EARTHQUAKE RESISTANT DESIGN NEEDS STRUCTURAL CONTROL

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ABSTRACT :

The development of earthquake engineering in the past was very much dependent from the information gathered for the behaviour of the structures during real earthquake ground motion. Analysing on a long term the effect of the gathered information it is evident that the scientists were recommending that the earthquake design spectra in the codes have to be increased. The consequences of this approach will be that the structures in the future will be more rigid, as well as heavier and during real strong ground motion will generate higher inertia forces and as a consequence of that we may expect again high level of vulnerability. Contrary to this approach is the structure to be designed as a flexible one and for controlling of deformations special devices to be added on the structure by which its dissipation capacity will be increased and deformations will be decreased. To demonstrate this concept of structural design a hypothetical five story steel frame structure is designed for ground acceleration of 0.2g and 0.4g based on EC 3 and EC 8 regulations. Analyses have been performed varying the viscous damping of the dampers from 10-30%, as well as increasing the peak ground motion up to 0.6 g. In this paper will be presented a set of data by which it will be demonstrated that by adding of devices at the floor levels more advanced and safer structure could be designed and the response of the structure exposed to time dependent loads could significantly be improved.

KEYWORDS: steel structures, controlled structures, added damping

1. INTRODUCTION

The behaviour of structural systems when responding to dynamic loads is mainly associated with their ability to dissipate the kinetic and the potential energy through hysteretic and viscous mechanisms of the structures. Vibration of structures and associate forces could be reduced and controlled through damping of the structure. The damping compensates for structural nonlinearity through which the external energy should be dissipated (absorbed). Also, the effect of damping can be has influence on the vulnerability of the structure, particularly that of the non-structural elements, which means that the overall cost for retrofitting is also decreased.
In general, the damping in steel structures consists of inherent-viscous damping, which is usually low (1%-5% of critical), hysteretic damping through nonlinear behaviour of structural elements and damping as a result of added different energy dissipation systems.
According to Eurocode 8 (EC8) requirements, earthquake resistant steel buildings shall be designed in accordance of two concepts: a) low-dissipative structural behaviour and b) dissipative structural behaviour. In concept a), the action effects may be calculated based on an elastic global analysis without taking into account a significant non-linear material behaviour and this concept is recommended for designing of steel structures in low seismicity regions. In concept b) the capability of parts of the structure, so called “dissipative zones”, to resist earthquake actions through inelastic behaviour is taken into account. Structures designed in accordance with this concept belongs to structural ductility classes medium or high, which correspond to increased ability of the structure to dissipate energy in plastic mechanisms of the main structural elements.

Contrary to this approach, for proper seismic design, the amount of hysteretic energy dissipated by the structure has to be minimized, which means that additional damping has to be introduced in the structure.
Possibility of introduction of additional energy dissipating mechanisms into the structure, either passive or semi-active, which should be designed to consume a portion of the input energy, reduces the damage to the main structure caused by hysteretic dissipation. What should be the total damping capacity of these devices and their contribution in the overall effective damping of the structure in order to meet the EC8 design requirements, such as total top displacement, inter-story drifts, stress level in the main columns, beams, diagonal elements etc. represents a crucial question.
In this paper is presented a set of data by which it will be demonstrated that by adding of devices at the floor levels more advanced and safer structure could be designed and the response of the structure exposed to time dependent loads could significantly be improved. Comparative analysis of moment resisting frame and the same frame with added dampers, exposed to time dependent loads in linear and non-linear range is conducted. As much as the added damping is increased the structural response is improved resulting in a decrease of storey drifts and hysteretic damping demand towards its vanishing and linear structural response. Through an amount of added damping, it is possible to control the nonlinearity and storey drifts to a required level defined in the Eurocode 8.

