Seismic Retrofit of Cut-out Weakened Precast RC Walls by Externally Bonded CFRP Composites

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SUMMARY:
The seismic performance of the reinforced concrete wall systems was reportedly one of the most satisfactory as opposed to the performance of the frame systems. Nevertheless, structural alterations by doorway cut-outs impair the seismic response of a reinforced concrete wall member. In order to enhance the seismic performance of the cut-out weakened walls the retrofitting technique by externally bonded fibre reinforced polymers (FRP-EBR) was investigated in this paper through seven near full-scale quasi-static cyclic tests performed on precast large panel models constructed according to the 1980 Romanian practice. The key feature of the experimental program is the variable axial loading procedure in order to model the outrigger effect. The results indicated improved overall seismic response for the retrofitted walls. However, the analysis of the performance characteristics showed that the strengthening effect was different in terms of strength, stiffness, displacement and energy dissipation. It was concluded that the CFRPs subjected to alternating tension-compression forces are susceptible to premature failure.

Keywords: RC wall, precast, FRP, seismic retrofit, cut-out opening

1. INTRODUCTION

Reinforced concrete shear walls are one of the most reliable lateral load resisting systems. The good seismic performance of the wall structural systems was proved by several earthquakes (Fintel 1990). This can be attributed to the large (wide) inclined load path available for the shear transfer mechanism, see Fig. 1.1. Despite this excellent performance the widespread use of the shear-walls is hindered by a certain degree of antagonism regarding the functional rigidity of this system. However, in East-European countries, and especially in Romania, the precast large panel systems were extensively used. In the 1950-1990 period more than 40 000 five-storey large panel residential blocks of flats were constructed (National Inst. of Statistics 2002). The architectural drawback is resolved by piercing the existing solid walls with new openings, refer to Fig 1.1. According to the common sense of structural engineering, this remodelling implies a weakening which should be addressed by a certain retrofitting method. This paper intends to shed some light on the extent of weakening caused by cut-out openings and the strengthening attainable by externally bonded carbon fibre reinforced polymers (CFRP-EBR). Furthermore, the much more generic issue of the shear transfer mechanism in concrete walls is addressed. This latter aspect is an intensely disputed topic involving basic seismic design philosophies.

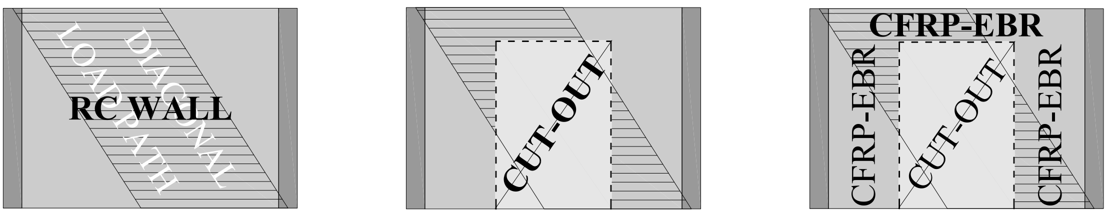

Figure 1.1. Weakening and strengthening

2. EXPERIMENTAL PROGRAM

The experimental campaign involved seven quasi-static cyclic tests on near-full scale (1:1.2) precast concrete wall specimens. In order to assess the weakening and strengthening effects the test variables included the opening condition and the strengthening condition, see Table 2.1. For a detailed presentation of the test program the reader is referred to Demeter et al. 2010 and Demeter 2011; hereafter only the main aspects are described. The wall specimens were constructed according to the 1980 Romanian large panel manufacturing practice. Each specimen was composed of a web-panel and two T-shaped boundary wings as shown in Fig. 2.1. The reinforcement details were common for all specimens: the web was reinforced by a single curtain of ϕ4/100 mm welded wire mesh and ϕ10/265 mm horizontal ribbed bars providing 0.42% and 0.13% reinforcement ratio in the horizontal and vertical direction, respectively. Except the solid reference wall, the specimens were pierced by door openings as depicted in Fig. 2.1. The locations and the dimensions of the two opening types relative to the web-panel are indicated in Table 2.2.

