FRP RETROFIT FOR INSUFFICIENT LAP-SPLICE IN RECTANGULAR HOLLOW SECTION RC BRIDGE PIERS: NUMERICAL SIMULATION AND LARGE SCALE TESTING

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ABSTRACT :
Most existent bridges do not reflect the recent earthquake design rules particularly for the aspects related to the definition of the seismic input, the energy dissipation capacity and the capacity design concepts. Several efforts have been addressed in last years both developing assessment methods and testing retrofit measures to improve the seismic response of RC bridge piers.
The research presented in this paper focused on the loss of resistance and ductility due to insufficient lap-splice, leading to the slippage of the longitudinal bars, in the critical area of hollow rectangular section bridge piers. First, a simple and reliable model for assessment and design purposes was developed and validated on the tests results of a previous experimental campaign. This model has then been used to investigate the efficiency of the FRP retrofit in case of insufficient lap-splice. The retrofit design has been optimized through a parametric analysis considering different fibers and geometrical characteristics.
Quasi-static cyclic tests will then be performed on four 1:2 scaled piers allowing for the evaluation of the efficiency of the FRP strengthening through comparison of the performances of the as-built and the retrofitted specimens. During the realization of the retrofit, particular attention will be paid to the anchorage of the FRP at the base of the pier. Furthermore, it is also very important not to violate the shear capacity consequently to the increase of the lateral strength deriving from the upward shift of the plastic hinge obtained as a consequence of the chosen retrofit solution.
KEYWORDS:
hollow sections; bridge piers; FRP seismic strengthening; lap-splice; under-designed.

1. INTRODUCTION

The majority of the European highway bridges have been constructed between the ‘50s and the ‘70s when many areas had not yet been recognized to be earthquake prone and seismic provisions were not enforced. Consequently, most of the existent bridges do not reflect the recent earthquake design rules particularly for the aspects related to the definition of the seismic input, the energy dissipation capacity and the capacity design concepts. The seismic events of the past years underlined that the major lacks limiting the seismic performances of the existing bridge stock are: (i) the reduced lateral confinement; (ii) the unbalanced flexural-shear resistance; (iii) the early interruption of the longitudinal bars; (iv) the insufficient lap-splice. From here the necessity for efficient and reliable methods to assess and techniques to improve the seismic response of a huge number of bridge piers.
This research focuses on the problems deriving by the insufficient lap-splice at the critical section of the piers. Two previous experimental campaigns on 1:4 scaled hollow section bridge piers designed accordingly to non-seismic standards and tested both before and after FRP seismic retrofit (Calvi *et al*., 2005 and Pavese *et al.*, 2004) are the starting level of knowledge of the present work. The results obtained from those tests have been used to validate a finite element model of easy use during both the assessment and the retrofit design phase. This model has then been used to carry out a parametric study considering carbon, glass or aramid fibers and different geometrical characteristics of the FRP layers with the objective of optimizing the strengthening for each pier. Experimental quasi-static cyclic tests will be performed in an attempt of estimating the efficiency of the FRP retrofit towards the restoring of the desired flexural, shear and energy dissipation capacity.