2. DESIGN AND ANALYSIS OF HYPOTHETICAL STRUCTURE

Two hypothetical 5 stories steel frame structures, with three bays, have been designed as Moment Resisting Frame (MRF) according to EC8 and EC3 requirements for ultimate limit states and serviceability limit states, taking into account design response spectrum calculated for soil type B, PGA=0.20g, and PGA=0.40g damping of 2% and for two different q-factors, q=2 and q=4. So, total three steel frame structures have been designed. For all structures the bay length is 6.0 m and storey height is 3.0 m.
The SAP2000 computer program has been used for modelling and optimizing of the structural sections during the design phase and preliminary linear response history analysis only. The masses representing the weight at each floor level, including the weight of the beams and the columns and a portion of live load (24%), are concentrated at the beam-column joints. The total storey mass is 62.10 t. Beams and columns are modelled as frame elements with specified end length offsets and rigid-end factors, typically taken as 0.7. For such a model, at the final design phase it was found that the period of the first mode of vibration for the 5 storey structures designed for PGA=0.2g are T_1=1.00 s and T_1=1.38 s, designed for q=2 and q=4 respectively. For the 5 storey structure designed for PGA=0.4g and q=4 the first period of vibration T_1=1.011 s
Modelling and analysis of the designed steel frame structures for the purpose of non-linear response history analysis was done using computer program NONLIN-Pro, Ref [4], in which the analysis engine is the DRAIN-2DX computer program. Both steel frame structures are designed as 2D structures, using several types of elements that are available in DRAIN-2DX program. Namely Beam and columns are modelled as plastic hinge beam-column elements Type 02, taking into account the axial-flexural interaction for columns. The columns were modelled using the built-in yielding functionality of the DRAIN-2DX program, wherein the yield moment is a function of the axial force in the column. All beams are modelled to respond linearly, since for beams with flexural yielding that is independent of axial force, it better to explicitly model the hinges using simple connection element Type 04 as explained ad in the following context. The plastic hinges, located 30 cm away from the column face, in all beams are modelled using zero length rotational connection elements (springs), which means that 100% of the inelastic rotation is assumed to occur in the rotational plastic hinges. It has to be pointed out that DRAIN-2DX does not have the capability to model loss of strength after first yielding, so it is assumed bilinear, inelastic moment-rotation behaviour for the spring having 3% post yielding stiffness ratio. Since a very significant portion of the total story drift of a moment-resisting frame may be due to deformations that occur in the panel zone region of the beam-column joint, the panel zones are modelled using an approach developed by Krawinkler, Ref [4]. In DRAIN-2DX it consists of a “frame” of Type 02 beam-column elements connected at the four corners by compound nodes. The upper left compound node utilizes a rotational Type 4 spring to represent the panel zone web stiffness and strength. The upper right compound node utilizes a Type 04 rotational spring to represent column flange contributions. The other two

compound nodes are simple flexural hinges. So, each panel zone was modelled using 12 elements and 12 nodes, and has 28 degrees of freedom. Also, in each panel zone 20 mm thick doubler plate has been taken into account. Both rotational springs were modelled assuming bilinear, inelastic moment-rotation behaviour for the spring having 3% and 1% post yielding stiffness ratio for flange component and for panel component, respectively.

