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BEHAVIOR OF CONCENTRICALLY BRACED FRAMES WITH FRICTION DAMPERS

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Abstract. *The papers investigates the behavior of steel frames with friction dampers connected to steel braces Both numerical and experimental analyses have been conducted in order to characterize in terms of energy dissipation capacity such a type of systems and obtain an equivalent hysteric model to be used in numerical simulations. Tests have been realized for two series of brace members equipped with strain hardening friction dampers: 1st series with the brace designed to avoid buckling; 2nd series with the brace working and prone to post-elastic buckling after the damper consumed its stroke. The equivalent brace-damper model experimentally calibrated has been applied in numerical simulation of multi-storey frames in order to observe their performance in comparison with conventional centric braced systems. In the paper the test results and numerical simulations are summarized as well as the resulted conclusions.*

1 INTRODUCTION

In general damping devices can be classified according to their behavior as follows:

1. Velocity dependent devices

These devices are dependent of the velocity of application of the load. They modify their hysteretic behaviour according to velocity (Figure 1). As an example we can mention here fluid viscous dampers and fluid spring dampers.

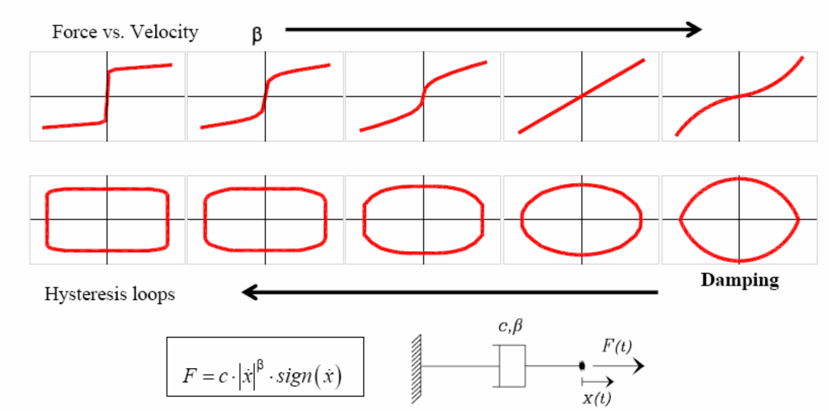


Figure 1: Influence of velocity on hysteretic behavior of fluid viscous dampers [1]

2. Displacement dependent devices

In the category enter devices non-linear behaviour such as: steel hysteretic dampers, shape memory alloy devices, and with linear behaviour such as: elastomeric viscoelastic devices. The damper to be used in the research is a strain hardening friction damper of SERB type manufactured in Romania with the hysteretic behaviour described in Figure 2.

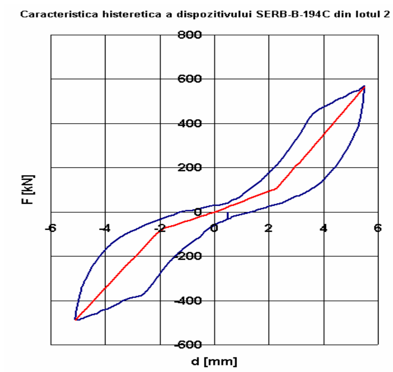


Figure 2: Hysteretic behavior of SERB type friction damper prototype

The paper analyses the behavior of steel dual frames with centrically braced frames in the mid span. The aim is to study and analyze new systems to improve the seismic behavior of steel structures

The studied frame is a dual frame with moment resisting frames and concentrically braced frames equipped with friction dampers at the base of the braces to improve their seismic response. The frame was designed according to EC3, EC8 and special considerations from the Romanian seismic design standard P100-2006 [3] for response spectra with TC=1.6.

For the experimental program a part of the braced frame was extracted and tested in laboratory both with and without damping devices (Figure 3).

