Nonlinear Mechanical Model of Seismic Behaviour of Beam-Column Pin Connections

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SUMMARY:
The present study is part of an experimental research project whose objective is to quantify the characteristic parameters of the seismic behaviour (in terms of strength, stiffness, ductility, dissipated energy, deformability, degradation, damage) of the dowelled connections commonly adopted in Italian precast structures for the beam-column joints. The monotonic test has been carried out up to the failure of the specimen, in order to measure the entire load-displacement curve ("skeleton" curve) and to identify the various displacement levels characterizing the following cyclic tests. The results obtained by test are then compared with the scanty bibliographical data. A nonlinear mechanical model of seismic behaviour of the connection is also proposed in order to implement the same one in partial and global numerical models.
The results of the experimental campaign may have a direct industrial relevance, since they will be used for the development of guidelines for a reliable seismic design of precast structures.

Keywords: Precast structures, beam-column connection, experimental tests, nonlinear mechanical model

1. INTRODUCTION

The hierarchy of resistance is one of the innovative principles of the most recent seismic codes, as also shown by the Italian seismic codes, the OPCM 3431 (Presidenza del Consiglio dei Ministri, 2005) and the Norme Tecniche per le Costruzioni (Ministero delle Infrastrutture, 2008). The aim is to guarantee the development of ductile mechanisms capable to dissipate energy and to avoid any fragile mechanism leading to sudden structural collapses.
The particular nature of cast-in-situ frame structures conditioned the development of consolidated design rules, that are an appropriate answer to the afore mentioned necessities. However, reinforced - concrete precast structures are a different case, since their specific nature makes them closer to metallic structures (both R/C precast and metallic structures require the assembling of different components).
In addition to the general indications concerning single elements and floors, the new Italian design provisions (Ministero delle Infrastrutture, 2008), according to European ones (CEN, 2003), also deal with the connections, which play a central role in seismic design. The following connection typologies are considered: connections placed far from the sections where the plastic demand is likely to be high and connections placed in such zones; in the latter case, the connection should be designed in agreement with either an overstrength criterion (with respect to the nearby zones) or a high- ductility criterion.
It is evident that the application of the strength-hierarchy criterion to precast industrial buildings requires the knowledge of both the monotonic and cyclic behaviour of the connections. However, there are still scanty experimental information in this field, with specific reference to the connections commonly used in Italy.
Obviously, studying beam-column connections is quite critical, because these connections joint major structural members, that guarantee seismic strength. Italian typologies generally consist in connections made of a rubber support and steel dowels (Fig. 1), embedded both in the beam and in the column;

their main function is the absorption of the horizontal forces, in order to avoid any differential displacement between the connected elements. Nowadays, in Italy, the connection resistance is computed by means of simplified formulae (CNR, 1984), which do not consider many parameters, such as - for example - the cover, the direction of the action and the distance between dowels.

