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Rehabilitation of existing structures by optimal placement of viscous dampers

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ABSTRACT: The aim of this paper is to study the optimal placement of viscous dampers in existing structures, in order to minimize the displacement response, for pulse like seismic conditions of Romania. The optimal behavior is determined using the dynamic equation of motion, written in the frequency domain. The optimization process is based on minimizing the sum of the mean square response of interstorey displacement. Using the sensitivities of the transfer function, the optimal position of the viscous dampers will be evaluated for a structural configuration and increased in an iterative process. A case study on an existing concrete frame structure is presented. The structure is modeled with finite elements, and the optimal viscous damper placement is determined. The seismic responses of the existing structure and of the structure rehabilitated with optimal viscous dampers are compared. Also the influence of the dampers is assessed on the adjacent connecting elements.

1 INTRODUCTION

The current approach to seismic design of buildings supposes three different levels of performance. The structure needs to behave differently for different earthquake intensities. In the event of a minor earthquake the structure must not sustain damage, while in the event of a moderate earthquake the structure can sustain damage in certain parts of the structure. In the event of a severe earthquake, buildings must not collapse, however considerable damage is allowed. This principle requires buildings to be repaired after each moderate or severe earthquake. A modern approach to rehabilitation is to increase the ability of the structure to absorb energy through passive devices.

The viscous damper is a passive device, which has been shown to significantly improve the response of a structure during an earthquake. References to these devices have been implemented in design codes like the FEMA 356 (2000). It is shown that, in general, this type of device improves the seismic behavior of structures.

While there are some studies on the effect of passive damping devices in the structure, their optimal placement is not as well researched. Most studies on the optimal distribution use iterative nonlinear trial and error analyses to obtain an optimal damper distribution. In their study, Martinez and Romero (2003) distribute the highest damping capacity in the stories with the maximum relative velocity. The evaluation of this parameter is, however, dependent on a certain seismic action, thus a more theoretical approach needs to be studied. Such an approach has been developed by Takewaki (2009) using optimal design theory, and will be used in the following study.

In addition, the study aims to address optimal damper placement for the particular seismic conditions of pulse like, long period earthquakes produced by the Vrancea source.

Currently, design codes in Romania want to increase the level of seismic hazard, to a more suitable mean return period of 475 years. It is argued that for these types of earthquakes the forces in the viscous dampers need to be exceedingly high, introducing additional internal forces into the adjacent member.

The main objectives of the study are the following:

1. Test the Optimal Damper Placement method developed by Takewaki (2009).
2. Determine the opportunity of using viscous dampers for the rehabilitation of structures under the particularities of Vrancea earthquakes and the new design provisions.

2 VISCOUS DAMPERS

Viscous dampers are passive dissipation devices. This type of damper is very robust and has been

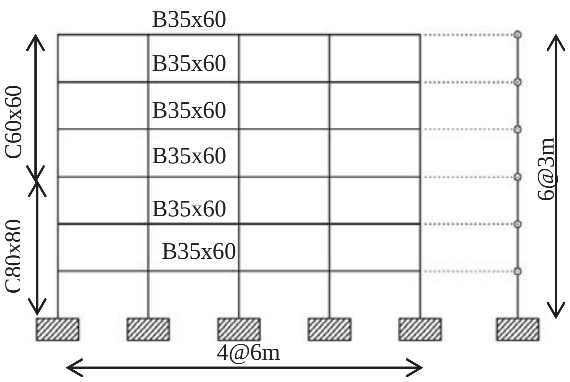

Figure 1. Elevation of original structure.

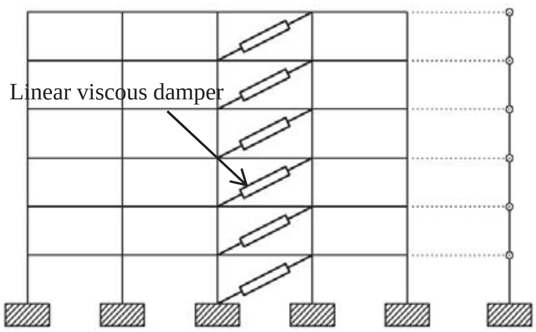

Figure 2. Elevation of Retrofitted Structure.

used in both new and existing projects. The viscous damper is built like a piston with two chambers, one of which is filled with viscous fluid. When the piston moves, it forces the liquid through an orifice generating a resisting force. The value of the force developed in the damper F_{vs} is:

$$F_{vs} = sign(v)Cv^{\alpha} \qquad (1)$$

where v is the relative speed between the ends of the damper, C is the damper constant and α is a power exponent of relative speed, between 0.3-1.5. The article will refer only to the linear viscous damper, for which α=1.

