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BEHAVIOUR OF A BEAM TO COLUMN "DRY" JOINT FOR PRECAST CONCRETE ELEMENTS

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ABSTRACT :

In Italy, precast concrete structures are traditionally designed as moment resisting frames with plastic hinges occurring at the column base and beams hinged to the columns. A ductile moment resisting connection between the column and the beam can provide the advantage of designing a seismic resisting frame which can develop plastic hinges at the beam-column joints, besides those at the column base. This paper aims at presenting the results of experimental tests concerning the cyclic behaviour of a particular beam-column "dry" connection for precast concrete elements. The joint is characterized by the use of high strength steel bars and of a fibre reinforced grout pad in the "Z" shaped beam-column interface, increasing the shear resistance of the connection. The experimental results show a good performance of the joint, in term of resistance, ductility and energy dissipation, with little damage observed in the connected members.

KEYWORDS: Precast Concrete Structures, Beam-Column Joints, Test Results

1. INTRODUCTION

In Italy, precast concrete structures are traditionally designed as moment resisting frames with plastic hinges occurring at the column base and beams hinged to the columns. A ductile moment resisting connection between the column and the beam can provide, with respect to a simply supported precast beam, the advantage of designing continuous beams with a reduced beam depth, or with an increase of either span length or carried load. Furthermore a seismic resisting frame can develop plastic hinges at the beam-column joints, in addition to those at the column base.

Since the seventies, it is possible to find in the literature a lot of researches and examples of precast structures emulating the behaviour of reinforced concrete cast in-situ seismic resistant frames [1, 2]. In Figure 1 an example of an equivalent monolithic moment resisting structure is shown: the cruciform prefabricated beam-column elements are joined one to each other in the middle of the beam length, so that the plastic hinges can develop at the beam ends. In this solution, according to the capacity design criteria, the beam joints located in the middle of the beam are stronger than the plastic hinges which form at the beam ends. Other equivalent monolithic solutions need casting in place of concrete to develop a joint between the precast beams and the column and the need to ensure the reinforcement anchorage of the jointed elements for moment transmission causes an unavoidable joint reinforcement congestion (Figure 1.(b)).

The main feature of the monolithic equivalent solution is that plastics hinge form at the beam ends. The hysteretic behaviour is equivalent to that developed in cast in-situ concrete structures due to the ductility and energy dissipation allowed by the yielding of the longitudinal mild steel reinforcement. An irreversible damage of the jointed elements is the unavoidable consequence.

In Japan and in New Zealand, several prefabricated buildings have been built since the sixties adopting post-tensioned dry joints. They showed a good seismic behaviour due to limited residual structure deformations, which allowed immediate occupancy of the buildings after a seismic event. The post-tensioned tendons act as elastic springs re-centering the joint and in this way resetting the initial conditions existing before the seismic event. On the other hand, the dissipative properties of such joints are limited [1, 2].

Since the middle of the nineties, on the basis of the Performance-Based Design philosophy which considers the maximum allowable drift of the structure the starting point of the design procedure [3, 4], hybrid connection systems have been developed allowing an energy dissipation through yielding of mild steel, as well as a self-centring capacity due to a reduction in residual displacement of the structure [5, 6, 7]. In the hybrid solutions, the beams are jointed to the column by ordinary longitudinal reinforcement, dissipating the energy, and post-tensioned unbonded tendons allowing a controlled rocking of the beam (Figure 2). The typical effect is a "flag-shaped" hysteretic behaviour (Figure 2.(b)), with a reduction of the damage of the jointed elements due to opening and closing of the pre-existing gap at the beam-column interface. An innovative solution proposed in [8] also suggests the utilization of special external dissipation devices to be positioned on the lateral joint surfaces. A well-known example of utilization of hybrid joints with post-tensioned tendons is the prefabricated Paramount 39-storey building in San Francisco, California [9].