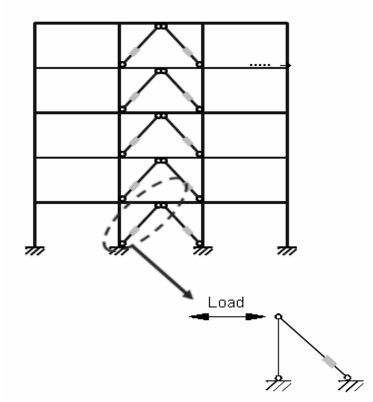


Figure 3: part of the braced frame extracted for experimental tests

2 DESIGN PRINCIPLES

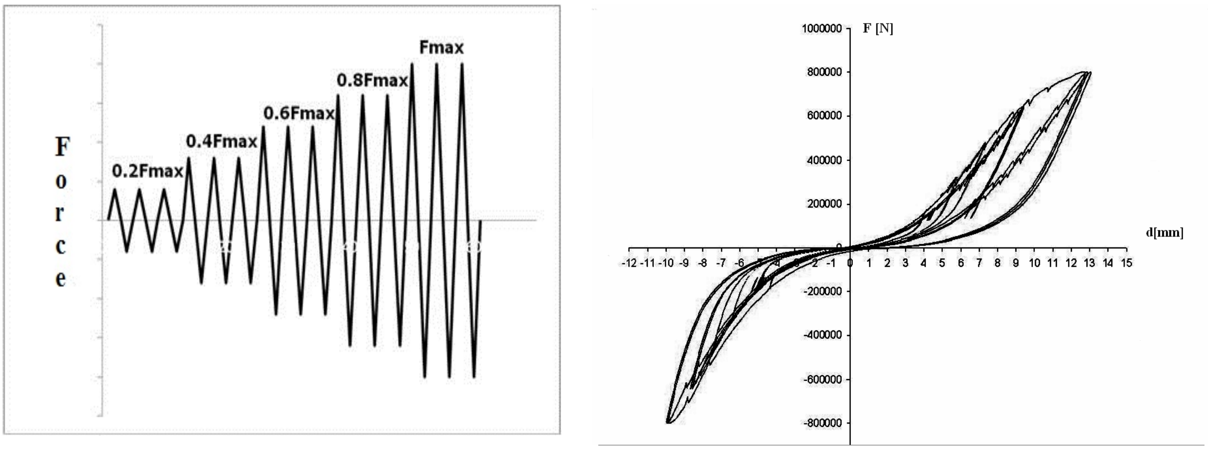
For braced structural systems the seismic design concept translates in designing the braces to dissipate the energy induced by the earthquake through the formation of plastic hinges protecting the elements that are considered non-dissipative from degradation. This concept leads to the introduction of the behavior 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 analyzed here presents a particular pseudo-elastic behavior. 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 structures equipped with this particular type of dampers can be designed using two different concepts. A first concept is to design the braces to remain in elastic domain controlling the response of the structure solely through the friction dampers. In this case the structure has no ductile elements and is designed with a behavior factor corresponding to low dissipative structures of $1<q<2$ and benefits from the reduction of design seismic forces due to the increase in global damping. However, introducing supplemental damping in the structure leads to a much smaller reduction of design seismic loads compared to the reduction that comes from using a higher behavior factor value that corresponds to a dissipative design approach in which the brace itself is the main energy consuming element. For example an increase of damping in the structure to 15% critical damping leads to a reduction of the loads with only 35% [2]. Furthermore these types of dampers have a brittle failure that must be avoided in all configurations. All the above mentioned lead to a second design concept in which the damper has sufficient over strength compared to the brace to assure that the brace has deformation in the plastic domain and is the weaker element in the configuration. This concept should benefit in theory from both the energy dissipation capacity of the brace and the supplemental damping from the device, and the failure will occur in the brace and not in the device. For seismic motion levels corresponding to ultimate limit state the brace is the „active" element according to the dissipative design concept and for service limit state the damper is the „active" element ensuring that the brace remains in elastic domain and providing an overall damping increase. According to P100/2006 [3] the relative story drift criteria for SLS is 0.008h, where h is the story height. For the structure analyzed here this translates in a drift value of 28mm which leads to a displacement of 20mm in the brace. The damping devices were selected to satisfy this displacement criteria corresponding to SLS. Both design concept presented above will be used in the configuration of the experimental tests that will be presented further on.

3 EXPERIMENTAL PROGRAM

The experimental program is divided in two parts:
(1) experimental tests on friction dampers;
(2) experimental tests on single brace configuration with and without dampers.

3.1 Experimental tests on friction dampers

Experimental tests were performed on two dampers with maximum capacity of 800kN and 1500kN. The tests were performed in the CMMC laboratory using a cyclic load protocol. The tests were done using a load control protocol having as reference the maximum capacity of each damper Three cycles were considered at each force level (Figure 4a).



a. b.

Figure 4: Cyclic force control load protocol (a) and force displacement curve obtained for 800kN damper (b)

The hysteretic curves obtained experimentally were in accordance to those supplied by the manufacturer (Figure 4b).

3.2 Experimental tests on single brace configuration with and without dampers

The experimental model is made from half of the beam and the brace, hinged at both ends with or without dampers installed. In the experimental program the two design concepts presented before can be found. The first concept is based on a design of the system so that energy dissipation occurs in the damper alone. This was achieved by designing the brace with sufficient over strength relative to the maximum capacity of the device. The second design concept follows both the behavior of the damper and of the beam in post-elastic domain. This was achieved by choosing the device with sufficient over strength with respect to the brace. According to these principles the cross sections of the brace were chosen as follows:
(1)„Strong” brace configuration (HEA240);
(2)„Weak” brace configuration (CHS D133x5 and HEA100)
The experimental program is detailed in Table 1:

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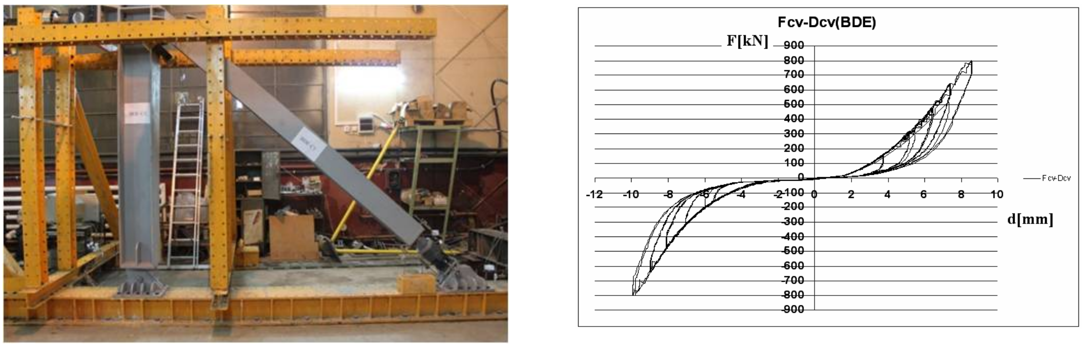
No	Brace	Specimen	Damper	Type	Tests	Measured Parameters
1.	HEA 240	BDE-C	YES	cyclic	2	-relative displacement of the brace -total displacement -brace force -damper displacement -global behavior
2.	CHS, D133,t=5	B-MT, B-MC	NO	monotonic	2	
3.	CHS, D133,t=5	B-C	NO	cyclic	2	
4.	CHS, D133,t=5	BDY	YES	cyclic	2	
5.	CHS, D133,t=5	BDY	YES	cyclic	2	
6.	HEA100	HB-MT, HB-MC	NO	monotonic	2	
7.	HEA100	HB-C	NO	cyclic	1	
8.	HEA100	HBDY-C1	YES	cyclic	1	
9.	HEA100	HBDY-C2	YES	cyclic	1	