3 NUMERICAL PROCEDURES

The study aims to present and apply an optimal damper placement strategy and test it to the specific, pulse like, Vrancea earthquake.

First of all, the dynamic analysis of the structure is computed without dampers. The code requirements are assessed using the performance levels expressed in FEMA 356 (2000), which will also be adapted to the Romanian code P100 (2006).

Two sets of constraints on the viscous dampers are imposed. Firstly, using equivalent viscous damping, an overall sum of damping coefficients (C_{tot}) is determined for the structure. Secondly, for technical and economic reasons a limit value is imposed on the damping constant, $C \leq C_{lim}$.

From the full finite element model of the structure a shear building model is constructed, in order to simplify the amount of calculation in the optimization process. Using the original structure and the constraints determined for the dampers, the optimal damper distribution is determined, considering the March 4th, 1977 Vrancea earthquake.

Once the optimal distribution is obtained, the model with a uniform damper distribution and the optimal damper distribution are subjected to an incremental dynamic analysis (IDA). For the incremental dynamic analysis (IDA) 4 accelerograms are generated using Vanmarke (1967) method. Each of these accelerograms PGA is scaled using 4 levels (Sf={0.6, 1, 1.5, 2}).

For each one of the cases, a nonlinear dynamic analysis is run. A total of 3 damper distributions (no dampers, uniform dampers distribution, optimal damper distribution) are tested against 5 accelerograms, each with 4 scaling coefficients, resulting in 60 nonlinear dynamic analyses. Each dynamic nonlinear analysis is performed using SAP2000 v14 software. The results of the analyses are compared and the performance levels are assessed for the 3 distributions.

4 THE TEST STRUCTURE

For the numeric experiments, a symmetric concrete structure is used. Only one of the central frames of the structure is studied and its elevation is shown in Figure 1. The structure has 6 stories of 3m each and 4 spans of 6m each. The building is checked and complies with current design specifications corresponding to a 100 year mean return period PGA. The rebar grade considered is S235 and the concrete class is C20/25. The design includes dead loads (5 kPa), which comprise slab finishing layers, partitions, and live loads (2 kPa).

The inelastic response of the structure is modeled using plastic hinges which can form in both ends of each bar element. The plastic hinges are assigned a Takeda type hysteretic behavior. For the beam plastic hinges only the moment curvature relation is considered and idealized for bilinear behavior. For column plastic hinges, the interaction between axial force and moment is considered. The relation moment curvature for each level of axial force is considered bilinear.

Each of the moment curvature relations are deduced using the average strength of the concrete and steel used. Additionally, for the concrete strength and strain, the effect of confinement is considered using formulas given by the P100 (2006) design standard. The acceptance criteria for the plastic hinge rotation are extracted from FEMA 356 and are:

Table 1. Acceptance criteria for plastic rotation angle (rad)

Performance Level	IO	LS	CP
Beams	0.010	0.020	0.025
Columns	0.005	0.015	0.020

All of the elements have adequate transverse reinforcement. Also, it is considered that the beam column connection is strong enough to avoid any shear deformation or yielding. The model is considered fixed to the ground at the base of the bottom storey. In order to determine the optimal distribution of the viscous dampers, the current frame is further simplified to a shear frame model.

5 DESIGN METHOD

In the following chapter, the proposed design method is developed using theory by Takewaki (2009). Firstly, some theoretical considerations are presented and explained. In the second part of the chapter the logical steps for programming are presented and in the last part of the chapter the used accelerograms are discussed.

5.1 *Theoretical considerations*

The problem which needs to be solved can be formulated in the following manner. Given a structure, its dynamic characteristics and the power spectral density (PSD) of the input accelerogram, the optimal position of the viscous dampers needs to be evaluated, so that it minimizes the sum of mean squares of interstorey displacements. The problem needs to be solved while accounting for two constraints. Firstly, the sum of the damper coefficients (C_i) is equal to a set value (C_{tot}). Secondly, each of the damper coefficients will be smaller than a certain value (C_{lim}).

