Rehabilitation of Existing Structures Using Optimal Viscous Damper Placement in the Seismic and Soil Conditions of Romania

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SUMMARY:
The aim of this paper is to study the opportunity of optimal damper placement in existing structures, in order to minimize the displacement response, for specific seismic conditions in Romania. The optimal behavior will be determined using the dynamic equation of motion, written in the frequency domain. The purpose of the optimization process is to minimize the sum of the transfer functions of interstorey displacements for the fundamental period of the structure. 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 for an existing structure will be examined for seismic and soil conditions in Romania. The method will be applied to an existing frame structure, modeled using finite elements and the optimal damper placement will be determined. In the last part of the paper the behavior of the existing structures and the optimal damped ones will be compared to assess improvement.

Keywords: Optimal damper placement, soil structure interaction, retrofit, concrete frame

1. INTRODUCTION

The current seismic design theory supposes, for a structure, three different levels of performance. The structure needs to behave differently for different earthquake intensities. For a minor earthquake the structure must not sustain damage, while in the event of a moderate earthquake some localized damage is permitted. In the event of a severe earthquake the buildings can sustain considerable damage but cannot collapse. This principle requires buildings to be repaired after each moderate or severe earthquake. A modern approach to rehabilitation is to dissipate an important part of the seismic motion through the aid of passive devices.

The viscous damper is a passive device which has been shown to significantly improve the response of a structure during an earthquake. The use of viscous dampers for structures has prompted introduction in codes such as FEMA356 (2000). It has been proven that the introduction of viscous dampers improves the seismic behavior of structures.

While there are some studies on the effect of passive damping 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 distribution. In their study, Martinez and Romero (2003) distribute the highest damping capacity to the stories with the maximum relative velocity. This parameter is highly dependent on the seismic action considered. A more theoretical approach has been studied extensively by Takewaki (2009) using optimal design theory. This theory will be also used in this study as it provides an optimal distribution of dampers, considering also the soil structure interaction.

The study aims to address optimal damper placement for particular seismic conditions of pulse like, long period earthquakes produced by the Vrancea source. Currently, design codes in Romania will be rewritten to increase the level of seismic hazard from 100 years to a more suitable mean return period of 475 years. It is argued that for these types of earthquakes viscous dampers are not as well suited, introducing considerable forces to the adjacent members. Moreover, the paper will study the effect of different soil conditions in Bucharest on the optimal distribution.

The main objectives of the study are as follows:

1. Test the Optimal Damper Placement developed by Takewaki (2009), and extrapolate the results to nonlinear behavior of structures.
2. Determine the opportunity of using viscous dampers for the rehabilitation of structures under the particularities of Vrancea earthquakes and new design provisions.
3. Study the effect of soil structure interaction on the optimal distribution of the dampers for different sites in Bucharest.

2. VISCOUS DAMPERS

Viscous dampers are passive dissipation devices. Viscous dampers have been 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. As the piston moves the viscous liquid is forced through an orifice generating a resisting force. The force developed in the damper F_v is:

$$F_v = sign(v) C v^{\alpha} \tag{2.1}$$

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 to linear viscous dampers 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. The dynamic analysis of the structure is computed without dampers. The code requirements are assessed using the performance levels expressed in FEMA 356 (2000). Two sets of constraints are imposed on the viscous dampers. Firstly, the sum of all the damping coefficients (C_{tot}) will be fixed. Secondly, for economic and technical 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. Once the optimal distribution is obtained, the model with the optimal damper distribution and a model with a uniform damper distribution is subjected to an incremental dynamic analysis (IDA). For the IDA, 4 accelerograms are used, the recorded principal component of the March 4th 1977 earthquake (recorded at INCERC) and 3 generated accelerograms obtained using Vanmarke(1967) method. Each of the accelerograms PGA is scaled using 4 levels (Sf={0.6,1,1.5,2}). Lastly, all of these analyses are carried out for two considered soil profiles and for the building without the considered soil layers. For each of the cases a nonlinear dynamic analysis is run. A total of 3 damper distributions (no dampers, uniform damper distribution, optimal damper distribution) are tested against 4 accelerograms, each with 4 scaling coefficients, resulting in 144 nonlinear dynamic analyses. These are performed using SAP2000 v14 software. The results of the analyses are compared and the performance levels are assessed for the distributions.

