

SEISMIC BEHAVIOUR OF R.C. HOLLOW SECTION BRIDGE PIERS RETROFITTED WITH FRP

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SUMMARY

The research work presented in this paper concerns the seismic assessment of hollow bridge piers strengthened with fibre-reinforced polymer (FRP). The scope of the strengthening is to overcome some common deficiencies deriving from the use of non-seismic design rules and leading to a not adequate cyclic response. The strengthening design was studied by means of a parametric analysis considering different fibres and geometrical parameters applied to standard case studies. Quasi-static cyclic tests were performed on five 1:4 scaled piers designed according to old non-seismic Italian codes and strengthened according to the previous analytical study. Efficiency of FRP strengthening was evaluated comparing the experimental results with those obtained in a previous experimental research performed on similar not strengthened specimens. Base shear vs. lateral deflection curves, dissipated energy and collapse mechanisms comparison shows the achievable effectiveness once the debonding risk has been overcome.

INTRODUCTION

Over the past 20 years, fibre-reinforced polymers (FRP) have been increasingly employed for strengthening and repairing interventions in seismic prone countries. This is primarily due to their remarkable mechanical properties, the low weight and the manageability, which allows low-effort interventions. On the opposite, the relatively high costs and the bonding problems are the main shortcomings. Though extensive research has been focused on the calibration of design rules, there still is a lack of knowledge on strengthened bridge piers; in particular, the seismic response of hollow square piers strengthened with FRP is poorly investigated and it is not common in literature.

The relevance of this research is due to the large number of bridges built in Europe in the past 30 years, when seismic zones were often not recognized, resulting thus in structural elements designed for gravity loads only. Several existing hollow-section bridge piers require strength and ductility enhancements in order to meet the prerequisites of modern seismic regulations.

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A large number of hollow piers subjected to recent earthquakes have suffered failures due to insufficient shear strength, insufficient ductility capacity or to poor reinforcement detailing (insufficient lap-splice, lack of confinement etc). It is thus clear that the development of efficient structural intervention measures to be applied for repairing or strengthening of square hollow bridge piers is of great importance and relevance, in particular within the framework of European transport infrastructures.

OBJECTIVES AND METHODS

A previous experimental campaign on 1:4 scaled prototypes designed accordingly to non-seismic standards, Calvi [1], is the starting level of knowledge of the present work. The results obtained from those tests have pointed out that collapse of under-designed piers is often governed by brittle mechanisms due to unbalanced flexural-shear strength, premature bars buckling or loss of bonding in lap-splice. Moreover, preliminary investigations can be found in Calvi [2] and Pinto [3].

In the research presented in this paper, FRP strengthening solutions have been adopted on five prototypes, similar to those tested in the previous research [1], in order to enhance both strength and ductility and to possibly accomplish the capacity design criteria. A parametric study using numerical simulation considering carbon, glass or aramid fibres and different geometrical characteristics of the FRP layers has been performed to establish the optimal strengthening for each pier typology.

Experimental quasi-static cyclic tests were then performed and base shear vs. lateral displacement curves and damage patterns were compared with those obtained from the previous research. The enhancement due to the FRP strengthening is evaluated in terms of flexural and shear strength and dissipated energy.

SPECIMENS AND TEST SET-UP

Two series of 1:4 scaled specimens were used for the experimental studies; the former is composed of five under-designed specimens tested in a previous research work, Calvi [1], the latter, tested in the current research, consists of similar specimens (same geometrical properties and reinforcement details, similar material properties) strengthened with FRP. Details of the geometry and of the reinforcement are shown in Figure 1 and summarized in Table 1.



Figure 1 - Geometrical properties of S-type and T-type specimens (a); reinforcement details (b)

Pier Id	Collapse mode of not strengthened specimens	Height [mm]	Long. and transv. reinforcement (ρ_L ; ρ_V)	f_c^st , f_c^{stst} [MPa]	v^{*} , v^{**}			
S250 [*] , S250FRP ^{**}	Shear	900	Type 1 (1.07;0.13)	35.0, 36.9	0.06, 0.06			
S500 [*] , S500FRP ^{**}	Shear	900	Type 1 (1.07;0.13)	23.7, 33.4	0.19, 0.13			
T250L [*] , T250LFRP ^{**}	Loss of bonding in lap-splices	1350	Type 2 (1.76; 0.25) long. bars spliced on 16 cm	23.6, 30.3	0.09, 0.07			
T250 [*] , T250FRP ^{**}	Flexural - low ductility	1350	Type 2 (1.76; 0.25)	30.3, 32.8	0.07, 0.07			
T500 [*] , T500FRP ^{**}	Flexural - low ductility	1350	Type 2 (1.76; 0.25)	29.7, 33.1	0.15, 0.13			