Table 1 : Experimental program for brace tests with and without dampers

3.3 „Strong" brace configuration with damper (BDE)

In order to validate the experimental behavior of the test configuration in this stage is based on the „strong" brace concept in which the brace is designed to remain in elastic domain. The aim of these tests is to study the behavior of the damper with the brace as the element with over strength. The load protocol used is identical with that used for the single damper having as reference the maximum force capacity of the damper. The experimental test configuration (Figure 5a) and total force-displacement recorded for the HEA240 brace (Figure 5b) are presented below.



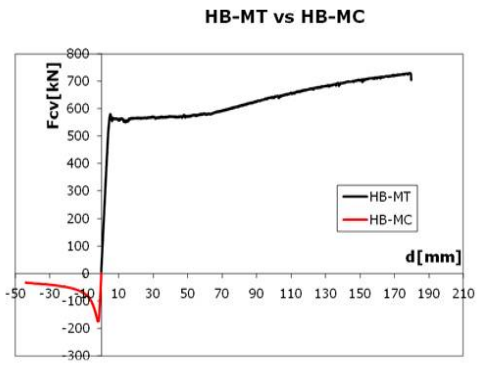
a. b.

Figure 5: Experimental test configuration (a) and global force-displacement curves for specimen BDE (b)

The total response of the system is governed by the behavior of the damper resulting in a symmetrical behavior both in tension and in compression without strength and stiffness degradation. The brace remains in elastic domain for the entire test duration. The test was stopped when the device reached its maximum capacity. For the "weak" brace configuration two types of cross-sections for the brace were used as it was presented in the summary of experimental program. The procedure of experimental investigation was the same for the two brace types and the results showed the same global behavior of the brace with and without damper. For this reason only the experimental data obtained on HEA100 braces will be presented here.

3.4 HEA100 brace with and without damper

Monotonic tests were conducted at first for the brace without damper in order to determine the yield displacement and yield force which was used to construct the load protocol according to ECCS procedure that was used for the cyclic tests that followed. The experimental test setup had the same general configuration as the one described in the paragraph above. The behavior curves obtained for the monotonic tests and buckling of brace is presented in Figure 6a,b.

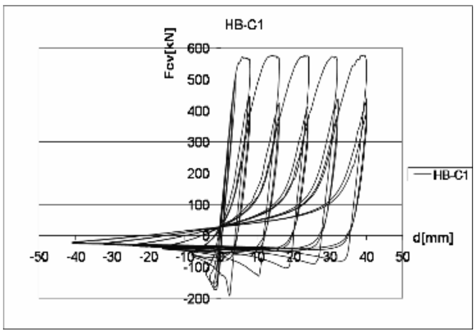


a.

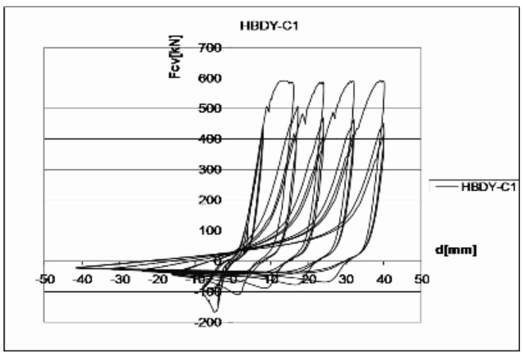
b.

Figure 6: Monotonic test results on HEA brace without dampers (a) and buckling of the brace in compression (b)

The load protocol for the cyclic tests was constructed using the results from the monotonic tests. The loading protocol is made according to ECCS procedure with cycles at steps of magnitude 0.25, 0.5, 0.75, 1.0, 2, 4, 6, 8 times ey with 3 cycles at each step following ey (ey-yield displacement of brace obtained from monotonic tests). The hysteretic behavior of the HEA brace without damper under cyclic load with ey=4 mm is presented in Figure 7a and the brace equipped with damper under cyclic load in Figure 7b.



a.



b.

Figure 7: Hysteretic behavior of the HEA100 beam without damper under cyclic load(a) and with damper under same cyclic load(b)

4 DISCUSSION

In order to analyze the influence of the damper on the global behavior of the brace the hysteretic behavior of the brace without damper is taken as reference curve. The behavior of the brace with damper obtained for the two design concepts of „weak" and „strong" brace is therefore compared with the hysteretic behavior of the brace without damper.