(a) (b)

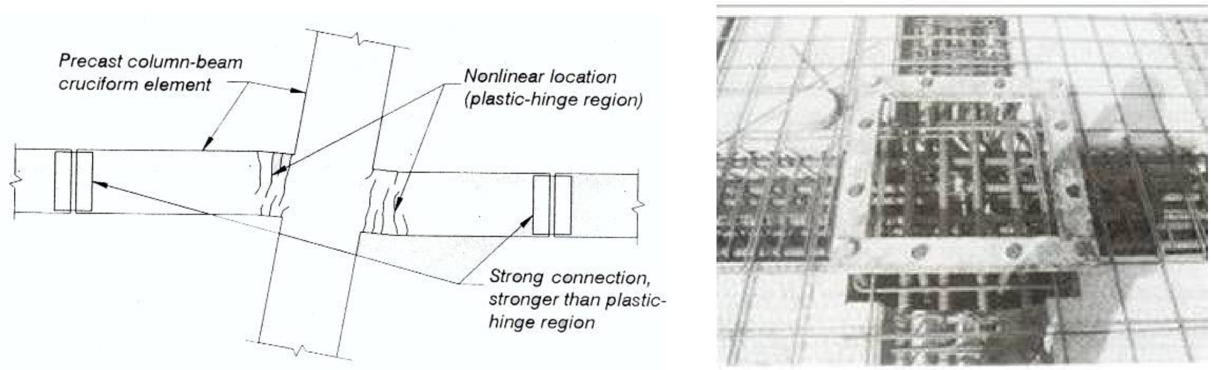


Figure 1 Example of equivalent monolithic system for precast moment resisting frames (a); Structure realized in Japan: detail showing the joint extrados after the floor positioning [2]

(a) (b)

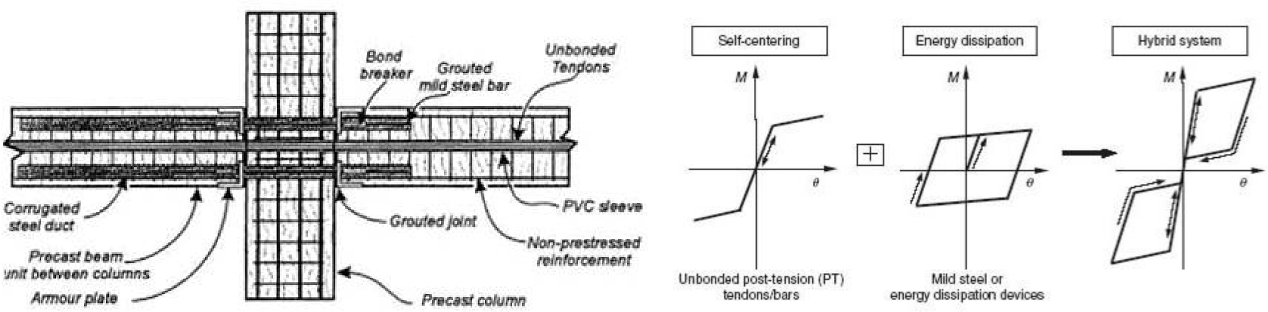


Figure 2 Hybrid joint with post-tensioned unbonded tendons and mild steel bars (a), and its "flag-shape" hysteretic behaviour (b)[2]

This paper aims at presenting the first results of an experimental research concerning the cyclic behaviour of a particular beam-column "dry" connection for precast concrete elements. The column and the beam are jointed together by the use of high strength threaded steel bars and of fibre reinforced concrete, grouted in a "Z" shaped beam-column interface, thus increasing the shear resistance of the connection. This research is therefore aimed at developing a ductile beam-column joint able to absorb a high "negative" moment (upper tensioned fibres) in case of earthquake. The joint damage might be localized at the beam-column interface due to both the unbonded bar elongation and the opening of the pre-existing crack at the beam-column interface. This feature might reduce the inelastic mechanism and damage in the connected members, when compared to the emulated cast in place connection, and might allow an easier member repair after a seismic event. The joint is also designed to carry a positive moment only due to seismic effects, which is approximately equal to 50 % of the maximum negative moment.
This work or is part of a wider research program started at University of Brescia and University of Bergamo (Italy), concerning the seismic behaviour of prefabricated reinforced concrete structures: numerical analyses of beam-column joints [10], experimental researches regarding innovative column-foundation joints [11, 12] as

well as both experimental and numerical researches concerning an interaction between cladding panels and structure have been carried out [13, 14].