Table 1 - Properties of the specimens (fc is the average cylindrical concrete compression strength; v =N/Acfc is the dimensionless axial load ratio, *first series, **second series)

Each series consists of two different typologies of specimens, identified in the following with the acronyms reported in Table 1, where *S* and *T* stand for short (height = 900 mm; aspect ratio = 2) and tall (height = 1350 mm; aspect ratio = 3) piers, the number indicates the applied axial load expressed in kN, the suffix *L* identifies the piers with lap-splice, while FRP indicates the strengthened specimens.

Both steel and concrete used for the preparation of the specimens of the two series reflect the common characteristics of the materials employed in the past for the construction of Italian bridges, with relatively poor concrete strength (average value of 33 MPa) and relatively high strength steel ribbed bars (yield and ultimate average strengths of 550 MPa and 670 MPa).

Both strength and collapse mode of the *S-type* specimens of the first series were predicted with enough accuracy using finite element modelling, for the flexural response, and some of the most popular resistance assessment models, for the shear response (Moehle [4], Priestley [5] and Kowalsky [6]).

The specimen T250L had a failure mode governed by the slippage of the longitudinal bars due to an insufficient lap splice at the base of the pier. An overlapping equal to 20 longitudinal bars diameters (160 mm), allowed by several old codes of practice, was adopted. This specimen was tested showing an overall response in good agreement with the prediction model proposed by Priestley [7].

The *T-type* specimens showed collapses with low ductility and wide openings on the side walls, rather well predicted by the same aforementioned analytical models.

On the base of the experimental information given by the first series of tests, in the current research a specific strengthening was designed *ad hoc* for other five prototypes adopting high modulus carbon (C), alkali resistant glass (G) or aramid (A) fibres: the following Table 2 shows their main mechanical and geometrical properties. The strengthening material consists of dry unidirectional FRP fabrics applied with the wet lay up system on sandblasted surfaces characterized by rounded corners. Due to the presence of a little concrete cover (about 8 mm, scale 1:4) the corners of the section were smoothed with a radius of curvature equal to 25 mm.

Table 2 - Mechanical properties of the fibres used in the experimental campaign

Type of fibre	Density [kg/m³]	Effective thickness [mm]	Tensile strength [MPa]	Modulus of elasticity [GPa]	Ultimate strain [%]
High modulus carbon	1820	0.165	3000	390	0.8
Aramid	1440	0.214	2800	105	2.7
Alkali resistant glass	2600	0.230	1700	65	2.8

An experimental campaign was performed in quasi-static regime by applying series of three cycles at increasing drift levels (0.4%, 1.2%, 2.4%, 3.6% and 4.8%, when possible) at the top of the piers using a horizontal hydraulic actuator. The vertical load was applied using a ϕ 32 mm post-tensioned high-strength steel bar connected to a hydraulic jack used to balance the axial load variation during the test execution. The specimens footings were design to avoid any possible failure and fixed at the strong floor with a system of steel beams and post-tensioned bars (Figure 2a).

The data acquisition system included global measurements of horizontal, vertical and diagonal displacements, measurements of longitudinal deformation in the plastic hinge zone, strain-gauges on stirrups and longitudinal bars, verification of possible base translation and rotation (Figure 2b).



Figure 2 - Test set-up (a) and transducers arrangement on T-type specimen (b)

STRENGTHENING DESIGN

Collapse mode of the not strengthened *S-type* piers was characterized by shear rupture after a limited penetration in the non-linear field. The scope of the strengthening intervention is to restore an optimal flexural-shear strength ratio. This is achieved through FRP layers wrapped all over the specimen height, which improve the confinement of the concrete and act as additional transversal reinforcement. A parametric study has been conducted considering 1 or 2 layers of high modulus carbon (C-FRP), aramid (A-FRP) or glass (G-FRP) fibres for each level of axial load considered.