2. INSUFFICIENT LAP-SPLICE

Tensile stress in the longitudinal bars are transmitted in lap-splice with the development of a strut-and-tie mechanism in the concrete (Priestley and al., 1996). When the longitudinal bars superposition is limited, tensile failure can occur in concrete leading to the opening of a big single crack and to the slippage of the longitudinal reinforcement strongly limiting the energy dissipation capacity. Furthermore, if the tensile force developed in the bars is lower than the steel yielding limit, a significant loss of the flexural capacity will take place.
Several possible solutions to this problem can be found in literature aiming to restore the stress path across the lap-splice hence allowing for the development of a higher strength as well as longitudinal steel yielding (plastic hinge is expect to form above the retrofitted region) leading to a most efficient energy dissipation mechanism. One of the most common solution is jacketing the critical zone. Adding confinement to the lap-splice region through jacketing it is possible to increase the concrete tensile capacity leading to an improvement of the stress transfer between the two overlapped bars. Different materials can be used to realize the jackets: concrete, steel or composite materials. In seismic applications, problems can arise using a concrete jacket since a meaningful variation of the element stiffness is caused by the increase of the section leading to undesirable variation of the modal characteristics of the retrofitted structure. The effectiveness of a steel jacket is on the other hand limited by the achievement of its yielding stress. Once yielded and plastically deformed the additional confinement given by the jacket is lost. Furthermore, the efficiency of a steel jacket applied on a rectangular pier is very low away from the corners of the section. Last option is to use composite material jacketing. This is quite effective when the retrofit material is applied to the structure with a certain level of prestress. Cons of this method are the difficult application of prestressed jackets, that clearly can not be done by hand, and the low efficiency of wrapping on a non-circular section. In any case, the effectiveness of the additional confinement on rectangular hollow section piers remains doubtful. Another already experimented solution is the use of near-surface mounted FRP bars in the attempt to create an alternative path for the tensile stresses crossing the lap-splice. These bars need to be placed close to the pier surface and wrapped to them to guaranty their anchorage to the pier. Critical is their anchorage in the foundation.
The retrofit solution that we chose follows the idea of creating an alternative stress path across the spliced region using longitudinal FRP strip applied to the critical region. Their minimum length should equal the overlapping region plus the anchorage length. Particular attention should be paid to the anchorage of the retrofit material at the base of the pier. With a correct design the retrofitted region should remain elastic, while a plastic hinge should form above the retrofitted region allowing for an efficient energy dissipation. Clearly the upward shift of the plastic hinge will lead to an increased shear capacity, hence it is of primary importance not to violate the capacity design criterion protecting the retrofitted elements from shear collapse: At this scope FRP transversal wrapping can be used increasing both the shear capacity and the concrete confinement, always useful when a RC structure is exposed to seismic loading.

3. DEVELOPMENT OF A MODEL FOR ASSESSMENT AND RETROFIT DESIGN

Because of the large number of bridges built before the enforcement of seismic design rules that need to be assessed and that potentially calls for an adequate seismic retrofit to satisfy the actual code requirements and to withstand seismic loading, the development of an easy to use model applicable within these scopes is critical. This was the first objective of this research. A quite simple though effective finite element (FE) model was developed using Seismostruct (2006): a finite element program featuring fibre elements automatically accounting for both material and geometric non-linearity. The following Figure 1 shows the adopted FE model. Clearly, modelling the bridge pier in the as-built configuration does not represent a difficult problem. Once the retrofitted pier has to be modelled, things are a bit more complex. The longitudinal FRP layers have to be represented like an element itself. Rigid links has been used to place the FRP at the right distance from the longitudinal axis of the retrofitted member. Each FRP element must have pinned connection at both ends in order to be subjected to pure axial load. It has to be noted that the FRP has been applied, in this case, only to one side of the member since this model was used to run unidirectional push over analyses. Furthermore, it is worth mentioning that in this example the rigid link are not superimposed: this has been done only to underlined that two pins must be used at the end of each rigid link. Two pins are required, one for each of the FRP element joining the node, since they will have

different rotations during the analysis to allow the FRP to work under pure axial load. On the other hand, the FRP wrapping can be modelled without adding elements to the model, since its main effect is the increased concrete confinement that can be represented by the confinement factor already present in adopted concrete stress-strain representation.
The only care that one must have using this model to design the FRP longitudinal strips is to correctly choose the length of the base element. Since the FRP element will have a constant axial load (*i.e.* a constant axial stress) along its length, while the maximum stress will be at the base, the shorter the FRP element the higher its stress. On the other hand, a too short base element will certainly produce numerical instability. For this reason, few trials are required to try to find a balance between these two problems, hence estimating with a reasonable accuracy the maximum stress in the FRP avoiding at the same time loss of convergence from a numerical point of view.

