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SEISMIC PERFORMANCE OF MULTISTOREY STEEL FRAMES WITH STRAIN HARDENING FRICTION DAMPERS

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The general aim of the research program is to establish the seismic performance of multistorey steel concentrically braced structures equipped with strain hardening friction dampers in the braces. Previous companion paper in print (Norin FILIP-VACARESCU et al.) presented the experimental program conducted in the CEMSIG research center of the Politehnica University of Timisoara to validate the behaviour of the damper and to determine the behaviour of a brace with damper assembly. The experimental results are used to calibrate a numerical model for the damper and damper-brace assembly in order to determine the performance of this structural system. This paper presents the calibration and implementation of a numerical model based on experimental results, to determine the performance of concentrically braced frames equipped with strain hardening friction dampers in the braces.

Key words: friction damper, strain-hardening, seismic performance, numerical model.

1. INTRODUCTION

For braced structural systems the seismic design concept is based on designing the braces to dissipate the energy induced by the earthquake through the formation of plastic hinges protecting the elements that are considered non-dissipative from degradation. This concept leads to the introduction of the behaviour factor q that reduces the design seismic forces. Introducing damping devices in the structure leads to an increase in energy dissipation capacity of the structure. For these structures the energy dissipation devices represent "sacrificial" elements that assume the role of energy consumers entirely by plastic deformations that occur in the devices. The device prototype that is being analysed here presents a particular pseudo-elastic behaviour. This device does not have elements that yield. Instead, it consumes energy through friction from the elongation and compression of a set of steel rings around a steel core. The damper studied is a strain hardening friction damper and has a different concept to "classical" friction dampers. The aim of the research program is to determine the seismic performance of concentrically braced frames equipped with such dampers. To this end an experimental program was conducted to validate the behaviour of the damper and to determine experimentally the behaviour of the brace-damper assembly using two design concepts. The experimental tests were conducted in the CEMSIG laboratory of the Politehnica University of Timisoara with relevant results presented in the companion paper (Norin FILIP-VACARESCU et al.). Based on the experimental data numerical models for damper and brace-damper assembly were validated and implemented as presented in this paper.

2. NUMERICAL MODELLING OF ELEMENTS

2.1. Numerical model for damper

The main issues with modelling the behaviour of the damper are the pinching effect of hysteretic curve, stiffening behaviour ($K_2 > K_1$) and lack of degradation of the loops. For modelling of damping devices

SEISMOSTRUCT software offers the use of link elements that have the possibility of defining different hysteretic behaviour for each of the 6 degrees of freedom. Several hysteretic behaviours were tested in an attempt to model the behaviour of the SERB damper. These behaviours were defined for the degree of freedom corresponding to axial deformation having a linear elastic behaviour defined for the other 5 with sufficient stiffness to ensure their restraint. Some the trial hysteretic behaviour models were used in the first step using common behaviour curves for friction dampers. A conclusion of these trials was that to model the behaviour of the SERB damper a combination of two link elements was needed in order to obtain a larger stiffness for the second branch of the device. The final damper model was constructed using a two link elements working in parallel namely a bilinear symmetric behaviour type link (Fig. 1) combined with a gap-hook element that is employed to model the pinching of the curve (Fig. 2). The coupled stiffness of these two elements in parallel yielded the desired stiffness for the device.

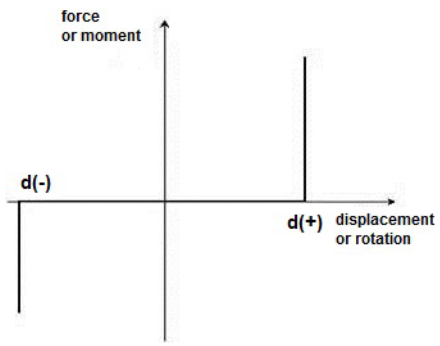

Fig. 1 – Gap-Hook link behavior for damper model [0].

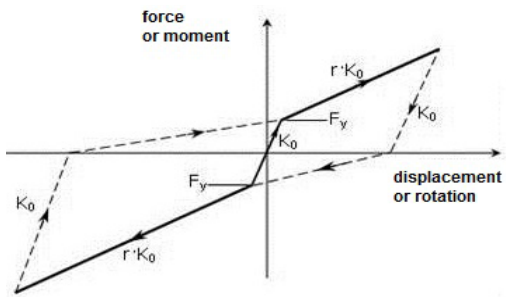

Fig. 2 – Bilinear Symmetric link behavior for damper model [0].

