# EXPERIMENTAL STUDY ON THE CYCLIC RESPONSE OF AN EXISTING RC BRIDGE PIER

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**ABSTRACT :**

This paper deals with experimental activity on piers belonging to an old viaduct of the Italian highway network. The tests consist of cyclically imposed displacement to a 1:4 scale of one of the piers, performed in the laboratory of the department of structures of the University of Roma Tre. The objectives of the experimental campaign is twofold: 1) evaluation of the failure mechanisms and collapse strength of the pier, 2) calibration of a numerical non-linear model using Opensees code, in order to simulate the seismic behaviour of the analyzed bridge. The experimental test performed on the first mock-up has shown that the shear behaviour is extremely relevant. In particular, the shear failure of the transverse beam highly influences the global cyclic behaviour of the specimen, with a pronounced pinching effect that reduces the dissipation capability of the pier. Moreover, other less relevant phenomena are observed: bond-slip of the reinforcing bars and buckling of the longitudinal steel bars of the column. A numerical model built in Opensees has been calibrated in order to reproduce the experimental results and provide a reliable model for further seismic analyses. Using literature models to reproduce the experimentally observed phenomena, the model has been able to simulate the real behaviour of the specimen in a satisfactory way.

**KEYWORDS:**
Existing bridges, experimental activity, plain steel bars, cyclic response, shear modeling, non-linear analysis

## 1. INTRODUCTION

The assessment of the seismic vulnerability of bridges is a complex process, which requires an adequate background, often not achievable for the lacking of information, hardly available, especially for reinforced concrete structures (adequate knowledge of the mechanical and geometrical characteristics of structure and materials, information about any structural modification occurred during the life of the bridge, a reliable estimation of the gravity load, etc..). These difficulties have an important consequence: the practical impossibility to estimate the seismic risk of such kind of structures.

In particular, the evaluation of the expected seismic response of the bridge structures is still conditioned by many uncertainties about the main mechanisms which characterize the post-elastic response. For this reason an effective utilization of methods and calculus instruments which imply non-linear analysis is difficult. Because the latter is becoming a popular analysis method, there is the necessity to correctly model the plasticization zones providing reliable indications about the performance required to the structure.

In addition, the steel reinforcement of old constructions was often realized with plain steel bars, whose behaviour, especially regarding bond and anchorage, has not been widely investigated in the past. In fact, the technical literature about the plain bars behaviour and relative anchorage, realized by circular hooks, highlight experimental campaign is very dated. Only recently, this problem has been re-evaluated under a scientific point of view and a systematic approach to the study of bond-slip and anchorage efficiency of plain steel bars has been adopted (Fabbrocino et al. 2005, Feldman and Bartlett 2005).

The bridges of the Italian highway network often show the above mentioned characteristics. Moreover, recalling the strategic nature of this kind of constructions, an adequate further study of the problem appears necessary.

Among the research activities devoted to this topic, the experimental one assumes a very important role for quality and quantity of information that is able to provide. Furthermore, a scarce experimental activity on old reinforced concrete structures makes this subject even more interesting (Verderame et al. 2004).
In the present paper some results of an experimental campaign on reduced-scale bridge framed piers are illustrated. The piers belong to an old Italian highway viaduct on the Firenze-Bologna section, which has been realized in the sixties. The 1:4 scaled specimens are three one-bay two-floors reinforced concrete frames built with plain steel bars. The objectives of the experimentation are: 1) identification of the most relevant failure mechanisms and the ultimate strength of the piers, 2) calibration of a numerical non-linear model using Opensees code for simulating the seismic behaviour of the piers. Because of the tests are still in progress, in what follows only the results about the first pier will be illustrated and discussed, postponing in further works the considerations on the results about the second and third pier.