2. THE JOINT SYSTEM

The joint system consists of the following components (Figure 3.(a)): a “Z” shaped steel plate (1) with four upper and two lower bushes to connect the threaded bars to the column; the plate, which is embedded in the column, is equipped with two pockets (2) in order to insert a supporting bracket consisting of two steel plates (25x135mm section) (3) carrying the slab self weight; (4) a “Z” shaped steel plate embedded in the beam; (5) four upper threaded M24 bars and (6) two lower threaded B7 high strength steel M24 bars transfering the bending moment in the joint; four upper steel dowels (7) with threaded hole, welded to the column-side “Z” shaped plate, allow the connection between the longitudinal upper bars and the column, while the lower bars are positioned in 50mm diameter metal corrugated sheathes (8) inclined of 0.5 %; during the assembling operation, both bars are partially taken off from the beam sleeves and screwed into the bushes (9) welded to the column pocket; the fibre-reinforced concrete grouting of ducts allows the bar bond to the beam. Two lateral venting tubes (10) are provided to ensure that the corrugated sheathes have been completely filled with grout; plastic ducts are used to deactivate the bond of both the four upper and the two lower bars for a length of 20 cm and 10 cm, respectively, in order to provide adequate ductility and dissipative capacity of the joint in case of cyclic actions. Two vertical M24 (11) bars go through the beam head and join the beam to the lower bracket, thus providing the torsional stability of the beam during the assembly of the floor. The shear resistance of the connection is provided by a fibre-reinforced grout at the beam-column interface (12); thanks to the particular “Z” shape of these plates, the grout improves the shearing strength of the joint as well as accommodates construction tolerances. Figure 3.(b) shows the assembled joint and the joint during the grouting operation.

(a) (b) (c)