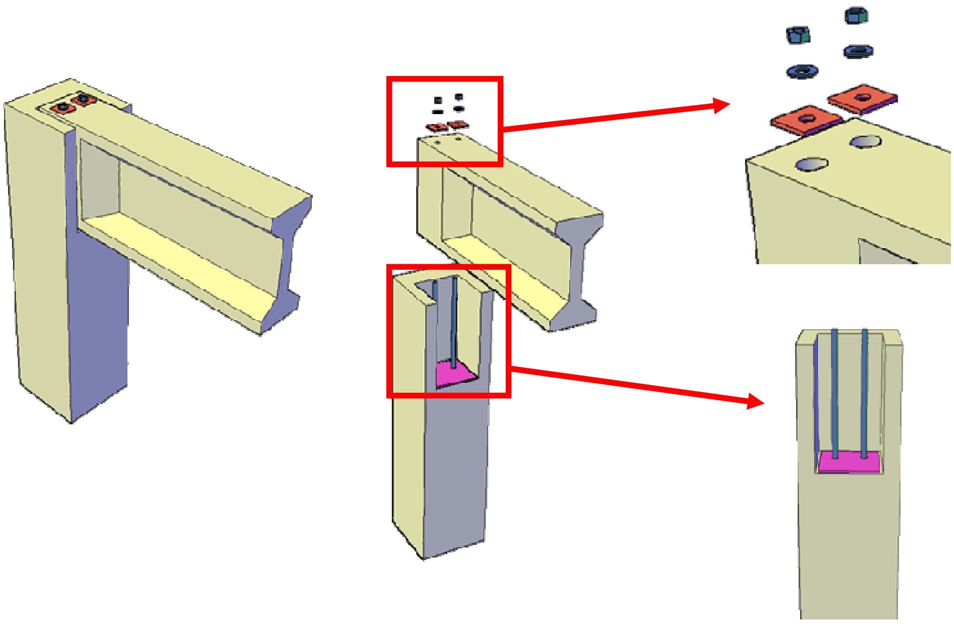


Figure 1. Beam-column dowelled connections

Several studies in the past in order to define the shear resistance of connection between concrete precast elements (starting by Rasmussen, 1963 passing by Dulácska, 1972 and Engström, 1990 and ending with Vintzeleou and Tassios, 1987, Soroushian et al, 1987a, 1987b, 1989, Tsoukantas and Tassios, 1989, Dei Poli et al, 1992, 1993) are carried out with several formulations. They define very well the shear transfer mechanism considering the submechanisms mobilized by slippage, friction and dowel action. A very efficient resume of this study is present in fib Bulletin 43 (2008) but always related to the only strength parameter. Furthermore, connection ductility, stiffness and hysteretic behaviour, necessary to completely mechanically define the connection, are still highly debated.

Considering the available literature, most of the recently papers are aimed to improve the understanding of the seismic response of (a) monolithic precast beam-column connections (i.e. Ersoy and Tankut, 1993, Restrepo et al, 1995, Alcocer et al, 2002, Khaloo and Parastesh, 2003, Blandón and Rodríguez, 2005), or (b) innovative connections seldom used in Italy (i.e. Pampanin, 2005). Only few paper are on pinned connection (El Debs et al, 2010, Dotreppe et al, 2006, Ferreira and El Debs, 2000) but they do not completely define the mechanical behaviour of the connection subjected to shear or bending action.

Hence, specific design-oriented studies on beam-column connections based on dowels and rubber layers, subjected to seismic excitation, are hardly available. On this topic another European project is starting entirely devoted to the connections in precast buildings underlining the importance of this aspect. The project is entitled “Performance of Innovative Mechanical Connections in Precast Building Structures under Seismic Conditions” (acronym: SAFECAST) and runs in framework of the Seventh Framework Programme (FP7) (Kramar et al, 2010).

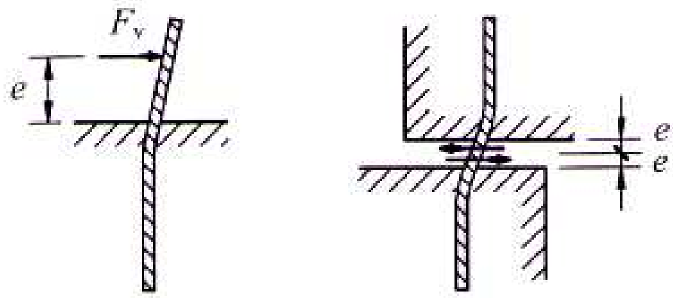
For this reasons, the main objective of our research project is the experimental characterization of some typologies of the dowelled beam – column connections often found in both one-storey and multi-storey precast buildings. Shear and bending tests under both monotonic and cyclic loading have been planned as presented in Capozzi et al (2010). The experimental information will be instrumental in developing suitable numerical procedures for the analysis of complex structural systems and in providing design guidelines, according to the strength-hierarchy criterion.