## 2. TEST SPECIMENS AND SET-UP

The Department of Structures of the University of Roma Tre is involved in a wide Italian research project regarding the assessment of existing bridges (Reluis Project), and in particular about the definition and execution of experimental tests for the evaluation of the seismic behaviour of high-dimension structural elements with and without retrofitting systems. In particular, tests on the piers belonging to an old Italian viaduct have been carried out in the laboratory of the department. The structure, built in Italy in the sixties along the Firenze-Bologna highway (Fig. 1), is characterized by continuous deck pinned over 12 piers. The twelve supports are realized by a couple of reinforced concrete framed piers. Three 1:4 mock-up's of the piers n° 12 (Fig.1) have been built in order to carry out quasi-static cyclically imposed displacement tests. Each pier is a reinforced concrete frame 3.5 m high, composed by two circular columns of diameter D=30 cm, a transverse beam with rectangular section 10x30 cm placed at middle of the height (1.75 m), and finally a cap beam with a c-shaped section (Fig. 2). The foundation is a rectangular beam with section 30x60 cm.


Figure 1 – "Rio Torto" viaduct

The mix-design of concrete has been performed using the Bolomey's curve for the aggregate design mix, adopting as maximum aggregate dimension 2 cm, in order to have short-term properties compatible with the awaited behaviour of the pier. Subsequently, using direct compression tests on cylinder, the entire constitutive law of the concrete has been determined.
The reinforcing of the mock-up's consist of smooth rebars. Direct tensile test for the evaluation of the constitutive law of steel has been carried out on steel bars with diameter of 2 and 6 mm, which are used in the experimental campaign (Fig. 2). In Table 1 are shown the mechanical characteristic of the concrete and steel bars used in the tests.

Table. 1 – Mechanical Characteristic of materials (average values)

| | **Strength (MPa)** | **Elastic Modulus (MPa)** | **Poisson Coefficient** |
|---|---|---|---|
| Concrete | 26 | 27000 | 0.16 |
| Steel | 360 | 205000 | 0.30 |

In order to characterize the bond mechanism of straight steel bars and bars with end circular hooks, pull-out tests have been performed. The tests consist of displacements applied to a steel bar anchored in a concrete block; force and slip between bar and concrete are continuously measured during the tests. The results have shown the efficiency of the anchorage device, even if a relevant slippage of the steel-bar has been observed. Furthermore, the significant parameters of the normal tension-slip law of the anchorage device have been identified.

As far as the experimental test on the mock-up concerns, horizontal displacement imposed in quasi-static regime to the top of pier has been applied using a 250 kN hydraulic actuator in displacement control (Fig. 3). The foundation of the mock-up is anchored to the laboratory base concrete slab using anchor bolts. Vertical loads are simulated using prestressing forces applied to the columns using post-tensioned bars. The displacements along the height and the horizontal force applied at the top have been measured using, respectively, wire LVDT transducers and 500 kN load cells. The mean curvature at the column and beam edges has been monitored using longitudinal displacements sensors (Fig. 4)