4. THE MODEL

For the numeric experiments a symmetric concrete structure is used. Only one of the central frames of the structure is analyzed. The structure has 6 stories (3 m each) and 4 spans of 6 m (Figure 1(a)). The building is an existing building and needs to be rehabilitated. The concrete used in the structure is C20/25 and the rebar grade considered is S235. The design loads are comprised of dead loads (5kPa) and live loads (2kPa). 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

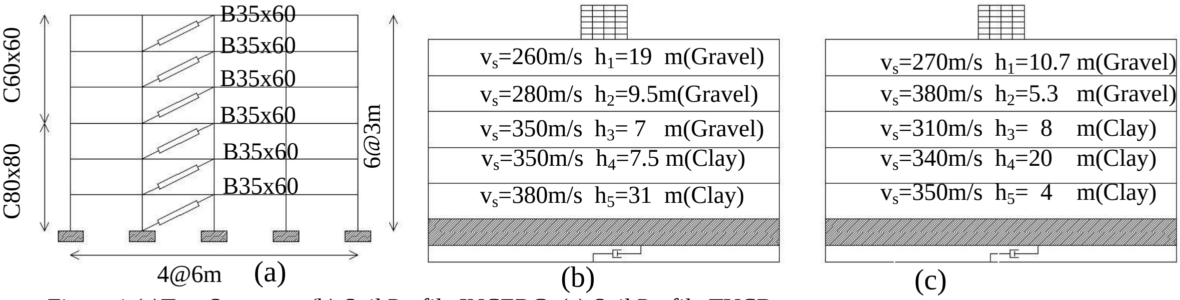

Figure 1 (a)Test Structure; (b) Soil Profile INCERC; (c) Soil Profile TUCB

behavior. For the beam plastic hinges only a moment curvature relation is considered, while for the columns plastic hinges the moment curvature relation is dependent on the axial force. Both moment curvature relations are considered bilinear, and are deduced using the average strengths of the materials. For the concrete strength and strain, confinement is considered. The acceptance criteria for the plastic hinge rotations are extracted from FEMA 356. All of the elements are considered to have adequate transverse reinforcement. Also, it is considered that the beam column connection is strong enough to avoid shear deformation or yielding.

The study considers the building supported by 6 layers of soil. The soil is modeled using area elements and it is assumed to follow an equivalent linear model similar to Schnabel (1971) Shake model. The model and the accuracy have been studied by Takewaki (2002). The stiffness of each soil layer is strain dependent, as is its damping (Figure 5). Two soil profiles are used for two sites. The first site is the INCERC site, where the above mentioned earthquake has been recorded; the second site corresponds to the one at the Technical University of Constructions Bucharest (TUCB). The hatched soil layer represents the engineering bedrock (v_s=400 m/s), underneath which a linear viscous damper ($c_r=\rho_0 v_s A$) has been placed to account for radiant damping (Lysmer and Kuhlemeyer, 1969).

5. OPTIMAL DAMPER DISTRIBUTION

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

5.1. Theoretical Considerations

The problem which needs to be solved is the following. Given a shear building model of a structure, its dynamic characteristics and the power spectral density (PSD) of the input accelerogram, find the optimal story in which the dampers should be placed, and their characteristics, so that the sum of the interstory displacements is minimal. Firstly, the problem needs to be addressed such that 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}).

$$\sum_{i=1}^{5} C_i = C_{tot} \tag{5.1.1}$$

$$C_i < C_{\lim} \tag{5.1.2}$$

$$d = \sum_{i=1}^{5} \sigma_{d_i}^2 \tag{5.1.3}$$

The method uses optimal design theory, to a linear system. The article aims to apply this algorithm and study whether the results can be extrapolated for the nonlinear response of the structure. The problem can be formulated using generalized Lagrange formulation and multipliers (λ, μ, η):

$$L(C_i, \lambda, \mu, \eta) = \sum_{i=1}^{6} \sigma_{d_i}^2 + \lambda(\sum_{i=1}^{6} C_i - C_{tot}) + \sum_{i=1}^{6} \mu_i (0 - C_i) + \sum_{i=1}^{6} \eta_i (C_i - C_{\lim}) \tag{5.1.4}$$

