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Cyclic shear test on a dowel beam-to-column connection of precast buildings

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Abstract. This paper aims at developing the knowledge on the seismic behavior of dowel beam-to-column connections, typically employed in precast buildings in Europe. Despite the large diffusion of the industrial buildings, a high seismic vulnerability was exhibited by these structures, mostly due to the connection systems deficiencies, during some recent earthquakes (Emilia 2012, Turkey 2011).

An experimental campaign was conducted on a typical dowel connection between an external column and a roof beam. In this paper, the performed cyclic shear test is described. According to the experimental results, the seismic response of the system is evaluated in terms of strength, stiffness and failure mechanism. Moreover, the complete damage pattern of the test is described by means of the instrumentations records.
The connection failure occurred due to the concrete cover failure in the column (splitting failure). Such a mechanism corresponds to a negligible energy dissipation capacity of the connection, compared to the overall seismic response of the structure. The experimental results are also compared with the results of a similar monotonic shear test, as well as with some literature relationships for predicting the strength of dowel connections under horizontal (seismic) loads.

Keywords: dowel connection; precast buildings; experimental test; cyclic behavior; seismic response

1. Introduction

Precast RC structures are a code recognized structural system, increasingly used in Europe and in several countries all over the world. However, until 2005 no specific rules regulated the precast buildings design in Eurocode 8; even in the current Eurocode 8 (CEN 2005), the section on precast RC structures is still not complete, particularly for the design of the connection systems (Fischinger *et al*. 2011).

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During recent European seismic events (Emilia 2012, Turkey 2011, L'Aquila 2009) several failures were recorded in many industrial structures, even recently constructed, mainly due to the failure of the connections between the structural elements (Saatcioglu *et al*. 2001) and between the structural and nonstructural components (Magliulo *et al*. 2014b). The main causes of the exhibited seismic vulnerability in precast buildings were the inadequacy of both the building codes and the seismic hazard map (Magliulo *et al*. 2008).

The first recommendations concerning the shear strength evaluation of dowel beam-to-column connections were stated by Vintzeleou and Tassios (1986). They studied the behavior of dowel beam-to-column connections under horizontal shear forces. Different formulas were proposed in order to evaluate the strength of the connection, taking into account two main typologies of collapse mechanism. The first mechanism involved the yielding of the dowel and the crushing of the compressive concrete (failure mode 1); the other mechanism involved the tensile concrete, causing the splitting of the concrete covers of the connected elements (failure mode 2). The proposed relationships were also compared with some experimental results, as reported in Vintzeleou and Tassios (1987). In this paper the authors presented an experimental investigation on the dowel connection behavior under different loading histories, i.e., monotonic and cyclic, on different specimens, i.e., 3 values of concrete compressive strength, 3 dowel diameters and 3 dowel bottom concrete covers. The test specimen consisted of three concrete blocks, connected by two deformed bars; the friction between the blocks was removed and no transverse reinforcement was included in the concrete elements. The cyclic tests evidenced that, if the bottom concrete cover was small (20 mm or 40 mm), the recorded hysteresis loops showed an asymmetric behavior. In particular, the response in the negative loading direction (against the cover) was considerably lower than the response in the positive loading direction (against the core). Furthermore, a cyclic strength degradation was evidenced: the authors suggested to evaluate the dowel strength under cyclic actions by multiplying the monotonic strength value (Vintzeleou and Tassios 1986) by a coefficient equal to 0.5.

Some critical issues can be found in this work: a) such a degradation coefficient was proposed only for the failure mode 1 strength; no indications were given for the strength evaluation of the connection under cyclic loads if the failure mode 2 occurs; b) the tested specimens did not provide any steel reinforcement in the connected concrete elements; c) the friction between the connected concrete elements was not included, neither in the specimen nor in the strength evaluation. According to these considerations, the tested specimens do not fully represent the typical connections in precast buildings, in which the connected elements provide high longitudinal and transversal reinforcement ratios.