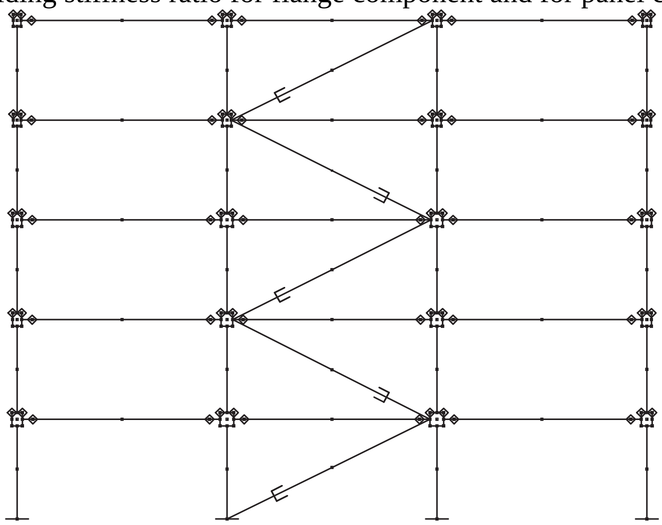


Figure 1**:** Mathematical models for non-linear response history analysis

The mathematical model with viscous dampers (VISC) is presented in Figure 1. The main structural elements beams, column and panel zones are the same as for MRF model. The viscous dampers are located in the middle bay along the height of the model and are modelled using two inelastic truss bar elements Type 01 in parallel. Both springs are modelled to respond linearly having very low stiffness, k = 0.01 kN/cm, with that difference that one has very high beta value (element stiffness proportional damping factor), and the other has beta value zero. The product of the stiffness and the beta value is equal to the desired damping coefficient, C. The use of a very low stiffness is consistent with the behaviour of a viscous fluid damper which has a near zero storage stiffness (if excited below its cut off frequency).
The diagonal viscous dampers have linear force versus velocity relationship, so the damper force is expressed as:

$$F_{dj} = C_j \dot{u}_{dj} \qquad (1)$$

where C_j is the damping coefficient for the damper at level j, while $\dot{u}_{dj}$ is the relative velocity between the damper joints along its axis. The added viscous damping, for each separate vibration mode can be calculated according to: Ref [5]

$$\beta_{vm} = \left(\frac{T_m}{4\pi}\right) \frac{\sum_j C_j \cos^2\theta_j \phi_{rj}^2}{\sum_i m_i \phi_{im}^2}, \quad \phi_{rj} = \phi_{jm} - \phi_{(j-1)m} \qquad (2)$$

where, T_m undamped natural frequency of the m-th mode, $\cos\theta$ is the damper inclination angle, ϕ_{im} are the ordinates of the m-th undamped mode, ϕ_{rj} is the modal drift and m_i is the storey mass. For the analysis the C_j coefficient is the same for all dampers along the height of the structures and it is derived from Eq. (2) in order to introduce a predetermined additional damping in the first fundamental mode (10, 20 and 30% of the critical for each analysed case respectively.
Modelling of inherent damping of 2% of critical in the first fundamental mode was done through mass proportional damping only, for both MRF and VISC models. The values of stiffness proportional damping factor for all elements in both models, excluding the viscous damping elements, are taken to be zero.

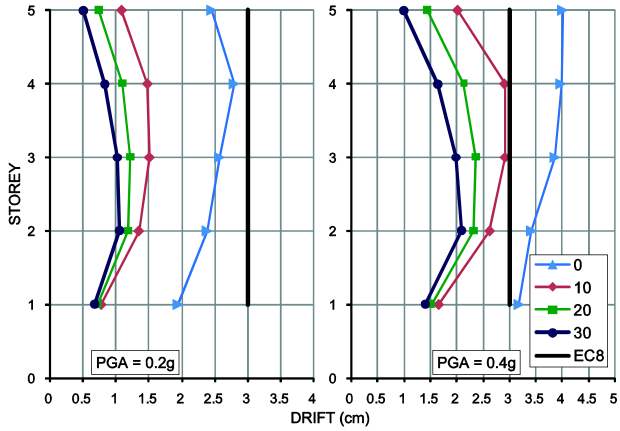
Table 1: Earthquake records used in analysis

Earthquake	Registration	PGA (g)	Name
Taiwan 1999	Chi-Chi CHY028 NS	0.82	CHI
Imperial Valley 1940, California	El Centro Copmp180	0.35	ELC
Erzincan 1992 Turkey	Mudurlugu N279	0.51	ERZ
Loma Prieta 1989 Santa Cruz Mountains	CSMIP 4725 Comp 0	0.37	LOP
Hyogo-Ken Nan-Bu , Kobe Japan 1995	Kobe University N-S	0.28	KOB
Montenegro 1979	Petrovac-Oliva NS	0.45	PET
Montenegro 1979	Ulcinj-Albatros EW	0.22	UAL
Montenegro 1979	Ulcinj-Olimpic NS	0.29	UOL
San Fernando 1979	OWNER 0241 Comp 360	0.25	SAN
Northridge 1994 Sylmar	LADWP 306 Comp S38E	0.75	SYL