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

$$\left(K + i\omega C - \omega^2 M\right) v(\omega) = -Mr\ddot{v}_g(\omega) \tag{5.1.5}$$

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:

$$A = K + i\omega C - \omega^2 M \tag{5.1.6}$$

The equation of motion is written:

$$Av(\omega) = -Mr\ddot{v}_g(\omega) \tag{5.1.7}$$

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

$$\begin{bmatrix} d_1(\omega) \\ d_2(\omega) \\ d_3(\omega) \\ d_4(\omega) \\ d_5(\omega) \\ d_6(\omega) \end{bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ -1 & 1 & 0 & 0 & 0 & 0 \\ 0 & -1 & 1 & 0 & 0 & 0 \\ 0 & 0 & -1 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & 0 \\ 0 & 0 & 0 & 0 & -1 & 1 \end{bmatrix} \begin{bmatrix} v_1(\omega) \\ v_2(\omega) \\ v_3(\omega) \\ v_4(\omega) \\ v_5(\omega) \\ v_6(\omega) \end{bmatrix} \tag{5.1.8}$$

$$d_i(\omega) = -TA^{-1}Mr\ddot{v}_g(\omega) \tag{5.1.9}$$

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:

$$\sigma_{d_i}^2 = \int_{-\infty}^{\infty} \left|H_{d_i}(\omega)\right|^2 P_g(\omega) d\omega \tag{5.1.10}$$

In order to find the strain in the soil layers ($2.5\sigma_{d_i} 0.65$) a peak factor is employed (Der Kiureghian, 1980). To assess the first order sensitivity of the mean square response of the interstory displacement to each damper (C_j) the following equation is used:

$$\frac{\partial \sigma_{d_i}^2}{\partial C_j} = \int_{-\infty}^{\infty} \frac{\partial H_{d_i}(\omega)}{\partial C_j} \overline{H_{d_i}}(\omega) P_g(\omega) d\omega + \int_{-\infty}^{\infty} H_{d_i}(\omega) \frac{\partial \overline{H_{d_i}}(\omega)}{\partial C_j} P_g(\omega) d\omega \tag{5.1.11}$$

And for the second order sensitivity:

$$\frac{\partial^2\sigma^2_{d_i}}{\partial C_j\partial C_l}=\int_{-\infty}^{\infty}\frac{\partial H_{d_i}(\omega)}{\partial C_j}\frac{\partial\overline{H_{d_i}}(\omega)}{\partial C_l}P_g(\omega)d\omega+\int_{-\infty}^{\infty}\frac{\partial H_{d_i}(\omega)}{\partial C_l}\frac{\partial\overline{H_{d_i}}(\omega)}{\partial C_j}P_g(\omega)d\omega+\int_{-\infty}^{\infty}\frac{\partial^2 H_{d_i}(\omega)}{\partial C_j\partial C_l}\overline{H_{d_i}}(\omega)P_g(\omega)d\omega+$$
$$+\int_{-\infty}^{\infty}H_{d_i}(\omega)\frac{\partial^2\overline{H_{d_i}}(\omega)}{\partial C_j\partial C_l}P_g(\omega)d\omega \tag{5.1.12}$$

After determining the first and second order sensitivities, the algorithm described in the next chapter is used to solve the optimal distribution problem.

5.2. Optimal Distribution Algorithm

The software MATLAB is used to program an algorithm based on the above theory. It is shown by Takewaki (2009) that solving a similar algorithm solves the Lagrange problem.
Step 1. Initialize all damping constants C_j=0;
Step 2. Define/Update Dynamic Characteristics (M,K,C) and constraints (C_{tot},C_{lim}), PSD function (P_g) and number of steps (n); Update strain dependent soil stiffness and damping;
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:

$$\frac{\partial D}{\partial C_{l1}}+\sum_{i=l1}^{lk}\frac{\partial^2 D}{\partial C_{l1}\partial C_{li}}\Delta C_{l1}=...=\frac{\partial D}{\partial C_{lk}}+\sum_{i=l1}^{lk}\frac{\partial^2 D}{\partial C_{lk}\partial C_{li}}\Delta C_{lk} \tag{5.1.13}$$