The Italian association of the precast industry (ASSOBETON) is supporting the experimental

campaign by supplying the components and sub-assemblages.
In this paper, monotonic shear test results along the longitudinal beam axis direction, performed at the laboratory of Department of Structural Engineering of University of Naples, on beam-column pin connections are reported; specimen characteristics, setup, test procedures and first results are describes in details also critically considering the bibliographical data. A nonlinear mechanical model of the investigated connection subjected to shear action is proposed for design purpose. This is relevant considering that research is performed in order to asses the seismic vulnerability of the most spread Italian beam-column connection in precast buildings.

2. ON THE SHEAR DESIGN STRENGTH OF DOWELLED CONNECTIONS

Shear forces can be transferred between concrete elements by adhesion or friction at joint interfaces (due to external compression or pull out forces), shear-key effect at indented joint faces, dowel action of transverse steel bars, pins and bolts, or by other mechanical connections devices. The typical Italian precast beam - column connection, as previously said, is made of a rubber support and steel dowels (Fig. 1), embedded both in the beam and in the column and then the transmission of forces between the beam and the column is realized by means of two components: (1) neoprene bearing pad; (2) dowels.
In the shear mechanism, the neoprene component has an important role at the connection ultimate condition.
Dowel action is the fundamental mechanism in transfer of shear force in a beam-column pin connection (Fig. 2). The dowel is loaded by shear in front of the joint and its condition normally results in considerable flexural deformations and flexural stresses in the dowel. Depending on the strength and dimensions of the steel and the position of the bar relative to the element boundaries, three main failures modes are possible: (1) steel shear failure; (2) concrete splitting failure; (3) combined steel and concrete failure.



(a) (b)

Figure 2. Shear transfer by dowel action in bolt, pin or bar: (a) one-sided dowel; (b) double-sided dowel (Fib, 2008)

The first failure mode happens when a weak bar is in a strong concrete element and then might fail in shear of bar itself. The shear design capacity of steel bar loaded in pure shear can be estimated through the yield criterion by Von Mises. This typology of collapse is of little importance because for normal condition the last two mechanisms happen before than the first one.
The second failure can be defined “strong mechanism” while the third one “weak mechanism”. In the “strong mechanism”, a collapse mechanism by formation of one or more plastic hinges in the dowel is developed; simultaneously, local crashing occurs in the surrounding concrete where the contact pressure is high. Through the dowel, high concentrated forced are introduced in the concrete where the dowel is placed and considerable tensile stresses may appear in the area around the dowel. This resistant mechanism is, then, ductile. Instead, when the dimensions of the concrete element are small, or the dowel is placed near the free edges of the element, splitting cracks may appear even for small shear forces. They can cause premature brittle failures that limit the shear strength of the connection.

This behaviour defines the "weak mechanism". Clearly the cover, and then the distance to free edges, defines the transition by one to other resistance mechanism. In Tsoukantas and Tassios (1989) and fib (2008) the various mechanisms and their physical behaviour are explained very well.

2.1. Failure Mode I: Strong Mechanism (yielding of the bar and crushing of the concrete)

Several studies have been devoted to understand the dowel action in order to characterize the resistance and the behaviour of a bar anchored at both sides (Fig. 2b). The first model, used to study the strong mechanism, was proposed by Rasmussen (1963). He analysed the phenomenon adopting theory by plasticity. In fact, because concrete and steel reach a plastic behaviour, the state of equilibrium and the resistance in shear of connection can be analysed by considering the dowel action as a pile in a Winkler's material. He derived his theory under the hypothesis of no shear load eccentricity. All other formulations, developed in the following, derive from this one.
The Rasmussen's equation, after also adopted by CNR, is as follows:

$$V_{Rd} = \alpha_0 \cdot d_b^{\,2} \cdot \sqrt{f_y \cdot f_{cc}} \tag{2.1}$$

where α_0 is equal to 1.2, d_b is the bar diameter, f_{cc} is the design compressive strength of concrete under uniaxial compression and f_y is the dowel bar yielding stress. Eq.(2.1) is only valid for any shear-force eccentricity smaller than 0.5 d_b.
Tsoukantas and Tassios (1989) and Vintzeleou and Tassios (1987), based on the same model, assume α_0 equal to 1.3. These last two papers are very interesting on the topic. They generalized the dowel action problem also identifying, for the first time, the weak mechanism and the border with the strong one. In the first paper, authors provide an equation for computing the shear strength, for strong mechanism, when the load is eccentric respect to the joint face:

$$V_{Rd}^2 + \left(10 \cdot f_{cc} \cdot d_b \cdot e\right) \cdot V_{Rd} - \alpha_{0c}^{\,2} \cdot d_b^4 \cdot f_{cc} \cdot f_y \cdot \left(1 - \zeta^2\right) = 0 \tag{2.2}$$

where e is the load eccentricity (Fig. 2). The eccentricity, in a beam-column pin connection, is due to the neoprene packet thickness t. The eccentricity, in this case, is equal to t/2. α_{0c} is a factor ($\leq$1.3) depending on the available concrete cover of the bar in the direction of the shear force.
$\zeta = \sigma_s/f_y$ is used to calculate the decrease of the dowel response in the case of bar simultaneously subjected to an axial stress σ_s due to other actions.
According to the authors' suggestions, the design dowel force under cyclic loading in case of failure for strong mode V_{Rdc} can be taken approximately to half the dowel values under monotonic loading V_{Rdm}.
Soroushian et al (1987a) also provide an Equation to define the shear strength of dowel bars:

$$V_{Rd} = 0.5 \cdot f_b \cdot \left(0.37 \cdot \gamma \cdot d_b - c'\right)^2 + 0.45 \cdot f_y \cdot d_b^2 \cdot \left(1 - T^2/T_y^2\right)/\gamma \tag{2.3}$$

where $\gamma = \sqrt[4]{E_s / K_f \cdot d_b}$, K_f is the concrete foundation modulus ($\equiv$ 271.7 MPa/mm), $f_b = 37.6 \cdot \left(\sqrt{f_{cc}} / \sqrt[3]{d_b}\right)$ is the concrete bearing strength, $c' = 0.05 \cdot \dfrac{f_y \cdot d_b}{f_{cc}} \cdot \sin\alpha$ is the length of the crushed concrete zone and takes into account the inclination α of the bar, T is the dowel bar axial force, T_y is the dowel bar yield axial force.

2.2. Failure Mode II: weak mechanism (concrete splitting)

According to Vintzeleou and Tassios (1987), for concrete cover smaller than 1/6-1/8 of the bar diameter, the dowel mechanism fails by concrete splitting (weak mechanism). Side or bottom splitting may occur, depending on the side-to- cover ratio. The formulations proposed by authors are based on equilibrium of system forces in cracked reinforced concrete.
For large values of the side-to-cover ratio, bottom splitting occurs and the dowel capacity has the following expression:

$$V_{Rd} = 5 \cdot d_b \cdot c_2 \cdot f_{ct} \cdot \frac{c_2}{0.66 \cdot c_2 + d_b} \tag{2.4}$$

where c_2 is the concrete cover in the force direction and f_{ct} is the concrete tensile strength.
For small values of the side-to-cover ratio, side splitting occurs and the dowel capacity has the following expression:

$$V_{Rd} = 2 \cdot d_b \cdot b_{ct} \cdot f_{ct} \tag{2.5}$$

where:
b_{ct} is the net width of the section (b-d_b) being b the beam width.
Also Soroushian et al (1987b) provide an equation to calculate the strength of the connection in case of weak mechanism:

$$V_{Rd} = 0.83 \cdot \psi \cdot b_{ct} \cdot f_{ct} \tag{2.6}$$

where:
$\psi = \pi \Big/ 2 \cdot \sqrt[4]{\dfrac{K_f \cdot d_b}{4 \cdot E_s \cdot I_s}}$ is the distance from the crack face to the inflection point (see Fig. 12b (Souroushian et al, 1987b)).