The combined behaviour of the two hysteretic behaviours presented above is shown in (Fig. 3). The damper model was compared with the behaviour obtained experimentally (Fig. 4).

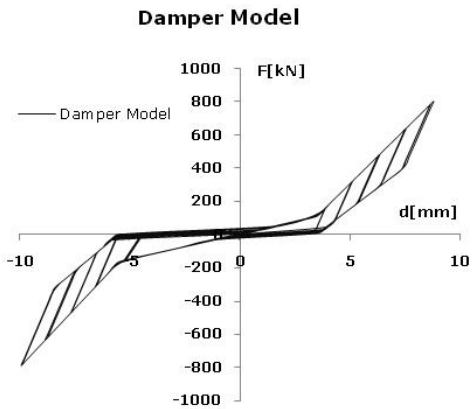

Fig. 3 – Behaviour of damper model.

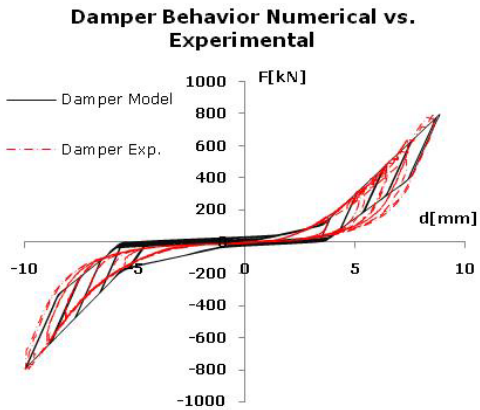

Fig. 4 – Comparison between the damper behaviour of the model and the damper behaviour obtained experimentally.

2.2. Numerical model for brace

Using as reference the experimental behaviour of the HEA100 brace a numerical model that could model with sufficient accuracy the cyclic behaviour of the brace was developed. The main issue that arises with brace modelling is the accurate modelling of brace behaviour at buckling. For the numerical simulation SEISMOSTRUCT version 5.5 Build 10 software [1] was used, a finite element package that uses fibre formulation. The buckling behaviour of brace was modelled using geometric imperfections computed according to EN1993 1-1 [2]. The brace element was divided into segments with each point having corresponding values of the imperfections computed based on a parabolic shape of the deflection with the value of the imperfection computed at midpoint of the element e_0 = 26.54 mm. The material model used for

the steel was Menegotto-Pinto steel model with isotropic hardening (Fig. 5), with parameters obtained experimentally from tensile tests on steel samples from the brace and calibrated parameters as follows: $A_1 = 17$, $A_2 = 0.1$, $A_3 = 0.025$, $A_4 = 8$.

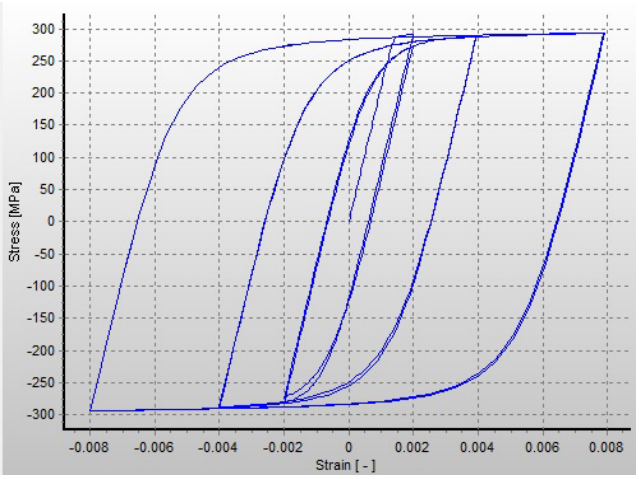

Fig. 5 – Menegotto-Pinto steel model with isotropic hardening [0].

A parametric study was conducted to determine the optimum number of elements in which the brace is to be divided and the value of the imperfections to be adopted comparing the cyclic behaviour of the brace with the behaviour obtained from experimental tests. The brace was divided in 2 and 4 elements (Fig. 6 and Fig. 7) and for each of the two models 4 values of the imperfections were considered: e0, e0/2, e0/3, e0/4 and length of the plastic hinge of 16.66%, 20% and 25% (Table 1).