5.3. Input Accelerograms

The most important accelerogram which will be used is the record of March 4th, 1977 Vrancea Earthquake (N-S principal component, recorded at INCERC). The mentioned earthquake is the only strong motion record, and will be the basis for the optimal design. Using EERA, an EXCEL code of SHAKE, and introducing the INCERC soil profile, where the mentioned accelerogram has been recorded, the input motion at the engineering bedrock is obtained. In order to account for the

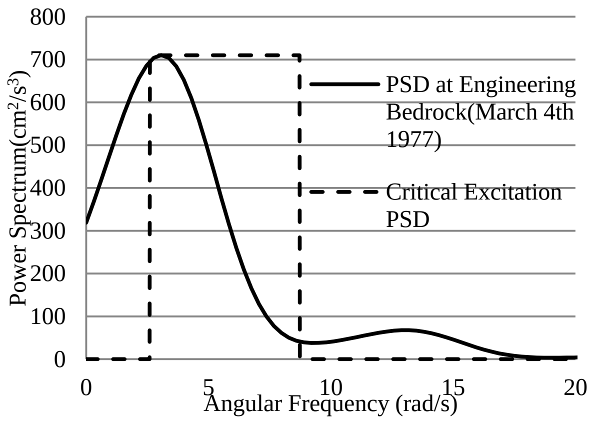

Figure 2 PSD of recorded motion, critical excitation

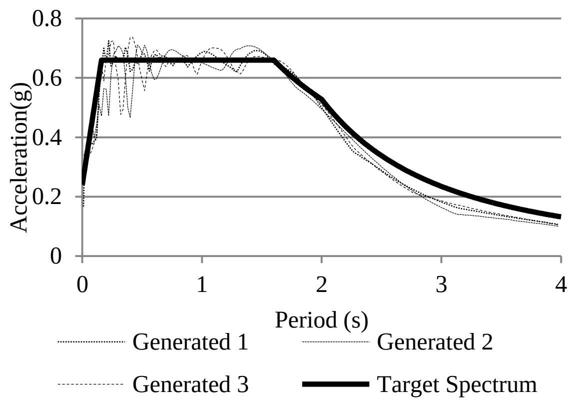

Figure 3 Generated accelerogram spectrum, target spectrum

Table 1 Mean Return Period, Required Performance Levels for Structure

Mean Reurn Period (years)	50	100	475	975
Scaling Factor (Sf)	0.6	1	1.5	2
Required Performance level	IO	LS	LS	CP
Beam Plastic Rotation	0.010	0.02	0.02	0.025
Column Plastic Rotation	0.005	0.015	0.015	0.020

variability of the input motion, a critical PSD is computed according to Takewaki (2007). It is considered that the critical excitation PSD has the same total power as the recorded earthquake, with its maximum intensity. 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=710 cm^2/s^3 and the total area of the PSD is S=4340 cm^2/s^4. The critical excitation PSD has a constant amplitude of 710 cm^2/s^3 on a 6.12 rad/s interval centered on the first period of the studied frame ω_1=5.86 rad/s (T_1=1 s) for the INCERC site and ω_1=7.38 rad/s (T_1=0.85 s) for the TUCB site. There is a change in the systems period as the damping capacity increases as the response of the structure influences the strain in the soil layers. This variation is taken into account but in this case is not significant.

In order to confirm the results through nonlinear time history analyses, another 3 accelerograms are generated using Vanmarke (1976) algorithm. In Figure 2 the spectrum is presented along with the spectra of the generated accelerograms. It must be noted that a series of 50 accelerograms were generated and only 3 have been chosen, based on similarity with the recorded accelerogram in terms of PGA and Arias Intensity. All the accelerograms have a PGA of 0.24g, corresponding to current design requirements for the city of Bucharest. The PGA 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.

6. RESULTS OF OPTIMAL DAMPER DISTRIBUTION ALGORITHM

From the complete finite element model, a reduced shear building model is produced. The characteristics of the reduced shear building model are presented in Table 2

Table 2. Characteristics of the shear building model

	m_i (10^3kg)	k (kN /m)	$\bar{c}$ (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 of the model is assumed to be Rayleigh proportional to the story stiffness. The damping matrix results considering a 5% fraction of critical damping for the first period of the structure.