3. SHEAR TEST ON CONNECTIONS WITHOUT WORKING SLAB

The tested specimen (Fig. 3) is characterized by three principal components: two vertical blocks (representing the columns; section size 60 cm x 60 cm; height 100 cm) and an inverse-T extremity for restraining the blocks to the strong floor; an horizontal member, representing the beam (section size 60 cm x 60 cm; length 210 cm). The beam-column connection is placed at the left block (two M27 class 8.8 dowels and one 10 mm-thick neoprene support (15 cm x 60 cm)). The dowels' dimensions are typical of the first Italian seismic zone (PGA 0.35 g) according to the OPCM 3431 (Presidenza del Consiglio dei Ministri, 2005). The right-end side column acts as a mere support to the beam; two teflon layers are placed on it, in order to avoid any friction. The dowels are distant 100 mm and 150 mm from the side (c_1=3.7 Φ) and the bottom cover (c_2=5.55 Φ) respectively. The concrete covers are typical of technical practice; according to Vintzeleou and Tassios (1987), then, being concrete cover smaller than 1/6-1/8 of the bar diameter, the dowel mechanism should fail by concrete splitting (weak mechanism).
The mechanical characteristics of the concrete used in prototypes were determined at the same day of the tests, through compression tests on cubic specimens. The characteristic compressive cylinder concrete strength is resulted equal to 34 MPa.
The beam is loaded by means of a shear force provided by a horizontal hydraulic and by a vertical force provided by a vertical jack restrained to a prestressed metallic beam, that crosses the beam to be tested through a special hole; the vertical load is fixed during the test at 450 kN. A sleigh anchorage system is placed at the other side of the metallic prestressed beam, in order to avoid undesirable restraining effects. The vertical force simulates the weight of a real beam in seismic condition; it activates the neoprene – concrete friction force, which above all conditions the ultimate seismic strength of the connection.
The shear force is increased monotonously increasing the displacement with a low speed equal to 0.02 mm/s (shear loading rate). The test is executed with dowel action due to the bar pushing against the concrete cover.
Two LVDT transducers are placed horizontally at the beam-end cross sections, at different heights, in order to evaluate any displacements and rotations of beam ends.
The mechanical curve obtained by monotonic tests and damage patterns are already presented in detail and critically discussed in Capozzi et al (2010). In the following paragraph a preliminary analytical model for weak mechanism based on executed test is proposed.

