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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF NON-SEISMIC REINFORCED CONCRETE FRAMES STRENGTHENED WITH CONCENTRIC STEEL BRACES

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Abstract. *Many reinforced concrete structures built before 1960's were designed to resist mainly gravity loads and wind, even if they were located in seismic zones. That is why when subjected to earthquakes of intensities even below the design level, they are at risk because of poor detailing and lack of capacity. Evaluation of such type of structures according to the present seismic design provisions leads, in most cases, to the necessity of strengthening.*

In the paper, the strengthening of non-seismic reinforced concrete frames with dissipative inverted V bracing systems is presented. Conventional concentric V braces and buckling restrained V braces are used. Portal frames with and without bracings are isolated from a real structure and tested experimentally under monotonic and cyclic loading. The tests aimed to offer information related to the real dissipation capacity of the initial unretrofitted structure and the retrofitted structures, including the effectiveness of connecting details of pre-stressed tendons, specifically designed for connecting braces in an existing frame. Both test results and numerical simulations on portal frames and on strengthened building frames have been used to evaluate the q factors.

1 INTRODUCTION

In many areas with seismic risk, the reinforced concrete structures (RC) built before 1960's were designed to resist mainly gravity loads. The main deficiencies of these structures are linked to low quality of material (poor concrete) and insufficient detailing (insufficient confinement and anchorage of the reinforcements), and thus leading to reduced local and global ductility and, finally, to a poor seismic response. When such types of structures are evaluated according to the present seismic design provisions, one finds out that, in almost all cases, strengthening is needed. Intervention strategies must be appropriately selected and applied but at the same time they must be linked to the available capacity of existing structure in terms of strength, stiffness and ductility. Reinforced concrete jacketing or FRP wrapping of existing members are among the most used strategies. Disadvantages of these strategies are linked to their irreversibility and lack of efficiency when lateral stiffness is insufficient. In these cases, the system can be improved by adding new structural elements, e.g. steel bracings, with or without local strengthening of elements with deficiencies. The main objective of the study is to validate a seismic strengthening technique for non-seismic reinforced concrete frame buildings that consists of conventional concentric V braces (CBS) or buckling restrained V braces (BRB). Numerical and experimental investigations were carried out at Politehnica University of Timisoara, within CEMSIG Research Centre (http://cemsig.ct.upt.ro) in order to study and realize this intervention [1]. The case study is represented by a historical building, erected in the first half of the XXth century. The reinforced concrete building was designed according to the Italian design code at the time but is characteristic of many reinforced concrete buildings constructed before 1970 both in Italy and in other southern European countries.

2 PERFORMANCE BASED EVALUATION AND RETROFIT OF A REINFORCED CONCRETE FRAME BUILDING

The 3 story building has plan dimensions of 23.4 by 18.4 m and 11.95 m height (Figure 1). The characteristics of rebars and concrete were considered those used in that period, i.e. concrete characteristic compressive strength f_{ck}=20N/mm^2, rebars with a characteristic yield strength f_{sk}=230N/mm^2. The specific detailing of the reinforcement is also characteristic for the design practice at that time, with poor anchorage length of the rebars at the external beam-to-column joint, the use of plane rebars (not ribbed), inclined reinforcement used for shear resistance and large spaced stirrups (15cm for columns and 25cm for beams) in plastic zones.

In the first step, the evaluation of the structural system of existing building was performed, to decide the locations of intervention. The seismic response was first evaluated using a response spectrum analysis. The seismic load for ultimate limit state verifications was defined through an elastic spectrum with the following parameters: peak ground acceleration (PGA) of 0.23g, γ_I=1.0, T_B=0.15s, T_C=0.5s, T_D=2.0s and S=1.2, where the periods T_B, T_C and T_D and the soil factor S describe the shape of the elastic response spectrum and depend upon the ground type. Considering the very poor ductility of the structure, a seismic behavior factor q = 1.5 was used. For structure strengthened with BRB, the seismic behavior factor q amounted 4, while for structure strengthened with CBS, the seismic behavior factor q amounted 2. For the steel braces, the cross sections requirements were: in X direction, ground floor = 8 cm^2, 1st level = 4 cm^2, 2nd level = 3 cm^2; in Y direction, ground floor = 6 cm^2; 1st level = 5 cm^2 and 2nd level = 3 cm^2.

