The permanent deformations provoked by strong earthquakes could be so large, particularly for very deformable structures, that the buildings restoration may result not convenient in comparison with a full reconstruction. For this reason, although the great part of Codes does not give explicit prescriptions on this issue, the residual drifts control is an important goal that the use of dampers could allow to reach.

In figures 13, the average residual drifts, obtained for a peak ground acceleration of 0.9g, excluding those bare frames affected by global instabilities or hinges failures, are depicted. Although the design procedure has been addressed only to the achievement of specific reductions of storey drifts, the applied dampers have been able of producing significant reduction of residual deformations, namely from 1.0%-1.5% registered for bare frames to values of about 0.5%. Obviously, this type of effect is more evident for the case of frames with hysteretic dampers, due to the restoring forces they have been able to transmit, but it is noticeable that also the use of viscous devices can positively contribute in this sense due to their damping capacity, as well as to the fact that they did not lead to amplification phenomena of the seismic input.

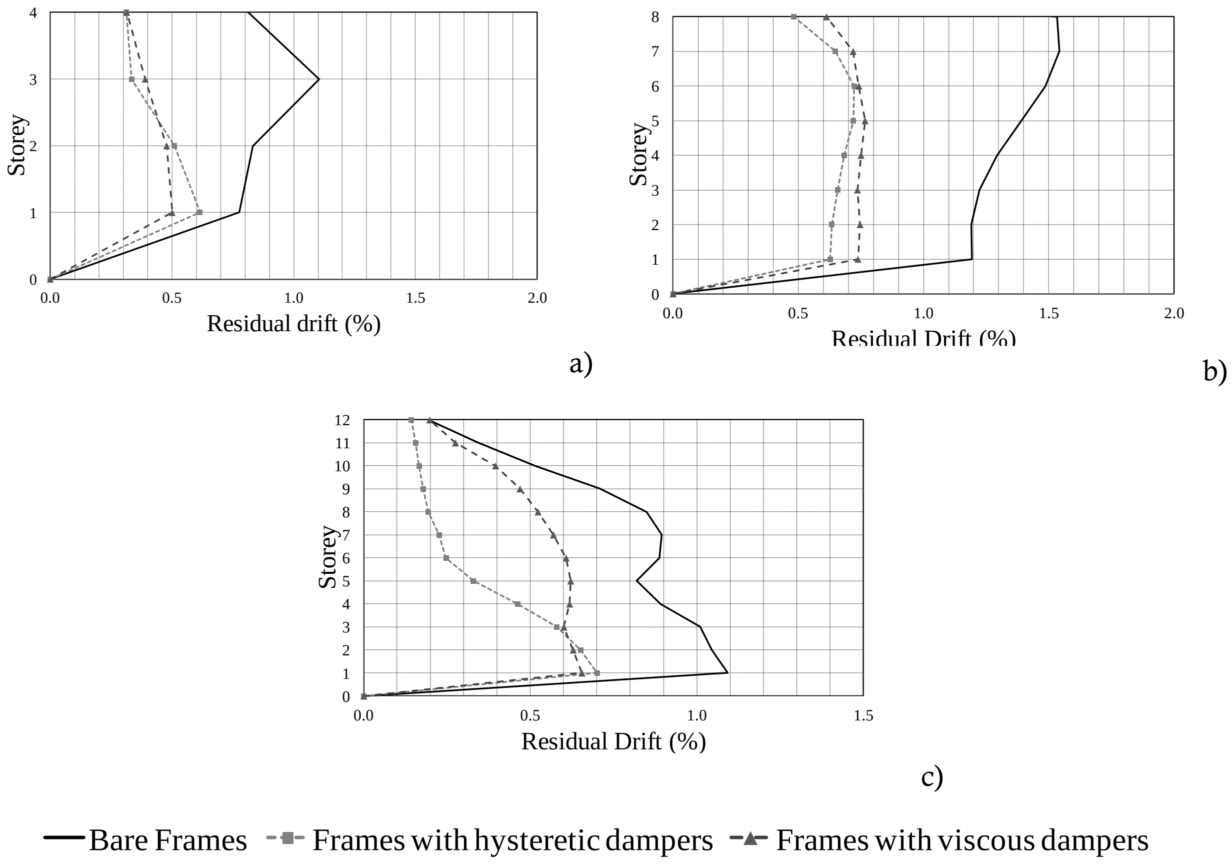


Figure 13. Residual drifts (average) measured, for a PGA of 0.9g, on the a) 3 bays- 4 storeys, b) 3 bays- 8 storeys and c) 5 bays- 12 storeys frames.