The FRP contribution to the shear resistance was calculated accordingly to the following equation by Priestley [7], where wrapped fibres are considered as additional equivalent stirrups.

$$V_s = A_i \cdot \varepsilon_i \cdot E_i \cdot D' \cdot \cot \vartheta$$
 Eq. 1

where A_j , ε_j and E_j are respectively the effective area, the strain and the modulus of elasticity of the fibres, D' is the core dimension and ϑ is the angle of the inclined cracks (30° in the current case study).

The contemporaneous confining action effects due to the steel hoops and to the FRP wrapping have been evaluated accordingly to Wang [8], adding the relative lateral pressures f_l in the x and y directions:

$$f_{lx} = \rho_{sx} \cdot f_{sh} + \rho_{jx} \cdot f_j$$
 and $f_{ly} = \rho_{sy} \cdot f_{sh} + \rho_{jy} \cdot f_j$ Eq. 2

where the indexes *s* and *j* are referred to the steel hoops and to the FRP wrapping respectively, ρ is the reinforcement ratio and f_{sh} and f_j are the confining stresses calculated using the following Eq. 3 and Eq. 4, assuming that the transverse strain in the concrete ε_t , in the fibres and in the hoops are all equal:

$$f_{sh} = \min \left\{ E_s \cdot \varepsilon_t; \ f_{sy} \right\}$$
 Eq. 3

$$f_j = E_j \cdot \varepsilon_t$$
 if $0 \le \varepsilon_t \le \varepsilon_{ju}$ or $f_j = 0$ if $\varepsilon_t > \varepsilon_{ju}$ Eq. 4

where E is the modulus of elasticity, f_y is the yielding stress and ε_u the ultimate strain. The transverse strain is determined from the axial one assuming a Poisson's ratio value equal to 0.5 (Wang [8]).

Increased flexural strength due to the confining effect of the FRP wrapping has been determined using an iterative procedure (Spoelstra [9]), also considering the effect of the radius of curvature of the smoothed corners (Samaan [10]) and the reduction of lateral pressure caused by the geometry of the section. A concise scheme of the aforementioned iterative procedure is the following:

- the vertical strain is given and a trial value of the lateral stress f_l is imposed;
- the corresponding confined peak strength and strain of the concrete are determined using the confined-concrete model by Mander [11] (Eq. 5 and Eq. 6);

$$\frac{f_{cc}'(f_l)}{f_{c0}'} = 2.254 \cdot \sqrt{1 + 7.94 \cdot \frac{f_l}{f_{c0}'}} - 2\frac{f_l}{f_{c0}'} - 1.254$$
 Eq. 5

$$\varepsilon_{cc} = \varepsilon_{c0} \cdot \left[1 + 5 \cdot \left(\frac{f'_{cc}(f_l)}{f'_{c0}} \right) \right]$$
 Eq. 6

- the current vertical stress is carried out using the equation by Popovics [12] (Eq. 7);

$$f_c' = \frac{f_{cc} \cdot x \cdot r}{r - 1 + x^r}$$
 Eq. 7

- the lateral stress and strain are updated (the uniaxial stress response of plain concrete under compressive vertical strain is represented in accordance with Pantazopoulou [13])
- the procedure ends if the lateral pressure calculated in the last step is sufficiently close to the initial trial value; otherwise a new cycle with an updated value of the lateral stress is required

Then, the force-displacement curves have been obtained by double integration of the curvature distribution over the pier height. In Figure 3a the flexural domain related to the higher concrete confinement (2 layers of high modulus carbon fibres) is shown. The shear domains have been calculated using the model proposed in Kowalsky [6], with the addiction of the fibres contribution:

$$V = V_c + V_s + V_h + V_f$$
Eq. 8

where V_c , V_s , V_N , and V_f are respectively the contributions of: concrete, transversal reinforcement, concrete related to the extension of the compressed area of the cross section and FRP according to Eq. 1.

Examining the lateral force-displacement curves in Figure 3a, it can be observed that the response of the strengthened prototypes should be characterized by notable increments of shear strength and displacement capacity while the increase in flexural strength is negligible.

Among the examined solutions, the strengthening made of aramid or glass fibres are sufficient to achieve the shear strength requested to improve the post yielding behaviour. They could be preferable with respect to the high modulus carbon fibre solution essentially for economical reasons; moreover the strengthening with aramid fibres is also preferable, if compared to glass and both high modulus and normal modulus carbon fibres, for their capacity to develop relatively high stress concentrations at the corners of the pier section without damaging itself.