$$\sum_{i=1}^{5} C_i = C_{tot} \tag{2}$$

$$C_i < C_{\lim} \tag{3}$$

$$d = \sum_{i=1}^{5} \sigma_{d_i}^2 \tag{4}$$

This problem has been studied by Takewaki (2009). The method uses optimal design theory, supposing the structure remains elastic. The article will apply the method to the present problem and extrapolate the results for the nonlinear response of the structure. The problem can be formulated using generalized Lagrange formulation and Lagrange multipliers (λ,μ,η):

$$L(C_i,\lambda,\mu,\eta) = \sum_{i=1}^{6} \sigma_{d_i}^2 + \lambda(\sum_{i=1}^{6} C_i - C_{tot}) + \sum_{i=1}^{6} \mu_i (0 - C_i) + \sum_{i=1}^{6} \eta_i (C_i - C_{\lim}) \tag{5}$$

For the reduced model the equation of motion is written in the frequency domain:

$$\left(K + i\omega C - \omega^2 M\right) v(\omega) = -Mr\ddot{v}_g(\omega) \tag{6}$$

where M is the mass matrix, C is the damping matrix, K is the stiffness matrix, r is a column vector with 1 on every position, $v(\omega)$ is the Fourier transform of the displacement vector and $\ddot{v}_g(\omega)$ is the Fourier transform of the ground acceleration. In order to simplify the statement the following notations are made:

$$A = K + i\omega C - \omega^2 M \tag{7}$$

The equation of motion is written:

$$Av(\omega) = -Mr\ddot{v}_g(\omega) \tag{8}$$

The relation between displacement and interstorey displacement is expressed using a transformation matrix(T):

$$\begin{bmatrix} d_1(\omega) \\ d_2(\omega) \\ d_3(\omega) \\ d_4(\omega) \\ d_5(\omega) \\ d_6(\omega) \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ -1 & 1 & 0 & 0 & 0 & 0 \\ 0 & -1 & 1 & 0 & 0 & 0 \\ 0 & 0 & -1 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & 0 \\ 0 & 0 & 0 & 0 & -1 & 1 \end{bmatrix} \begin{bmatrix} v_1(\omega) \\ v_2(\omega) \\ v_3(\omega) \\ v_4(\omega) \\ v_5(\omega) \\ v_6(\omega) \end{bmatrix} \tag{9}$$

$$d_i(\omega) = -TA^{-1}Mr\ddot{v}_g(\omega) \tag{10}$$

or writing $Hd(\omega) = -TA^{-1}rM$ as the transfer functions for each of the interstorey displacements. Using random vibration theory, the mean square response of the interstorey displacement σ_{di}^2 can be expressed:

$$\sigma_{d_i}^2 = \int_{-\infty}^{\infty} \left|H_{d_i}(\omega)\right|^2 P_g(\omega) d\omega \tag{11}$$

Where, P_g represents the power spectral density function of the input acceleration. The next step is to assess the first order sensitivity of the mean square response of the interstory displacement to each damper (C_j):

$$\frac{\partial \sigma_{d_i}^2}{\partial C_j} = \int_{-\infty}^{\infty} \frac{\partial H_{d_i}(\omega)}{\partial C_j} \overline{H_{d_i}}(\omega) P_g(\omega) d\omega + \int_{-\infty}^{\infty} H_{d_i}(\omega) \frac{\partial \overline{H_{d_i}}(\omega)}{\partial C_j} P_g(\omega) d\omega \qquad (12)$$

And the second order sensitivity:

$$\begin{aligned} \frac{\partial^2 \sigma_{d_i}^2}{\partial C_j \partial C_l} = & \int_{-\infty}^{\infty} \frac{\partial H_{d_i}(\omega)}{\partial C_j} \frac{\partial \overline{H_{d_i}}(\omega)}{\partial C_l} P_g(\omega) d\omega + \\ & + \int_{-\infty}^{\infty} \frac{\partial H_{d_i}(\omega)}{\partial C_l} \frac{\partial \overline{H_{d_i}}(\omega)}{\partial C_j} P_g(\omega) d\omega + \\ & \int_{-\infty}^{\infty} \frac{\partial^2 H_{d_i}(\omega)}{\partial C_j \partial C_l} \overline{H_{d_i}}(\omega) P_g(\omega) d\omega + \int_{-\infty}^{\infty} H_{d_i}(\omega) \frac{\partial^2 \overline{H_{d_i}}(\omega)}{\partial C_j \partial C_l} P_g(\omega) d\omega \end{aligned} \qquad (13)$$

After determining the first and second order sensitivities, the following algorithm is used to solve the optimal distribution problem.

5.2 *Optimal distribution algorithm*

A routine has been programmed into MATLAB to solve the optimal linear viscous damper placement. It is shown that the following algorithm solves the Lagrange problem.