Table 2.1. Experimental program outline

Element designation	As-built / cut-out opening type	Strengthening condition
PRCWP 1-S-T	Solid (S)	Bare (T)
PRCWP 3-S/E1-T	Solid / narrow door (S/E1)	
PRCWP 5-S/E3-T	Solid / wide door (S/E3)	
PRCWP 3-S/E1-T/R	Solid / narrow door (S/E1)	Post-damage strengthened (T/R)
PRCWP 4-S/E1-R/T		Prior-to-damage strengthened (R/T)
PRCWP 5-S/E3-T/R	Solid / wide door (S/E3)	Post-damage strengthened (T/R)
PRCWP 6-S/E3-R/T		Prior-to-damage strengthened (R/T)

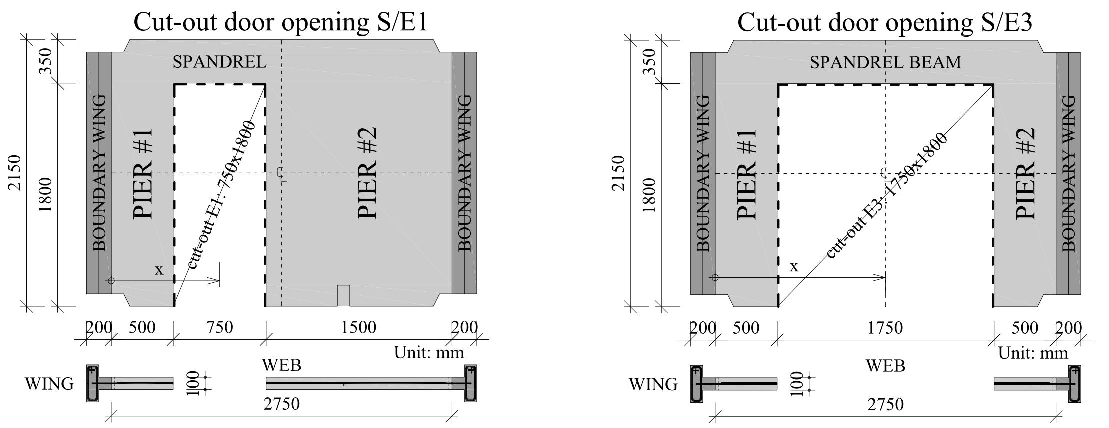

Figure 2.1. Cut-out openings

Table 2.2. Opening ratios

Opening type	Narrow door (E1)	Wide door (E3)
Location x/l_w	0.32	0.50
Length ratio l_o/l_w	0.27	0.64
Height ratio h_o/h_w	0.84	0.84
Diagonal ratio $\sqrt{(l_o^2+h_o^2)/(l_w^2+h_w^2)}$	0.56	0.72
In-plane area ratio A_o/A_w	0.23	0.53
Peripheral ratio $\sqrt{A_o/A_w}$	0.48	0.73

Note ï x: opening centreline position from the left edge of the web-panel; l_w: web panel length; h_w: wall height; l_o: opening length; h_o: opening height; A_o: in-plane area of the opening (A_o=$l_o h_o$); A_w: in-plane area of the web-panel (A_w=$l_w h_w$).