Although a single layer wrapping should be sufficient to match the strengthening requirements, a twolayer wrapping solution was adopted in order to remove possible problems of low impregnation of the aramid fibres with resins and debonding in the joining zone. Another reason that induced the choice of two layers is related to the uncertainty of the parameters used to determine the shear strength.

Continuous wrapping (with an overlapping of 200 mm) was provided all over the pier height paying attention to the position of the bolts used to fix the linear potentiometers (Figure 3b).



Figure 3 - S-type specimens: response prediction using different type of fibres and number of FRP layers with axial load equal to 500 kN (a); strengthening details (b)

The specimens of the first series with insufficient lap splice are characterized by a limited flexural strength due to the loss of bonding and the slippage of the longitudinal bars. As expected, the failure mechanism was dominated by a wide crack crossing horizontally the base of the pier, in the region where the longitudinal bars are joined, and took place when was still in the elastic range of the loading curve.

The purpose of the retrofit intervention is to overcome problems of bars slippage, achieving a response similar to the case of continuous reinforcing bars. High modulus carbon fibres were used because of their high stiffness, in order to deviate tensile stress from the longitudinal bars to the external FRP reinforcement. Three longitudinal layers of CFRP, each one with different length to realize a progressive stress-transfer protecting the upper end of the FRP from debonding, were applied in the zone of bars overlapping. It is believed that the base of the pier remains elasticid and the critical section shift upwards, where the steel bars could yield without slippage phenomena, being well anchored. A CFRP wrapping is provided in order to improve the shear strength and to prevent debonding problems (Figure 4b).

The anchorage of the fibres at the base of the pier was the crucial problem. The longitudinal sheets were fixed at the foundation by means of a cut filled with epoxy resin and a ϕ 7 mm CFRP transversal bar, solution adopted basically for research reasons, because it is commonly used in several practical cases.

The force-displacement curve (Figure 4a) was predicted by a double integration of the curvature (fibre analysis of the section), dividing the pier height in a lower part (about 300 mm) considered elastic and in an upper part, where the non-linar behaviour of the materials was considered.



Figure 4 - T250L FRP: response prediction using carbon fibres (a); strengthening design details (b)

The *T-type* specimens showed a mixed shear-flexural collapse mechanism, characterized by limited post yielding branch, longitudinal bars buckling, inclined cracks crossing the sides of the specimens and rupture of the transversal reinforcement.



Figure 5 - T500: response prediction for different fibres and number of layers; 450mm - width (a) and 200mm - width (b) longitudinal layers; GFRP strengthening (c); additional anchoring system (d)

First goal of the strengthening strategy was the shear strength improvement adopting transversal FRP wrapping using the same design procedure presented for *S-type*. The second objective was the flexural strength improvement. Even if experimental tests performed on the not strengthened specimens did not shown remarkable lack in this aspect, it was decided to explore this field of application, poorly investigated particularly for bridge piers.

The study was developed considering the following parameters:

- materials: high modulus carbon fibres, aramid fibres, glass fibres;
- number of transversal layers: 1 or 2;
- longitudinal-layer width: 200, 350 and 450 mm;
- axial load: 250 kN, 500 kN.

An analytical fibre model, hypothesizing a perfect concrete-FRP adhesion, with plain cross-sections, was used to determine the base shear vs. lateral displacement prediction.

The results obtained using the numerical model described above for different FRP solutions in terms of flexural behaviours compared to the corresponding shear domains are shown in Figure 5a and 5b.

Shear strength improvements obtained with the different material solutions are adequate to the main object of the design and the same comments of the *S-type* apply. For the flexural behaviour it should be noted that glass fibres, with high deformability (ultimate strain > 2% and elasticity modulus of about 70 GPa), appears to be more appropriate with respect to high modulus carbon fibres, which allow greater strength improvement but the overall response of the specimen is more fragile. Similar comments can apply also for aramid fibres, which are substantially equivalent to the glass type.

The success of the strengthening measure is totally influenced by two crucial problems:

- debonding phenomena of the FRP layers along the pier height;
- anchorage of the longitudinal fibres at the base of the pier (similarly to the T250LFRP case).