Step 1. Initialize all damping constants C_j=0;

Step 2. Define Dynamic Characteristics (M,K,C) and constraints (C_{tot},C_{lim}), PSD function (P_g) and number of steps (n);

Step 3. Find "l" damper so that $\partial D / \partial C_l$ is minimum and increase the damping constant of damper "l" $\Delta C_l = W / n$

Step 4. Update the objective function using a linear approximation $D + \Delta C \partial D / \partial C_l$
and its sensitivity $\partial D / \partial C_i + \partial^2 D / \partial C_i \partial C_l$;

Step 5. If there is another damper "m" such that $\partial D / \partial C_m \geq \partial D / \partial C_l$, compute the increment ΔC_m

Step 6. Update C matrix and continue from step 3 for the remaining number of steps.

If in step 3 there are multiple dampers "*l1…lk*" with the same sensitivity, all of their damping coefficients are increased using the following relation:

$$\frac{\partial D}{\partial C_{l1}} + \sum_{i=l1}^{lk} \frac{\partial^2 D}{\partial C_{l1} \partial C_{li}} \Delta C_{l1} = \ldots = \frac{\partial D}{\partial C_{lk}} + \sum_{i=l1}^{lk} \frac{\partial^2 D}{\partial C_{lk} \partial C_{li}} \Delta C_{lk} \qquad (14)$$

5.3 *Input accelerograms*

The most important accelerogram to be used is the record of the March 4th, 1977 Vrancea Earthquake (N-S principal component, recorded at INCERC). For this accelerogram the PSD is plotted in Figure 3.

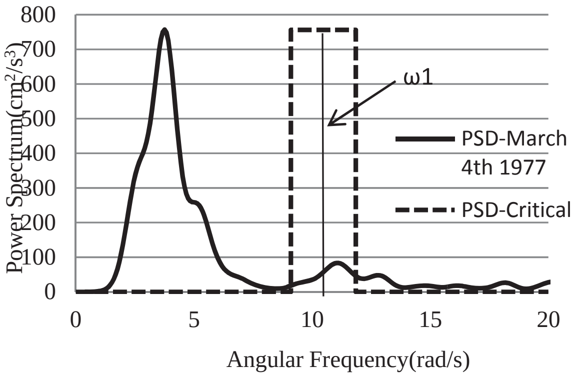

Figure 3. PSD of March 4th, 1977 recorded accelerogram and the Critical PSD.

This record, being one of the only strong motions recorded in Romania, will constitute the basis for the optimal design procedure. In order to account for the variability of the PSD function, a critical approach method is used, also proposed by Takewaki(2007). It is considered that the critical excitation PSD has the same total power as of the March 4th, 1977 accelerogram and the amplitude is equal to the peak value of the recorded accelerogram PSD. The critical PSD is centered on the first angular frequency of the structure. Thus, for the proposed critical excitation, the response is almost resonant. The peak value of the PSD is s=756 cm^2/s^3 and the area of the PSD S=2075cm^2/s^4. The critical excitation PSD used for the optimal design process has a constant amplitude of 756 cm^2/s^3 on a 2.7 rad/s interval centered on the first period of the studied frame ω_1=10.4 rad/s (T_1=0.6s). Figure 3 plots the critical excitation PSD and the PSD of the March 4th, 1977 earthquake. In order to confirm the obtained results through the nonlinear time history analysis, another series of 4 accelerograms are generated. The accelerograms are spectrum compatible and have been generated using Vanmarke (1976) algorithm. In Figure 4 the target

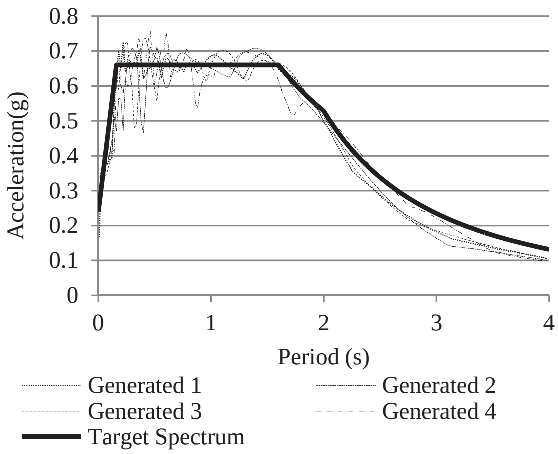

Figure 4. Target Spectrum and Generated Accelerogram Spectra.

spectrum is presented, along with the spectra of the generated accelerograms. It must be noted that a series of 50 accelerograms were generated out of which 4 have been chosen to resemble the March 4th, 1977 accelerogram in terms of PGA and Arias intensity. All the accelerograms used, have a PGA of 0.24g, which corresponds to current Romanian design codes for conditions in Bucharest. This PGA level corresponds to a mean return period of 100 years. The next generation of Romanian codes aims to raise the mean return period of the design earthquake to 475 years. Table 2 presents the levels of seismic hazard and the acceptance criteria which need to be fulfilled for each hazard level.