Then, a detailed seismic evaluation using a static nonlinear analysis, both for initial structure and for strengthened structure, was performed. The static nonlinear procedure was based on the N2 method (EN 1998-1, 2004) [2]. Three limit states, defined as immediate occupancy (IO), life safety (LS) and collapse prevention (CP) were used. The performance based seismic evaluation implied the verification of behavior at three performance levels introduced above (i.e. IO, LS, CP), using three target displacements, one for each level. The design seismic hazard (associated to LS) corresponds to a recurrence interval of 475 years. For IO and CP, the recommendation is to use hazards with 95 years return period and 975 years return period, respectively. Table 1 shows the simplified coefficients for conversion of the peak ground acceleration (PGA) corresponding to a recurrence interval (IR) of 100 years to values of PGA corresponding to IR of 30 and 475 years. The 2D analysis was done separately on X and on Y directions for all three structural systems (MRF, MRF+BRB and MRF+CBS). The concrete material was considered unconfined and modeled using nonlinear model of Kent and Park, with no tension. Reinforced concrete members were modeled with plastic hinges concentrated at both ends. The bracing system was applied on the external frames of the RC building as an inverted V system pinned at the ends. The inelastic behavior of BRB system was modeled considering the concentrated tri-linear plasticity curve with strain hardening and strength degradation of 0.8 from maximum capacity, according to FEMA356/ASCE-41 [3]. For steel braces in tension, the modeling parameters and the acceptance criteria were based on [3] and on previous experimental tests on BRB elements (see [3]) and are summarized in Table 2. Modeling parameters and acceptance criteria for CBS were also based on [3], see Table 3.

Ratio of seismic hazard/design seismic hazard	a_g(95 years)/ a_g(475 years)	a_g (975 years)/ a_g (475 years)
Conversion coefficient	0.5	1.50

Table 1: Coefficients for conversion of PGA

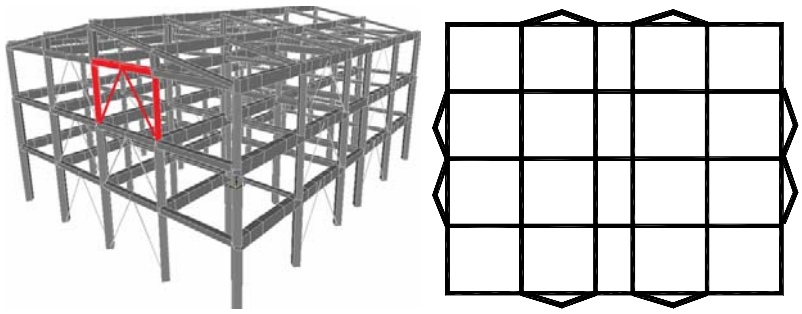


Figure 1: RC building model, with location of brace systems

BRB properties		Values of parameters
Modeling curve	Type	Tri-linear
Material	Steel	S235
Area of the core steel element	A_c [cm^2]	1x3
Core length	L_c [m]	1.7
Yielding displacement	Δ_y [mm]	1.9
Ductility displacement	μ	22
Strain hardening adjustment factor	ω	1.9
Compression adjustment factor	β	1.2
Acceptance criteria (modified FEMA356/ASCE41 for braces in tension)	IO	0.5Δt
	LS	14Δt
	CP	18Δt

Table 2: BRB modeling parameters for the analysis

CB properties		Parameters value
Material		S275
Cross section		Tube 101.6x3.6 mm
Area of the steel element	A_c [cm^2]	11.08
Length	L [m]	3.4
Acceptance criteria FEMA356/ASCE41 for braces in tension/compression	IO	$0.25\Delta_t/0.25\Delta_c$
	LS	$7\Delta_t/4\Delta_c$
	CP	$9\Delta_t/6\Delta_c$