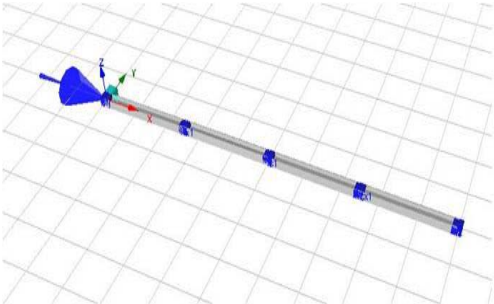

Fig. 6 – Brace discretisation in 4 elements.

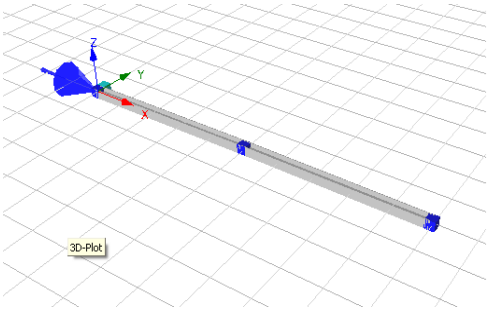

Fig. 7 – Brace discretisation in 2 elements.

Table 1

Parametric study to determine optimum number of elements and plastic hinge length

No. of elements	Imperfection values				Plastic hinge length
2	e_0	$e_0/2$	$e_0/3$	$e_0/4$	16.66%
4	e_0	$e_0/2$	$e_0/3$	$e_0/4$	
2	e_0	$e_0/2$	$e_0/3$	$e_0/4$	20%
4	e_0	$e_0/2$	$e_0/3$	$e_0/4$	
2	e_0	$e_0/2$	$e_0/3$	$e_0/4$	25%
4	e_0	$e_0/2$	$e_0/3$	$e_0/4$	

The best results were obtained for the 2 element brace with a value of imperfection at midpoint of e0/2 and plastic hinge length of 20%. The behaviour of this brace model is presented in Fig. 8, in comparison with the behaviour of the same brace obtained experimentally. Parametric studies conducted by Landolfo et al. [3] also recommended the use of 2 element division for modelling cyclic behaviour of brace.

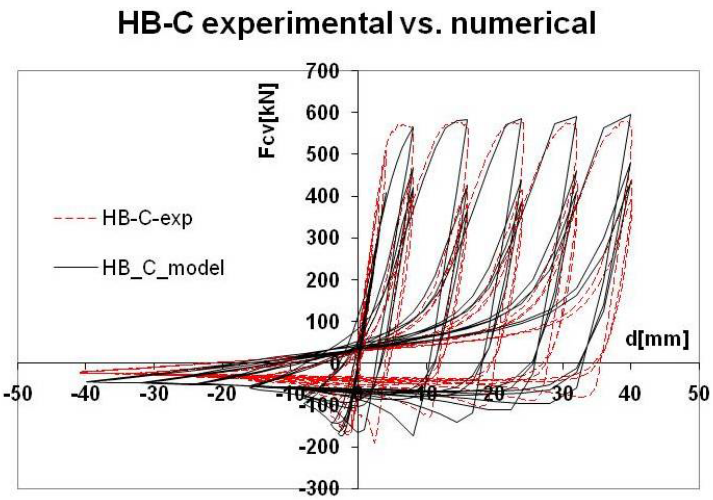

Fig. 8 – Comparison between cyclic behaviors of brace from the numerical model with the one obtained experimentally.

2.3. Numerical model for brace-damper assembly

The two numerical models detailed above, calibrated on experimental test data for damper and brace were then combined to obtain the brace-damper behaviour. The results from the numerical model were compared to the experimental results (Fig. 9).

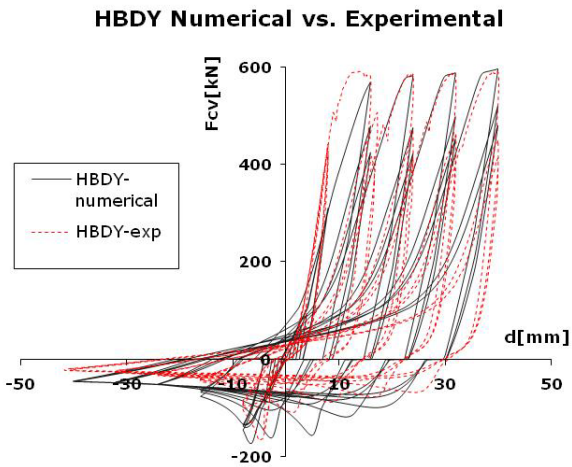

Fig. 9 – Comparison between numerical and experimental behaviour of brace with damper.