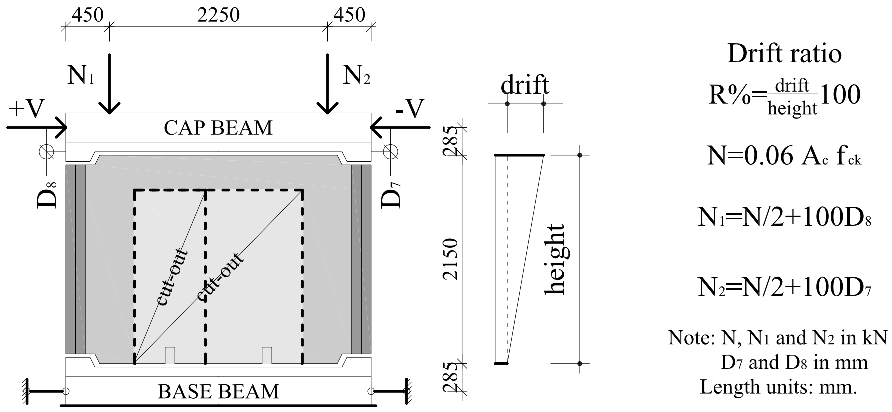

Figure 2.2. Test set-up and loading procedure

Table 2.3. Drift amplitudes

Cycles	**Initial**	**2**	**2**	**2**	**2**	**2**	**etc.**
Drift (mm)	load control	2.15	4.3	6.45	8.6	10.75	+2.15
Drift ratio (%)	(drift Ò1 mm)	0.1	0.2	0.3	0.4	0.5	+0.1

The opening ratios, see Table 2.2., are extremely meaningful to assess the degree of weakening due to cut-outs. Previous research related to walls with openings employed the length ratio (Kakaletsis and Karayannis 2009; Warashina et al. 2008), height ratio (Warashina et al. 2008), in-plane area ratio (Yanez et al. 1992) and peripheral ratio (Warashina et al. 2008; Umemura et al. 1980).

The material tests carried out on concrete and steel reinforcement samples showed the following results: measured cube strength ($f_{cm,cube}$) of the web-panel concrete: 17.5 MPa (for specimens 1, 3 and 6), 28.4 MPa (for specimen 5) and 40.2 MPa (for specimen 4); measured yield strength (f_y) of the reinforcement: 450 MPa (for the ϕ10 mm ribbed bars) and 618 MPa (for the ϕ4 mm welded wire mesh); measured tensile strength (f_t) of the reinforcement: 564 MPa (ϕ10) and 667 MPa (ϕ4).

The loading procedure is a key aspect of the experimental program. It is composed of static cyclic lateral loads and variable axial loads. Except the initial cycles, the lateral loads were applied in drift control (refer to Fig. 2.2.) performing two load-displacement cycles at each drift amplitude shown in Table 2.3. The slope of the loading history, defined by the 0.1% drift ratio, can be looked at as rather moderate in comparison with other concrete wall tests reported in the literature. As regards the axial loading, the base value (N) was computed considering 6% normalised axial load. In addition to this constant axial load level, variable axial loads were applied in uplift displacement control at 100 kN/mm rate, see Fig. 2.2. This latter aspect is quite unusual and may need some further explanation.

In order to compare the testing conditions applied in previous experimental programs, the authors undertook an extensive literature survey with particular attention on the loading, test-set-up and boundary conditions (Demeter 2011). As of the end of 2011, more than 150 experimental programs were included in this database. Depending on the shear span (moment to shear ratio) with respect to the wall height, the experimental programs can be grouped in three categories: cantilever ($M/V=h_w$), additional moment ($M/V>h_w$) and restrained rotation ($M/V<h_w$), where M is the moment at the wall base, V is the base shear and h_w is the wall height. Even though the number of cantilever tests prevail per total, there is clear the increasing number of restrained rotation tests starting from the 1990s. The additional moment and the restrained rotation tests are intended to realise greater or smaller shear ratios (M/Vl_w), respectively, than the aspect ratio (h_w/l_w) of the wall, where l_w is the wall length. The majority of the researchers agree in that the axial loads should be kept constant during the test; nevertheless, the additional moment and the restrained rotation tests are practically achieved by variable axial loading. In order to avoid the violation of the constant axial load concept, these loads are acting in pairs, one contrary to the other, generating the required moment for the desired shear ratio. In a rather small number of research programs was admitted the existence of variable axial loads, e.g. in

the tests reported by Muto 1974 and by Abrams 1991. So as to sustain this standpoint, one can argue by saying that in a real spatial structure the uplifting end of a laterally loaded wall attracts additional axial loads from the adjacent orthogonal structural members; this phenomenon was referred to as the outriggering action (Abrams 1991). The additional eccentric axial loads in the present experimental program were intended to model the outrigger effect. It is also worthy of notice that the variation of the axial loads during a seismic event is recognised in Eurocode 8 (CEN/TC250 2004) at the following places: in clause 5.1.2 in relation to the definition of the large lightly reinforced walls; in paragraph 5.2.3.6(3) regarding the membrane reaction of the slabs mobilised by upward deflections of structural walls; in 5.4.2.4(2) in relation to the axial force fluctuation in coupled walls; and in paragraphs 5.4.2.5(3)P and (4) regarding the additional dynamic axial forces in large walls due to uplifting.