Table 3: CBS modeling parameters for the analysis

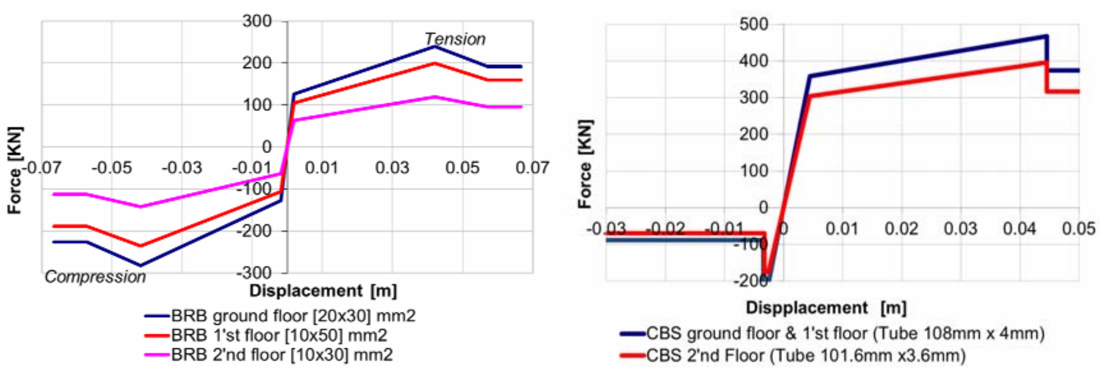


Figure 2: a) Tri-linear modeling on Y direction of: a) BRB system; b) CBS system

The results of the pushover analysis on Y direction are presented in Figure 3.a. Figure 3.b, Figure 3.c and Figure 3.d show the history of plastic hinge in the structures for LS performance level. Different symbols were used to plot the plastic rotation demand in elements: triangle shape is associated to IO, circular shape to LS and rhomb shape to CP.

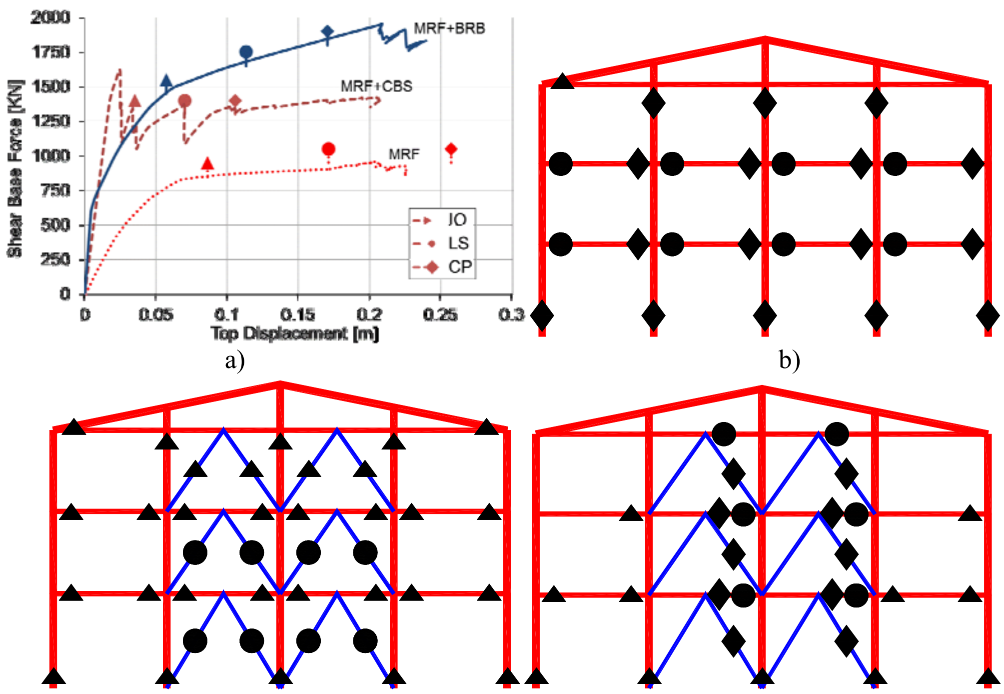


Figure 3: a) Pushover curves on Y direction; Location and stage of plastic hinges corresponding to a LS performance level for; b) MRF (frontal view on Y direction); c) MRF+BRB; d) MRF+CBS