The numerical model presents the same global behaviour as the one obtained from experimental data with a damper governed behaviour up to 2ey and a brace governed behaviour afterwards, reaching the same peak values of force for each tension cycle and with sufficiently accurate modelling of sliding of the damper at zero force point transition. These two models for the brace and for the damper as presented above are employed in the overall assessment of the behaviour of the full frame by implementing them for studied concentrically braced frames.

3. EXAMPLE OF APPLICATION OF NUMERICAL MODEL FOR CBF FRAMES

The numerical model calibrated as detailed in the previous chapter is used to determine the performance of this system coupled with concentrically braced frames. The structure analysed is a 5 storey plane frame with an underground level extracted from a 3×3 layout (Fig. 10) with 3 spans of 6 m with chevron bracing in the mid-span and a storey height of 3.5m (Fig. 11). The frame was design according to

EC3 and EC8 with some special considerations from the Romanian seismic design code P100/2006[0] considering the design spectra for Bucharest with a corner period of $T_C = 1.6$s and peak ground acceleration $a_g = 0.24$g.

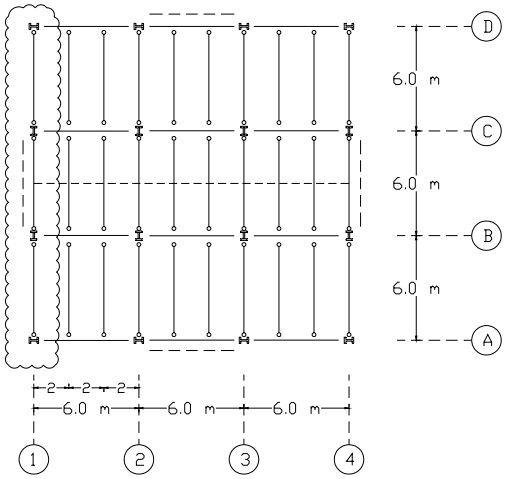

Fig. 10 – Plan layout.

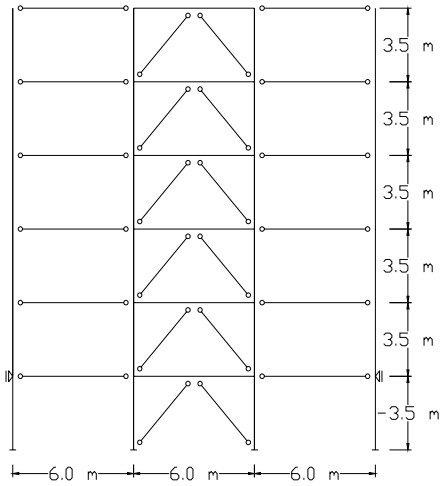

Fig. 11 – Selected frame geometry.

Extensive time-history analyses were conducted using two sets of seismic motions recordings scaled to the design spectra as follows: 7 semi-artificial seismic motion characteristic for soft soil type (Bucharest) and 7 artificially generated seismic motions characteristic for stiff soil (Class B soil according to SREN1998-1[5]) both with and without dampers. The two target spectra were scaled to the fundamental period of vibration of the analysed structure, so as to yield roughly the same design seismic forces (Fig. 12 andFig. 13).

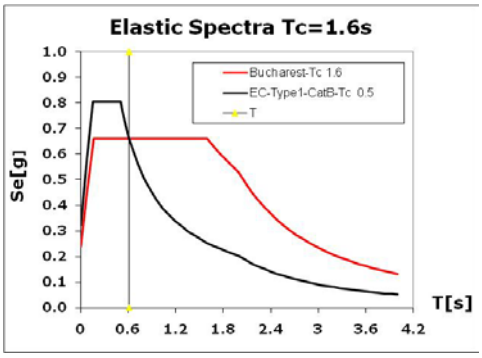

Fig. 12 – Elastic spectra for soft soil type $T_C = 1.6$s.