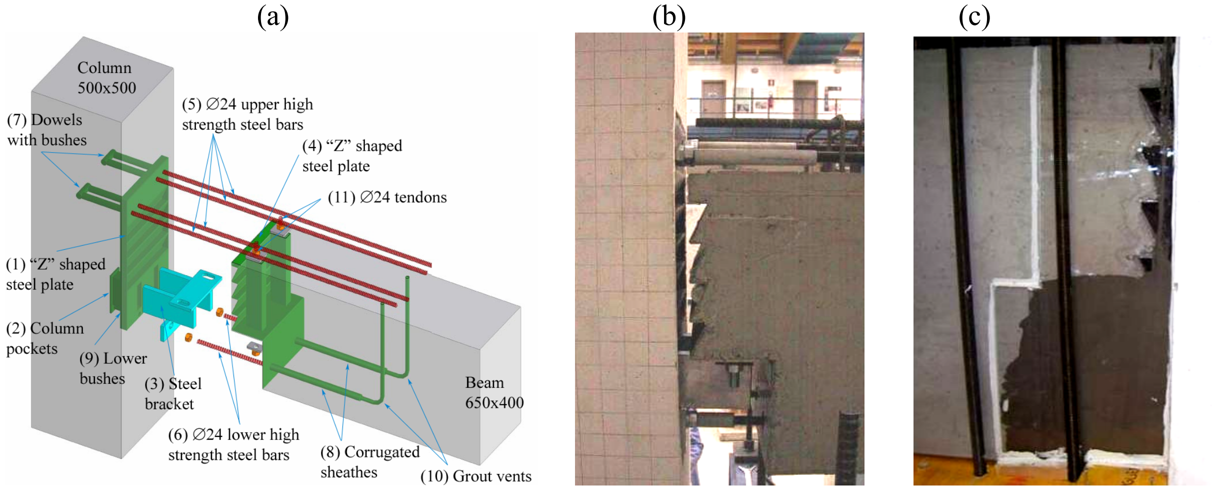
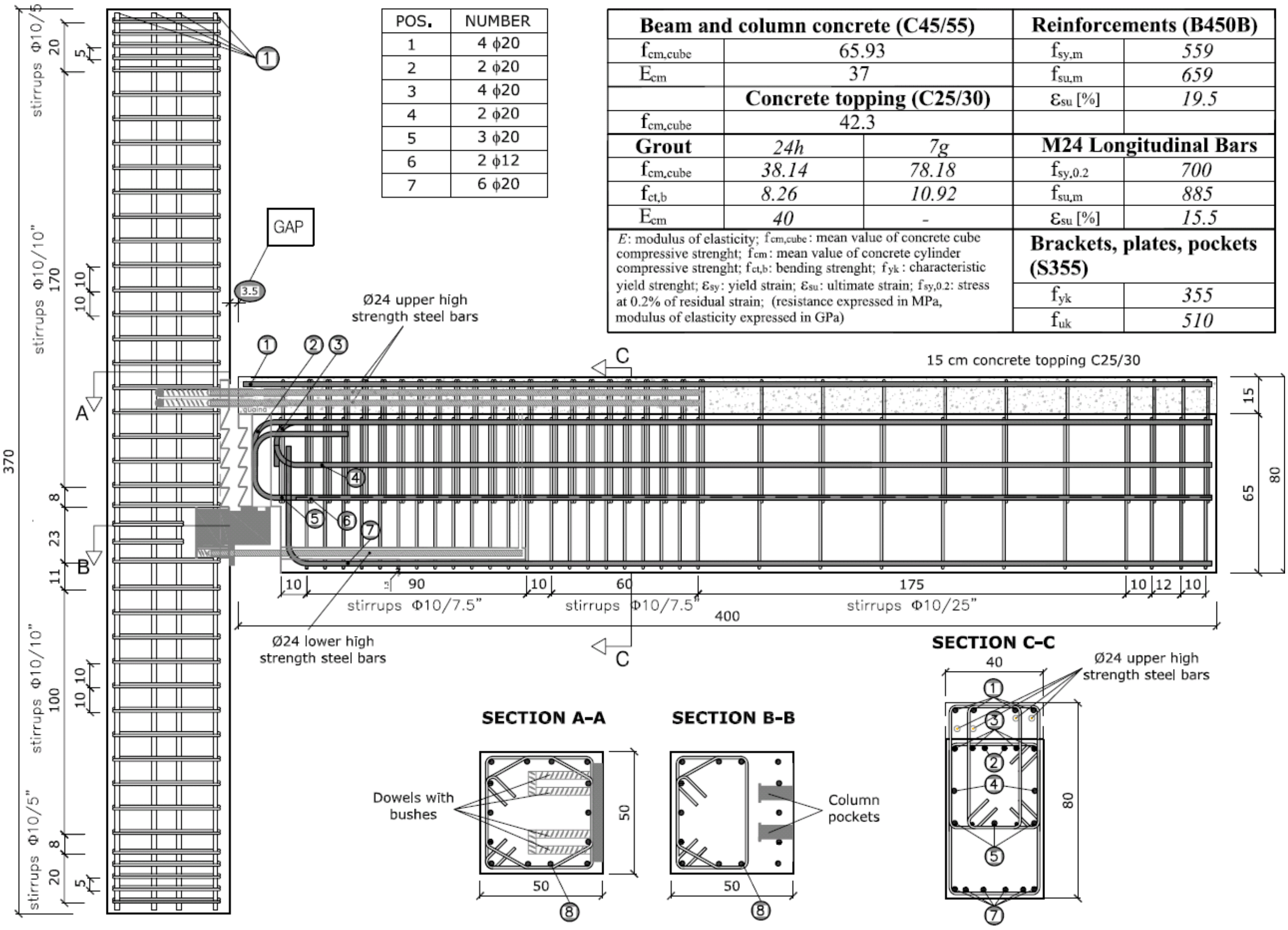


Figure 3 Edilmatic system components for beam-column joints in precast reinforced concrete structures (a); the assembled joint (b) and during the grouting operation (c)

3. TEST SET UP

The tested specimen represents a typical outer joint of a frame in a precast multi-storey reinforced concrete building. The column has a 50x50cm section and a 3.7m height, while the beam has a 4.0m length and a section equal to 40x80cm, including 15 cm concrete topping, where the upper joint reinforcements are embedded (Figure 4). The main mechanical properties of steel, concrete and grout are reported in Figure 4.