3. RESULTS AND DISCUSSION

As general remarks regarding the load-displacement responses recorded during the tests one can recall the pinched shape of the hysteresis loops, exhibiting quasi-parallel loading and unloading branches, and the significant degradation of the reloading stiffness. The crack development was marked by flexural cracking at the pier-to-spandrel joints and at the wall base, followed by inclined cracking across the piers (primarily on the wide ones). Regarding the FRP distress it is noteworthy the early fibre fracture at the pier-to-spandrel connection and at the pier-to-foundation anchorage; the bulging of some vertical strips above the confined pier toes; and the debonding in the vicinity of the inclined cracks extending across the piers. The failure occurred by concrete crushing in the critical regions located at the pier-to-spandrel connections and at the pier toes. The lateral load vs. displacement response and the load and displacement histories for the PRCWP 4-S/E1-R/T specimen are shown in Fig. 3.1. Further details regarding the responses of the other specimens were reported in Demeter et al. 2010 and Demeter 2011.

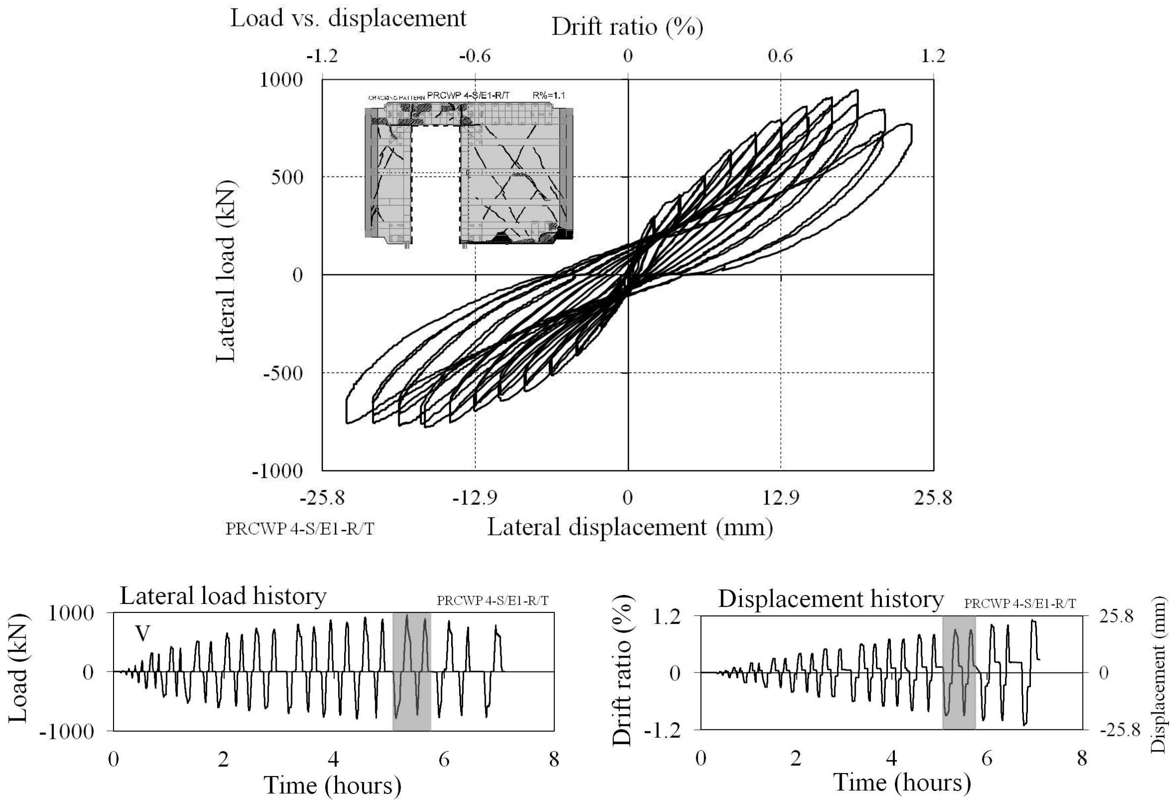

Figure 3.1. Lateral load response

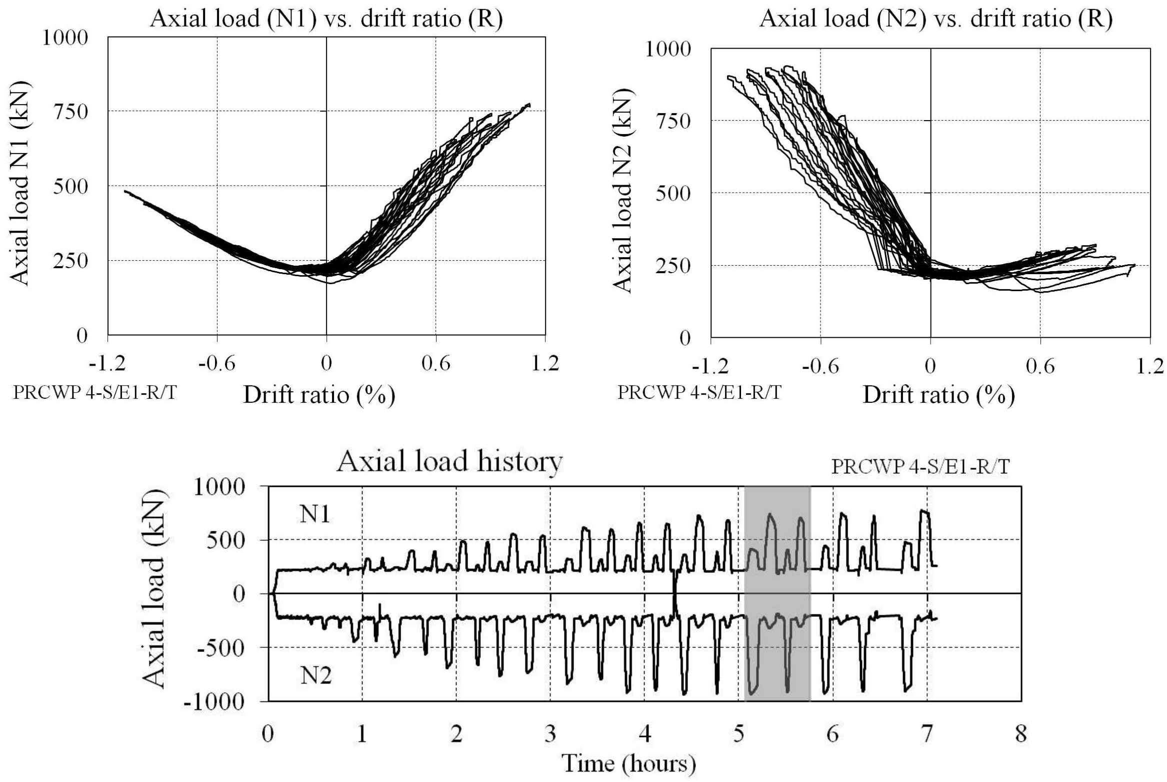

Figure 3.2. Axial load response

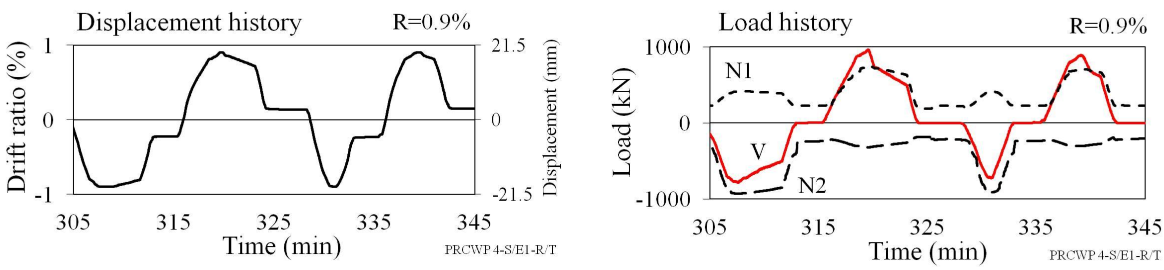

Figure 3.3. Expanded load and displacement histories