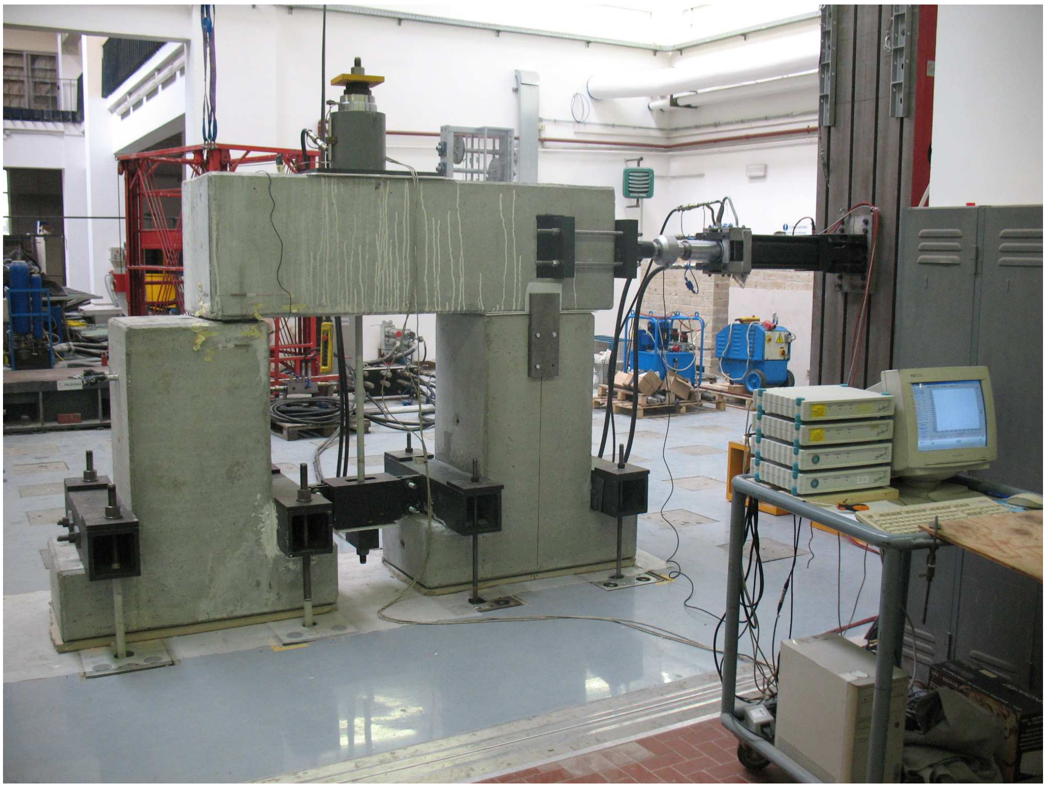


Figure 3. Set-up of the shear tests

4. ANALYTICAL NONLINEAR MODEL FOR WEAK MECHANISM

In this paragraph, a numerical model of the beam-column pin connection for weak mechanism is presented. As said, in dowelled connection the transmission of forces between the beam and the column is realized by means of two components: (1) neoprene supporting pad; (2) dowels. This model considers the two components and their interaction inside the mechanical behaviour of the pin beam-column connection; the system strength is given by the sum of neoprene and dowel components. The model derives from eqn. 2.5 of Vintzeleou and Tassios theory and extends considering phenomena typical of pin beam-column connection in precast structures.
In Figure 4, the cracks on columns are reported. We observe the formation of two concrete cones near the two dowels with a crack inclined of about 23°. Under the beam an inclination of about 35° is observed. The Etag001 (2008), that prescribes the rules for metal anchors for use in concrete, defines an inclination of cracks equal to 33°.

Figure 4. Crack on the top column surface

Their development is very important on the strength calculation of the connection. Indeed, the dimension b_{ct} (eqn. 2.5), has to be calculated considering the length of the cracks and not equal, as assumed in the Vintzeleou & Tassios (1987) and Soroushian et al. (1987b), to the net width of the section. The authors, in fact, assumed, in their papers, this value for the specific case but in general it has to be assumed equal to the length of the crack. By experimental results, the following relationship to calculate the maximum dowel shear strength (Point B in Figure 5), for weak mechanism interesting the side-cover is proposed modifying the Equation 2.5 (proposed by Vintzeleou and Tassios, 1987):

$$V_{RddB} = 2 \cdot d_b \cdot b_{cl} \cdot \left(\beta_1 \cdot f_{ctu} + \beta_2 \cdot f_{ctt} \right) \tag{4.1}$$

where d_b is the bar diameter, b_{cl} is the length of the expected cracks obtained as before explained, β1 and β2 are two parameters that define the regions where there are respectively a tri-axial or uniaxial state of tensile stress. Indeed, for concrete subjected to high bearing stresses under a local loading area, a tri-axial state of stress is obtained. For such a case compressive stresses can reach values that are several times the uniaxial concrete compressive strength.
In this case, β_1 is equal to 0,10 and β_2 = 0,90 and then the linear development of the "tri-axial" region ($\beta_1 \cdot b_{cl}$) for each dowel results to be equal to about 1.25 d_b; clearly the tri-axial region decreases decreasing the bar diameter.
f_{ctu} and f_{ctt} are respectively the concrete uniaxial and tri-axial tensile strength. Clearly for design purpose, the design value of strengths had to be considered.
The connection strength is clearly obtained summing to this value the friction strength V_{Rdn}. This last value can be evaluated according to Magliulo et al (2008) equations.