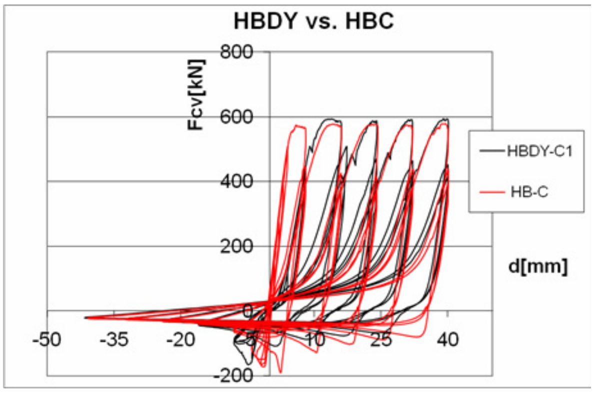
4.1 „Strong" brace with damper configuration

The behavior of the brace with dampers taken as reference is considered that recorded for double T section profile of the brace (HB-C) mainly following two parameters: recorded total force in the brace and total displacement of the brace. This is compared to the behavior recorded for the system comprised of „strong" brace (HEA240) with damper (BDE).

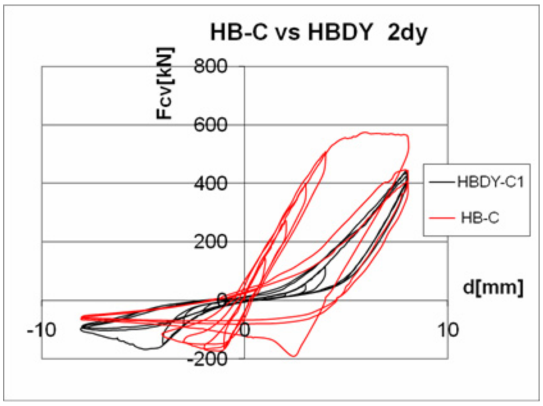
In this design concept the global behavior of the system of brace and damper is completely governed by the constitutive law of the damper and its properties. The system does not suffer any degradation in terms of strength and stiffness these being strictly dependent on the damper properties. The system will continue to take on load until the maximum capacity of the device is reached, with the brace remaining in elastic range. This high load carrying capacity without strength and stiffness degradation represents the advantage of this type of design concept but can also lead to an increase of the load levels in the beams and columns of the braced frame due to the pseudo-elastic behavior of the damper. Furthermore failure of this type of system is a brittle one due to failure of the device and must be avoided.

4.2 „Weak" brace with damper configuration

In this design concept the brace is allowed to have plastic deformation and the global behavior of the damper brace system is a mixed one. The weak element in this configuration is the brace which will ultimately fail. The behavior of this system is presented in Figure 8a in comparison with the behavior of the same brace, under the same load protocol but without damper.



a.



b.

Figure 8: Comparison between the total hysteretic behaviors of the "weak" brace with damper and without damper (a) and hysteretic behaviors up to 2ey (b)

In both configurations the force level drops significantly after the fist cycle at each load step and the next two cycles of the same load step. The brace with damper has a higher flexibility and yields at the same load step but at a displacement of approximately 50% higher. For this system up to a level of 2ey the global behavior is governed by the behavior of the damper and by the behavior of the simple brace at higher load steps. The difference between these two systems can be observed more closely up to a level of 2ey (Figure 8b).

Up to this level the behavior is that given by the damper parameters. At tension cycles the brace remains in elastic domain and the load level in the system is significantly smaller then that of the brace without damper with a higher overall flexibility. For compression cycles the brace with damper buckles at the same load level as the one without damper but has a higher deformation capacity due to the damper properties. The experimental results are in agreement with the two design concepts considered. For the starting load levels of up to 2dy the brace remains in elastic domain and has a lower level of energy dissipation but there is a significant decrease in load level due to the damper and also an increase in flexibility. After this level the hysteretic behavior of the system is very similar to that of the brace without damper, with energy dissipation due to the formation of a plastic hinge in the brace. Failure in this design concept is represented by the failure of the brace in compression. As a preliminary conclusion it is expected that this type of damper could improve the behavior of rigid structures that are sensitive to formation of plastic hinges at levels corresponding to service limit state.

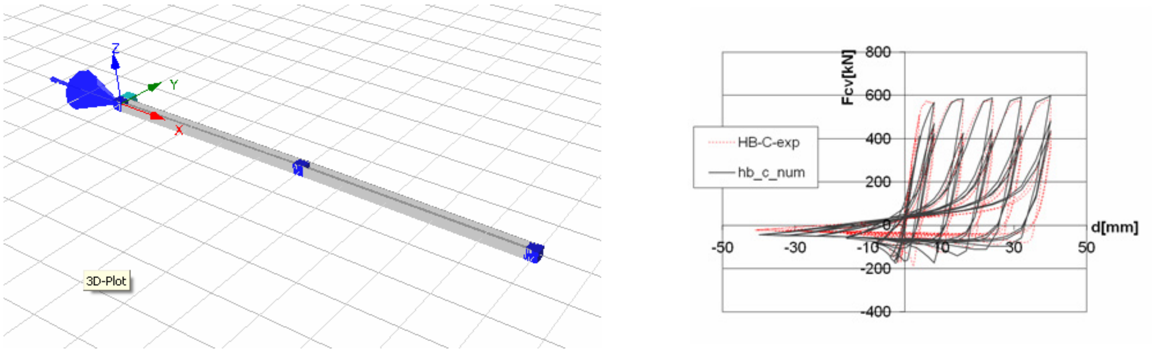
5 NUMERICAL MODELLING

The numerical modeling can be split mainly in two independent parts or stages. The first stage consists of numerical simulation of the behavior of the two elements, the brace and the damper separately, but most importantly their behavior as a whole. The second stage consists of a series of numerical simulation on the full dual frame with and without dampers in the braces. Numerical time-history analysis will be conducted using a set of recorded seismic motions scaled to the design spectra. The final stage consists of performance base evaluation of the structure with this type of damping devices and the comparison with other types of damping devices used for seismic protection.