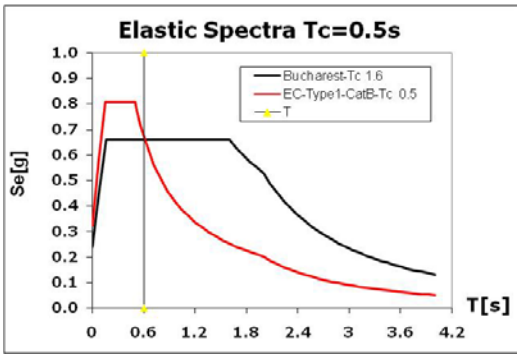

Fig. 13 – Elastic spectra for stiff soil type $T_C = 0.5$s.

Performance based evaluation was performed using acceptance criteria for plastic axial deformation in the braces and plastic rotation for beams and columns according to FEMA356 [6]. Three performance levels were considered for each seismic motion having an acceleration multiplier of 0.5 (30 years return period), 1.0 (100 years return period), 1.5 (475 years return period) corresponding to serviceability limit state (SLS), ultimate limit state (ULS) and collapse prevention (CP):

- SLS: $a_{g,\mathrm{SLS}}=0.5\ a_{g,\mathrm{ULS}}$
- ULS: $a_{g,\mathrm{ULS}}=0.24$ g
- CP: $a_{g,\mathrm{CP}}=1.5\ a_{g,\mathrm{ULS}}$

Maximum drift levels (Fig. 14), maximum drift at each storey (Fig. 15, 16, 17) and top displacement for the structure (Fig. 18, Fig. 19, 20) without dampers are presented as mean values of recorded values for all 7 seismic motions at levels corresponding to SLS, ULS and CP in comparison with the same values recorded for the structure with dampers in the braces.

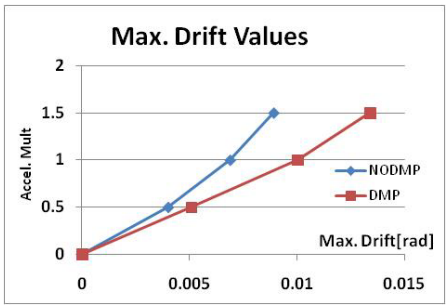

Fig. 14 – Maximum drift values for the structure with and without dampers (stiff soil).

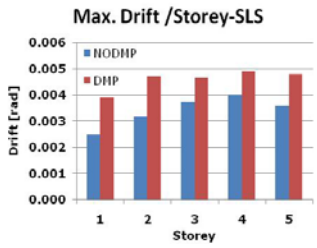

Fig. 15 – Maximum drift at each storey at SLS for the structure with and without dampers.

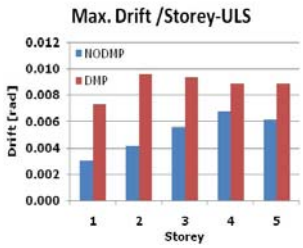

Fig. 16 – Maximum drift at each storey at ULS for the structure with and without dampers.

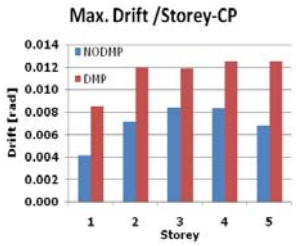

Fig. 17 – Maximum drift at each storey at CP for the structure with and without dampers.

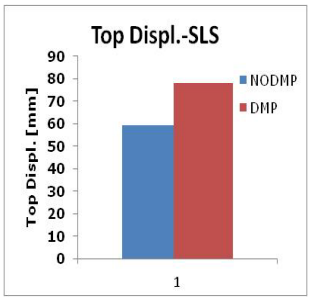

Fig. 18 – Top displacement at SLS for the structure with and without dampers.

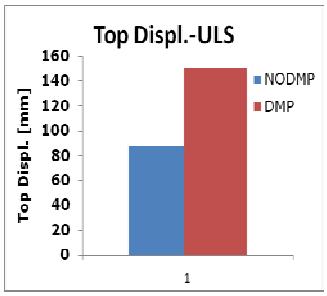

Fig. 19 – Top displacement at ULS for the structure with and without dampers.

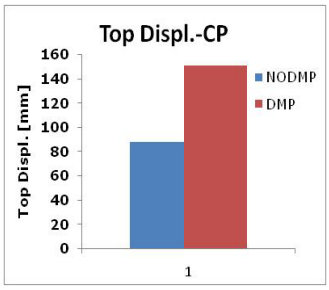

Fig. 20 – Top displacement at CP for the structure with and without dampers.