It may be seen the initial structure MRF (Figure 3.b) has a limited ductility and does not attain the performances required for LS, as the plastic rotation demand in beams and columns exceed even CP criteria. When the structure is strengthened with BRB, the behavior is much improved. The stiffness and the strength increase and for LS performance level (Figure 3.c) there are no elements where the acceptance criteria are exceeded. When the strengthening system with CBS is used, the result is a structure with a good stiffness and strength. After the compression brace buckles, there is a reduction of the capacity. It may be noticed that even if less plastic hinges occurred in columns, beams requirements are beyond the acceptance criteria (Figure 3.d). Thus, the unbalanced vertical action effect is transferred to the beam and plastic hinges develops at the beam end and in the vicinity of brace-to-beam connection.

3 EXPERIMENTAL VALIDATION OF STRENGTHNENING SYSTEMS FOR SEISMIC RETROFIT OF A REINFORCED CONCRETE FRAME STRUCTURE

3.1 Test Specimen

Following the results of the nonlinear static analysis, a RC portal frame was isolated from the case study building (Figure 1). Six reinforced concrete frames of 3.2m height and 4.5m span were tested under monotonic and cyclic loads: two RC frames (MRF), two RC frames strengthened with BRB (MRF+BRB) and two RC frames strengthened with CBS (MRF+CBS). Results of materials testing are presented in Table 4. The BRB steel core plate (Figure 4) was divided into three main segments: the end segment (connection), the transition segment and the yielding segment. Based on the experimental results obtained on BRB elements only ([1], [5]), polyethylene foil of 1 mm thick was used as unbonding material. It should be mentioned here the BRB systems must be subjected to a technical validation procedure before intended use, which includes relevant type tests and factory control tests (see EN 15129, [4]). The core was designed for S235, but the material supplied shown larger values by more than 40% (or 100 N/mm^2).

Material		Nominal values [N/mm^2]	Test results [N/mm^2]
RC frame	Concrete (R_c)	20.5	35.5
	Beam rebars (f_y)	235	497
	Beam rebars (f_y)	235	402
	Stirrups (f_y)	235	290
BRB	Concrete (R_c)	20.5	35.1
	Steel Plate (f_y)	235	335
CBF	Tubular section (f_y)	235	248

Table 4: Material test results

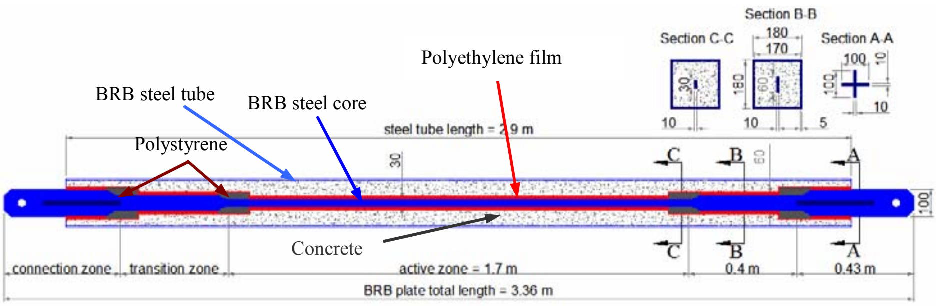


Figure 4: Details and geometry of BRB (unbonding material – polyethylene foil, 1mm thick)

The CBS were realized from circular hollow sections with S235 steel (Figure 5). The material supplied indicated the steel yield strength is very closed to the nominal values.