5.1 Brace modeling

The main issue that arises with brace modeling is the accurate modeling of brace behavior at buckling. For the numerical simulation SEISMOSTRUCT version5.5 Build 10 software was used, a finite element package that uses fiber formulation. The buckling behavior of brace was modeled using geometric imperfections computed according to EN1993 1-1[4]. 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. 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 behavior of the brace with the behavior obtained from experimental tests. The brace was divided in 2 and 4 elements and for each of the 2 models 4 values of the imperfections were considered: e_0, $e_0/2$, $e_0/3$, $e_0/4$ (Figure 9a). The material properties used were also obtained experimentally from tension tests on steel samples from the HEA100 brace.

The best results were obtained for the 2 element brace with a value of imperfection at midpoint of e0/2 (Figure 9b). Parametric studies conducted by Landolfo et.al.2010 [6] also recommended the use of 2 element division for modeling cyclic behavior of brace

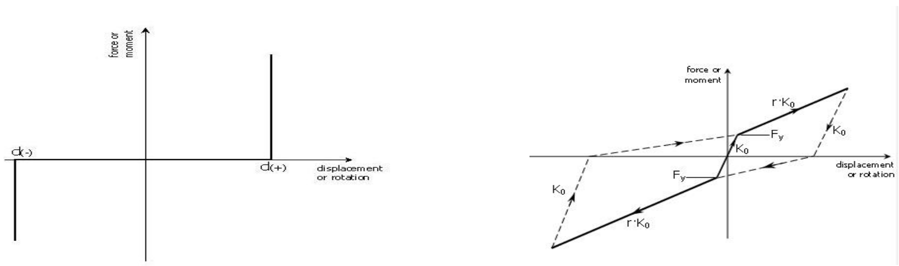


a. b.

Figure 9: Brace model in SeismoStruct (a) and comparison between cyclic behavior of brace from the numerical model with the one obtained experimentally

5.2 Damper modeling

For modeling of devices SEISMOSTRUCT software offers the use of link elements that have the possibility of defining different hysteretic behavior for each of the 6 degrees of freedom. To model the behavior of the SERB damper a combination of two parallel link elements was used. The hysteretic loops of the damper were modeled using a bilinear symmetric behavior type link (Figure 10b) combined with a gap-hook element that is employed to model the pinching of the curve (Figure 10a). These types of behavior laws were defined only for the degree of freedom corresponding to axial deformation, the other 5 degrees having a linear elastic behavior with sufficiently high stiffness as to ensure their restraint.



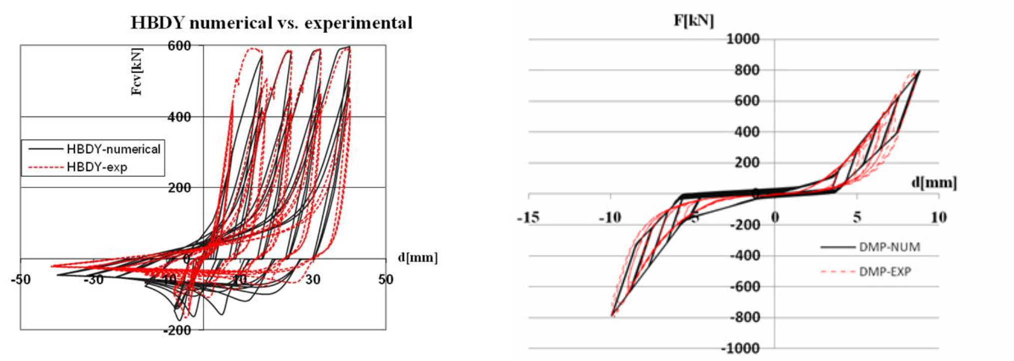
a. b.

Figure 10: Gap-hook behavior (a) and bilinear symmetric behavior of link element (b) [5]

The behavior obtained in the numerical model using the combined behavior these 2 types of link elements provide a satisfactory model behavior of the damper (Figure 11b)

5.3 Brace with damper model

The behavior of the brace with damper is obtained combining the models discussed above for the brace and the damper. The results from the numerical model were compared to the experimental results HBDY-C1 (Figure 11a).