Table 2. Levels of considered seismic hazard

Mean Return Period (years)	50	100	475	975
Scaling Factor (Sf)	0.6	1	1.5	2
Performance Level	IO	LS	LS	CP

6 RESULTS OF OPTIMAL DISTRIBUTION ALGORITHM

After the complete frame is modeled using finite elements, a reduced shear building model is produced.

The characteristics of the reduced shear frame model are presented in table 3.

Table 3. Characteristics of the shear building model

Setting	m_i	k	$\bar{c}$
	10^3 kg	kN /m	kN s/m
Storey 1	111	230367	3128
Storey 2	111	110856	1505
Storey 3	111	88911	1207
Storey 4	122	70183	953
Storey 5	122	67628	918
Storey 6	122	59768	811

The mass matrix is determined using the finite element model. The structural damping is assumed to be Rayleigh proportional to the stiffness of the structure. The damping matrix results considering 5% fraction of critical damping for the first period of the structure.

A certain level of damping is imposed. Because this study aims to prove the opportunity of using viscous dampers for particular seismic conditions, an average level of equivalent viscous damping is chosen for structures outfitted with viscous dampers. By choosing an equivalent viscous damping level of ξ_d=25% of critical damping the following formula can be used to compute an uniform distribution of linear viscous dampers (C_{unf}):

$$C_{unf} = \frac{4\pi\xi_d \sum_i m_i\phi_i^2}{T_1 \sum_i m_i\phi_{ri}^2 \cos^2\theta_i} \approx 4000 kNs/m \qquad (15)$$

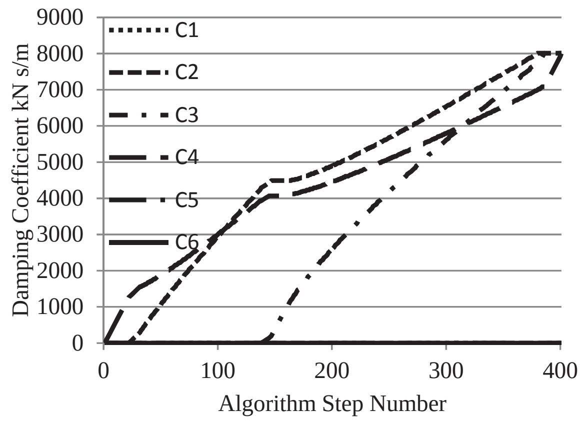

Figure 5. Evolution of damping coefficients with respect to design step.

Where ξ_d is the equivalent viscous damping introduced by the dampers, m_i ϕ_i ϕ_{ri},θ,T_1 are the storey mass, normalized displacement in the first mode, relative normalized displacement in the first mode, the angle between the damper and horizontal, respectively the first period of the structure. Thus, the uniform distribution uses 6 linear viscous dampers, each one with a damping constant (C_{unf}=4000 kNs/m).

For the optimal distribution of the dampers the following constraints are employed. Firstly, the sum of the damping coefficients for the whole structure will be the same as in the uniform distribution case (C_s=24000kNs/m). The second constraint of the algorithm needs to be chosen considering two aspects. Firstly, a very high damping constant results in large forces which can cause local structural failure, for the elements with which the damper is connected. Secondly, technical aspects need to be taken into account as producers usually have a limit force for their dampers. In the case of this study the limit on the damping constant considered is C_{lim}=8000 kNs/m). For these constraints, considering the critical PSD, shown in Figure 3, and choosing a number of steps (n=400), the damper distribution presented in Figure 5 is obtained.

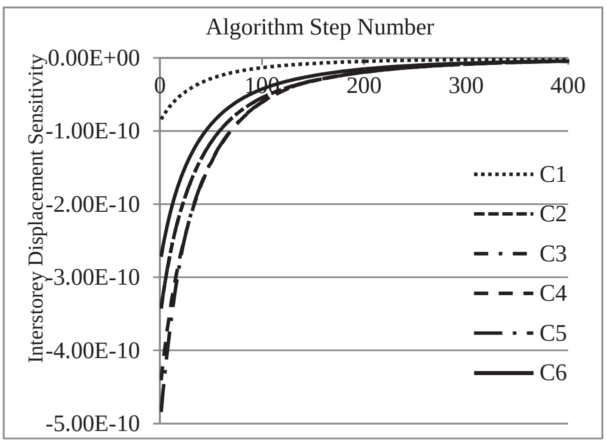

Figure 6. Evolution of Interstorey Displacement Sensitivity with respect to design step.