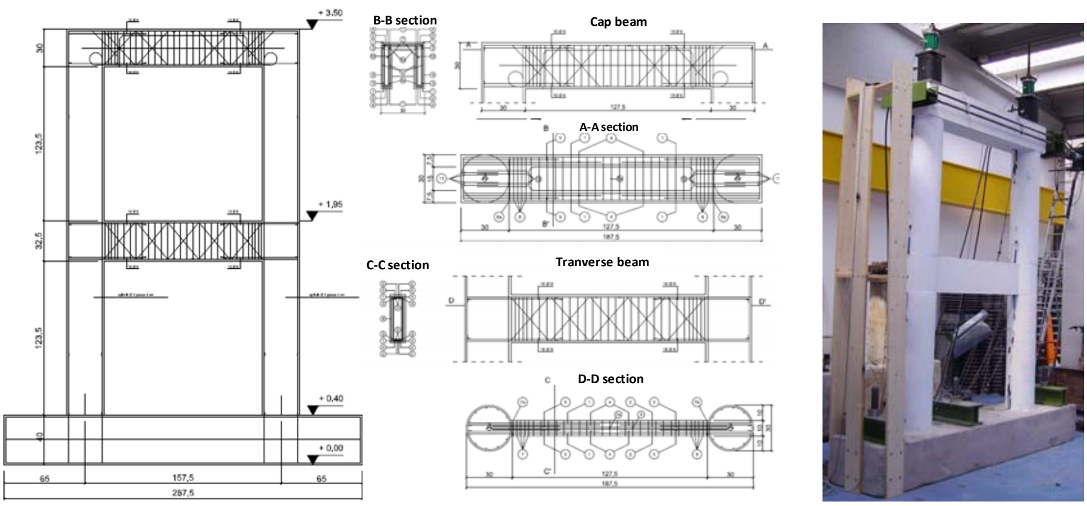



Figure 2 – Test specimen and reinforcement details

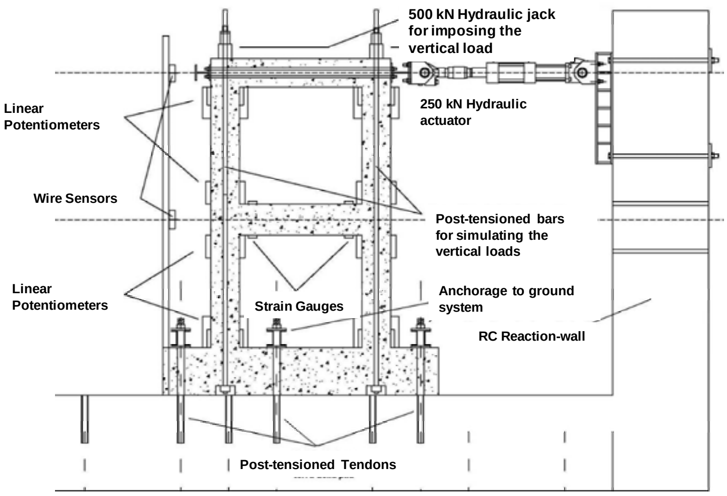



Figure 3 – Test specimen and experimental setup

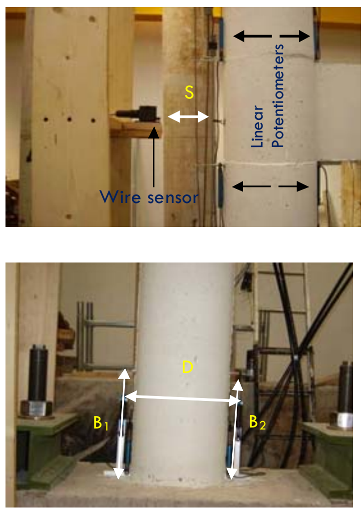



Figure 4 – Displacement transducers

## 3. ANALYSIS OF THE EXPERIMENTAL RESULTS

The experimental test consists of displacements cyclically imposed at the top of the specimen. The time-history of the displacements is shown in Figure 5. The amplitude of the cycles is variable from 0.1 mm to 60 mm, and for each amplitude three cycles of displacements have been imposed, for a total number of 52 cycles. The frequency of the applied signal is equal to 0.05 Hz in order to realize a quasi-static test. The dead load acting on

the real scale pier is equal to 6600 kN; therefore, a vertical load of 200 kN has been applied to each column of the pier, using the prestressing system previously described.
The dead loads were expected to be constant for simulating what happens during an earthquake, in case of the absence of vertical component of the acceleration. Actually, the load cells have recorded a time-variable force, with the same frequency of the horizontal applied displacements. The two vertical forces are out-of phase, which show an initial value of 200 kN and maximum and minimum equal to 260 kN and 190 kN respectively. Moreover, the time-history is not symmetric and shows an increasing average value.