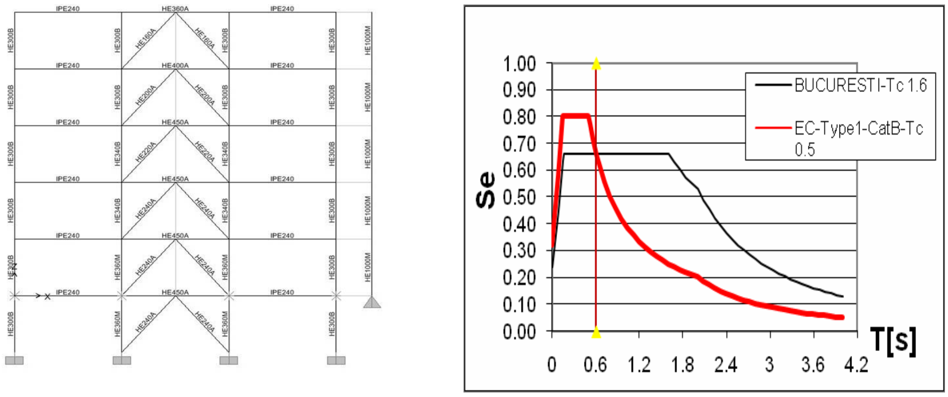
a. b.

Figure 11: Comparison between experimental and numerical behavior of HEA brace with damper (a) and comparison of the experimental and numerical damper behavior (b)

The numerical model presents the same global behavior as the one obtained from experimental data with a damper governed behavior up to 2ey and a brace governed behavior afterwards, reaching the same peak values of force for each tension cycle and with suficiently acurate modelling of sliding of the damper at zero force point transition. This two models for the brace and for the damper as presented above are employed in the overall assesment of the behavior of the full frame.

5.4 Numerical simulation on the full frame

The structure analyzed is a 5 storey plane frame extracted from a 3x3 layout with 3 spans of 6m with chevron bracing in the mid-span and a storey height of 3.5m (Figure 12a). The frame was design according to EC3 and EC8 with some special considerations from the Romanian seismic design code P100/2006 considering the design spectra for Bucharest with a corner period of TC=1.6s. Time-history analyses are conducted using two sets of seismic motions recordings scaled to the design spectra as follows: 7 recorded 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) both with and without dampers (Figure 12b). The two target spectra were scaled to the fundamental period of vibration of the analyzed structure, so as to yield roughly the same design seismic forces. Three performance levels were considered for each seismic motion having an acceleration multiplier of 0.5, 1.0, 1.5 corresponding to serviceability limit state (SLS), ultimate limit state (ULS) and collapse prevention (CP) respectively. Performance based evaluation was performed using acceptance criteria for plastic deformation in the braces and plastic rotation for beams and columns according to FEMA356



a. b.

Figure 12: Analyzed frame geometry (a) and design spectra used (b)

For the first set of time-history analysis the 7 seismic motion recordings for a soft soil type scaled for the Bucharest response spectra were used. For all 7 seismic motions used the results showed that for all performance levels the building with dampers exhibited a significant increase in drift for all 5 storeys. For SLS (0.5) the building without dampers does not form any plastic hinges in elements while the building fitted with dampers forms plastic hinges in the bracing with values of plastic deformation that check the acceptance criteria for immediate occupancy (IO) from FEMA. At ULS (1.0) both frames with and without dampers form plastic hinges in braces and in the central beams. At this level the structure with dampers has a higher number of plastic hinges in elements and a higher value of plastic deformation/rotation in elements then the structure without dampers. The values of plastic rotation for the beams exceed the acceptance criteria corresponding to life safety (LS) from FEMA 356. At collapse prevention (1.5) the behavior of both types of frames is considered unsatisfactory due to the formation of plastic hinges in central columns at most levels. As example max drift values for 3 of the 7 seismic motions with T_C=1.6s are shown in Figure 13 and values of plastic deformation/rotation for first seismic recording VR-77-INC-NS at SLS (Table 2) and ULS (Table 3, Table 4, Table 5) with and without dampers in the braces are presented.

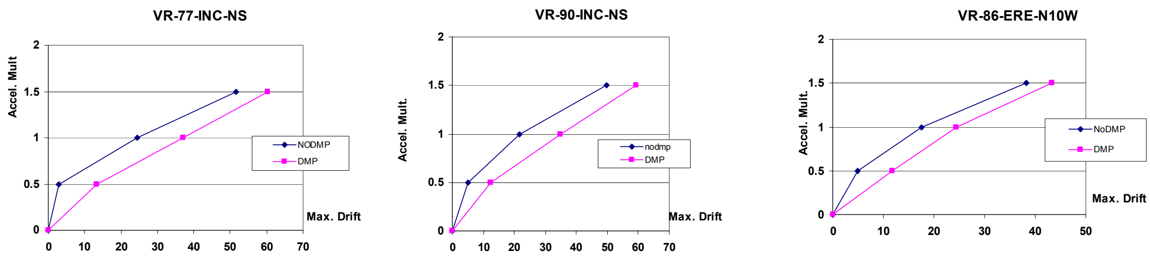


Figure 13: Comparison of the maximum drift levels for 3 of the 7 seismic motions with T_C=1.6s

BRACE Compr.	Plastic deformation demand-SLS , mm		Plastic deformation capacity, mm
LOC.	NODMP	DMP	IO
BR5R	-	0.43893	0.692
BR4L	-	0.75084	0.8945
BR2L	-	1.67516	1.1225

BR3L	-	1.18621	0.99125
BR5L	-	0.71606	0.692
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 2: Plastic deformation for braces in compression at SLS for VR-77-INC-NS seismic recording (soft soil)