4.5 Roof accelerations

The accelerations provoked by an earthquake on the top of a structure represent another important parameter for evaluating the structural performance. In fact, if uncontrolled amplification phenomena arise, the building functionality could be compromised, especially when strategic structures like hospitals or industries are of concern.

In figure 14 the roof accelerations registered for each frame, averaged on the seven imposed records, are expressed as a function of the peak ground acceleration. For low rise buildings, the application of hysteretic dampers provoked, with respect to the bare frames, amplification phenomena because of the induced reduction of periods due to the increased stiffness. On the contrary, viscous dampers lead to roof accelerations that are lower than the ones measured for bare frames. However, when the height of the frames increases, these types of discrepancies are less important: the 8 storey structure with hysteretic dampers presents the same accelerations of the bare frame, whereas the frame with viscous devices is characterized by slightly lower accelerations.

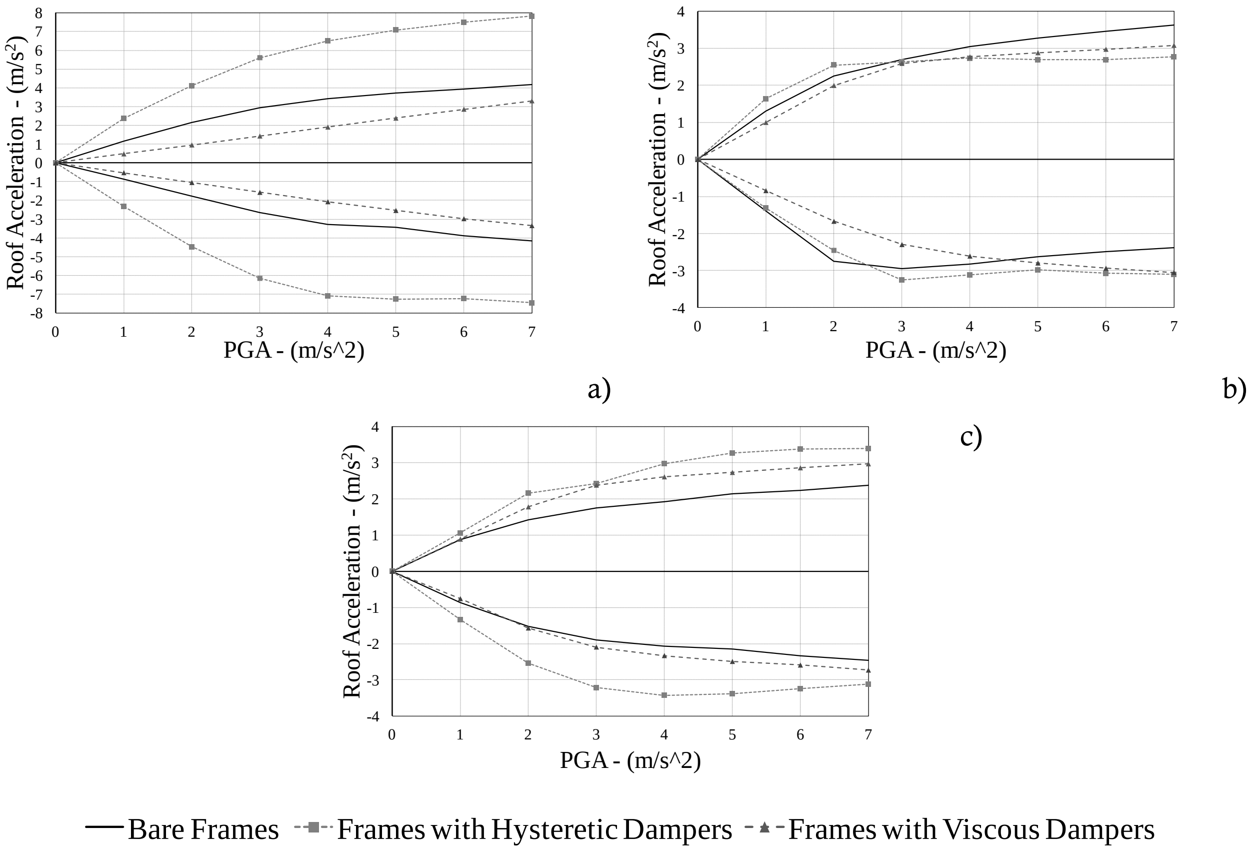


Figure 14. Roof vs. Peak Ground accelerations (averaged on the imposed records): a) 3 bays- 4 storeys, b) 3 bays- 8 storeys and c) 5 bays- 12 storeys frames.