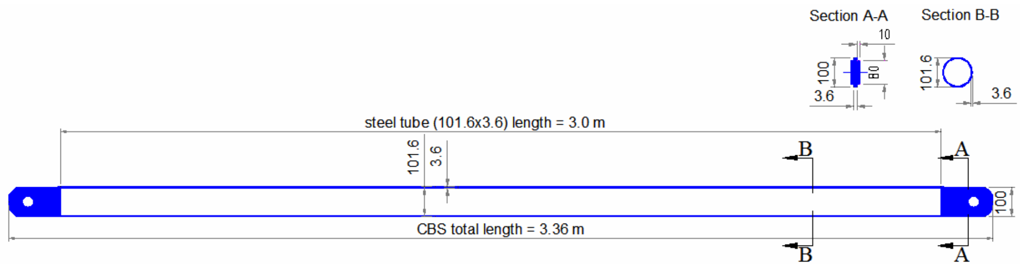
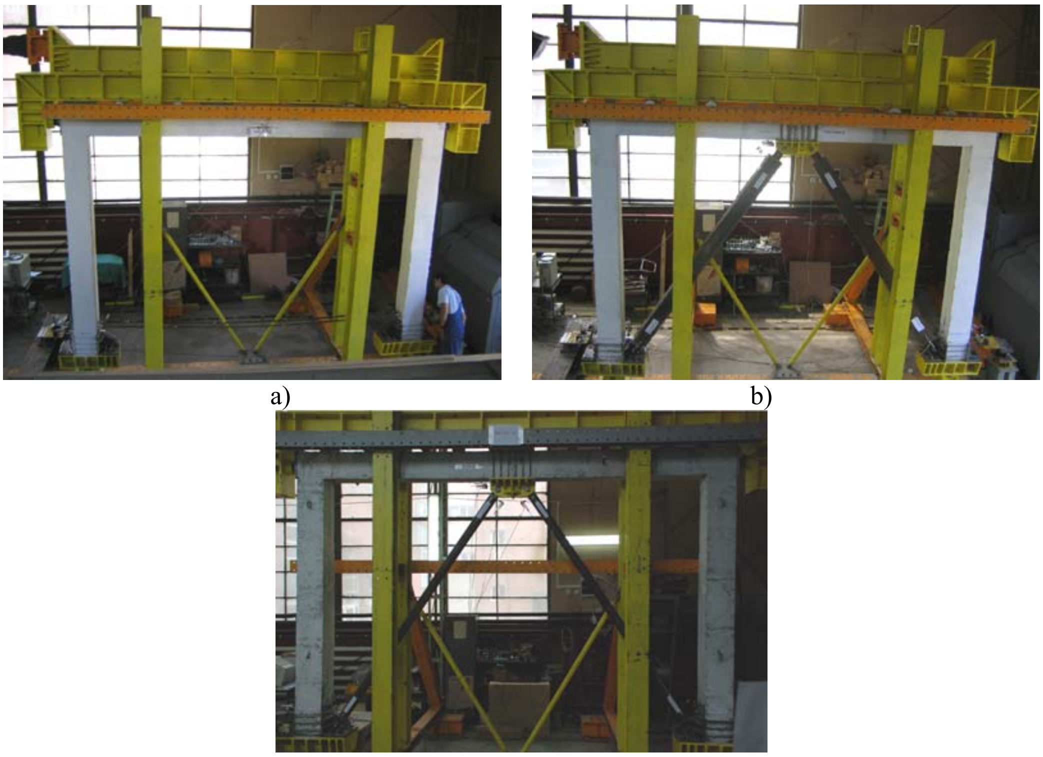


Figure 5: Details and geometry of conventional brace

3.2 Test set-up

The test set-up, the loading system and the specimens installed in testing rig are presented in Figure 6. The connection details for BRB and CBS systems are similar and use pinned connections between the brace elements and the beam and at the base of the columns. In order to prevent the slip of the connection between the braces and the RC beam, high strength pre-loaded ties have been used. The effectiveness of the connecting device has been preliminary checked by FEM simulation. Thus, a numerical simulation aimed to calibrate the level of pre-stressing forces applied in the brace - frame connecting device. Local pressure on the concrete was also checked, in order to keep the connection “elastic” (Figure 7, Figure 8).



a) b)

c)

Figure 6: Test set-up: a) initial RC frame without strengthening; b) RC frame strengthened with BRB; c) RC frame strengthened with CBS

During tests, the vertical and horizontal displacements of the connections were continuously recorded and plotted. Three monotonic tests have been carried out in order to evaluate the yielding point of each type of frame. The yielding displacements measured in the monotonic tests are then used in the subsequent cyclic tests to calibrate the cyclic loading history. The strain rate in the monotonic and cyclic tests was 5mm/min, so that the application of the load can be considered quasi-static.