The force values corresponding to a situation of incipient debonding, calculated for each material and geometric configuration using the simplified model by Van Gemert [14] for uncracked sections, are shown in Table 3. The presence of the wrapped layers give a relevant advantage to the system, improving the debonding resistance and delaying the detachment of the fibre from the pier. Although refined calculations have not been developed yet, an increment of at least 15% of the aforementioned forces is expected simply from the evaluation of the maximum confining pressure given by the adopted strengthening system.

presence of the mapped layers (van demon [14] for anotabled beetlene)							
	Axial load	l = 250 kN	Axial load	Axial load = 500 kN			
Fibres material	Layer width [mm]						
	200	450	200	450			
C-FRP	233	252	251	276			
A-FRP	216	235	239	257			
G-FRP	213	231	235	251			

Table 3 – Lateral forces [kN] corresponding to situations of incipient debonding without considering the presence of the wrapped layers (Van Gemert [14] for uncracked sections)

On the base of the experimental results of the test on the specimen with lap-splice, an additional anchoring system, consisting of L-shaped flanged steel beams fixed at the foundation with anchor bolts, was adopted as shown in Figure 5d. The additional anchoring system was designed to carry the whole maximum tensile force developed in the FRP, subtracting the contribution of the fibres fixed at the base by the cut filled with epoxy resin. In order to validate the effectiveness of the steel anchor bolts, opportune preliminary experimental pull-out tests on a single bolt were performed. It is fundamental to underline that

a gap large enough to avoid contact with the pier was left, since its only function is to guaranty the bonding between the fibres and the foundation, without any interaction with the pier.

EXPERIMENTAL INVESTIGATION

S-type specimens (shear failure)

The experimental investigation performed on the not strengthened *S-type* specimens highlighted the inadequate seismic behaviour, essentially due to the unbalanced flexural-shear strength. In addition in the pre-yielding loading phase, inclined shear cracks appeared, causing marked deviation in the response curves with respect the numerical prediction performed with FE models (Figure 6a and 7a)

The strength of the S250 specimen ($\nu = 0.06$) decreased rapidly after a limited post yielding displacement (Figure 6a) and hysteretic loops were poorly dissipative. During the cycles at 2.4% drift, the damages progressively increased, the loss of strength became excessive and the pier failed in shear with broken stirrups visible through the 30° inclined cracks crossing the lateral walls of the pier.

The wrapped specimen S250FRP ($\nu = 0.06$) showed a very stable behaviour: only few horizontal cracks arose with no effects on the overall response curve. Emphasis must be given to the fact that the increased stiffness observed during the first cycle at 3.6% drift was due to a malfunctioning of the servo-valve controlling of the axial load, which was higher than the expected level. At the 4.8% drift, a bulge blown up at the pier base (buckling of longitudinal bars), a horizontal crack cut the FRP and the pier resistance decreased progressively. The Figure 6c shows the specimen after removing the aramid wrapping at the end of the test. No shear cracks or broken stirrups were observed, but only a wide horizontal crack at the base and buckled longitudinal bars between the first two stirrups. The FRP wrapping changed the failure mechanism of the specimen bringing a meaningful ductility enhancement, twice of the original; hysteretic cycles were bigger and more stable, the loss of strength was more progressive.



Figure 6 – S-type specimens with axial load of 250 kN: test results and analytical envelope (a); failure of the piers S250 (b) and S250FRP (c) after the removal of the fibres at conclusion of test

Similar comments regards the S500 ($\nu = 0.19$) where the strength decrement appeared just after the elastic limit of the response curve (Figure 7a). This was essentially due to the fact that the imbalance between flexural and shear strength was bigger as a consequence of the increased axial load which was more effective in the flexural strength enhancement than in the shear one. Trying to increase the drift to 2.4%, the specimen failed prior to the achievement of the maximum target displacement and the test was interrupted. The concrete was spalled out, the longitudinal bars at the pier base were buckled, and several stirrups were broken in tension. The shear damaging at low levels of loading have determined the deviation from the pure flexural response.

On the S500FRP no evident damages appeared in the first part of the experimental test. Some horizontal cracks at the pier base appeared at the 2.4% drift level. In this phase a bulge blown up due to the buckling of the longitudinal bars. Increasing the drift to 3.6% and 4.8%, the already existing cracks kept enlarging as well as the bulge; moreover the FRP fabrics wrapped at the lower part of the pier tore for a height of about 5 mm at 3.6% drift and 15 mm at 4.8%. The bearing capacity started to decrease during the 3.6% drift and it was lost at the successive drift: for this reason the test was stopped.