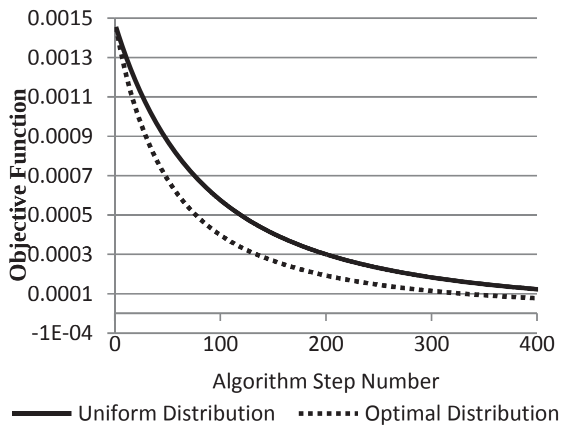

Figure 7. Evolution of objective functions with design step.

The optimal damper distribution uses 3 dampers on stories 2,3 and 4 each with a damping coefficient C_{lim}=8000 kNs/m. The results indicate that the 4th storey damper is the most useful for the structure. As its value increases the sensitivity of the 4th storey damper (Figure 6) reaches the sensitivity of the 2nd storey damper and the increase in damping coefficient is distributed between the two dampers. In the last steps of the process both the 2nd and the 3rd storey dampers reach the imposed constraint of the algorithm and their increase is transferred to the 4th storey damper which also reaches the constraint in the last step of the algorithm.

Using equation (15), the equivalent viscous damping can be assessed for both distributions. The equivalent viscous damping for the uniform distribution is equal to 26%, while the equivalent viscous damping for the optimum distribution is almost 1.5 times higher (38%).

In Figure 7 the objective function, the sum of interstorey displacement is plotted through the algorithm steps for the optimal distribution and for the uniform distribution. It is clear that through all the design steps, the optimal distribution obtained provides lower interstorey displacement than the uniform distribution.

The gap between the two curves starts to decrease towards the end as the constraints in the algorithm limit the values of the most useful damping coefficients. The difference between the two objective functions (sum of the interstorey displacements) at the end of the algorithm is 13%.

7 RESULTS OF NONLINEAR DYNAMIC ANALYSIS

The nonlinear dynamic analysis needs to establish the opportunity of using the linear viscous dampers for specified seismic conditions. The structure is analyzed without viscous dampers and with two distributions of viscous dampers (uniform distribution and optimal distribution).

In Figure 8, the results for the nonlinear analysis are displayed for the structure in the conditions of the March 4th 1977 accelerogram and all of the three damper distributions. The positive effect of the dampers is evident on the maximum drifts of the structure. From the Figure it results that the dampers are even more effective in reducing displacements as the level of the PGA increases. If the decrease from the no damper distribution to the optimal distribution is 22% for *Sf*=0.6, it increases to 50% for *Sf*=2. The differences in drift between the uniform distribution and the optimal distribution range between 8% and 16%, approximately the same as the elastic structure.

In Figure 9 the maximum results from the 4 generated accelerograms are presented. It is evident that the trends maintain. The differences in drift between the bare structure and the optimum distribution of dampers range from 25% to 50%, while the differences between the uniform distribution and the optimal distribution are between 7% and 20%. The maximum results of the generated accelerograms vary on average by only 7% with regard to the results obtained using the recorded accelerogram, which means that the generated accelerograms simulate with a satisfactory degree the behavior of

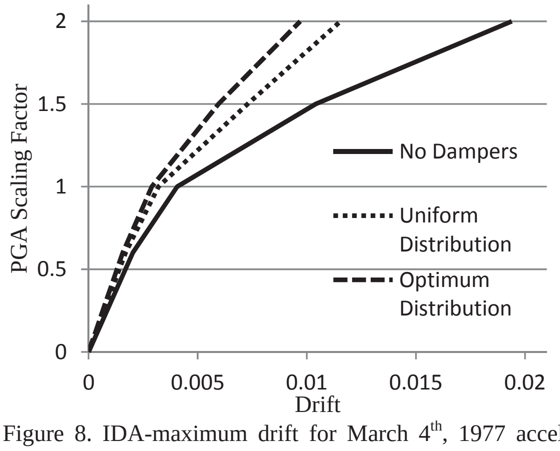

Figure 8. IDA-maximum drift for March 4th, 1977 accelerogram.