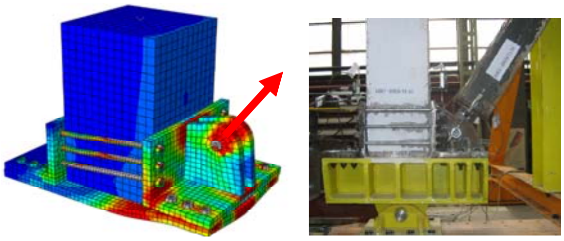


Figure 7: Column base and brace to column connection

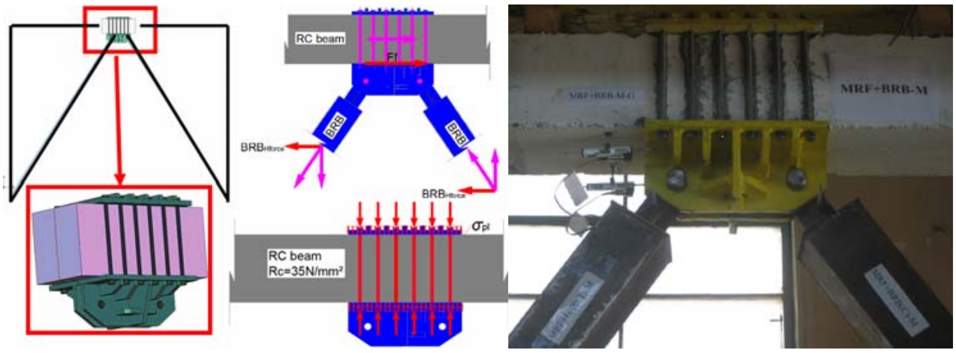


Figure 8: Beam – brace connection: pressure under the steel plate due to bolts pre-stressing

3.3 Results of monotonic tests

Figure 9 shows the force–displacement curves for the initial and strengthened frames. The effectiveness of BRB system is confirmed by the increase of both stiffness and strength. The ultimate load increases from 40kN to approximately 200 kN. The conventional brace system brings more strength and stiffness to the structure, but less dissipation capacity. The capacity increases to 230kN but the displacement corresponding to this ultimate capacity is about 50 mm, almost three times less then that corresponding to the BRB system. For the evaluation of the yield displacement (D_y) and the yield force (F_y), the ECCS methodology is used. According to this approach, yield displacement D_y and yield force F_y are obtained by intersecting the initial stiffness α_y and a tangent at the curve F - D with a slope of 10% of the initial stiffness. With the yielding point defined in this way, results D_y=29 mm and F_y=126KN for the MRF+BRB system and D_y=24mm and F_y=213KN for MRF+CBS system, respectively.

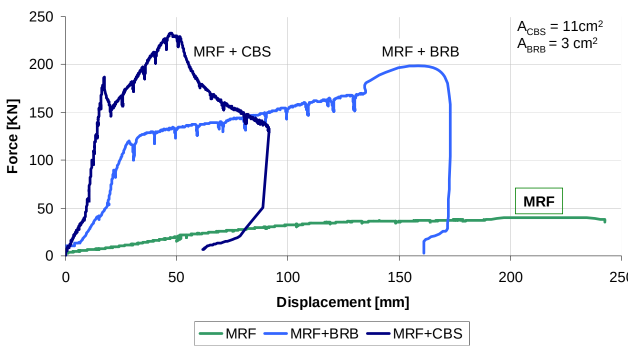


Figure 9: Force-displacement curves from monotonic tests

3.4 Results of cyclic tests

The modified ECCS loading protocol was applied in the cyclic tests. This modified procedure is characterized by a single loading at $D_y/4$, $2D_y/4$, $3D_y/4$ and D_y, followed by three repetitions of the cycles increased by $0.5D_y$ ($1.5D_y$, $2D_y$). The contribution of the strengthening systems is clearly indicated in Figure 10. The conventional brace system increases the capacity but, after few cycles in plastic range, there is an abrupt degradation in the hysteretic behavior due to the failure of the brace in compression. The frame strengthened with the BRB system has a much better behavior in terms of dissipation capacity.