The FRP wrapping was very effective: it allowed to reach a strength about 20% higher than the one of the previous test and a displacement ductility equal to 8 instead of 2. Further, it has to be noted that the failure mode has changed from a brittle shear to a flexural collapse even if characterized by the concrete crushing instead of steel rupture; the achieved deformation capacity and energy dissipation levels were very satisfactory. Numerical predictions are sufficiently close to the experimental curve envelopes.



Figure 7 – S-type specimens with axial load of 500 kN: test results and analytical envelope (a); failure of the piers S500 (b) and S500FRP (c) after the removal of the fibres at conclusion of test

Specimen with insufficient lap-splice (loss of bonding)

The base shear vs. lateral displacement curve of the not strengthened specimen T250L (Figure 8a) showed the effect of the slippage mechanism activated in the lap-splice with sudden loss of stiffness even in the elastic range. During the 1.2% drift level, the applied horizontal force reached the maximum value (about 130 kN), then the deviation from the pure flexural elastic response started and the strength rapidly decreased. The maximum sustainable force after the first cycles, when the bars slippage was more evident,

was less of 50 kN (the residual resistance given by the arch effect determined by the presence of the axial force).



After the FRP retrofitting the performances were not notably improved. At the first cycle at 1.2% drift level the slippage started and became more evident at the 2.4% drift. The hysteretic loops began to take the same shape shown in previous not strengthened case. The adopted retrofitting solution can be considered only partially successful, having involved initial steel yielding but being still unable to solve the lap splice problem. This inefficiency must be attributed to the debonding of the longitudinal fibres at the pier base. Being the problem of the seismic retrofitting of piers with insufficient lap splice of great interest, further experimental tests with additional anchoring systems are planned in the future.

T-type specimens (limited ductility with shear failure)

The experimental investigation performed on not strengthened *T-type* specimens pointed out that this typology is less sensible to the incorrect ratio between flexural and shear strength; the displacement reached is higher if compared to the *S-type* but still inadequate to the seismic requirements. Collapses were not reached gradually and often associated with broken stirrups.

The experimental response of the pier T250 showed evident stiffness degradation during the cycles at 1.2% drift, when the shear cracks appeared. At 2.4% drift the strength of the pier started to decrease throughout the 3 cycles. After that, the pier failed frailly. At the end of the test, it was possible to see the concrete spalled out at the base, with wide 30° inclined shear cracks on the whole height (Figure 9b).

Strengthened T250FRP specimen did not show damages during the first two drifts, in particular the preyielding loss of stiffness due to shear cracks did not appear. At the 2.4% drift level, some bolts in the additional anchoring system started to be pulled out. Going toward the target of 3.6% drift level, the pier showed crushing of concrete at the base, more or less at the same displacement of the T250. Despite of this, the confining action of the FRP wrapping allowed the pier to continue the cycles without meaningful loss of strength, and only after the last cycle at this drift level the concrete crushed on the opposite side. Though the concrete was crushed and the bars were buckled at the pier base, the FRP retrofitting intervention permitted the completion of the 4.8% drift. The base of the pier was damaged but no evident strength degradation could be found. Even the vertical bearing capacity was not compromised. Only pushing the pier toward the 6.0% drift level, the failure was reached showing a softening branch. At this point the FRP at the pier base was delaminated. Despite the very high drift level reached during the test, the specimen was not heavily damaged, with the exception of the pier base: in the plastic hinge zone the concrete cover was completely crushed and the steel bars were buckled.



Figure 9 – T-type specimens with axial load of 250 kN: test results and analytical envelope (a); failure of the piers T250 (b) and T250FRP (c)

The not strengthened pier tested with axial load equal to 500 kN (T500) showed a behaviour similar to that observed in the T250 specimen; in particular pre-yield stiffness reduction during the first cycle at the 1.2% drift due to shear cracks, limited ductility and fragile collapse were observed during the experimental test (Figure 10b).

After the retrofitting intervention (T500FRP), the overall response of the pier was significantly improved. Up to the 2.4% drift no damages could be detected, except for the concrete cracked and the longitudinal reinforcing bars yielded during the second drift, effects appreciable only looking at the force-displacement curve. Stiffness degradation during the 1.2% is again removed. At 2.4% drift the external bolts of the additional anchoring system started to be pulled out. During the first cycle at 3.6% drift, the concrete cover crushed at the pier base resulting in a little strength degradation. Despite of this, the confinement given by the FRP wrapping was enough to guaranty the pier surviving, in particular it could be noted that the following two cycles were very stable.