The structure is firstly outfitted with a uniform distribution of dampers in order to achieve 25% equivalent viscous damping. Knowing the amount of equivalent viscous damping the following formula can be used to compute the uniform damper constant (C_{unf}):

$$C_{unf} = \frac{4\pi\xi_d \sum_i m_i \phi_i^2}{T_1 \sum_i m_i \phi_{ri}^2 \cos^2\theta_i} \approx 4000 kNs/m \tag{6.1}$$

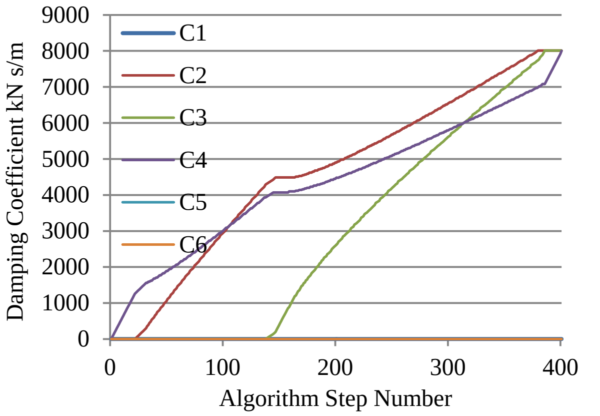

Figure 4 Evolution of damping coefficients with step (fixed base model)

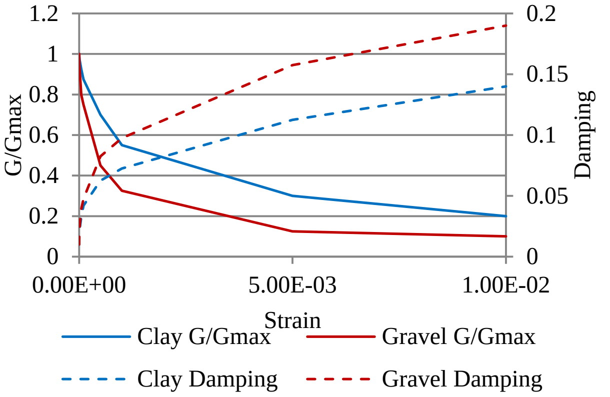

Figure 5 Evolution of G/Gmax and damping for considered soils

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. In order to obtain the optimal distribution of the dampers the following constraints are employed. Firstly, the sum of damping coefficients for the whole structure will be equal to the sum in the uniform distribution case (C_s=24000kNs/m). A second constraint is imposed on the damping constant of each damper. If the value of the damping constant is high the damper produces large forces which make the damper more expensive but also introduce significant forces in the adjoining members. Technical aspects need to be taken into account also, as manufacturers provide a certain set of 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 2, and selecting a number of 400 steps, the following damper distributions are obtained.

In figures 4, 6 and 7 the optimal distribution algorithm is presented for the INCERC site, TUCB site and the fixed base model. The optimal damper distribution for the fixed based model uses 3 dampers on stories 2, 3 and 4 each with a damping coefficient C_{lim}=8000 kNs/m. When the model takes into account the soil structure interaction, the optimal distribution changes. The 4th damper is the first which starts increasing in value until its sensitivity decreases to the point where it is similar to the sensitivities of the 2nd and 3rd story dampers, point at which all of the dampers increase in value until they all reach the limit C_{lim}. Although the soil layers are different, the optimal distributions vary only slightly between the two sites. Comparing with the fixed base model, the 4th storey damper is still the most useful damper to the structure, and the most important contributions are again made by the

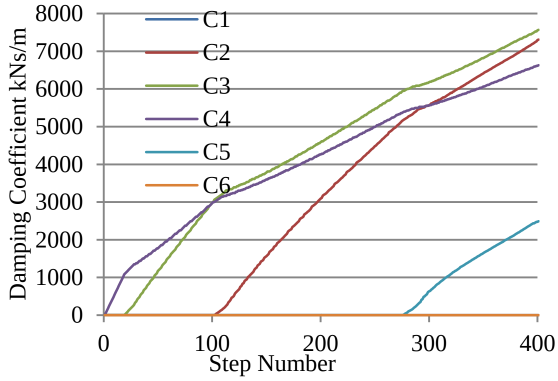

Figure 6 Evolution of Damping Coefficient (INCERC)