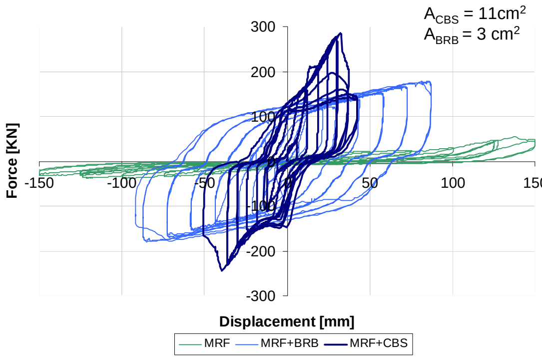


Figure 10: Hysteresis curves for initial RC frame and strengthened frames

Figure 10 shows the frames strengthened with BRB and CBS after the cyclic test. The brace in compression of the CBS system buckles and softens after few cycles and therefore the unbalanced vertical action effect is transferred to the beam. On contrary, the BRB system has a better behavior as the two braces have similar behavior and the brace in compression is protected from buckling.

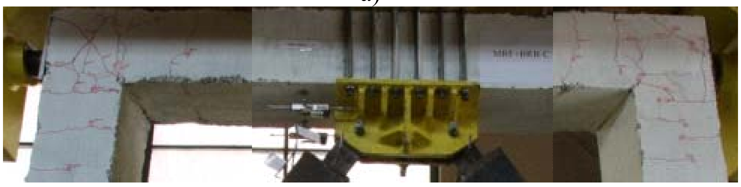
a)

b)

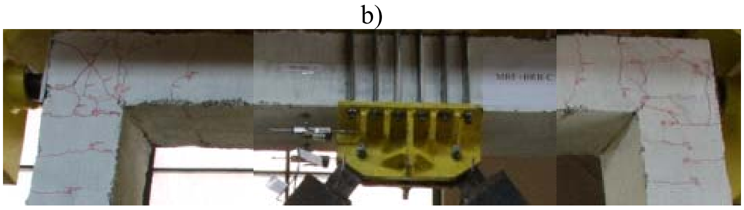
Figure 11: Photos after the test with frame strengthened with CBS a) and strengthened with BRB b)

The next photos show the cyclic damage that occurs in concrete elements, steel elements and their connections. Figure 12.a shows the crack development in initial RC frames. Bending cracks occurred first and were followed by shear cracks. The development of the shear cracks is mainly due to the inadequate distribution of stirrups. Figure 12.b and Figure 12.c show the crack development at both ends of the concrete beam and the brace connection after the test. The occurrence and development of cracks is similar to those of the initial RC frame, and were caused by the same inadequate confinement of the plastic zone. All the connections between BRB system and RC frame showed a very good behavior and there was no slippage recorded. In case of CBS system, the slippage of the brace to beam connection was about 25 mm in one direction and 10 mm in the opposite direction and is mainly due to the horizontal component of the unbalanced load that occurs after the buckling of diagonal in compression.

a)



b)



c)

Figure 12: Cracks in the concrete elements after the test: a) initial RC frame; b) frame strengthened with BRB; c) strengthened with CBS

4 EXPERIMENTAL EVALUATION OF Q FACTOR

The results of the experimental program were used also for the evaluation of the behavior factor q for tested structures. The q factor can be expressed as a product of the ductility factor q_μ, that accounts for the ductility of the structure and the overstrength factor q_s, that accounts for the reserve in strength of the structure (due to structural redundancy, material overstrength, member oversize due to design, other non-seismic load combinations and serviceability requirements). The overstrength may vary significantly and is affected by the contribution of gravity loads, material overstrength, etc. Therefore, in order to calibrate the behavior factor q, it is more important to focus on the ductility component, which can be taken equal to the displacement ductility factor μ. The displacement ductility factor q_μ is therefore defined as the ratio of the ultimate displacement D_u and the yield displacement D_y. Yield displacement D_y was evaluated with ECCS method (see section 3.3).