The axial load vs. drift response and the axial load history is shown in Fig. 3.2. There is more than evident the variation of the axial loads. One should bear in mind that if the axial loads were kept constant the load vs. drift ratio plot should be horizontal; whereas, one can observe that the axial load variation attained the magnitude of three times the initial value. This is a key aspect of the present program and represents the outrigger effect. Note that in the load-history plot the negative sign of the N_2 axial load is only for avoiding the curve congestion, it means compression too. In order to exhibit more clearly the variation of the axial loads, the expanded displacement and load histories corresponding the R=0.9% drift ratio (grey shaded on the full history plots) are shown in Fig. 3.3., where it is observable the alternate waving of the axial loads. Note that on the load history plot the N_2 axial load means compression; its negative sign serves only comparison reasons. Again, if the constant axial load concept would be followed, the axial load curves should be straight horizontal lines.

In order to assess the degree of pinching one should compare the energy dissipated during a complete load-displacement cycle (the area bounded within the hysteresis loop) with the energy which could have been theoretically dissipated within the same load-displacement limits assuming perfectly plastic behaviour (the area of the rectangle defined by the positive and negative peaks), see Fig. 3.4.; this ratio was referred to as energy dissipation ratio. Similar definition was reported by Hidalgo et al. 1996 and 2002, and by Olsen and Billington 2011.

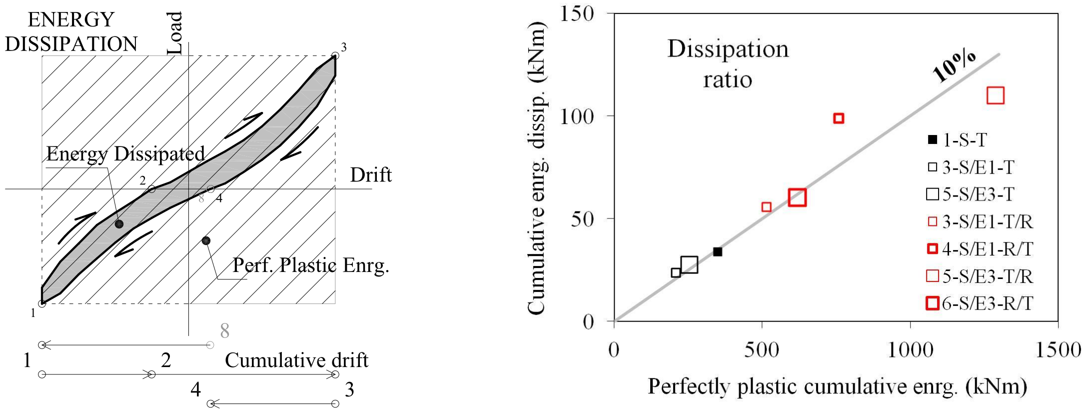

Figure 3.4. Energy dissipation

As it was expected, the dissipation ratio yielded quite small values, namely it varied within the (10÷15)% range. The cumulative energy dissipation ratio was defined as the ratio of the cumulative energy dissipated to the cumulative sum of the theoretical energies which might have been dissipated assuming perfectly plastic response. The cumulative dissipation ratio of the experimental specimens varied in the (8.5÷13)% range; it is noteworthy that neither the cut-out nor the strengthening condition affected significantly the dissipation ratio. Consequently, the value of 10% dissipation ratio can be considered as a general response characteristic of the wall panels in this experimental program, see Fig. 3.4. The energy dissipation rate can be defined as the ratio between the energy dissipated during a cycle and the cumulative drift through the same cycle, see Fig. 3.4. Similar, though not identical, definition was reported by Kakaletsis and Karayannis 2009. The dissipation rate was quasi-constant for a specific element, but it was significantly affected by the cut-out condition.