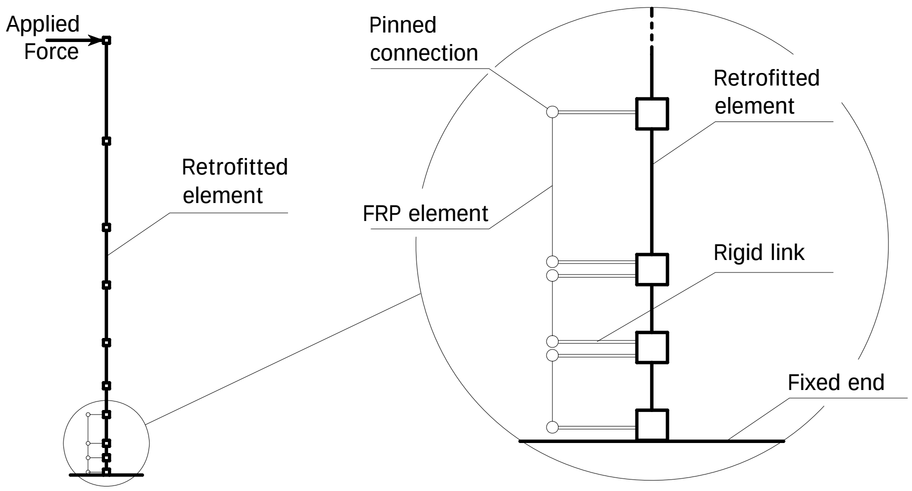


Figure 1 - FRP retrofitted pier model

To tests the effectiveness of the adopted FE model, the behaviour of 1:4scaled square hollow section piers from previous experimental campaigns (Calvi *et al.*, 2005 and Pavese *et al.*, 2004) has been reproduced through push-over analysis.
Experimental campaigns were performed in quasi-static regime applying series of three cycles at increasing drift level (0.4%, 1.2%, 2.4%, 3.6% and 4.8% when possible) at the top of the piers. Between the tested specimens, three couples of piers, with and without FRP retrofit, have been chosen involving both squat (aspect ratio 1:2) and tall pier (aspect ratio 1:3) with axial load equal to 250 kN or 500 kN. In the following, these pier will be referred to with their acronym: the first letter, S or T, stands for squat or tall; the number shows the applied axial load expressed in kN; the final letters "FRP" indicate the retrofitted piers. For instance the specimen T250FRP is the retrofitted pier with aspect ratio 1:3 and axial load 250 kN. Starting from the hysteretic loops experienced by the piers, the envelope have been drawn, as shown in Figure 2, keeping trace of the maximum of force and displacement achieved at each drift level.

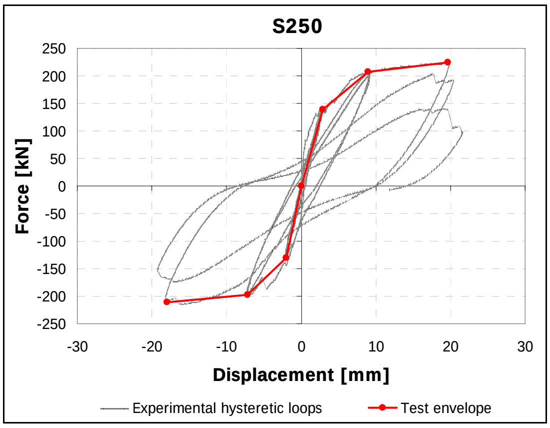


Figure 2 - Derivation of the backbone curve from one of the tests

This has been successively depurated from displacement due to the shear deformation, not accounted by the fiber element formulation implemented in Seismostruct. This "depurated" test envelope has been compared with the push-over curve obtained by the non-linear analysis. The results are reported in the next graphics (Figure 3) underlining that the adopted FE model is able to represent the behaviour of FRP retrofitted members, both for transversal and longitudinal retrofit. Note that, when the pier specimens failed in shear, the evaluated ultimate displacement exceed the real one due to the fact that the adopted model does not consider shear as a failure mode.