BRACE Tens.	**Plastic deformation demand-ULS , mm**		**Plastic deformation capacity, mm**
LOC.	**NODMP**	**DMP**	**LS**
BR1R	5.78823	2.61921	40.2283
BR2L	3.11884	0.27856	40.2283
BR2R	18.61805	11.77078	40.2283
BR3R	17.40089	18.75723	39.97
BR4R	2.69104	5.94618	39.48
BR3L	-	3.90426	39.97
BR5R	-	0.95701	39.039
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 3: Plastic deformation for braces in tension at ULS for VR-77-INC-NS seismic recording (soft soil)

BRACE Compr.	**Plastic deformation demand-ULS , mm**		**Plastic deformation ca-pacity, mm**
LOC.	**NODMP**	**DMP**	**LS**
BR1R	0.33586	0.90386	22.45
BR2L	0.59929	2.00463	22.45
BR2R	0.56675	2.27719	19.825
BR3R	0.52462	1.35489	17.89
BR4R	0.52487	2.18994	13.84
BR3L	0.45988	-	13.84
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 4: Plastic deformation for braces in compression at ULS for VR-77-INC-NS seismic recording (soft soil)

BEAM	**Plastic rotation demand-ULS , mm**		**Plastic rotation capac-ity, mm**
LOC.	**NODMP**	**DMP**	**LS**
grc3b	-	0.009598	0.007818
grc4a	-	0.007697	0.009118
grc2a	0.01832	0.028828	0.007969
grc3a	0.004891	0.012671	0.008091
grc1a	0.021767	0.03177	0.008009
grc2a	0.01832	0.028546	0.008532
grc2b	0.008985	0.019499	0.0073
grc1b	0.011624	0.01955	0.008447

Table 5: Plastic rotation at ULS for central beams VR-77-INC-NS seismic recording (soft soil)

For the second set of time history analysis the 7 seismic motion recordings for a hard soil type scaled for the Type 1 response spectra and Class B soil according to EN 1998 were used. For all 7 seismic motions used the results showed that the building with dampers exhibited a decrease in maximum drift values for SLS and an increase for the other 2 performance levels. For SLS the frame with dampers does not form any plastic hinges in the braces as opposed to the one without dampers (Figure 14a, b). At ULS the presence of dampers continues to im-prove the behavior of the structure by reducing the number of plastic hinges in elements but with higher values of drift at each storey (Figure 14c, d). As example max drift values for 3 of the 7 seismic motions with T_C=0.5s are shown in Figure 15and values of plastic deformation/rotation for first seismic recording at SLS (Table 6) and ULS (Table 7, Table 8, Table 9) with and without dampers in the braces are presented.

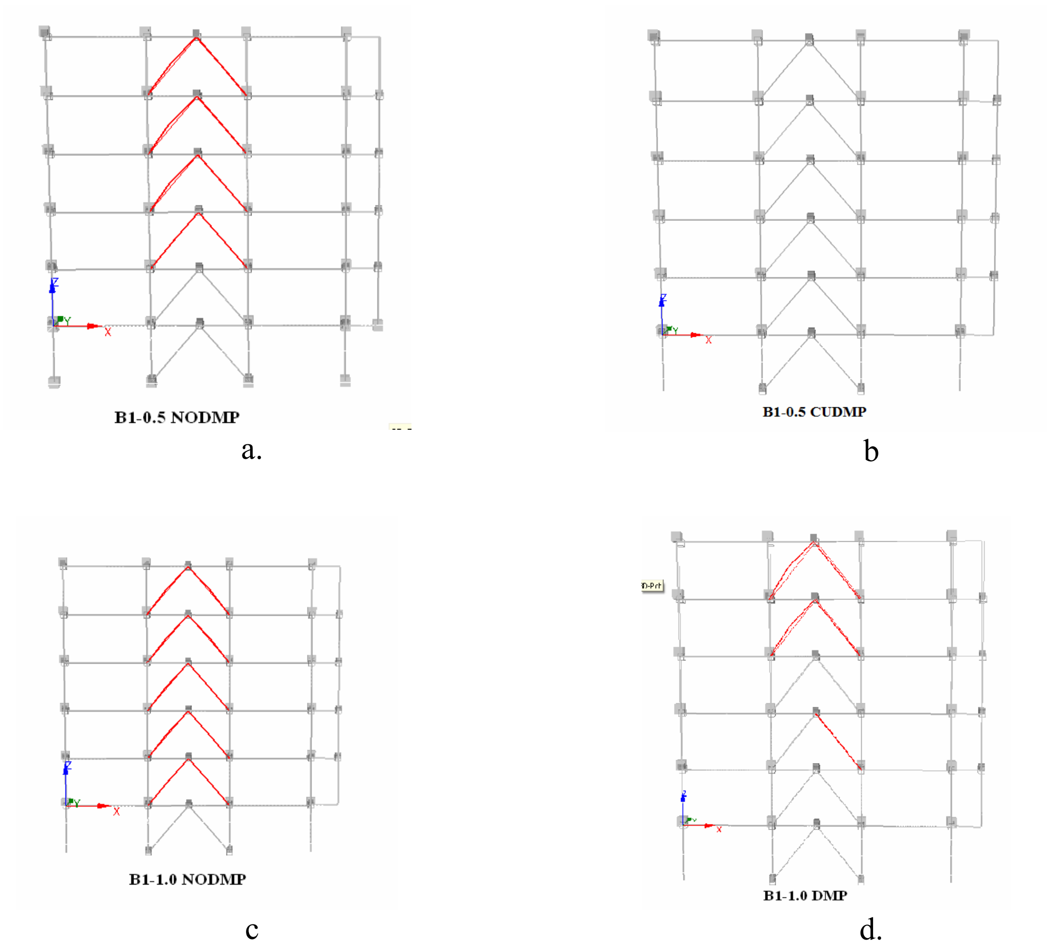


Figure 14: Plastic hinge location at SLS for the structure without dampers (a) and with dampers (b) and corresponding to ULS without dampers(c) and with dampers (d) for one of the 7 seismic motions with T_C=0.5s