Ten different time history records of real earthquakes, given in Table 1, taking the first 40 s of shaking duration have been used in the analysis. The earthquakes have been chosen based on their frequency content and elastic response spectra, as well as taking into account dynamic properties of the four designed MRF structures.
Non-linear response history analysis has been conducted on three structures two 5 storey, MODEL05_q2 and MODEL05_q4, and one 5 storey, MODEL05A_q4, where “q2 or q4” means which q-factor have been used in the design process. Further the response history analysis was done for constant time step dt = 0.001, for scaled earthquake time histories to PGA=0.2g, 0.4g and 0.6g, and all model have been analysed as MRF (0% of added viscous damping), while the first two models, MODEL05_q2 and MODEL05_q4 have been analysed for 3 values of added viscous damping, 10%, 20%, 30% of critical (VISC). Since for all 3 structures the first natural mode of vibration is dominant and has approx 80% participation in the structural response the damping coefficients for the viscous dampers are obtained from the condition to introduce additional damping 10, 20, and 30% of critical damping in the first mode of vibration. This means that for one structure total number of analyse are 120, and the total conducted analyses for all structures is 360.

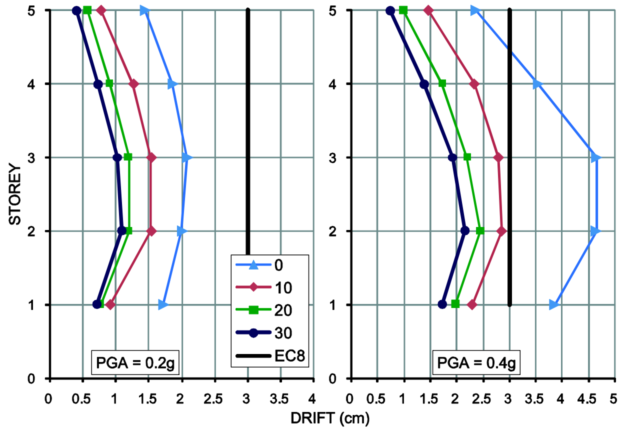
3. ANALYSIS RESULTS

Having in mind above, analysis results will be presented and discussed for maximum storey drifts, base shear forces, top displacements and top accelerations and for 5 selected earthquakes (CHI, ELC, ERZ, PET and SAN) from Table 1.
In general, for all selected five earthquakes scaled to 0.2g the storey drifts for the 5 storey structures (MODEL05_q2 and MODEL05_q4) are within the storey drift limit for elastic analysis defined in EC8 (0.01h = 3.0 cm), for MRF and VISC models. The exception from this is the case for MODEL05_q4 exposed to SAN earthquake scaled to 0.2g for MRF and VISC model with 10% of added viscous damping only. For MODEL05_q2, in the case for MRF, for all 5 earthquakes scaled to 0.4g all storey drifts are larger up to 1.5 times than EC8 limit. The inertial base shear force is within the range of 24% - 35%, the top acceleration is between 0.83g to 1.03g, while the top displacement is in the range 15.50 cm – 17.21 cm.
In Figure 2 are presented the envelopes of maximum storey drifts, along the height, as line graphs, as well as the extreme values of inertia base shear force, top displacement and top acceleration in tabular form, for MODEL05_q2 exposed to PET earthquake scaled to 0.2g and 0.4g for MRF case (0 added damping) and VISC case (10, 20, 30 % added damping).
In Figure 3 are presented the envelopes of maximum storey drifts, along the height, as line graphs, as well as the extreme values of inertia base shear force, top displacement and top acceleration in tabular form, for MODEL05_q4 exposed to ELC earthquake scaled to 0.2g and 0.4g for MRF case (0 added damping) and VISC case (10, 20, 30 % added damping).
For the same model, in the case for VISC, all storey drifts are smaller than EC8 limit, and for added 20% of damping the storey drifts are between 1.5 cm and 2.5 cm. For the same added damping the inertial base shear force is within the range of 18% - 27%, the top acceleration is between 0.62g to 0.79g, while the top displacement is in the range 8.32 cm – 10.74 cm.