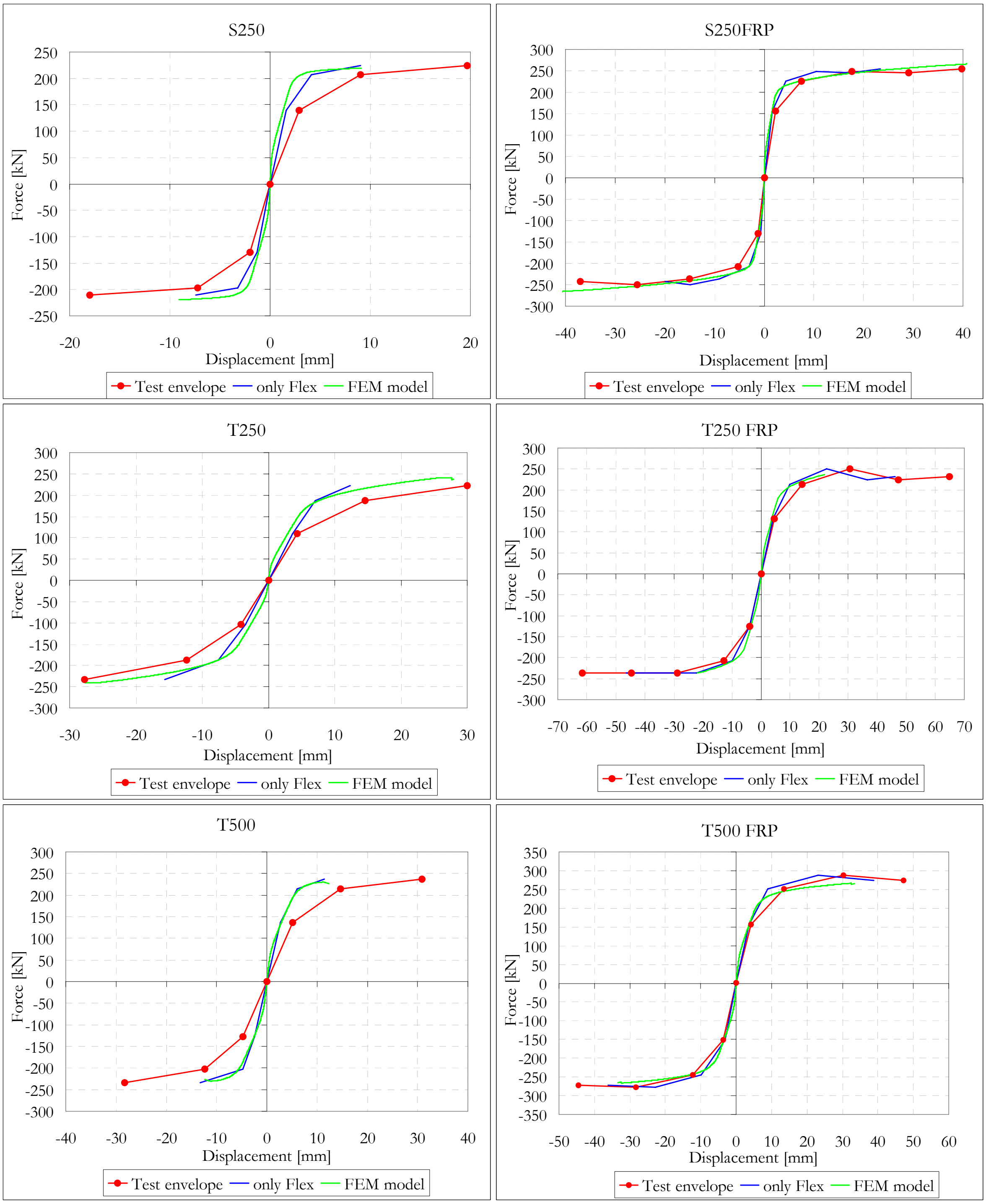


Figure 3 - Comparison between test envelope and FE model prediction

4. CURRENT EXPERIMENTAL CAMPAIGN

As part of the ReLUIS program, a research involving the Italian network of seismic engineering laboratories, four 1:2 scaled hollow section reinforced concrete bridge piers will be tested. Here is just a brief review of their principal characteristics. The four specimens have the same geometric properties, see Figure 4. The external cross section dimensions are 800 mm times 1500 mm with 150 mm wall thickness. The height of the specimens equals 6 m from the top of the foundation to the point of application of the horizontal load (aspect ratio equal 4). The specimens have 80 ϕ 10 mm longitudinal bars (ρ_L=1.05%). To investigate the problem deriving from an insufficient lap-splice, the longitudinal reinforcement is connected with the bars coming up from the foundation of the pier through a 200 mm (twenty times the bar diameters) superposition. The transversal reinforcement is designed following the Eurocode 8 prescriptions to avoid shear collapse in the as-built configuration. This reinforcement is made of rectangular stirrups having 6 mm diameters with 100 mm spacing (ρ_V=0.89%).