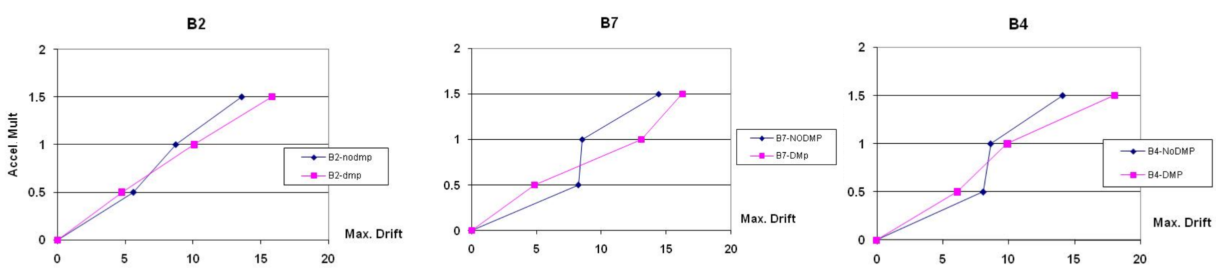


Figure 15: Comparison of the maximum drift levels for 3 of the 7 seismic motions with T_C=0.5s

BRACE Compr.	Plastic deformation demand-SLS , mm		Plastic deformation capacity, mm
LOC.	NODMP	DMP	IO
BR5R	0.271	-	13.84
BR3R	0.567	-	19.825
BR4L	0.682	-	17.89
BR5L	0.455	-	13.84
BR2L	0.291	-	22.45
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 6: Plastic deformation for braces in compression at SLS for B1 seismic recording (stiff soil)

BRACE Tens.	Plastic deformation demand-ULS , mm		Plastic deformation capacity, mm
LOC.	NODMP	DMP	LS
BR1R	2.642	-	40.2283
BR2L	1.336	-	40.2283
BR2R	5.497	-	40.2283
BR3R	2.627	-	39.97
BR4L	2.061	1.612	39.48
BR4R	1.979	-	39.48
BR5L	3.482	4.330	39.039
BR5R	3.308	5.975	39.039
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 7: Plastic deformation for braces in tension at ULS for B1 seismic recording (stiff soil)

BRACE Compr.	Plastic deformation demand-ULS , mm		Plastic deformation capacity, mm
LOC.	NODMP	DMP	LS
BR1L	0.022	-	22.45
BR2L	0.442	-	22.45
BR3L	0.351	-	19.825
BR4L	0.819	0.822	17.89
BR5L	1.216	1.395	13.84
BR5R	0.615	1.857	13.84
BR(storey no.)R- right brace for selected storey			
BR(storey no.)L- left brace for selected storey			

Table 8: Plastic deformation for braces in compression at ULS for B1 seismic recording (stiff soil)

BEAM	Plastic rotation demand-ULS , mm		Plastic rotation capacity, mm
LOC.	NODMP	DMP	LS
grc4a	-	0.003052	0.0091304

Table 9: Plastic rotation at ULS for central beams for B1 seismic recording (stiff soil)

At CP the global behavior of the two structures is similar, however for some of the seismic recordings for which the structure without dampers forms plastic hinges in central columns at the base of the structure the structure with dampers has no plastic hinges in columns. For all performance levels the recorded plastic deformations/rotations satisfy the acceptance criteria.

6 CONCLUSION

Experimental tests were conducted on SERB type friction damper and on damper with brace configuration. Two design concepts with "weak" and "strong" brace configuration were proposed and tested. The main purpose of the experimental program was to obtain the hysteretic behavior of the friction damping devices and the global behavior of the ensemble of brace together with damper in the two design concepts. A numerical model was developed for the brace with damper assembly and used in a performance based evaluation of the building under 2 sets of 7 recorded seismic motions scaled on 2 types of response spectra. The first set of numerical analyses showed that the frame equipped with dampers increases the flexibility of the structure, forming plastic hinges at SLS with a higher number of plastic hinges with higher values of plastic deformation/rotation in braces and beams respectively that no longer satisfy the performance criteria and generally a worse global behavior. The conclusion is that this particular type of damper is not efficient in reducing the seismic response of a building for earthquakes characterized by a high value of corner period T_C=1.6s (soft soil). The second set of numerical analyses showed that the frame equipped with dampers has a better performance avoiding the formation of plastic hinges at SLS and reducing the values of maximum drift, reducing the number of plastic hinges in elements at ULS and for some recordings avoiding the formation of plastic zones in columns at CP. The conclusion is that this particular type of damper is efficient in reducing the seismic response of a building for earthquakes characterized by short corner period T_C=0.5s (stiff soil).

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