5 *q*-FACTOR EVALUATION

As evidenced by the above outcomes, the application of seismic protection systems determines a deep variation of the structural inelastic response with respect to the bare frame. Linear analysis should contemplate such an effect by a proper seismic reduction factor, namely the q behaviour factor. For the studied frames, in both protected and un-protected configurations, this factor has been evaluated by means of two different procedures.

The former corresponds to the Setti method (Setti, 1985), which is useful for frames able to develop global collapse mechanisms and characterized by a dominant first vibration mode. On the basis of the outcomes obtained by the IDAs for each earthquake, the maximum value of the roof displacements (d), normalized to the displacement able to produce the first yielding on the structure (d_y), is put in relation to the relative peak ground accelerations (a), normalized to the acceleration (a_y) retrieving d_y. In this way, the so-called push-over dynamic curve can be obtained. The q-factor given by each record is expressed as the minimum a/a_y ratio among those giving back one of the following conditions: *i*) attainment of a limit interstorey drift (established, in the case being, as 2.5% and 1.5% for the braced and the bare frames respectively) or *ii*) loss of global stability for the whole frame. In particular, the a/a_y ratio corresponding to the last situation has been detected by the intersection of the dynamic pushover curve plotted in the (d/d_y, a/a_y) datum plane and the bisector of the datum plane itself, it representing the ideal elastic behaviour of the system (Fig. 15a). In fact the points (d/dy, a/a_y) belonging to the region upon this bisector represents unstable structural responses.

The second methodology has been based on an energy approach, according to Como and Lanni (1983), which express the behaviour factors as eq.(5):

$$q = \sqrt{\frac{W_U}{W_y}} \tag{5}$$

where W_u is the total energy inputted into the structure during the earthquake and W_y is its elastic aliquot according to figure 15b.

The results obtained by the above procedures are listed in tables 4, 5 and 6. In such tables, in some cases, the q-factor values have been not attributed when the Setti Method resulted un-applicable due to the fact that the trigger of local plasticity had significantly averted the frame behaviours from the response of a single degree of system, to which the Setti Method is referred.

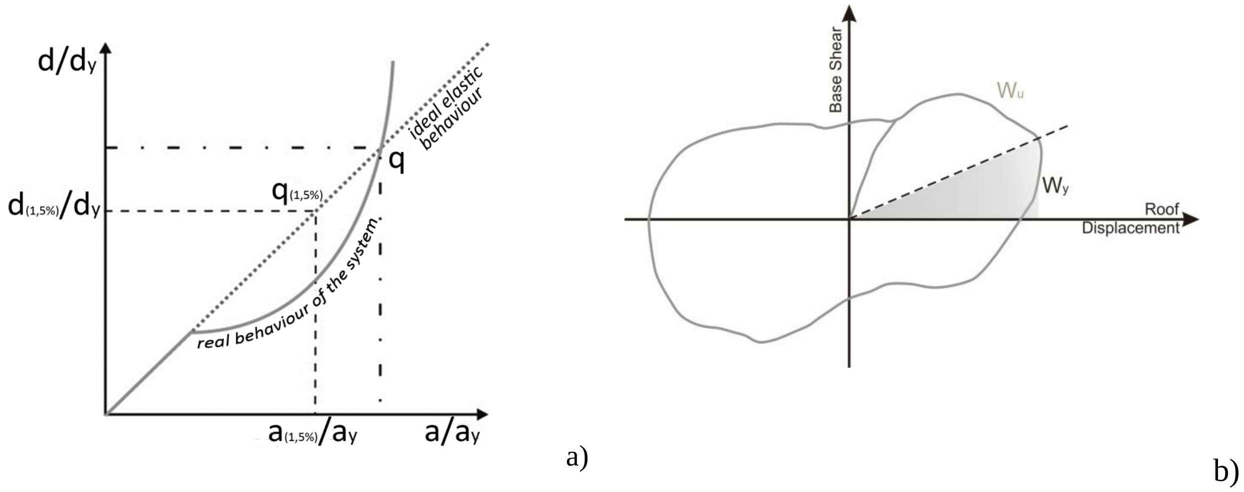


Figure 15. The approaches used for the evaluation of the *q* behaviour factor: a) Setti Method; b) Energetic Method.