In Psycharis and Mouzakis (2012b) the results of some monotonic and cyclic shear tests on common dowel beam-to-column connections were presented. The experimental results highlighted some significant issues. For small cover width the strength of the connection decreased in case the load was applied against the cover, due to the early spalling of the concrete cover. As in the previously described study, the proposed formula for predicting the cyclic shear strength provides a reduction of the corresponding monotonic shear strength by a factor equal to 0.5. Also these authors stated that the formula is valid if the failure mechanism involves the yielding of the dowels, e.g., if the dowels are properly anchored and bolted on their top and there is a distance of about 20 mm between beam and column at the joint, given by an elastomeric pad. If in the cyclic tests the splitting of the concrete cover occurred, no significant degradation of strength and stiffness was recorded up to the complete failure of the concrete cover in the element (beam). These results were also confirmed by a FE model of several pinned connections, calibrated on the

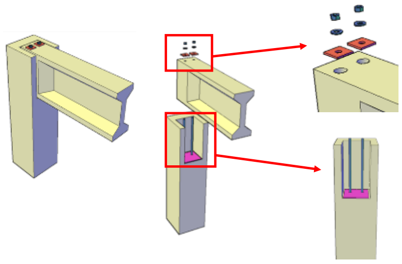


Fig. 1 Typical layout of a beam-to-column dowel connection

performed experimental tests under cyclic and monotonic loads (Kremmyda *et al*. 2014).

The same authors also performed some shaking table tests on pinned beam-to-column connections (Psycharis and Mouzakis 2012a). The dynamic tests showed that the overall structural behavior was highly influenced by the connection response during the excitations and that the inelastic response of the connections was generally compatible with the cyclic quasi-static tests on similar specimens. In the described test no axial forces were induced to the connections, considering that in the dowels only the additional loads of the roof are applied after the grout cast. However, large axial forces in the dowels may reduce the shear resistance of the connections; this phenomenon was not investigated in the research.

Other studies on the seismic design of dowel connections are related to some European research projects, e.g., the SAFECAST project (Toniolo 2012). This research project provided several monotonic, cyclic and shaking-table tests on different connection devices of European precast buildings. Within the SAFECAST project, cyclic and monotonic shear tests on dowel connection were conducted by Zoubek *et al*. (2013). In these tests the failure mechanism involved the flexural yielding of the dowel and the crushing of the surrounding concrete. The strength of the tested connections considerably depended on the position of the plastic hinge in the dowel. In the case of the cyclic loading, a smaller depth of the plastic hinge caused a smaller strength of the connections. However, the tested connections are representative of Slovenian precast structures connections: in all the performed tests a stiff steel tube included the steel dowel in the beam, increasing the confinement effect under shear loads. Moreover, the connections were characterized by very large dowel cover widths; when the dowel is closer to the edge of the beam or column cross-section, different failure mechanisms could occur, as shown in the damaged structures after some last earthquakes (Magliulo *et al*. 2014c).

The presented paper deals with a cyclic shear test of an experimental campaign, conducted in the Laboratory of the Department of Structures for Engineering and Architecture at the University of Naples, on a dowel beam-to-column connection (Fig. 1), that is typically employed in European precast industrial (one-story) buildings. This connection consists of one or more steel dowels embedded in the concrete column. The dowels are connected to the beam both by an in-situ cast grout and by means of steel plates, nuts and washers at top of the horizontal element. The tested specimen provides the typical features of the precast beam-to-column connections, i.e., percentage of longitudinal and transversal reinforcement required by the modern building codes, presence of axial force due to roof elements and live loads, acting on the connection after its complete installation, and realistic concrete cover width in the connected elements.

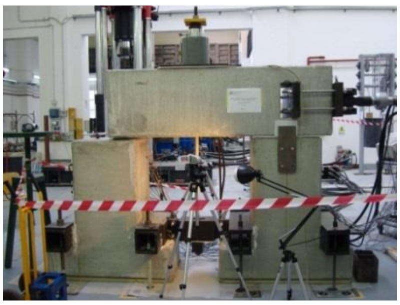
All the test details and the experimental results are reported in the following. The behavior under seismic forces is estimated in terms of strength, stiffness and damping properties. In order to justify the results and the failure mechanism, a detailed description of the recorded damage pattern is also provided. Finally, a comparison with a monotonic test and with some literature formulas is presented.

2. Test specimens, material properties, protocol and instrumentation

2.1 Specimens

In this paper the cyclic shear test on a connection between an external column and a beam is described. The test specimen (Fig. 2) consists of two vertical concrete columns (600 mm×600 mm) with a height of 1.0 m and a horizontal concrete beam (600 mm×600 mm) with a length of 2.10 m. The tested beam-to-column dowel connection is provided on one side of the specimen, i.e., the left side in Fig. 2(a). Since the unconnected column (i.e., the right column in Fig. 2(a)) only gives the support to the beam, two teflon sheets are placed between the right column and the beam in order to avoid undesired frictional resistances. The setup is fixed by means of steel profiles that connect the base of the two columns to the floor (Figs. 3(a)-(b)).