POS.	NUMBER
1	4 ϕ20
2	2 ϕ20
3	4 ϕ20
4	2 ϕ20
5	3 ϕ20
6	2 ϕ12
7	6 ϕ20

Beam and column concrete (C45/55)			**Reinforcements (B450B)**	
$f_{cm,cube}$	65.93		$f_{sy,m}$	*559*
E_{cm}	37		$f_{su,m}$	*659*
	Concrete topping (C25/30)		ε_{su} [%]	*19.5*
$f_{cm,cube}$	42.3			
Grout	*24h*	*7g*	**M24 Longitudinal Bars**	
$f_{cm,cube}$	*38.14*	*78.18*	$f_{sy,0.2}$	*700*
$f_{ct,b}$	*8.26*	*10.92*	$f_{su,m}$	*885*
E_{cm}	*40*	-	ε_{su} [%]	*15.5*
E: modulus of elasticity; $f_{cm,cube}$: mean value of concrete cube compressive strenght; f_{cm}: mean value of concrete cylinder compressive strenght; $f_{ct,b}$: bending strenght; f_{yk}: characteristic yield strenght; ε_{sy}: yield strain; ε_{su}: ultimate strain; $f_{sy,0.2}$: stress at 0.2% of residual strain; (resistance expressed in MPa, modulus of elasticity expressed in GPa)			**Brackets, plates, pockets (S355)**	
			f_{yk}	*355*
			f_{uk}	*510*

Figure 4 Tested specimens (dimensions expressed in cm)

The beam to column joint has been tested by fitting a reaction frame available at the P. Pisa Laboratory of the University of Brescia [11, 12] (Figure 5 (a)). The bench is self-balanced and consists of two reinforced precast concrete ring-shaped reaction frames designed to discharge whole bench weight to the ground, to lift the bench for assembling the joint to be tested and to allow an adequate resistance to overturning. The bench has properly been modified in order to test a beam-column joint with a hinge constraint at the base of the column as well as a roller constraint at the top of the column and at the free beam end. Therefore, the test piece represents a multi-storey frame part subjected to horizontal actions (Figure 5.(b)). The beam end is fixed to the bench by a pair of truss works fixed to the reinforced concrete reaction frames. The trusses are joined by two profiles supporting a roller allowing the longitudinal movement of the system (Figure 5.(c)). The profile dimensions are designed to limit the deflection of the free beam end to 1/1000 *L*, i.e. less than 4 mm. The roller constraint system is developed through a steel plate being fixed to the beam head and having two holed stiffeners to allow a pin to be inserted in. The pin is linked to a f 30mm high strength steel bar instrumented with strain gauges to measure the applied load *F*. The connecting rod end is threaded, and joined to two plates equipped with cylindrical bearings which allow the system to slide inside a slot. As shown Figure 5.(c), there are two symmetric rollers in order to obtain a bilateral constraint.

The test has been carried out by imposing a horizontal δ_x drift at the top of the column, with an average speed rate of about 7 mm/min. Twenty three cycles of increasing amplitude have been imposed, until a maximum δ_x drift equal to 91.3 mm has been reached. Next to a drift equal to 0.5 %, 1 %, 1.25 % and 2.5 %, three drift cycles have been carried out. Longitudinal d_x movements of the top of the column and the free beam end have been monitored by two wire potentiometers, while eight Penny & Gills potentiometers (four per joint surface) having a measurement base *b* equal to 580 mm have allowed to measure the beam-column joint deformation.

(a) (b) (c)

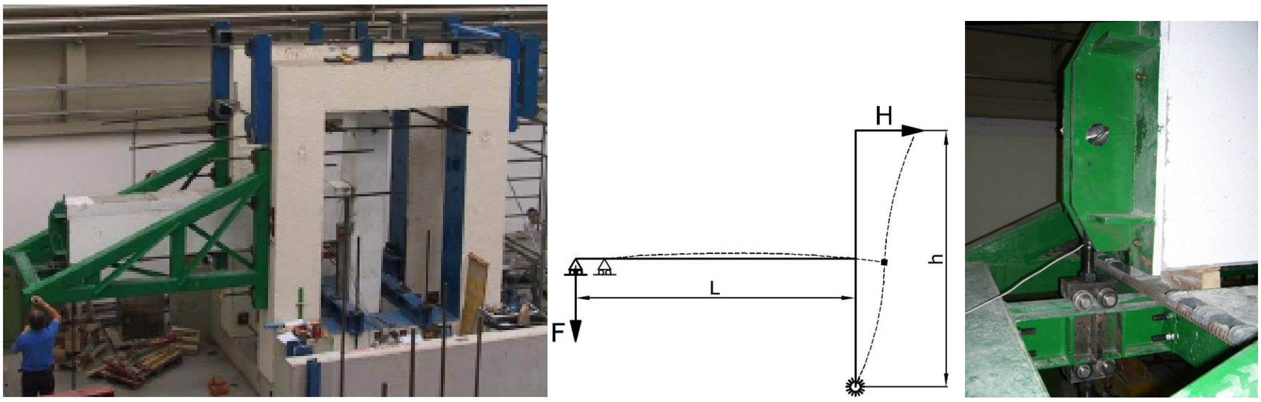


Figure 5 Test set up for corner beam to column joint (a), loading scheme (b) and beam end constraint (c)