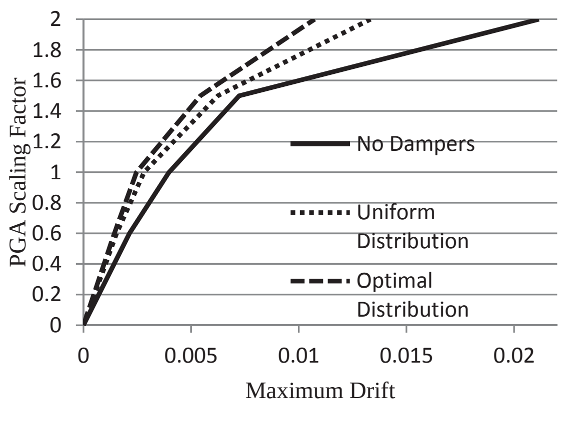

Figure 9. IDA Maximum drift for generated accelerograms.

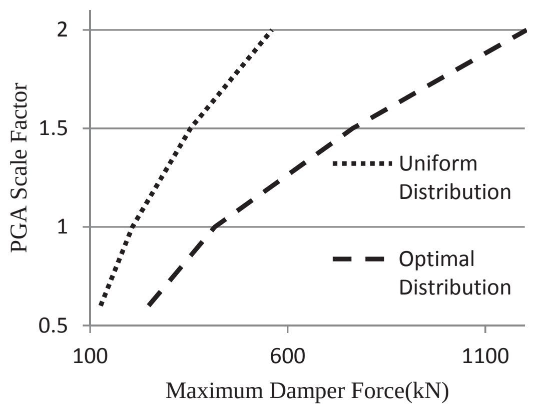

Figure 10. IDA Maximum Damper Force.

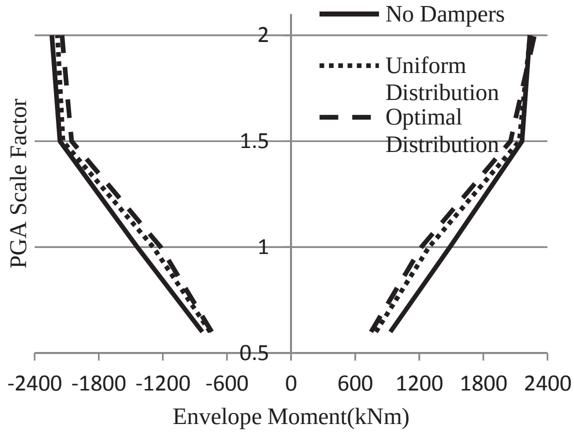

Figure 12. IDA Envelope of central column moment.

the recorded accelerogram.

In Figure 10 the IDA results for the maximum damper force are presented. The damper forces are much higher (50%) for the optimal distribution than for the uniform distribution, illustrating the effectiveness of the optimal distribution.

Another interesting aspect is the evolution of internal forces in the central column to which the damper is connected. In Figures 11 and 12 the envelopes of the axial force and moment for the central column and the three distributions are shown. The impact of the damper force on the axial force of the column has an influence proportional to the scale of the PGA. The axial force varies from the axial force in the structure without dampers to the optimal distribution by 15% to a maximum of 60%, for the highest PGA levels. With respect to the uniform damper distribution the initial axial force in the column varies from 14% to 47% from lowest to highest PGA levels.

The level of moment in the frame also changes but much less than the axial force. The envelope of the moment is presented in Figure 12. The moments decrease from the structure without dampers to the structure with the optimal distribution, however the maximum differences are only 20% and an average would be 8%.

Thus, it is safe to assume that the change in the maximum values of the moment is minimal. The decrease in axial force can cause damage to the column; however this was not the case for the current study. Although the axial force decreases and the moments remain constant, the value of the moment associated with the low axial force is not high enough to cause the appearance of plastic hinges. Rather the number of plastic hinges and their development is reduced from the model without viscous dampers to the model with viscous dampers.