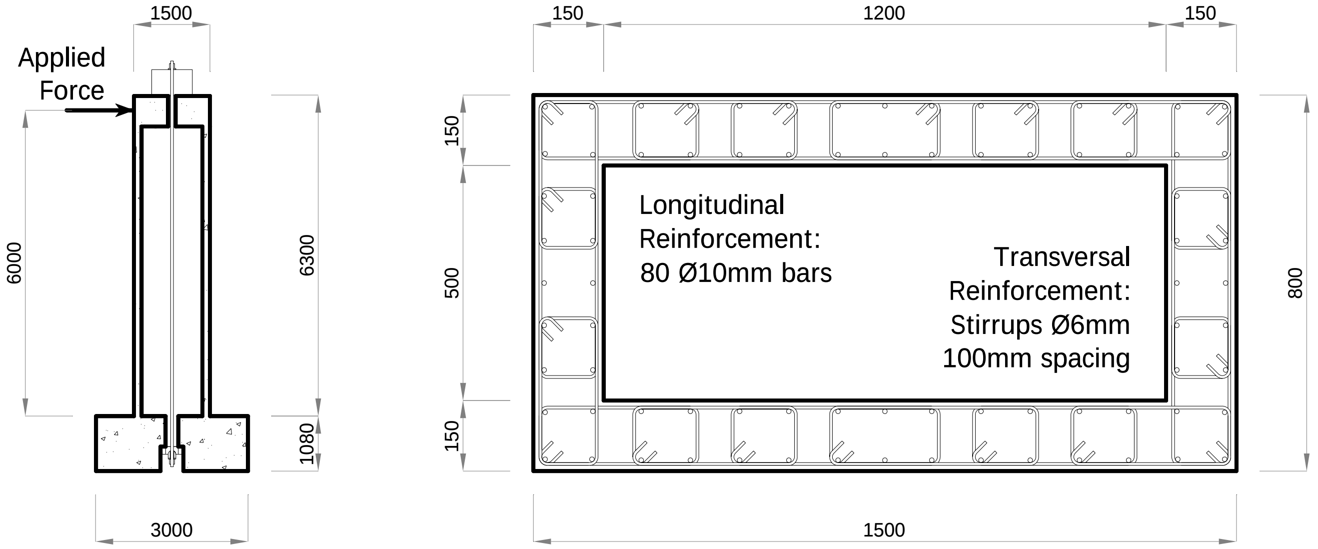


Figure 4 - Geometry of the specimens

Quasi-static cyclic test at increasing drifts levels will be applied to the pier up to a maximum drift level equal to 6% (*i.e.* 360 mm maximum top displacement) or until the pier collapse is reached. Two specimens will be tested in the as-built configuration with the objective of refining the assessment methods for the lap-splice capacity. The other two piers will be tested after the application of the FRP seismic retrofit aiming to the protection of the bar superposition region where plastic hinge with significant energy dissipation capacity can not form. Final goal of the retrofit will be the change of the collapse mechanism of the piers resulting in an upward shift of the critical section hopefully resulting in a higher lateral force capacity, displacement ductility and energy dissipation. The two couples of piers will be tested under two different axial loads, in particular the two axial loads will be 1000 kN and 2000 kN (ν = 4.9% and 9.8% respectively). The load will be applied by means of a post-tensioned high strength steel bar crossing the inner cavity of the specimen. The bar will be connected to the base of the pier using a hinge to avoid restrain moments, while at the top there will be an hydraulic jack used to keep constant the axial load during the test.