(a) (b)

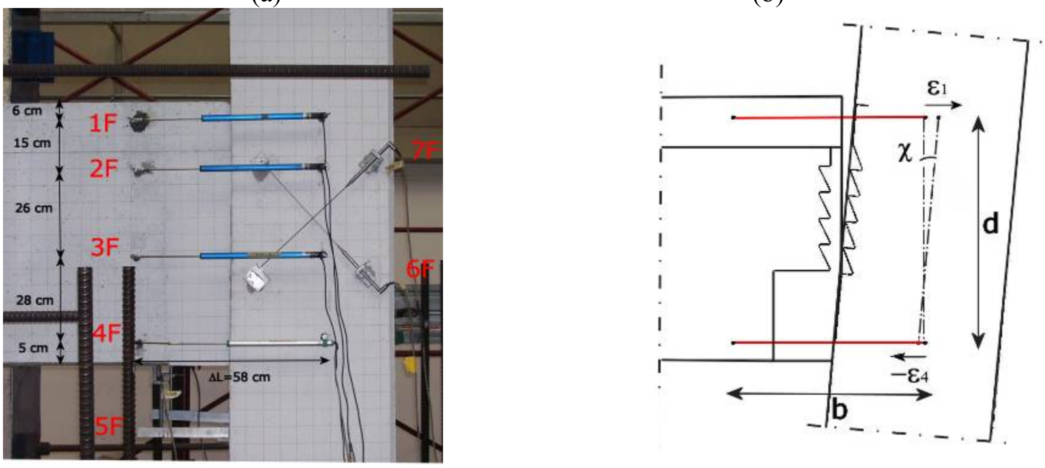


Figure 6: Detail of the instrumentation (a); evaluation of the joint curvature χ (b)

On the basis of simple geometrical relations, both pairs of horizontal outer potentiometers have allowed to evaluate the joint curvature c, which is given by the following equations:

$$c = \frac{e_1 - e_4}{d} = \frac{f}{b} \qquad (3.1)$$

$$f = \frac{w_1 - w_4}{d} \qquad (3.2)$$

where f is the rigid joint rotation; w_1 and w_4 are the averages of the relative displacements measured by each pair of outer potentiometers; e_1 and e_4 are the averages of the strains measured by each pair of outer potentiometers 1 and 4, placed at a distance d equal to 690 mm. The displacements given by the inner 2 and 3 potentiometers have been used to verify the rotation f calculated with the measurements of the outer 1 and 4 potentiometers

As already mentioned, the measurement base b of the eight horizontal potentiometers is much larger in comparison with the specific joint interface dimension (equal to 35 mm); for this reason, the calculated curvature c represents both the strain developed at the beam-column interface, due to the elongation of the unbonded bars, and the bending beam damage, as well as the local column damage around the joint.

With reference to Figure 5.(b), the bending moment in the joint is calculated as follows:

$$M = H \cdot h = F \cdot L \qquad (3.3)$$

where H is the force applied by the jack at the top of the column; F *is* the force measured in the cell at the beam end; h is the distance between the jack action application point and the hinge at the column base, equal to 3.7 m; L is the distance between the connecting rod at the free end of the beam and the column axis, equal to 4.375 m. The bending moment M tensioning the upper fibres has been considered as positive, and a positive rotation $\varnothing$ between beam and column corresponds to such moment M.