The fundamental improvement was an increase in strength and ductility. The strength increased approximately of 20%, in agreement with the numerical prediction. Although the test was interrupted before the failure, the obtained displacement ductility was about 4, but it was licit to expect larger ductility, considering that the pier was more or less undamaged (Figure 10c).



failure of the piers T500 (b) and T500FRP (c)

The numerical predictions given for T250FRP and T500FRP specimens are related to the flexural behaviour of the elements. The curves "FEM prediction" in Figures 9a and 10a have been obtained with a fibre model of the section with additional exterior fibres to account for the FRP longitudinal sheets (FRP constitutive model linear elastic up to rupture). The second curves named "FEM prediction (pulled out bolts)" have been obtained with a modified FRP constitutive model (bilinear with reduced stiffness in the last branch) to simulate the anchoring system pulled out during the last phase of the test.

Dissipated energy and damping

Further proof of the FRP strengthening effectiveness can be obtained considering the dissipated energy. The hysteretic damping or energy loss per cycle can be converted to an equivalent viscous damping ratio using the well known formula:

$$\xi_{eq} = \frac{A_h}{2\pi F_m \delta_m} = \frac{A_h}{4\pi A_e}$$
 Eq. 9

where A_h is the area included in a single hysteretic loop, F_m and δ_m represent the peak force and displacement values and A_e is the elastic energy stored in an equivalent linear elastic system.

An example of equivalent viscous damping ratios calculated from the experimental results of first and second series are compared in Figure 11a and 11b respectively for the *S-type* and the *T-Type* specimens with axial load of 500kN.

Dissipated energy for all the specimens at lower drifts (0.4% and 1.2%) is of the same order of magnitude both in first and second series. For higher levels of drift, it is confirmed that strengthened specimens can sustain higher demands. Total dissipated energy varies from 2 to 3 times of that obtained for the not strengthened cases. Exception is the lap splice case, being poorly meaningful.



S500 vs. S500 FRP (a) and T500 vs. T500 FRP (b)

CONCLUSIONS

In this research different FRP strengthening solutions have been considered to improve the cyclic response of bridge piers with deficiencies due to design for gravity loads only.

A first general comment should be given to observe that under-designed structures are often unable to sustain the acceleration levels prescribed from modern standards essentially for poor detailing (insufficient confinement, loss of bonding) and incorrect proportioning of flexural and shear strength. A critical review of the results obtained from this campaign allows some positive and encouraging conclusions but also underline that effectiveness of FRP wrapping could be still uncertain when trying to extend this technology to real hollow bridge piers.

The most impressive results have been obtained in *S-type* piers, where the lack of shear resistance is responsible of the fragile behaviour. The transversal wrapping improves the shear capacity, allows deep plastic deformations in the longitudinal bars, reduces the cracking, increases dissipated energy and reduces the fragile behaviour when the collapse is reached. There was no evidence of malfunctioning due to technical problems as local ruptures or debonding.

The lap splice case was not completely successful due to anchoring failure of the longitudinal sheets at the base of the pier. The idea of having an additional system to carry the entire tensile force on the section, bypassing the lap splice zone, appears to be promising in this case, where additional confinement is not enough to assure the force transfer in the joined bars. Nevertheless it needs further efforts to assess and to better calibrate the anchoring. A first attempt has been done adopting a mixed system for the other *T-type* samples strengthened with longitudinal FRP fabrics, which consists of two steel beams bolted to the foundation used to restrain the lower portion of the fibres.

Tests results on the *T-type* specimens with longitudinal and transversal strengthening have shown that flexural and shear strengths improve respecting the capacity design criteria. For the two cases considered, the additional anchoring system performed well enough to allow deep inelastic deformations (6% drift was reached) but there are still reasonable doubts about the applicability to real cases where large tensile forces must be anchored with bolts to the foundation, important technical problems must be solved.

Notable is the agreement between the experimental results and the numerical predictions obtained superimposing shear domains, calculated in accordance with Priestley [5], and flexural response curves, calculated with a fibre model of the cross section: both modified to account for the FRP enhancement.

ACKNOWLEDGMENT

The research work presented was funded by INGV-GNDT 2000-2003 framework project (VIA - Reduction of Seismic Vulnerability of Infrastructural Systems and Physical Environment) and by the European Community (SPEAR G6RD-CT-2001-00525).

The fibre reinforced polymer was provided by MAC S.p.a. (Modern Advanced Concrete), Treviso (Italy).

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