Finally, the response of the structure is studied with respect to the plastic rotation acceptance criteria. The results indicate that the higher the PGA of the earthquake, the more the dampers impact the formation of plastic hinges. For the lowest scaling factor of the PGA, the number of plastic hinges is almost the same in the model without dampers as in the model with dampers. However, for the higher levels of scaling of the PGA, the effect is more pronounced. For a *Sf*=2, the structure with no dampers develops a storey mechanism with plastic rotations above the limit of collapse prevention. This does not happen once the uniform damper distribution is installed. The difference is even more obvious for the optimal distribution. In the case of this distribution, although the number of plastic hinges is the same as

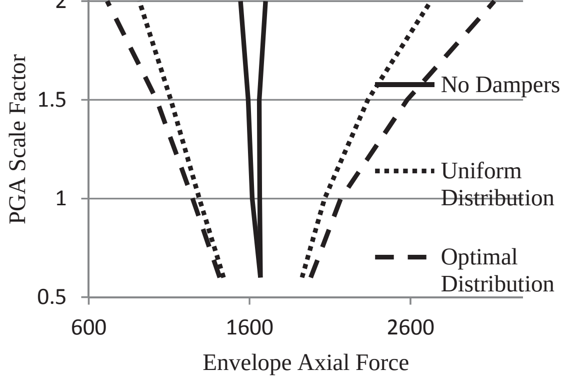

Figure 11. IDA Envelope of central column axial force.

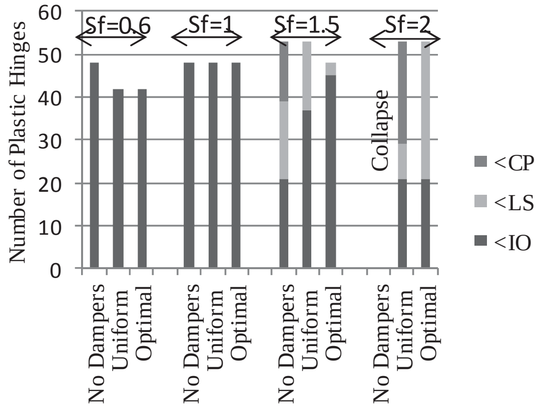

Figure 13. Number of plastic hinges and corresponding performance criteria for each Sf.

the uniform distribution the degree of their rotation ensures the life safety acceptance criteria. In the event of an increase of the seismic hazard in Romanian codes (from *Sf*=1 to *Sf*=1.5), the structure without dampers would be in need of rehabilitation. Both cases of distribution meet the design objectives which is life safety for *Sf*=1.5 and collapse prevention for *Sf*=2. The optimal distribution performs even better ensuring the life safety objective.

8 CONCLUSIONS

The seismic performance of a six-storey concrete frame to particular seismic conditions given by the Vrancea source is analysed in view of upcoming change of design codes. A comparative study is performed in order to study the opportunity of using linear viscous dampers to diminish the seismic response of the structure. At the same time the method developed by Takewaki (2009) to assess optimal damper distribution is tested. The proposed structure is outfitted with two damper distributions, a uniform distribution and an optimal distribution. The two configurations are studied using the only strong motion recorded earthquake from the Vrancea source (March 4^{th}, 1977). Moreover a set of 4 spectrum compatible accelerograms are generated in such a manner as to resemble as much as possible the recorded accelerogram. Four degrees of the PGA are considered corresponding to important mean return periods, aiming to prove that an eventual increase of seismic hazard can be compensated by the introduction of the linear viscous dampers.

The results show that the linear viscous dampers reduce the displacement of the structure in the event of a pulse like earthquake produced by the Vrancea source. It is shown that the introduction of a usual level of dampers reduces displacements from 22% up to 50% depending on the PGA of the earthquake.

The tested optimal distribution algorithm provides a decrease in displacements from the uniform distributions of about 50%.

Another interesting aspect is the change in internal forces for the members to which the dampers are connected. The results show that although the moment in the column does not decrease, the axial force varies extensively, by up to 60%. Although there is a change in axial force, the nonlinear analyses do not show the formation of additional plastic hinges on the columns adjacent to the dampers, which means that the decrease in axial force is not in phase with the maximum moments. One must note that for the current case study the change in internal forces does not affect the structural response, however the changes, being extensive, could have a negative effect on a different structure.

The eventual increase of the mean return period for the design earthquake in Romanian codes could render a lot of structures in need of rehabilitation. For the case study the structure does not meet the required acceptance criteria for the increased hazard, unless outfitted with viscous dampers. The viscous dampers prove their effectiveness in reducing displacements and plastic rotations. Furthermore, with the use of the optimal distribution algorithm, damper distributions can be improved. For the case study, the optimal distribution uses half of the number of dampers the uniform distribution does, limiting both costs and extent of intervention.

It is interesting to note that although linear viscous dampers provide a better structural response, the nonlinear viscous dampers are generally preferred because they limit the force which they develop. This study shows that the linear viscous dampers can successfully be employed, but a study of the nonlinear viscous dampers is already being developed and the results are promising.

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