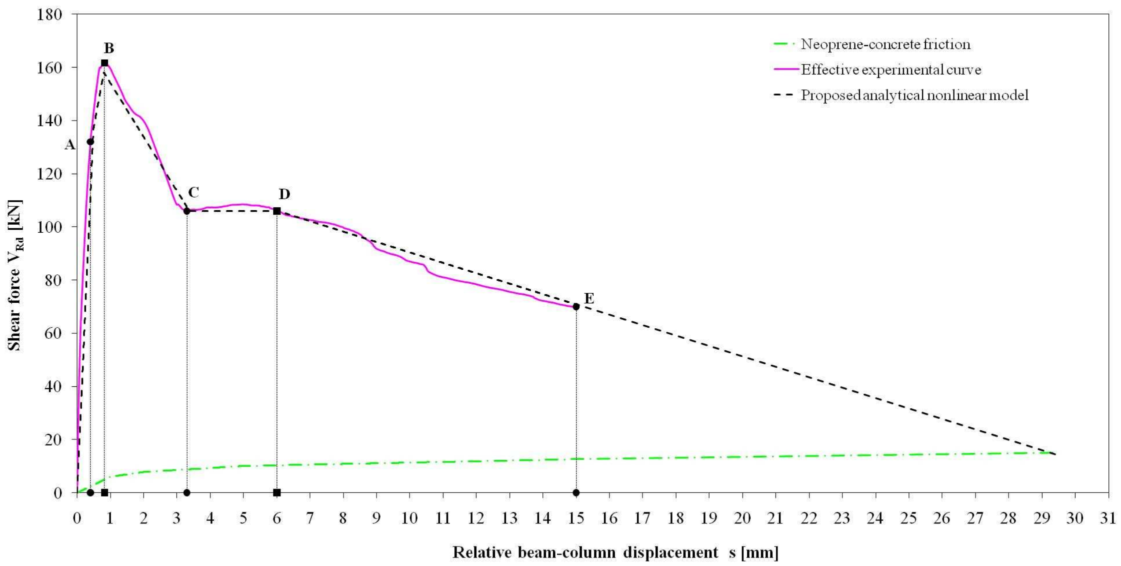


Figure 5. Comparison between the experimental results and the proposed analytical nonlinear model

The point A of experimental curve, representing the lost of the linear behavior, can be calculated according to the curve proposed by Tsoukantas and Tassios (1989) for the strong mechanism and then:

$$V_{RddA} = 0.5 \cdot V_{RddBs} \tag{4.2}$$

where V_{RddBs} represents, for strong mechanism, the dowel strength corresponding to the formation of the second hinge (eqn. 2.2).
The elastic stiffness k_{0A} can be obtained summing the dowel stiffness to neoprene one.
The elastic dowel stiffness can be easily calculated considering a scheme of beam with length equal to l_p with fixed ends with unitary settlement:

$$k_{d0A} = \frac{12 \cdot E_s \cdot I_s}{l_p - d_b} \tag{4.3}$$

where l_p is the distance between the two plastic hinge in the dowel ($l_p = 3{,}3 \cdot d_b$), d_b is the bar

diameter, E_s and I_s are Young's modulus of elasticity of the bar and I_s is, instead, the inertia moment of the bars.
The post-elastic stiffness k_{AB} can be expressed as:

$$k_{AB} = 0.25 \cdot k_{0A} \tag{4.4}$$

This value derives, instead, considering the same scheme but with an hinge and a fixed end.
The softening branch BC is defined by the post-capping stiffness, $k_{BC} = \alpha_{BC}\ k_{OA}$, which has a negative value.

$$k_{BC} = -0.075 \cdot k_{0A} \tag{4.5}$$

In the range CD there is a perfectly plastic behaviour with constant strength. This behaviour is given by the birth of the first plastic hinge in the two dowels calculated by BEF's equation proposed for strong mechanism (Vintzeleou & Tassios, 1987).
Knowing the position of the plastic hinge, it is possible to justify the strength registered in branch CD. Indeed, the connection, in this phase, can be schematized as represented in Figure 6 and imposing the equilibrium of forces, the strength of dowels can be obtained as:

$$V_{RddC-D} = 2 \cdot \frac{M_{pl}}{x_0 + e} \tag{4.6}$$

where $M_{pl} = \frac{f_{sy} \cdot d_b^3}{6}$ is the plastic bending moment of the single dowel and (x_0+e) is the distance between the plastic hinge and the shear force V_{Ed}.