4. EXPERIMENTAL RESULTS

The joint failed at a horizontal displacement d_x on the top of the column equal to 91.3 mm, which corresponds to a 2.5 % drift. The maximum positive moment is equal to 573 kNm, while the negative one is equal to -361 kNm (Figure 8). The dissipative capacity are limited due to the brittle failure of the joint column side because of the early failure of the longitudinal high strength steel bars anchorage system. The failure mode is confirmed by the form of the bending moment M versus curvature c diagram which points out that both the four upper reinforcement bars and the lower ones have not been yielded. Anyway, it is worth noting that the experimental ultimate bending moments M_u values slightly differ from the theoretical yielding moment M_y values, resulting from yield of the longitudinal M24 high strength steel bars (see Table 1). The joint is characterized by a stable behaviour during the first cycles, with a limited stiffness decrease. Assuming a grout modulus of elasticity equal to 40000 MPa, the values related to the theoretical 2nd stage stiffness can be compared to experimental values; a difference equal to 3.8 % regarding the tangent positive stiffness and 11.5 % concerning the tangent negative stiffness is found. Figure 7 shows the crack pattern evolution for subsequent cycles of increasing amplitude; the first crack development occurred at a drift equal to 0.5 % (4th cycle), with the formation of two pull-out failure cones, one upper cone next to the anchorage dowels and the other one next to both metal pockets where the supporting brackets are inserted in. During the 9th cycle (corresponding to a displacement imposed to the top of the column equal to 27.4 mm), initial flexural cracks appeared in the beam, with spacing equal to the stirrup distance; such cracks appeared in the column during the following cycle. The column damage around the joint area increases considerably with the rise of the loading cycle amplitude due to the lack of effective constraint of the transverse reinforcements of the column to the anchorage pull-out failure cone of the longitudinal joint bars. Such failure was also due to the lack of axial action in the column.

(a) - Drift: 0.5% - d_x:18.5mm (b) - Drift: 1.5% - d_x:55.5 mm (c) - Drift: 2.5% - d_x:91.3 mm