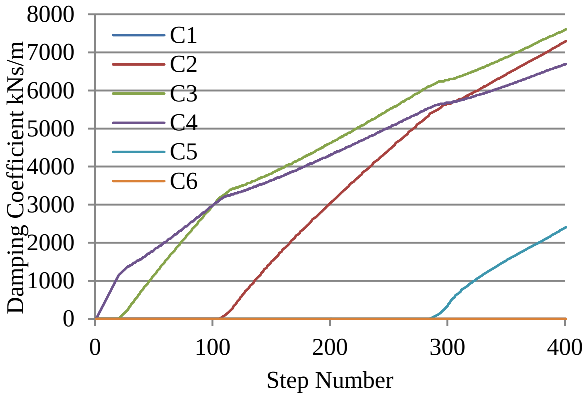

Figure 7 Evolution of Damping Coefficient (TUCB)

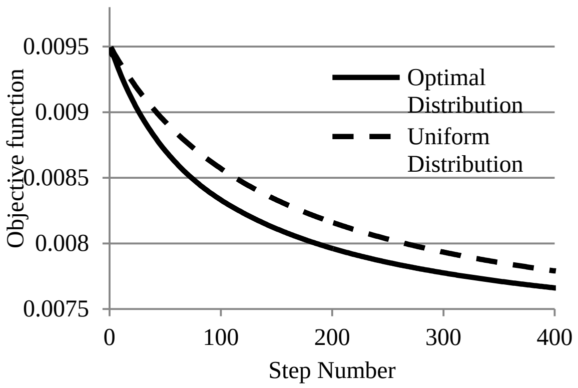

Figure 8 Evolution of objective function with algorithm step for INCERC site

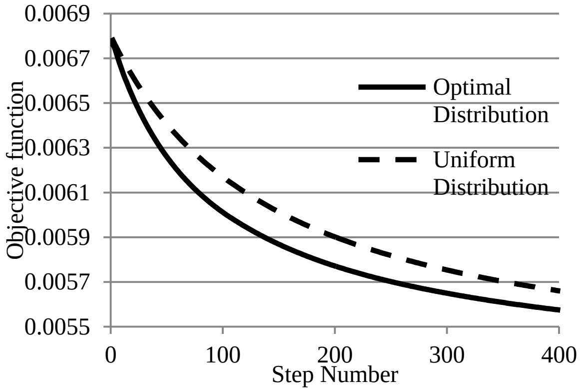

Figure 9 Evolution of damping coefficient with step (UTCB)

4th, 2nd and 3rd storey dampers. However towards the end of the algorithm the 5th storey damper also starts to play a role. In Figures 8 and 9 the variation of the sum of the mean square response of the drift is presented for both sites and for the optimal and uniform distribution. It is evident that the optimal distribution algorithm provides a better response in terms of the objective function.

7. RESULTS FOR NONLINEAR ANALYSIS

The purpose of the nonlinear dynamic analysis is to establish if the use of the viscous dampers can enhance the performance objectives for an existing structure. Three models are studied, one with fixed base, and two with different soil conditions. For each of the models three distributions are considered no dampers, uniform distribution and optimal distribution, also an IDA is carried out for each case.

In the following figures the maximum interstorey displacement is plotted for the March 4th 1977 earthquakes. These are also the maximum drifts obtained for all the accelerograms used. The beneficial influence of the viscous dampers on interstorey drift is evident, as it decreases from the no damper model to the optimal distribution. On average, the decrease between the no damper model and the uniform distribution is of 35%, while the difference between the uniform and the optimal distributions ranges between 6% for a *Sf*=0.6 to 20% for the highest *Sf*. Between the uniform and the optimal distribution the difference is more pronounced as the intensity (*Sf*) of the earthquake increases. In figure 8 the results are plotted for the fixed base model. With respect to the generated accelerograms the results vary only slightly for the fixed base model (on average drifts vary by 7% with respect to the recorded accelerogram), however when the soil structure interaction is considered, the results of the generated accelerograms consistently underestimate the results by up to 20% with respect to the recorded accelerogram.