For the initial RC frame, the ultimate displacement D_u corresponds to the attainment of the ultimate strength (shear strength, Figure 12.a) and amounts 150 mm. For the frame strengthened with BRB, D_u corresponds to the failure of the BRB in tension (Figure 12.b) resulting an ultimate cycle of 86 mm. Taken into account the effect of slippage, which produces pinching in the histerezis curve (see Figure 13), D_u needs to be reduced to a value of 71 mm. For the frame strengthened with CBS, D_u corresponds to the failure of the diagonal in compression (Figure 12.c) and amounts, after the correction due to slippage, 42.2 mm. For the definition of the envelope curve, the third cycle of each step was considered. The values of the yield displacements D_y are 69.4 mm for MRF, 16.3 mm for MRF+BRB and 22.5 for MRF+CBS. The values of the behavior factor q are then calculated and results are presented in Table 5. The original unretrofitted structure has a poor behavior, characterized by a low stiffness and low dissipation capacity. As expected, the stable behavior and large ductility of BRBs leads to a q factor that can classify the strengthened structure as a medium ductility structure. When strengthened with conventional braces (CBS), the structure gains rigidity but the q factor is low, classifying the system as low dissipative.

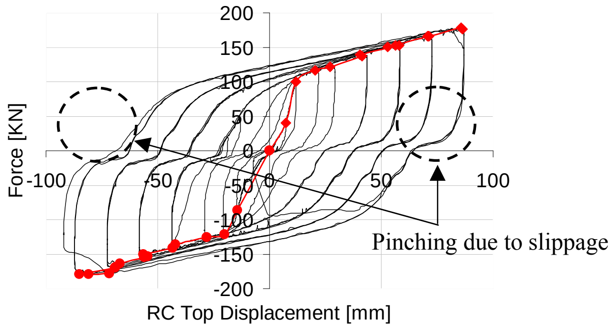


Figure 13: Envelope of the MRF+BRB cyclic test

MRF			MRF+BRB			MRF+CBS		
Dy [mm]	Du [mm]	q	Dy [mm]	Du [mm]	q	Dy [mm]	Du [mm]	q
69.4	150	2.16	16.3	71	4.3	22.5	42.2	1.8

Table 5: q factor values

In order to extend the investigation on the behavior factor q, a nonlinear dynamic analysis was employed. The results obtained so far confirmed the values obtained via experimental tests [6]. For example, for the structure strengthened with BRB, the results had shown a good agreement with experimental tests, with a mean value of q factor of 3.9.

5 CONCLUSIONS

Experimental and numerical investigations were conducted to evaluate the seismic behavior of RC frame buildings, designed for gravity loads, before and after strengthening with steel bracing systems. The preliminary analysis has shown that the RC structure is vulnerable and does not meet the seismic requirements for a moderate seismic zone. Therefore, the structural system should be adequately strengthened in order to attain the desired level of seismic resistance. Two types of strengthening techniques were considered, one with buckling restrained V braces (BRB) and one with conventional concentric V braces (CBS). The results have shown that the structure has a limited capacity, mainly due to poor detailing of the plastic zones (lack of stirrups). When the global strengthening technique is accomplished, the behavior is much improved.

A portal frame was then extracted from the building and studied experimentally. Six frames were tested, two for each type of frame, one monotonically and one cyclically. Tests have shown a very poor behavior of RC frame. The structural system should be strengthened in order to attain the desired level of seismic resistance.

Structure strengthened with BRB had a good behavior, larger rigidity, capacity and ductility compared to the initial structure. The failure was caused by the failure of the steel brace in tension. The connections between BRB and RC elements performed very well. It was also tested the workability of the system with pre-stressed ties. The connection devices used for installing BRBs within the frame took benefit from the friction resistant forces induced by the ties pretension and showed a very good behavior. In fact, reduced slips – with very small influence on the hysteretic loops of BRB system – were observed. Results recommend the application of this connecting system for such interventions. Moreover, in case of multi-story frames, such connecting systems also provide a beneficial confining effect at the frame joints, enhancing both strength and ductility of the MRF+BRB system.

Conventional brace system increases the strength and stiffness but is less ductile compared to BRB. In addition, the concrete beam should be able to support the vertical component of the unbalanced load when the compression brace buckles. The horizontal component of the same unbalanced load did also overstressed the brace-to-beam connection causing its slip. The application of such strengthening technique should therefore be done with caution, to avoid concentration of stresses and to reduce the ductility demands in critical sections.

Acknowledgement

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