At the end of each seismic recording used the structure was left to vibrate freely for 10 s. Recorded values of permanent displacement at top of the structure are presented as mean values for all 7 recordings in Fig. 21, 22, 23.

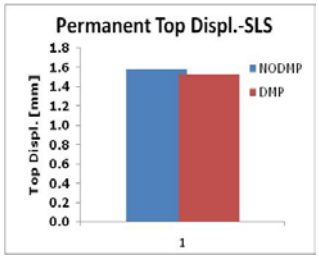

Fig. 21 – Permanent top displacement at SLS for the structure with and without dampers.

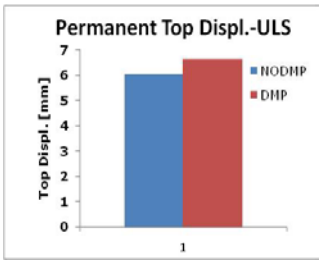

Fig. 22 – Permanent top displacement at ULS for the structure with and without dampers.

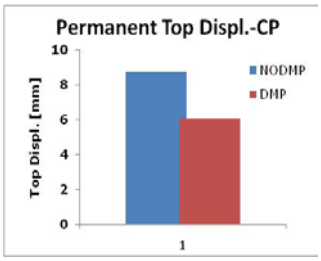

Fig. 23 – Permanent top displacement at CP for the structure with and without dampers.

For all 7 seismic motions characteristic for stiff soil type used the results showed that for all performance levels the building with dampers exhibited an increase in drift for all 5 storeys. The structure with dampers has lower values of permanent displacement at the top of the structure at SLS and CP.

At SLS the structure with dampers avoids almost completely the formation of plastic hinges in braces. At CP the structure with dampers has higher values of plastic deformation/rotation in elements. All plastic deformations/rotations satisfy the acceptance criteria at all levels.

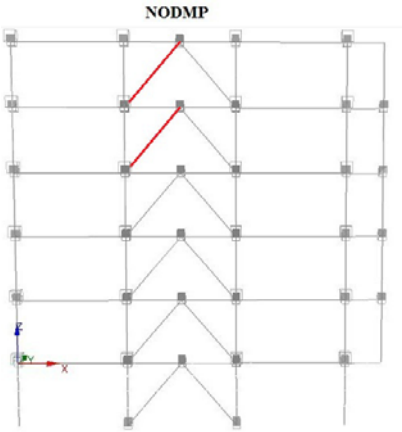

Fig. 24 – Plastic hinge formation for CBF without dampers at SLS.

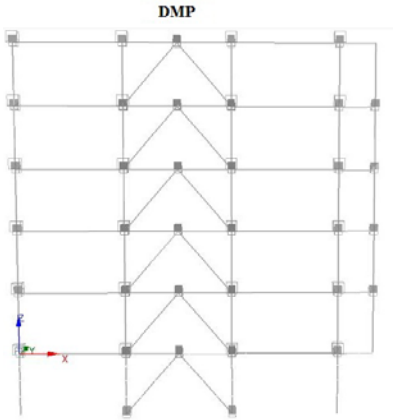

Fig. 25 – Plastic hinge formation for CBF with dampers at SLS.

At ULS both frames with and without dampers form plastic hinges in braces (Fig. 26, 27). The structure with dampers has lower values of axial plastic deformation in braces in compression but with slightly higher values for the braces in tension. At this level the structures have a similar behaviour with similar values of plastic deformation/rotation in elements. No plastic rotations of the central beams are recorded for either structure. All plastic deformations satisfy the acceptance criteria corresponding to life safety (LS) from FEMA 356 [6]. The structure with dampers has lower values of permanent top displacement then the structure without dampers

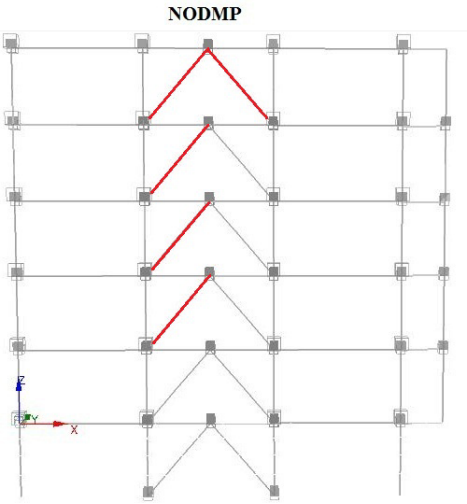

Fig. 26 – Plastic hinge formation for CBF without dampers at ULS.