It is evident that, apart from some specific cases, the application of the dampers is able to increase significantly the *q*-values with respect to the bare frame, and that the viscous damper systems often allows to retrieve more performing results.

Table.4. q-factors for the 3 bays–4 storeys frames

	Bare frames		**HD Frames**		**VD Frames**	
	Setti Method	Energy Method	Setti Method	Energy Method	Setti Method	Energy Method
Reykjavik	4.5	5.39	6.5	5.06	8	9.46
El Centro	2.5	5.33	6	6.4	8	6.06
Montenegro	3	3.94	6	5.77	-	6.9
Ano Liosia	5	5.13	-	4.64	-	4.76
South Isalnd	2.5	5.33	5.2	4.13	-	6.38
Ulcinj	3	5	-	5.52	-	4.76

Table.5. q-factor of the 3 bays–8 storeys frames

	Bare frames		**HD Frames**		**VD Frames**	
	Setti Method	Energy Method	Setti Method	Energy Method	Setti Method	Energy Method
Hachinohe	3	5.4	3.5	6.39	-	6.55
Darfield	3.5	2.95	4.5	3.11	-	3.38
Erzircan	3.6	2.8	4.5	4.26	4	4.75
Imperial Valley	1.9	1.84	2.5	5.81	2.6	2.75
Mijakejima Island	3.25	2.09	4.5	2.18	-	2.38
South Island	3	2.95	4.5	3.5	3.8	3.4
Southern Iwata	1.4	3.09	2	3.97	-	4.5

Table.6. q - factor of the 5 bays–12 storeys frames

	Bare frames		HD Frames		VD Frames	
	Setti Method	Energy Method	Setti Method	Energy Method	Setti Method	Energy Method
Hachinohe	4.5	4.46	6.5	6	6	6.07
Darfield	2.9	2.44	5.5	3	4	3.11
Erzircan	3.5	3.82	6.5	4.3	4.6	3.95
Imperial Valley	3	2	2	2	2	2.1
Mijakejima Island	3.92	2.3	4.5	3.93	4.5	3.9
South Island	5	2.8	6.5	4.15	9	4.14
Southern Iwata	3.5	3.92	4.5	5.13	-	5.42

6 CONCLUSIONS

In this paper, the seismic behaviours of dual steel frames protected by special devices have been analyzed. Three different frames and two (Hysteretic and Viscous) damper typologies have been considered. All the studied structures have been designed under specific earthquakes, according to a suitable design iterative procedure based on the use of the Ang and Park functional and imposing, for each storey, limit transient drifts of 2.5% and 1.5%, for the bare and the protected frames, respectively.

Incremental dynamic non linear analyses have been carried out on both the un-protected and the protected structures, this allowing to compare the main performance parameters with the aim of providing useful design indications.

Provided that the given results strongly depend on the applied design methodologies, the following main conclusions have been reached:

- with respect to the bare structures, the use of the proposed dampers allows of almost doubling the ground accelerations provoking the arising of the first damage phenomena on the other frame members, as well as the collapse of the whole frames. This result seems to be independent by the earthquake features when viscous dampers are employed. On the contrary, for low rise frames with hysteretic devices, some damage phenomena could be anticipated, with respect to the solutions with viscous devices, due to the increased stiffness of the whole structure;
- although the proposed design procedure does not comply explicit capacity design criteria, the considered dampers, when applied to medium-high rise buildings, allows to get global collapse mechanisms even in those situations in which the bare frames does not present this type of behaviour. This statement is not applicable to low-rise buildings.

- the adopted dampers, at least for not excessive seismic demands, have allowed to control the storey drift development, as well as to halve the residual drifts provoked by destructive earthquakes on the bare frames;
- the application of the protection devices allows to significantly increase the q-behaviour factors to be used in conventional linear analyses, with the viscous damper systems often retrieving more performing results.

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