The concrete structural elements (the beam and the columns) are designed according to the provisions of Eurocode 2 (CEN 2004), Eurocode 8 (CEN 2005) and D. M. 14/01/2008 (2008). The seismic action is evaluated according to a high seismic zone in Italy (0.35 g design peak ground acceleration). The column reinforcement details are reported in Fig. 4. The geometrical ratio ρ of the longitudinal reinforcement is the minimum requested by the code (ρ=1.0%) and it is assumed constant along the column height. The transversal stirrups are designed according to the provisions of CNR 10025/98 (2000), resulting in ϕ8 stirrups, i.e., 8 mm diameter bar, 150 mm spaced (see Section B-B in Fig. 4), and in two smaller ϕ8 stirrups, 50 mm spaced (see Section A-A in Fig. 4). The beam reinforcement details are reported in Fig. 5: eight 20 mm diameter bars are used as longitudinal reinforcement of the beam cross-section and ϕ8 stirrups, 75 mm spaced, are placed along the total length of the element. The two ϕ27 steel dowels are placed in the column before the concrete casting (Fig. 6(a)) and inserted in the beam ϕ60 holes (Fig. 6(b)). The connection between the column and the beam is provided by filling the beam holes with high strength grout (Fig. 6(c)) and fixing the dowels at the top of the beam by steel plates, nuts and washers (Fig. 6(d)), modifying the beam holes according to the steel plates rectangular shape. The filling grout is a compensated shrinkage and fiber reinforced mortar with a mean uniaxial compressive strength

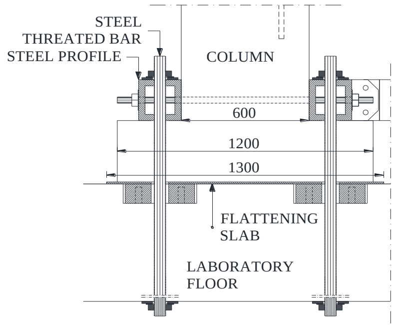


(a) West side

(b) East side

Fig. 2 Experimental setup of the cyclic shear test on a beam-to-column dowel connection (after 7 days ageing) equal to 65 N/mm^2. For both the column and the beam, the concrete covers of the steel dowels are equal to 150 mm and 100 mm in the load direction and in the orthogonal one, respectively. In order to distribute the normal stresses between the concrete elements, a neoprene pad (150 mm×600 mm×10 mm) is placed, designed according to CNR 10018 (1999).

(a)



(b)

Fig. 3 Connection of the column base with the laboratory floor

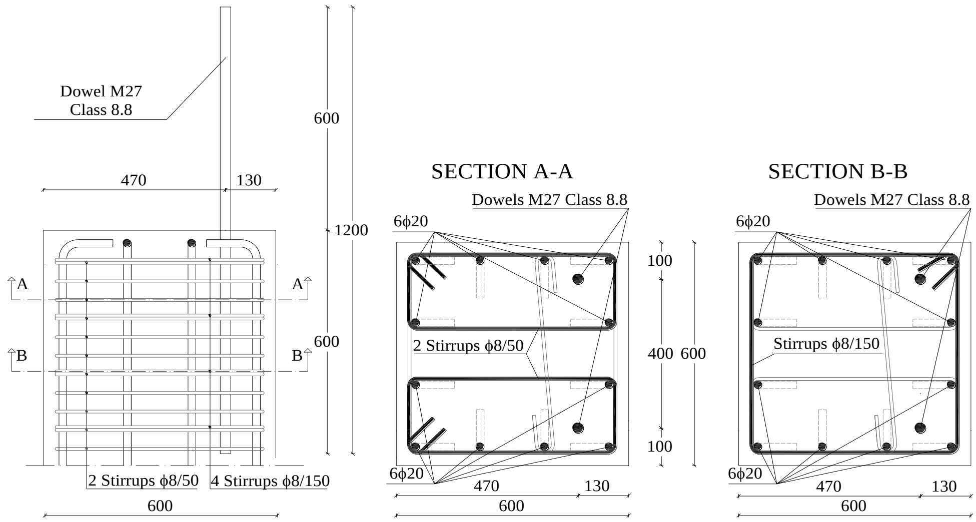


Fig. 4 Column reinforcement details (dimensions in mm)