The experimental results regarding the weakening effect of the door cut-outs on the seismic response of the solid reference wall are presented in Fig. 3.5. The cut-out ratio is a measure of the opening size relative to the solid reference wall either in terms of length ratio or peripheral ratio (see Table 2.2). The performance ratio indicates the response characteristic of the weakened specimen normalised to the corresponding characteristic of the sound (solid) reference. The situation of complementarity between the performance ratio and the cut-out ratio is represented by the line joining the unities of the two axes. One can observe that there is experimental evidence on the complementarity relationship between specific performance and opening ratios: the strength and stiffness performance ratios are the complement of the peripheral ratio, whereas the energy dissipation rate is the complement of the length ratio. The foregoing observations can be formulated in the following equations derived from the AIJ recommendation (AIJ 1999, quoted in Warashina et al. 2008):

$$(R)_{weak} = (R)_{sound} \cdot \alpha_p \tag{3.1}$$

where $(R)_{weak}$ is the response characteristic of the weakened structural member in terms of shear resistance, initial stiffness or energy dissipation rate; $(R)_{sound}$ is the response characteristic of the sound (solid) wall in terms of shear resistance, initial stiffness or energy dissipation rate; and α_p is the performance ratio, given by:

$$\alpha_p = 1 - \eta \tag{3.2}$$

The opening ratio η is given by:

$$\eta = \begin{cases} P = \sqrt{A_o / A_w} & for \quad (R): shear\ resist.\ and\ stiffness \\ l_o / l_w & for \quad (R): dissipation\ rate \end{cases} \tag{3.3}$$

where P is the peripheral ratio, A_o and A_w is the in-plane area of the opening and the wall, respectively; and l_o and l_w is the length of the opening and of the wall, respectively.

Note that expressions 3.1. to 3.3. were derived from the AIJ recommendation (AIJ 1999, quoted in Warashina et al. 2008). However, the AIJ equation is reportedly (Warashina et al. 2008; Taleb 2010) applicable only for peripheral ratios less than 0.4 and it refers only to the shear strength and stiffness. In the present experimental campaign the above equation was verified for two peripheral ratios greater than the upper limit given in the AIJ document, namely for 0.48 and 0.73, and it was generalised also for the dissipation rate response characteristic.

The effect of the FRP-EBR strengthening on the seismic response of the cut-out weakened specimens is presented in Fig. 3.6. The performance ratio indicates the response characteristic of the FRP-strengthened specimen normalised to the corresponding characteristic of the cut-out weakened bare reference. It can be remarked that the response characteristics were differently influenced by the CFRP-EBR strengthening; outstanding improvement was achieved in terms of energy dissipation. Furthermore, one can assess the differences the timing of the strengthening (post-damage or prior-to-damage) had on the response. Note that the results should be viewed in the light of the concrete strength performance ratio (in the range of 0.62 to 2.3) and of the loading and boundary conditions. As regards the performance of the FRP strips, it can be concluded that the confinement FRPs show the most stable performance; the shear FRP strips debond in the vicinity of the inclined cracks; and the flexural FRPs subjected to alternating tension-compression reversals parallel to the fibre direction are likely to fail prematurely.