BSF (%)	TD (cm)	TA (g)	
PGA = 0.2g			
0	19.08	10.43	0.81
10	14.87	6.05	0.44
20	13.63	4.76	0.38
30	12.64	3.95	0.36
PGA = 0.4g			
0	34.01	15.50	1.08
10	29.57	11.98	0.87
20	27.36	9.44	0.76
30	25.26	7.84	0.73

Figure 2: Maximum storey drifts, base shear force, top displacement and top acceleration for MODEL05_q2, Petrovac earthquake scaled to PGA=0.2g and 0.4g



BSF (%)	TD (cm)	TA (g)	
PGA = 0.2g			
0	11.25	7.24	0.52
10	10.49	5.97	0.45
20	10.43	4.61	0.41
30	10.13	3.87	0.38
PGA = 0.4g			
0	20.15	13.83	0.94
10	20.41	10.65	0.89
20	20.16	8.98	0.81
30	19.74	7.87	0.75

Figure 3: Maximum storey drifts, base shear force, top displacement and top acceleration for MODEL05_q4, El Centro earthquake scaled to PGA=0.2g and 0.4g

For MODEL05_q4, in the case for MRF, for all 5 earthquakes scaled to 0.4g all storey drifts are larger up to 2.4 times than EC8 limit. The inertial base shear force is within the range of 20% - 24%, the top acceleration is between 0.73g to 0.94g, while the top displacement is in the range 13.83 cm – 21.12 cm. In the case for VISC and for added 20% of damping, the storey drifts are smaller than EC8 limit for CHI, ELC and PET earthquake. For ERZ and SAN earthquake the drifts from first to third storey are above the EC8 limit, up to 1.5 times at second storey. The inertial base shear force is within the range of 17% - 24%, the top acceleration is between 0.64g to 0.81g, while the top displacement is in the range 8.98 cm – 17.30 cm.

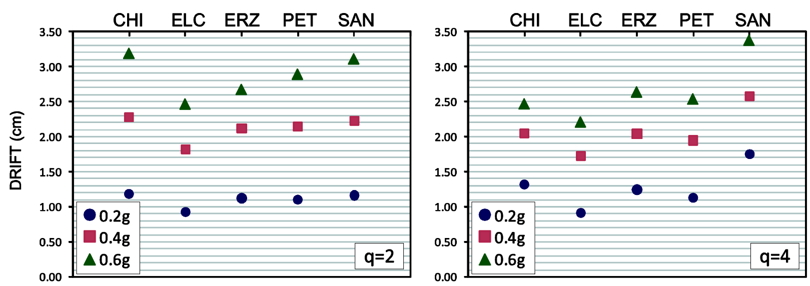


Figure 4: Maximum storey drift at 4-th storey, for MODEL05_q2 and MODEL05_q4, for 5 selected earthquakes scaled to PGA=0.2g, 0.4g and 0.6g and for added damping of 20%

In Figure 4 are presented the envelopes of maximum storey drifts, for the 4-th storey, for all 5 selected earthquakes scaled to 0.2g, 0.4g and 0.6g, for the MODEL05_q2 and MODEL05_q4 with added 20% of viscous damping. It can be seen that only for CHI and SAN earthquakes scaled to 0.6g the 4-th storey drift is slightly above the EC8 limit.