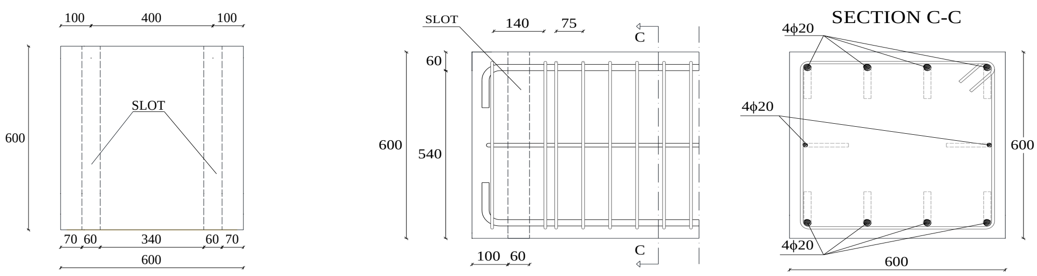


Fig. 5 Beam reinforcement details (dimensions in mm)

(a) Columns and dowels position (b) Insertion of the beam (c) Cast of grout in the beam holes (d) Dowels restrained at the top of the beam

Fig. 6 Construction phases of the specimen

2.2 Material properties

The material strength design values are used during the design phase, according to the mentioned building codes. In order to verify the assumed mechanical properties, some experimental tests were performed before the cyclic test on concrete, dowel and reinforcement steel.

The design concrete class is C30/37, i.e., the nominal cubic characteristic compressive stress is equal to 37 N/mm^2. Since the compressive strength of the concrete can have a significant variability (De Stefano *et al*. 2013), some compressive tests were performed on ten concrete cubic specimens in order to evaluate the effective strength. The experimental compressive strength of each specimen (R_{ci}) is reported in Table 1, with the corresponding mean (R_{cm}) and characteristic (R_{ck}) values, as well as the cylinder strength values (f_{ck} and f_{cm}). All these values are evaluated according to Eurocode 2 (CEN 2004).

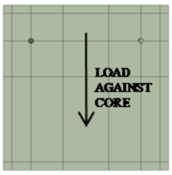
The characteristic yielding tensile strength of the reinforcing steel (B450C type) is equal to 450 N/mm^2. For each used diameter of the reinforcing bars, three tensile tests were performed and the results are reported in Table 2 in terms of: mean yielding strength (f_y), mean ultimate strength (f_u) and mean Young modulus (E_{sm}).

Table 1 Mean mechanical properties of the tested concrete specimens

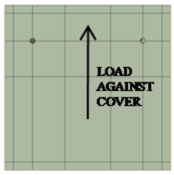
Specimen	R_{ci}	R_{cm}	R_{ck}	f_{ck}	f_{cm}
[-]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]
1	53.64	49.91	42.02	33.62	41.62
2	48.76				
3	46.80				
4	57.20				
5	54.05				
6	40.13				
7	51.52				
8	51.72				
9	46.82				
10	48.46				

Table 2 Mechanical characteristics of the steel used as reinforcement

Diameter	f_y	f_u	E_{sm}
[-]	[N/mm^2]	[N/mm^2]	[N/mm^2]
Φ8	523	639	201918
Φ12	492	621	198429
Φ20	502	607	195959



(a) Negative



(b) Positive

Fig. 7 Loading directions

The adopted steel dowels are two threaded bars with a yielding tensile nominal strength equal to 640 N/mm^2 (8.8 class). According to a performed tensile test on one dowel (ϕ27), the strength at yielding (f_y) is equal to 858 N/mm^2, the ultimate strength (f_u) is equal to 925 N/mm^2 and the Young modulus (E_s) is equal to 191658 N/mm^2.

2.3 Testing protocol

The cyclic test consists of two loading phases:

- the application of the vertical load, simulating the gravity load;
- the application of a cyclic horizontal load, simulating the seismic action.

The vertical load is provided by a vertical jack with a rate of 3 kN/s up to the maximum value of 450 kN, which remains constant during the application of the cyclic horizontal load. The vertical jack is restrained to a prestressed metallic bar, that crosses the RC beam through a special hole; a sleigh anchorage system is placed at the other side of the metallic prestressed bar, in order to avoid undesirable restraining effects (Fig. 3(a)). The applied vertical load was evaluated referring to typical geometrical configurations of an industrial one-story building; for an internal beam it can be considered representative of a tributary area of 90 m^2 and of a vertical load equal to 5 kN/m^2 (including self-weight, live loads, snow and roof loads in the seismic design situation). The vertical load activates the frictional contact between the concrete and the neoprene pad.