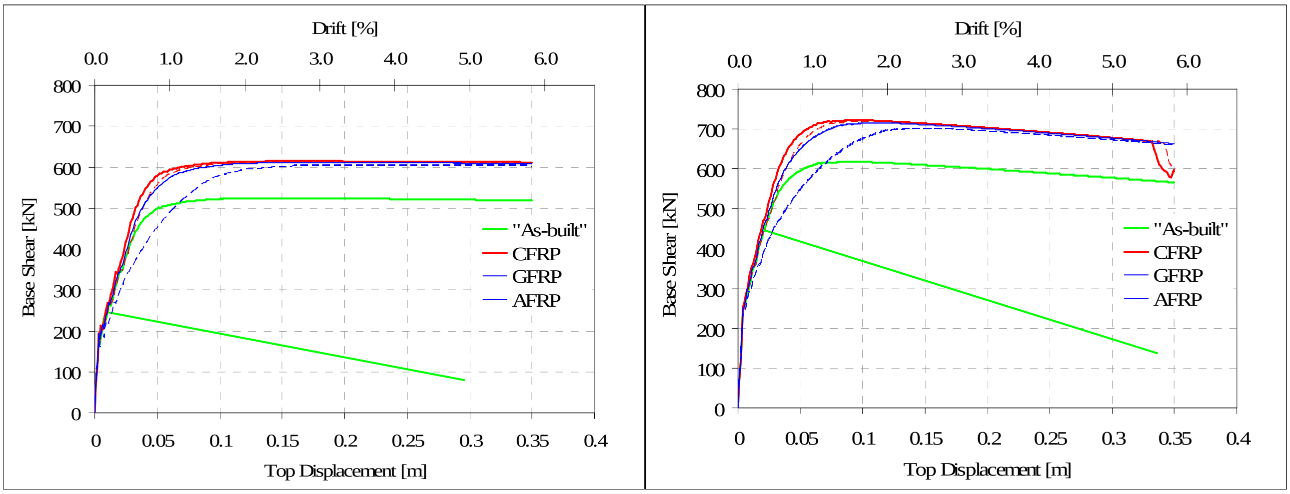
Table 4.1 Mechanical properties of the considered FRP materials

Material	Density	Effective thickness	Tensile strength	Elastic modulus	Ultimate Strain
	[kg/m^3]	[mm]	[MPa]	[MPa]	[%]
Carbon	1820	0.165	3000	390	0.8
Aramid	1440	0.214	2800	105	2.7
Glass	2600	0.230	1700	65	2.8

Although the actual mechanical characteristics of both concrete and steel will be determined just before the tests, the following values have been assumed for the numerical simulations accordingly to the design prescription: (i) concrete mean ultimate stress 37 MPa; (ii) steel mean yield stress 530 MPa. On the other hand, the mechanical properties of the FRP materials considered for the retrofit are reported in the previous Table 4.1.

5. NUMERICAL SIMULATIONS

The previously mentioned FE model was used to model the behavior of the four prototypes to be tested. Numerical simulations were required to estimate the expected pier behavior in terms of force displacement and to optimize the FRP retrofit intervention choosing the most suitable type of fiber: carbon, aramid and glass fibers were initially considered. Furthermore, also the number and the dimension of the FRP strips to be used had to be determined. The force-displacement capacity curve of the specimens with axial load equal to 1000kN and 2000kN obtained through numerical simulations are reported in the following Figure 5. The theoretical behavior of the as-built piers is shown by the green solid line. It has to be noted that this curve represents the behavior of the pier without lap-splice lacks. When the longitudinal bars start slipping, the monotonic force-displacement relation shows a linear softening branch (see Figure 5). This line is expected to start from the point corresponding to the achievement of the maximum stress that can be developed in the steel prior to the bar slippage: in this case this stress has been evaluated as 364 MPa following the indications given by Priestley *et al.* (1996). The descending line is estimated to reach a residual pier lateral strength, just due to the recentering capacity of the axial load, equal to 80 kN or 136kN for the piers subjected to 1000 kN and 2000 kN respectively. This residual force should be reached with a displacement ductility equal to 8.