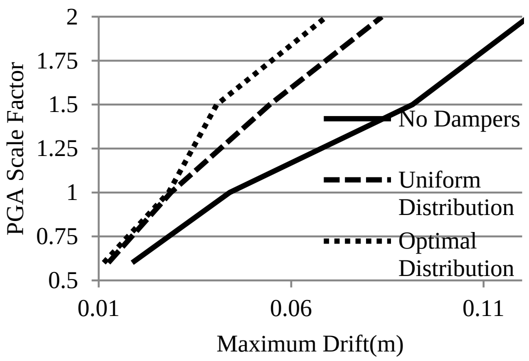

Figure 10 IDA for March 4th 1977 Earthquake, INCERC

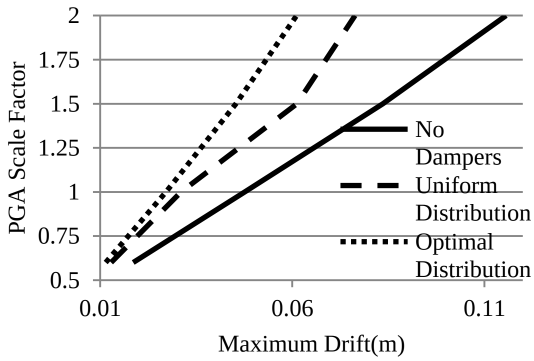

Figure 11 IDA for March 4th 1977 Earthquake, TUCB

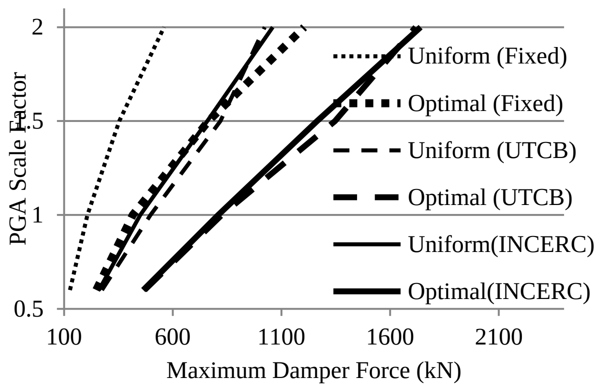

Figure 12 IDA Maximum Damper Force

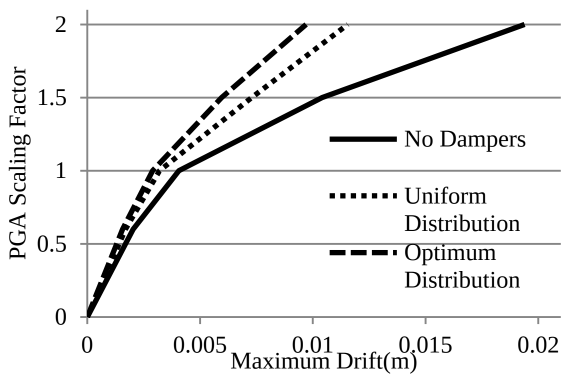

Figure 13 IDA for March 4th 1977 Earthquake, Fixed Base

In figure 12 the maximum damper force is plotted for the optimal and uniform distribution algorithm. It is evident from the plot that the optimal distribution makes better use of the installed dampers. Maximum damper force for the optimal distribution is on average, twice the value of the maximum force in the uniform distribution.

An interesting observation comes from the fact that the models which consider the soil structure interaction, although having dampers of similar capacity as the fixed base model, produce damper forces twice as large as the model which does not consider soil. This is even more interesting as the interstorey displacements are maximal for the fixed base model.

Another aspect of the nonlinear analysis is to study the opportunity of the use of linear viscous dampers in the conditions of pulse like earthquakes. For the studied model the variation in forces in the column next to the installed damper is studied. In Figure 14 the axial force envelope is presented for the IDA of the TUCB soil site. Because of relatively high damper forces introduced into the structure, the column axial force has a very pronounced variation for both damper distributions, with the maximum values attained for the optimal distribution. As the PGA *Sf* increases, the variation is also more pronounced as the damper force increases. Thus, it is clear that for *Sf* of 1.5 and 2 the force in the columns starts to produce tension at certain moments. However, if we observe the number of plastic hinges shown in Figure 15, it is evident that it decreases. This is due to the fact that although the column axial force varies widely for the case with mounted dampers it is not in phase with the maximum moment developed in the column. The variation of the moment for the same column is negligible for the cases when dampers are mounted.