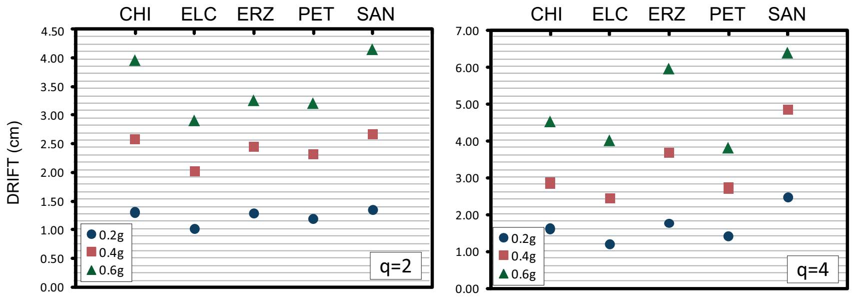


Figure 5: Maximum storey drift at 2-th storey, for MODEL05_q2 and MODEL05_q4, for 5 selected earthquakes scaled to PGA=0.2g, 0.4g and 0.6g and for added damping of 20%

Figure 5 shows the envelopes of maximum storey drifts, for the 2-nd storey, for all 5 selected earthquakes scaled to 0.2g, 0.4g and 0.6g, for the MODEL05_q2 and MODEL05_q4 with added 20% of viscous damping. As presented, for q=2, for CHI and SAN earthquakes scaled to 0.6g the 2-nd storey drift is above the EC8 limit; while for q=4 for all five earthquakes scaled to 0.6g the obtained drift for the second storey is above the EC8 limit.

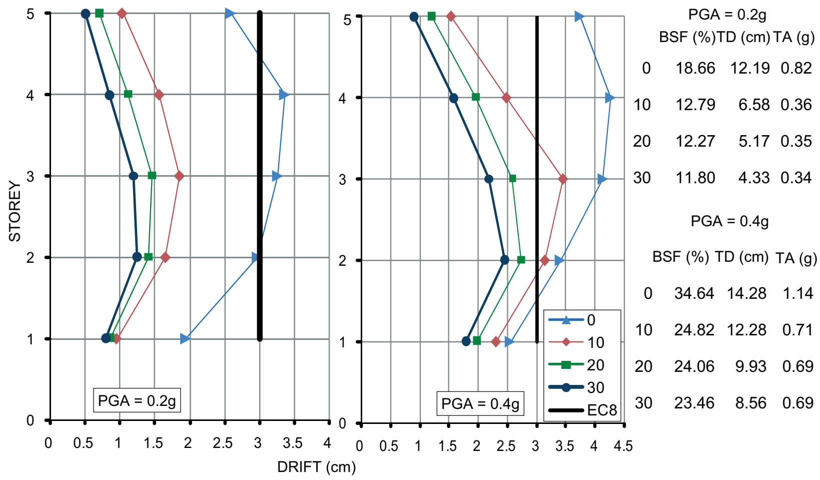


Figure 6: Maximum storey drifts, base shear force, top displacement and top acceleration for MODEL05A_q4, Petrovac earthquake scaled to PGA=0.2g and 0.4g

Figure 6 compares the envelopes of maximum storey drifts, along the height, as line graphs, as well as the extreme values of inertia base shear force, top displacement and top acceleration in tabular form, for the studied steel frame structure designed for 0.4g exposed to Petrovac earthquake scaled to 0.2g and 0.4g for MRF case (0 added damping) and the structure designed for 0.2g VISC case (10, 20, 30 % added damping).
In general, for the model designed to 0.4g, for all selected five earthquakes scaled to 0.2g the story drifts for the five story structure (MODEL05A_q4) are within the story drift limit for elastic analysis defined in EC8, for MRF and VISC model. For the same model in the case for MRF, for all 5 earthquakes scaled to 0.4g, all story drifts are larger up to 1.7 times than EC8 limit. The inertial base shear force is within the range of 23%-37%, the top acceleration is between 0.84g to 1.14g, while the top displacement is in the range 14.28cm – 19.74cm.
In the case for VISC all storey drifts are smaller than EC8 limit, and for added 20% of damping the storey drifts are between 1.5 cm and 2.5 cm. For the same added damping the inertial base shear force is within the range of 18% - 27%, the top acceleration is between 0.62g to 0.79g, while the top displacement is in the range 8.32 cm – 10.74 cm.