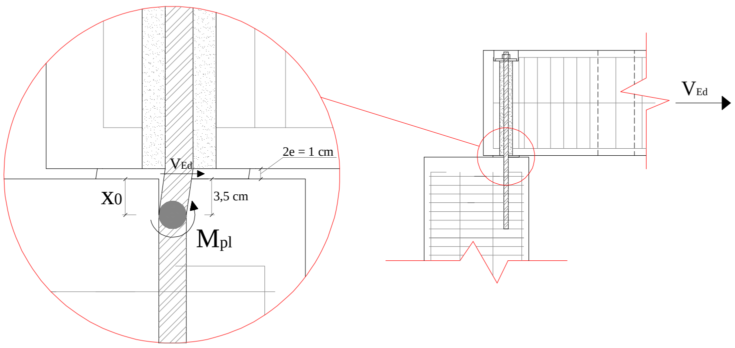


Figure 6. Plastic hinge modelling in the CD branch

The softening branch DE is defined by the post-capping stiffness, $k_{DE} = \alpha_{DE}\ k_{OA}$, which has a negative value:

$$k_{DE} = -0.0150 \cdot k_{0A} \tag{4.7}$$

The test is interrupted at beam-column relative displacement equal to 1.5 cm, in order to avoid the possible lost of the beam support (point E). This displacement represents an admissible displacement depending on the connection geometry. In general the connection strength, after the dowel collapse, tends towards the maximum friction strength obtained by Magliulo et al. [35] at slip of about 3t being t the neoprene thickness. In Figure 5 the comparison between the experimental results and the proposed monotonic nonlinear model is also represented. We can note the almost perfect overlapping between the two curve underlining the excellent modeling of monotonic behaviour. Also cyclic tests are in execution on dowelled connection in order to define: (1) as the mechanical parameters change passing by monotonic behaviour to cyclic one; (2) the hysteretic behaviour of the connection with the dissipated energy.
Other monotonic tests and simulations with FEM program are programmed varying the mechanical and geometric characteristics of the connection in order to define a generic model for monotonic and cyclic behaviour.

5. CONCLUSIONS

Current literature gives very limited information on the monotonic and cyclic behaviour of dowelled beam-column connections, that are extensively used in Italy by the precast industry and that should exhibit a favourable hysteretic behaviour in order to resist high seismic loads.

The lack of experimental and numerical knowledge concerning this connection is the driving force behind this mainly experimental project, that is in progress under the auspices of the Italian Precast Industry Association (ASSOBETON).

In this paper, the preliminary results obtained by monotonic shear test are presented and critically discussed.

A nonlinear analytical model for weak mechanism is presented that well approximates the experimental results of executed test. The obtained mechanical curve defines not only strength but also stiffness until the collapse. It may be used for computational purposes; as an example, it can be used when performing nonlinear static seismic analysis aimed at assessing seismic performance of new and existing precast buildings based on this typology of connection. Also cyclic tests are in execution on this connection in order to define also its hysteretic behaviour and the limit between weak and strong mechanism.

Other tests and simulations with FEM program are programmed varying the mechanical and geometric characteristics of the connection in order to generalize the proposed modelling and to extend to the cyclic behaviour.

The acquired knowledge can be used in order to develop numerical procedures for the analysis of complex structural organisms and to provide guidelines for the design of such connections according to the hierarchy of strength principle.

At the moment the planning of all tests and the design of the experimental set-ups is over and the cyclic shear tests are in execution.

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