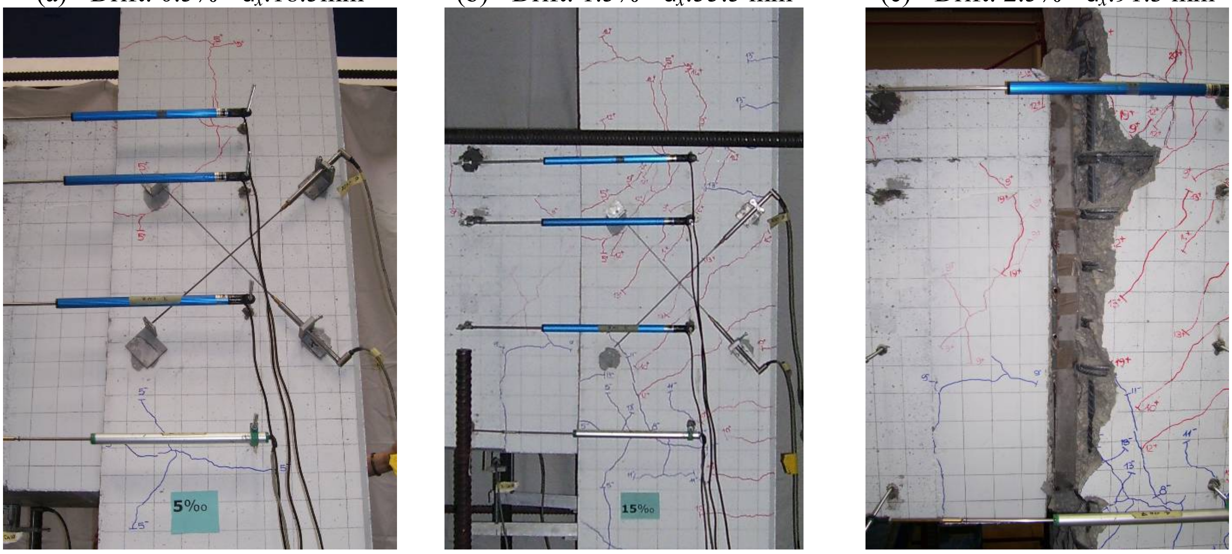


Figure 7 Evolution of the crack pattern

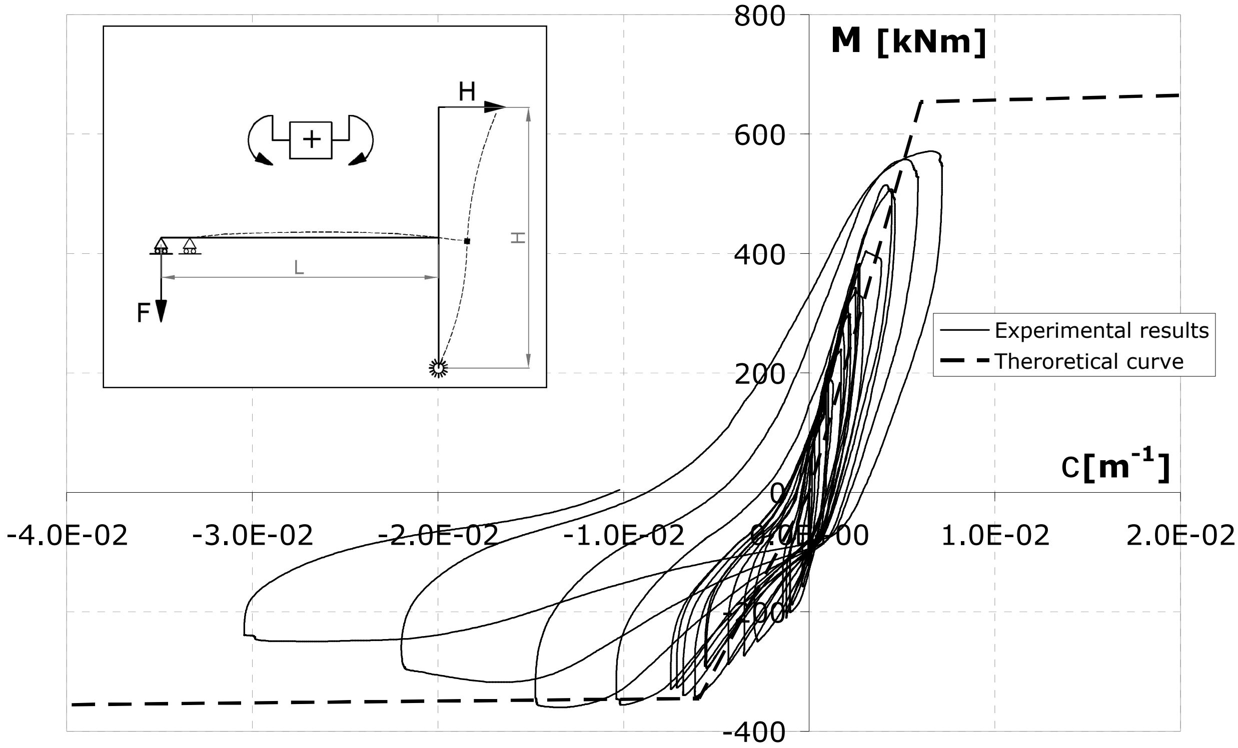


Figure 8 Bending moment M versus curvature χ diagram of all loading cycles applied to the joint

Table 1. Summary of the main experimental results

	M_u [kNm]	M_y [kNm]	χ_u [m^{-1}]	χ_y [m^{-1}]	K_{es} [Nm^2]	K_{th} [Nm^2]
+	**573**	**654**	**6.63x10^{-3}**	**6.02x10^{-3}**	**109**	**113**
-	**-361**	**-345**	**-1.34x10^{-2}**	**- 5.92x10^{-3}**	**58.3**	**65.9**

M_u: experimental ultimate bending moment; M_y: theoretical yield moment; χ_u: experimental ultimate curvature; χ_y : theoretical curvature at yielding; K_{es}: experimental tangential stiffness; K_{th}: theoretical 2nd stage stiffness.

5. CONCLUSION

The work presented shall be considered as a first step of an experimental research programme concerning a particular beam-column joint for precast concrete structures. The joint is featured by an obvious assembling ease due to the lack of reinforcement across the column of reinforced concrete brackets supporting the beam. The pre-existing crack at the beam-column interface should allow the damage to be localized without involving the jointed structural elements as well as an easy repair after a seismic event of mean intensity. The experimental results on a full-scale specimen show a good performance of the joint, in term of moment versus curvature response, characterized by a stable behaviour up to 2.0% drift. Concerning higher drift values, the joint has shown a limited dissipative capacity because of the early collapsed, reached during the cycle at 2.5% drift, due to the brittle failure of the connection on the column side with the pull-out of a conical fracture surface radiating from the anchored end. It is worth pointing out that the joint has shown significant potential for its application in the design of seismic precast structure, both due to the good behaviour exhibited by the joint on the beam side and for the ease of its assembly and repair. Further development of the joint detail on the column side is required in order to obtain an effective bar anchorage system allowing the bar yield which should provide the ductility and dissipative capacity of the joint, even in case of a high earthquake intensity.

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