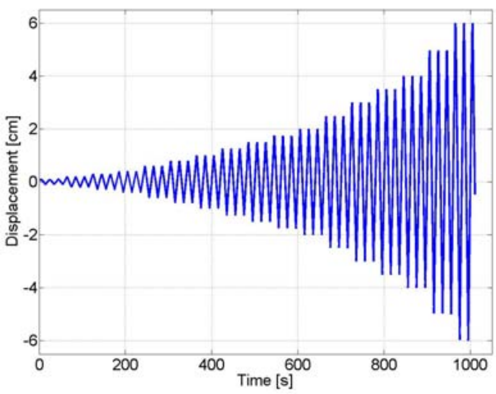



Figure 5 – Applied displacement time-history

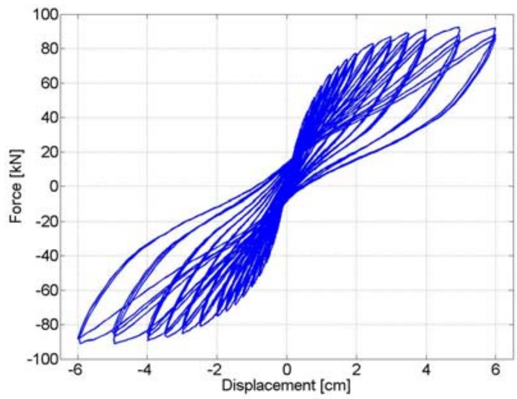



Figure 6 – Vertical load time-history

The global cyclic behavior measured in the experimental test during the first six cycles is, as expected, quite linear, even if there is a low dissipated energy probably due to an initial settlement of the specimen.
In Figure 6 the complete cyclic history is shown. Because the cycles are asymmetric, only the behavior for positive displacements and forces can be examined. For negative values similar conclusions can be drawn. As far as the shape of the cycle concerns, a marked pinching is observed, which indicates that the behaviour is dominated by shear failure.
In fact, Figure 7 shows the cracking pattern of the transverse beam, where the typical shear cracks are recognizable. However, the marked cracking of the transverse beam did not imply a fragile failure, but on the contrary the shear reinforcement has provided its contribution also in presence of evident buckling phenomenon of the longitudinal bars, with a consequent rupture of the stirrups.

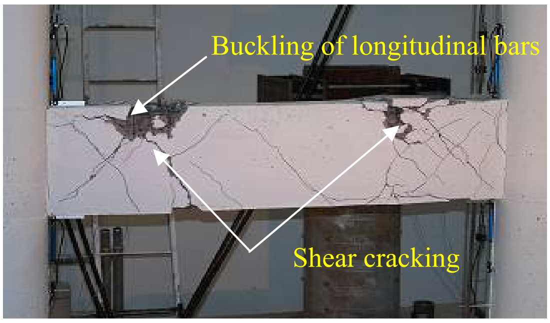



Figure 7 – Shear cracking pattern of the transverse

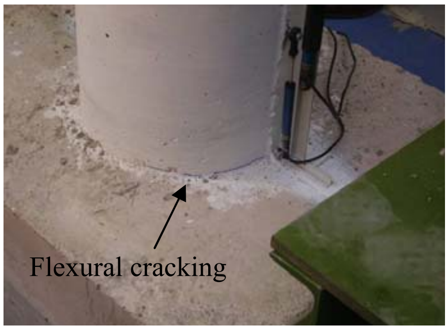



Figure 8 – Cracks at the base of a column

For high values of displacements a decreasing of the global stiffness has been observed up to half loading path and then an increasing of the stiffness up to the maximum applied displacement has been attained (Fig. 6). Moreover, degradation in terms of force has been identified for cycles with the same amplitude.
Another contribution to the particular shape of the global hysteresis cycles is due to the strain penetration effect, as will be highlighted with more detail in the following paragraph where numerical-experimental comparison will be shown. In fact, it is well known that on equal flexural moment, the bond-slip produces a local decreasing of the elements deformation and at the same time induces rigid rotations of the joints, making the

structure more flexible.
During the test, others local failure phenomena have been observed. In particular, cracks at the column-foundation, column-transverse and column-cap beam joints have been developed. As an example, Figure 8 shows a crack at the base of the right column. Moreover, a low buckling phenomenon of the longitudinal bars of the columns has been noticed, almost certainly due to too much high stirrup spacing.

## 4. FINITE ELEMENT ANALYSIS AND MODEL CALIBRATION

The experimental results illustrated in the previous sections for a bridge pier are herein compared with the simulation results in order to reproduce as precisely as possible the experimental test and to calibrate a finite element model.
The pier is modeled by using the finite element code Opensees (McKenna et al., 2007) developed for seismic non linear structural analysis able to take into account all nonlinearity sources in a reinforced concrete structure excited by earthquakes. The finite element scheme is illustrated in Figure 9a. The structural elements are modeled by 40 nonlinear beam elements characterized by flexibility formulation. All degrees of freedom are fixed at the base of the finite element model.