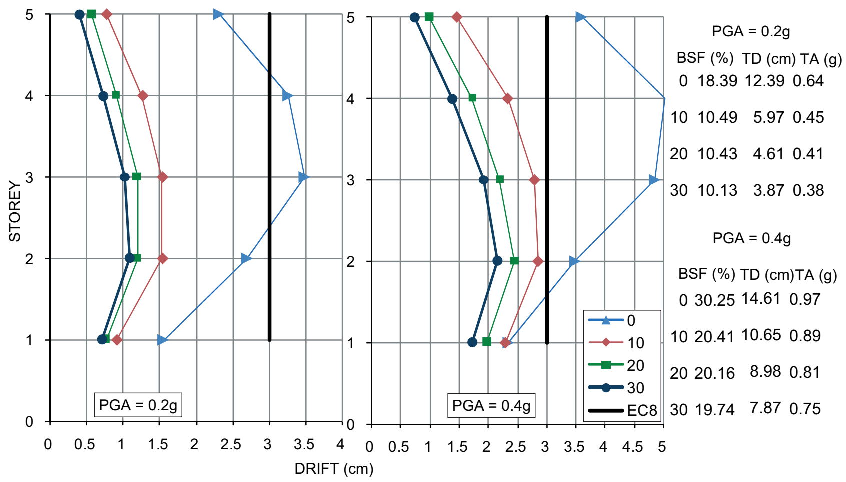


Figure 7**:** Maximum storey drifts, base shear force, top displacement and top acceleration for MODELL05A_q4, El Centro earthquake scaled to PGA=0.2g and 0.4g

In Figure 7 are presented the envelopes of maximum storey drifts, along the height, as line graphs, as well as the extreme values of inertia base shear force, top displacement and top acceleration in tabular form, for the studied steel frame structure designed for 0.4g exposed to El Centro earthquake scaled to 0.2g and 0.4g for MRF case (0 added damping) and the structure designed for 0.2g VISC case (10, 20, 30 % added damping).
It has to be pointed out that both structures with added viscous damping of 20%f and 30% for all 5 presented earthquakes when scaled to 0.2g responded in linear range. For the input level twice as much as designed one, 0.4g, both structures with added viscous damping of 20% and 30% respond approaching the linear limit, slightly above it or with minor damages. MRF structures, MODEL05_q2 and MODEL05_q4, for the input of 0.4g collapsed or suffer heavy damages which are not economically repairable. For the MODEL05A_q4 MRF structure, for the input of 0.4g the structure is not in the high damage limit, which is not case for the 0.6g where the structure suffers from heavy damages and collapsed. This point out that the added 20% of viscous or any other type of damping, by introducing passive or semi-active devices, in existing or new structures, compensates for the structural nonlinearity by which the external energy should be dissipated (absorbed) and could withstand earthquake which is twice as much as designed one. Also, the effect of added damping can be expressed through decreasing of vulnerability of the structure, particularly that of the non-structural elements, which leads to a decrease of the overall cost for retrofitting.

4. CONCLUSION

Possibility of introduction of additional energy dissipating mechanisms reduces the damage of the structure caused by hysteretic dissipation. By analytical simulation it was demonstrated that the optimal damping capacity of these devices in order to meet the EC8 design requirements, such as inter-storey drift is around 20%. It was demonstrated also, that the structures with added dampers that can introduce at least 20% of damping can withstand earthquakes with intensities twice as much as designed one, having no or minor damages. It was also noticed that when the dampers has the same mechanical properties along the height of the structure the storey drift decreases as the height increases. This means, by proper optimization of damping devices more favourable reduction of storey drifts could be achieved, which leads to lower cost solution for the damping devices. Also, the effect of added damping can be expressed through the decrease of the vulnerability of the structure, particularly that of the non-structural elements, which leads to a decrease of the overall cost for retrofitting.

Unfortunately, although this technology has been proven, both analytically and practically, to be very effective still it is not covered by the new European Codes for earthquake resistant design. On the other hand, USA and Japan have advanced in this field by making provisions, guidelines and recommendations.

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