(a) (b)

Figure 5 - Predicted force-displacement curves: (a) N = 1000kN; (b) N = 2000kN

Together with the ideal pier behaviour, that should be the target of an optimal retrofit, the force-displacement relations after the retrofit intervention are shown: red lines for carbon FRP and blue lines for aramid and glass FRP. For each material two lines are plotted showing the upper (solid line) and the lower bound (dashed line). The upper bound behaviour has been evaluated considering that FRP would be able to prevent bar slippage, hence considering the longitudinal steel as able to sustain tensile stresses. On the other hand, the lower bound has been estimated considering no longitudinal steel in the retrofitted region, thus neglecting the tensile contribution of the longitudinal steel as it was slipping from the beginning. Those two lines are therefore expected to bound the real behaviour that the retrofitted specimen will experiment during the tests.
Changing the number and width of the FRP strips applied longitudinally to the piers as reported in the following Table 5.1 the numerical simulations shown that it should be possible to reach the same level of lateral force. The increase of the maximum shear with respect to the “as built” situation is due to the fact that the FRP retrofitted region behaves linearly and the plastic hinge move upwards where the FRP ends: in this case it has been considered to apply the FRP up to 800 mm from the foundation. Since the longitudinal reinforcement is constant over the height of the piers, the yield moment will be constant too, but having a shorter lever arm the force

applied to the top of the pier must increase to cause the steel to yield. It is worth mentioning that this increase of the shear required to develop flexural yielding should be followed by an adequate increase of the shear capacity of the retrofitted element in order to respect the correct strength hierarchy and to avoid undesirable shear collapses that would compromise the effectiveness of the retrofit intervention. Depending on the transversal reinforcement of the pier, it could be necessary to apply also a transversal FRP reinforcement. This last would imply multiple advantages: beside avoiding shear collapse, the transversal layers will also help to avoid the debonding of the longitudinal strips and to confine the concrete increasing its deformation capacity.

Table 5.1 Number and width of the longitudinal FRP layers

Material	Number of layers	Strip width [mm]
Carbon	4	750
Aramid	3	750
Glass	5	700

The only consequence depending on the choice of one of the three retrofit material is the resulting initial stiffness. From this point of view carbon FRP seems to behave in the best manner keeping the initial stiffness as close as possible to the original value. This characteristic is quite important in seismic application since this parameter as a strong influence on the structural period of vibration, *i.e.* on the seismic input to the structure.

6. CONCLUSION

Within the scope of developing an easy way to assess the performances of exiting bridge piers and to design eventual FRP seismic retrofit, a simple finite element model has been developed. Validation of the FE model was done reproducing the behavior of 1:4 scaled piers as recorded from tests performed at the University of Pavia. Those tests involved hollow rectangular section bridge piers with and without the FRP retrofit showing different collapse mechanisms.
Once the FE model was tested, it was used aiming to the evaluation of the effectiveness of the FRP retrofit in case of piers suffering for insufficient lap-splice. Four 1:2 scaled specimens will be tested in the current experimental camping and their behavior has been assessed through numerical simulations. Beside estimating their performances in terms of force vs. displacement, the FE analyses were used to optimize the design of the retrofit intervention. Three different types of fibers were initially considered: carbon, aramid and glass. From the performed simulations it seems that using FRP retrofit could be possible to increase the lateral strength of piers suffering for insufficient lap-splice overcoming the problem of bar slippage. Only the initial stiffness of the ideal pier behavior (*i.e.* corresponding to a situation without bar superposition) could not be fully restored. From this point of view CFRP seems to be the most attractive material.
Nevertheless two criticalities arise. First of all, particular attention should be paid during the design of the retrofit intervention to avoid violation of the capacity design criteria. Since the application of the longitudinal FRP strips to the base of the pier will result in an increases of its flexural capacity (*i.e.* the shear corresponding to the yielding moment), the evaluation of the shear capacity is essential and if required transversal FRP reinforcement should be applied to avoid shear collapse. On the other hand, to properly anchor the longitudinal FRP to the pier foundation is fundamental. This point could be critical since from it depends the effectiveness of the retrofit intervention and very high forces must be anchored: in this case the maximum force is about 1500kN.
Experimental tests will be soon performed as they clearly remain essential in the attempt of evaluating the improvements deriving from the adoption of the FRP retrofit intervention. It has to be considered that several problems such as debonding of the longitudinal FRP at crack locations or tearing of the fibers along the corners of the section can not be included in the finite element model but they still remain of primary importance as they could jeopardize the effectiveness of the adopted retrofit solution.

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