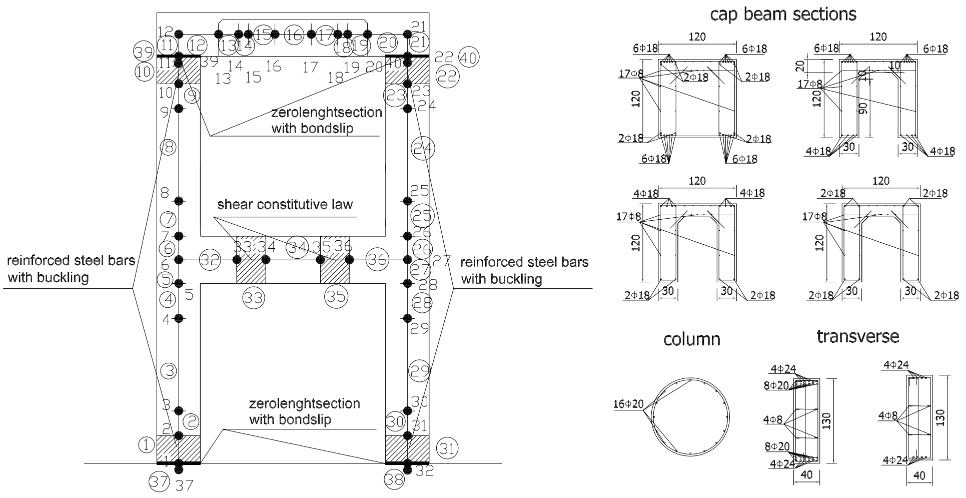
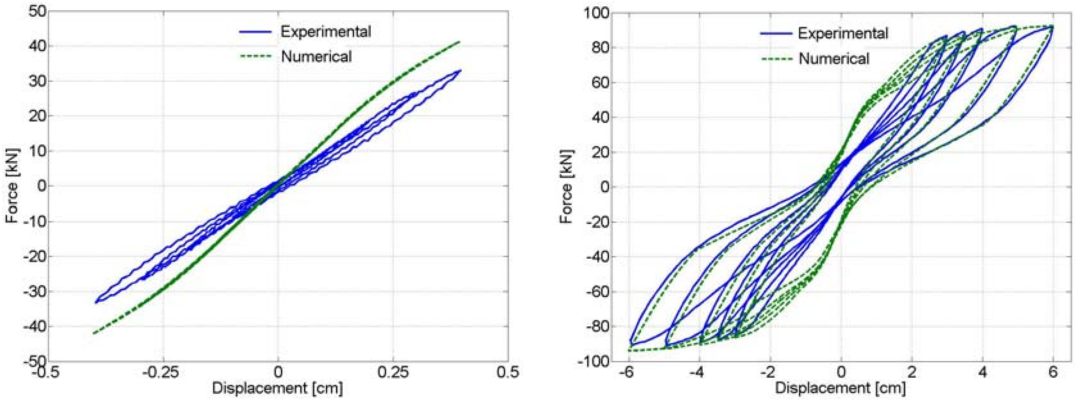



Figure 9 (a) Finite element model scheme. (b) Element sections (actual dimension and reinforcing steel).

The section of each element is subdivided in fibers such that it is possible to assign for each material the constitutive model and the exact position and dimension of the reinforcing bars. Six element sections are used as indicated in Figure 9b. A Kent-Scott-Park model is chosen for the concrete behavior (Kent and Park, 1971). This constitutive law for uniaxial material response has a first parabolic trend up to compression peak stress equal to 30 MPa with a corresponding strain equal to 0.25 % and a decreasing linear trend up to 26 MPa with corresponding strain of 0.6%. The concrete contribution for tension stresses is neglected and then its constitutive law has zero strength when the strain assumes positive values.
The reinforcing steel bars are modeled according to Menegotto-Pinto constitutive law (Menegotto and Pinto, 1973). A yield stress equal to 360 MPa is assumed, along with a modulus of elasticity equal to 205000 MPa and a hardening parameter equal to 0.025. The transition parameters from elastic to plastic behavior are set according to Menegotto and Pinto (1973).
The reinforcing bars at the bottom and top zone of the column (dashed zones of Fig. 9a) are modeled by using a constitutive law able to simulate the buckling phenomenon between two consecutive stirrups when the compression strain attains large values. In the literature, the importance of the instability phenomenon of compressed reinforcing bars has been underlined by several researchers (Cosenza and Prota, 2006) (Monti and Nuti 1992). In the finite element model herein used, the constitutive law for steel bars due to Gomes and Appleton (1997) is chosen from the available materials in the Opensees library (Mazzoni et al, 2006).