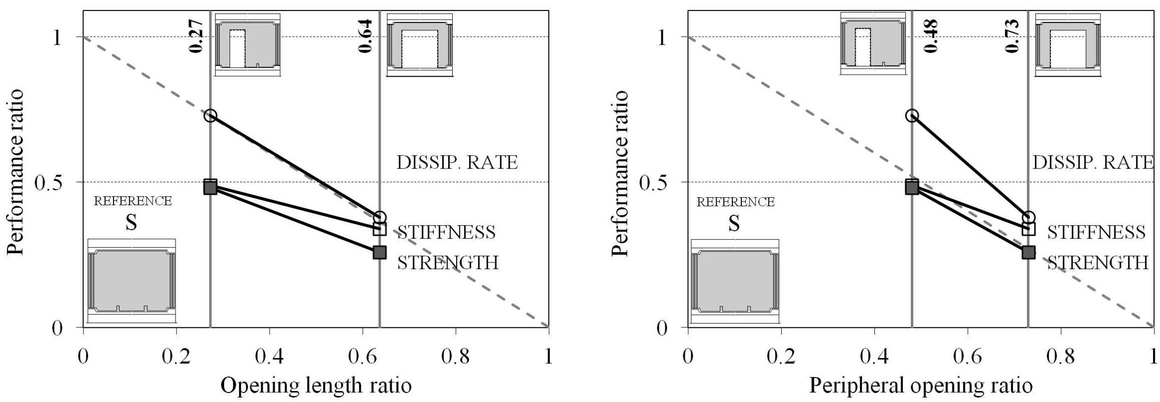

Figure 3.5. Weakening effect of the cut-out openings

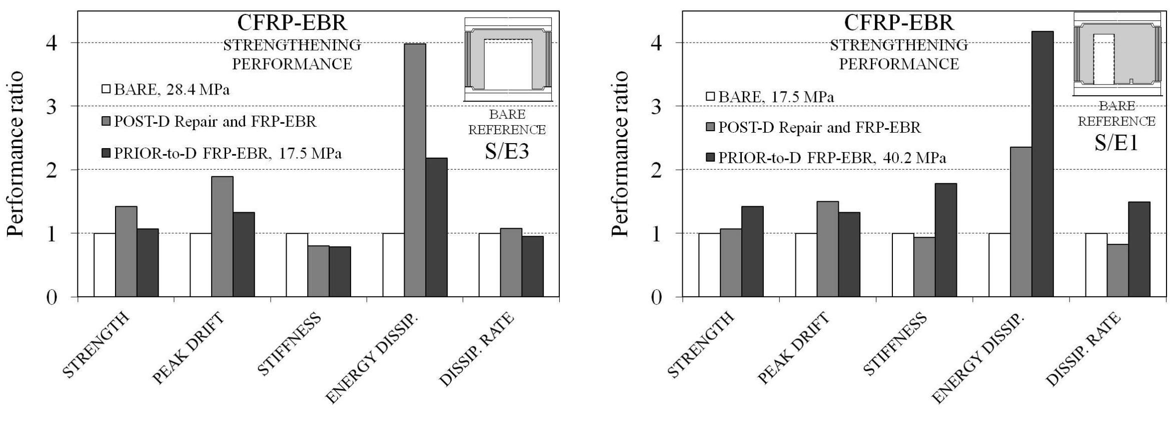

Figure 3.6. Strengthening effect of the FRP-EBR

4. CONCLUSIONS

In this study the authors addressed deliberately the outrigger effect by additional eccentric axial loading. It was shown that in these circumstances the response of the reinforced concrete large wall panels is characterised by very high shear resistance and about 10% energy dissipation ratio. The weakening effect of the cut-out opening was found to be in agreement with the predictions provided by the AIJ equation. The application range of the AIJ equation was extended and another response characteristic, namely the dissipation rate was introduced. One should bear in mind that the relationships given in Eqns. 3.1. through 3.3. were experimentally validated for the specific loading and boundary conditions applied in the present program; further investigations are required to widen the loading and boundary conditions range. Regarding the externally bonded FRP strengthening, the experimental results indicated that the energy dissipation capacity of the walls retrofitted by this technique increased significantly, whereas the other response characteristics were influenced in a smaller degree. One should bear in mind that this improvement of the seismic performance should be attributed primarily to the confinement and shear components of the strengthening system. The flexural FRPs were found to be susceptible to fail prematurely; however, it is not clear whether this type of failure is triggered by concrete substrate deterioration, i.e. local spalling and crushing, or directly by the adverse loading conditions (tension-compression reversals).

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