In figure 15 the number of plastic hinges is studied for the INCERC site through the IDA. The positive effect of the dampers is observed, as both the number of plastic hinges and the performance objective for the hinge decrease from the no damper model to the optimal distribution. The presented trends maintain for all models, and all the studied accelerograms.

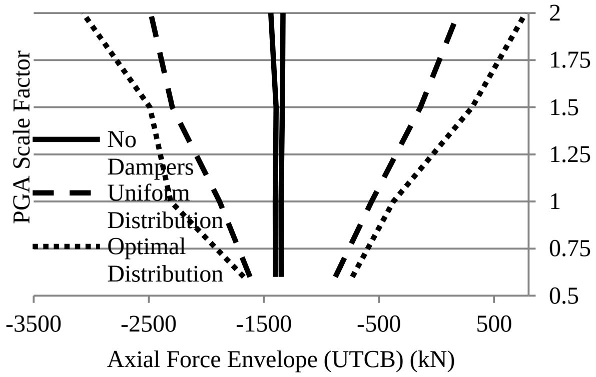

Figure 14 IDA Envelope of Axial Force (TUCB)

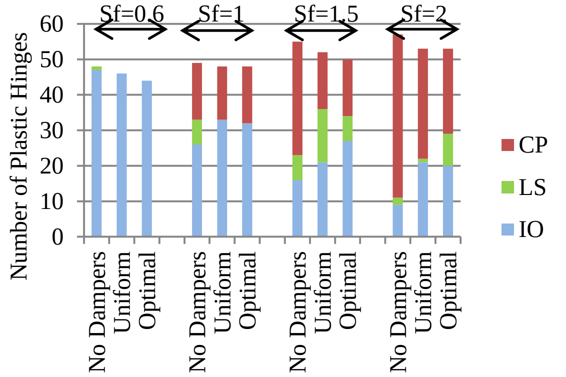

Figure 15 Number of Plastic Hinges for each Sf and distribution (INCERC)

8. CONCLUSION

The seismic performance of a six-storey concrete frame has been analyzed for the particular earthquake conditions given by the Vrancea source and for the particular soil conditions in Bucharest. A comparative study has been performed to assess the opportunity of using viscous dampers in order to rehabilitate existing structures. Also, the method developed by Takewaki (2009) to obtain an optimal distribution was tested, and its validity checked for the nonlinear response of the structure. The proposed structure was outfitted with two damper distributions, a uniform distribution and an optimal distribution. The two configurations were tested using an IDA and the only strong motion record from the Vrancea source. Finally, a set of 3 spectrum compatible accelerograms has been generated and used for the analysis to confirm the results given by the recorded accelerogram.

The results show that the linear viscous dampers reduce the relative displacement of the structure in event of a pulse like earthquake. The reduction in relative displacement is more pronounced as the PGA of the earthquake increases. The differences between the no damper model and the uniform distribution range from 20% to 30%. The optimal distribution of the dampers produces a further decrease from the uniform distribution of 9%-25% depending on the PGA. The results hold for both the soil profiles, with minor differences between them.

The study concluded that the optimal damper distribution used introduces important forces to the adjacent structural elements. Although in this case the effect of introducing dampers was positive, reducing displacements and performance levels, the structural detailing needs to thoroughly take into account the additional forces introduced to the structure by the viscous dampers.

The influence of considering soil structure interaction is shown to produce only minor differences with respect of the used soil site. Although the used sites, both corresponding to conditions in Bucharest, are somewhat different, the nonlinear response shows little difference between the two. Considering the current example where an equivalent viscous damping of 25% was used to size the uniform distribution, the introduction of the viscous dampers reduces the response of the structure, however it is not enough for the structure to accomplish more strict performance objectives which would be introduced by the modification of the mean return period.

The current study shows a sizeable decrease in displacements and plastic hinge performance levels when linear viscous dampers are used. Considering specific soil and earthquake conditions it shows that the optimal distribution algorithm employed by Takewaki can be used successfully to find an optimal distribution, which in this case although uses 2 less dampers than the uniform distribution, provides better results.

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