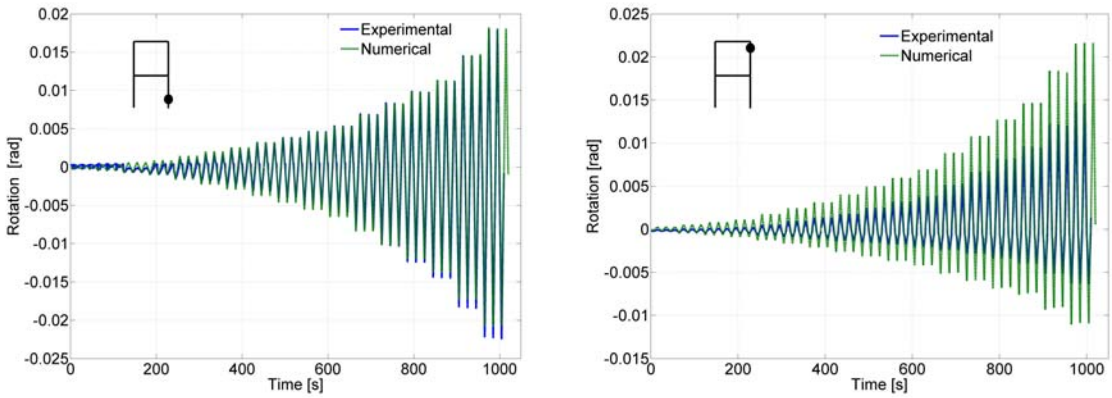
Another aspect to be taken into account is the bond-slip effect in proximity of the bottom and top of the columns. This phenomenon is due to the difference between the deformation of the bars and concrete which yields a typical crack. In the literature, the bond-slip problem and its contribution to the lateral flexibility of structures for horizontal forces have been widely investigated (Zhao and Sritharan, 2007), (Limkatanyu and Spacone, 2003). It is worth to point out that this effect may be pronounced for plain bars due to the low adhesion between concrete and steel. Following the approach proposed by Zhao and Sritharan (2007), a way to account for the bond-slip effect consists of concentrating the rotation due to the slip in a section. This may be done in Opensees by using a *zeroLengthSection* element which has a length equal to 1 with a single integration point. This implies that element deformations correspond to section deformations and then the moment-curvature is equivalent to moment-rotation relation. In this way, the rotation due to bond slip effect may be evaluated by defining a properly stress-slip relation for the steel, describing the interaction between concrete and bar. It is used to model the sections at the top and bottom of the columns (Fig. 9a). The bond-slip model parameters are chosen according to experimental results of pull-out tests as described above.
In order to calibrate the numerical model of the pier, a shear behavior for the weakest reinforced concrete elements must be implemented. It is well known that shear response plays an important role especially for existing structures that do not satisfy seismic engineering design criteria. In the literature, several studies concerning the shear behavior of reinforced concrete beams or walls and their interaction with flexural response are reported and compared with experimental results (Ceresa et al. 2007), (D'Ambrisi and Filippou 1999), (Hildago et al. 2002), (Lee et al. 2005).
Considering these formulations in the literature and the relatively scarce information about experimental results for shear behavior, especially for plain longitudinal bars, a phenomenological shear-strain hysteretic relation for shear behavior of the transverse is assumed. It consists of trilinear envelope curve with stiffness and strength degrading with pinching response which is always observable in the reinforced concrete elements subjected to shear forces. The model is similar to the one proposed by D'Ambrisi and Filippou (1997) and Lee et al. (2005) except for both influence of axial force on shear relation and trilinear assumption for the backbone curve.
The shear relation is implemented by using the option *section Aggregator* which groups different materials behavior into a single section force-deformation model; in this way, the shear and flexural behavior are linked by mean of the equilibrium equations, even though their mechanical formulations are uncoupled. In this case, a uniaxial material is chosen to represent the sectional shear behavior. It is defined through three points of the envelope curve. The first point has the force value $V_c$ equal to 43 kN computed according to the approach proposed by Priestley et al. (1996) which corresponds to the concrete contribution to the overall shear section strength and the deformation $\gamma_y$ which is the corresponding shear deformation equal to $8 \cdot 10^{-4}$. The second point has the force value $V'_c + V_s$ equal to 60 kN and the shear strain $\gamma_u$ equal to 0.00378; $V'_c + V_s$ is computed according to the formula proposed by Priestley et al. (1996) which corresponds to the concrete ($V'_c$) and transversal steel ($V_s$) contribution to the overall shear section strength; $V'_c$ is equal to 26 kN which accounts for degradation of the concrete strength ($V_c$) when the deformation increases. The third point has coordinates $V'_c + V_s$ and $10 \cdot \gamma_u$ which implies that the overall shear force remain constant for the last cycles.
In Figure 10a,b a comparison between experimental and numerical hysteretic cycles is illustrated; it is obtained plotting the global shear force in the columns versus the cap beam displacement. In Figure 10a the first cycles are shown for global displacement ranging from 0 to 0.4 cm; for this displacement range the structural behavior can be assumed as linear, even though the experimental cycles exhibit low energy dissipation. The initial stiffness is overestimated by the numerical model. This result is due to the particular constitutive law chosen for the concrete which does not allow separating the strength and elastic modulus.
In Figure 10b the cycles for global displacement between 3.0 and 6.0 cm are shown; the pinching effect due to the shear element in the transverse beam matches the pinching observed in the experimental results especially for the unloading branch. The global pier resistance for the numerical model matches the experimental results also for the largest displacement range. The total energy dissipation of numerical model is quite similar to energy dissipated during the experiment, even though the reloading part of the hysteretic cycles does not exactly follow the experimental cycles.
In Figure 11a,b a comparison between experimental and numerical local response is reported. Figures 11a depicts the numerical and the experimental rotation at the bottom of the right column. As can be seen the numerical results follows the experimental rotation trend in a very good way. In Figures 11b the numerical and