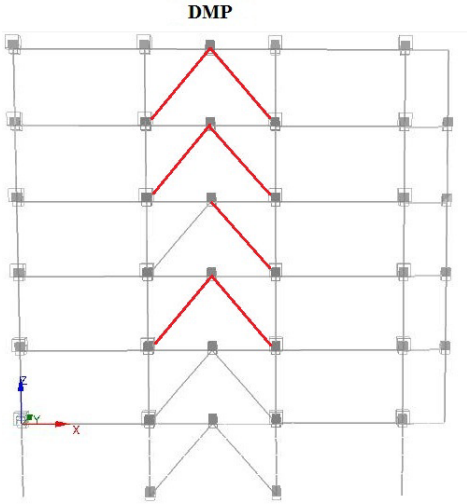

Fig. 27 – Plastic hinge formation for CBF with dampers at ULS.

At CP both frames with and without dampers form plastic hinges in braces and central beams. Structure with dampers has lower values of plastic axial deformation for compression braces and slightly higher for tension braces then the structure without dampers. No plastic rotations of the central beams are recorded for either structure with slightly lower values of permanent top displacement for the structure with dampers (Figs. 28, 29). All plastic deformations satisfy the acceptance criteria corresponding to collapse prevention (CP) from FEMA 356 [6].

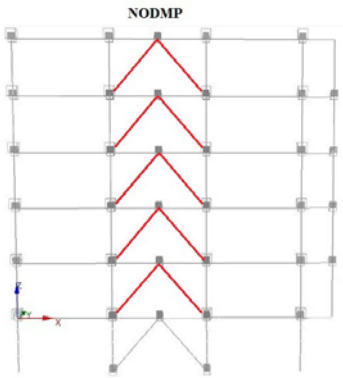

Fig. 28 – Plastic hinge formation for CBF without dampers at ULS.

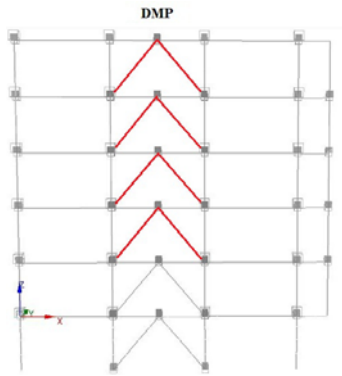

Fig. 29 – Plastic hinge formation for CBF with dampers at ULS

4. CONCLUDING REMARKS

Based on the results of the experimental program a numerical model was developed for the damper analysed in this paper and for the brace with damper assembly. Numerical time-history analysis were made on CBF frames using 7 seismic motions characteristic for stiff soil type with $T_C = 0.5$ s. The structure was analysed with and without SERB type dampers in the braces.

The structures with dampers were more flexible in all cases with drift levels and values of maximum top displacement higher than the ones of the structures without dampers at all performance levels. The introduction of the damper in the braces lead to a reduction of permanent drift values for structure.

Plastic hinge formation in elements and the values of plastic axial deformations and plastic rotations were as follows:

- At SLS plastic hinges appear exclusively in the braces. The damper avoids almost completely the formation of plastic hinges in braces keeping the structures in elastic domain.
- At ULS plastic hinges are limited to braces for both structures. The structures with and without dampers have a similar behaviour with values of all plastic axial deformation that satisfy the acceptance criteria for LS.
- At CP, for the CBF frame, plastic hinges form only in braces with only a few exceptions when plastic rotations are recorded in central beams but with very low values.

As a conclusion this type of damper is efficient in reducing the seismic response of a building for earthquakes characterized by short corner period $T_C = 0.5$ s (stiff soil) by preventing the formation of plastic hinges at SLS and reducing the permanent displacement of the structure. In this article we will present only relevant results for analyses on CBF structures, with and without dampers for the seismic motion characteristic to stiff soil. As a conclusion for earthquakes characterized by long corner period $T_C = 1.6$ s (soft soil) was that this type of damper is not effective in improving the behaviour of the structure. Under this type of seismic motions the structures with dampers form plastic hinges in non-dissipative elements with values that exceed the acceptance criteria for the corresponding performance levels.

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