the experimental rotation at the top of the right column is plotted. A numerical rotation larger than experimental one can be observed. The results are in any case satisfactory.




(a) (b)

Figure 10 (a) Experimental and numerical cyclic response $D_c = 0 - 0.4$ cm. (b) Experimental and numerical cyclic response $D_c = 3.0 - 6.0$ cm.




(a) (b)

Figure 11 Experimental and numerical rotation. (a) Bottom right column. (b) Top right column.

## 5. CONCLUSIONS

In this paper the first results of an experimental campaign on physical models of bridge framed piers have been illustrated and discussed. The test consists of cyclic imposed displacements on a one-bay, two-floor reinforced concrete frames, that is a 1:4 mock-up of a pier belonging to an old viaduct of the Italian highway network. The viaduct, situated on the Firenze- Bologna highway, has been built in the sixties and was realized using reinforced concrete with plain steel bars.

The experimental test has been performed in the laboratory of the Department of Structures of the University of Roma Tre. The experimental results have shown that the shear failure is the most relevant failure mechanism. In particular, the shear failure of the transverse beam at first floor has been observed with a typical cracking pattern. Other less important failure mechanisms have been noticed: bond-slip of the rebars, essentially due to smooth reinforcing steel and buckling on the longitudinal reinforcement due to high stirrups spacing.

On the basis of the experimental results, a numerical model has been built using Opensees code, in order to well simulate the behaviour of the specimen and for further seismic analyses on the pier. The shear mechanism has been modeled using a literature model, including a suitable hysteric law, already implemented in Opensees. In particular, the section aggregator command has been used in such a way that the shear and flexural behavior are linked by mean of the equilibrium equations, even though their mechanical formulations are uncoupled.

Moreover, bond-slip and buckling of the longitudinal rebars have also been included in the model, using literature formulations. The response of the model to cyclically imposed displacements is very satisfactory and matches quite well the experimental response of the physical model. Only the stiffness in the reloading phase is not very accurately reproduced.

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