The horizontal load is provided by a hydraulic actuator with a displacement control method. The loading history adopted in the cyclic test is defined on the basis of the results of a monotonic test (Magliulo *et al*. 2014a), previously performed in the same experimental campaign. The horizontal displacements are applied in two directions (Fig. 7): against the concrete core (positive direction in the following) and against the concrete cover (negative direction in the following). Fig. 8 shows the displacement loading history provided by the horizontal hydraulic actuator: it consists of 16 steps, each defined by three complete cycles with the same displacement amplitude. The loading rate is equal to 0.02 mm/s up to the 12th step and then equal to 0.04 mm/s up to the end. The displacement increment per step is equal to 1 mm for the first 12 steps and then it is equal to 2 mm up to the test end.

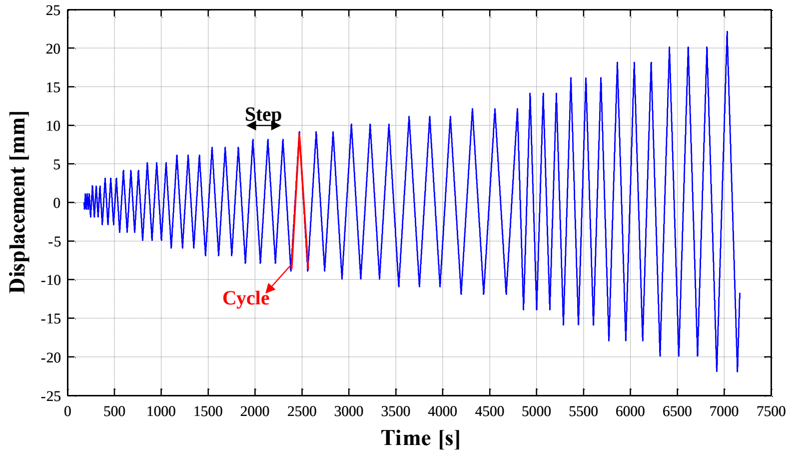


Fig. 8 Displacement loading history

2.4 Instrumentation

In order to record the connection behavior during the test, different instruments are installed on the specimen.

The testing loads are controlled by:

• one load cell in the vertical jack that measures the applied column axial forces;

• one load cell in the horizontal actuator, that measures the forces related to the imposed displacements.

The local deformation in the structural materials and elements are recorded by means of several strain gauges. Since the setup configuration is symmetric with respect to the column axis, these instruments are placed only on one side of the specimen:

• 3 strain gauges along the perimeter of the first three stirrups from the top of the column: at the center of the stirrup in the direction orthogonal to the load (no. 11, 21, 31 in Fig. 9(a)); at the dowel depth in the direction orthogonal to the load (no. 12, 22, 32 in Fig. 9(a)); at the dowel depth, in the load direction (no. 13, 23, 33 in Fig. 9(a));

• 2 strain gauges on a steel dowel: in the column (D1 in Fig. 9(b)), at a 100 mm depth from the element surface; in the beam (D2 in Fig. 9(b)), at a 100 mm depth from the element surface;

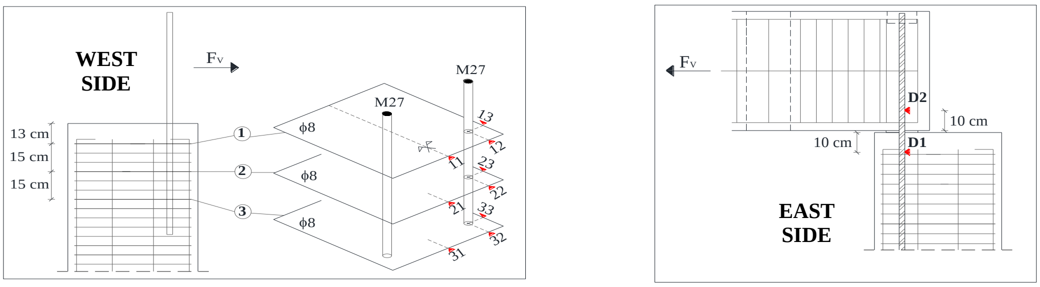
• 4 biaxial strain gauges on the top surface of the column: 2 strain gauges in the direction of the horizontal load, symmetrically placed with respect to the dowel (CC1 and CC2 in Fig. 10(a)); 2 strain gauges in the direction orthogonal to the load, symmetrically placed with respect to the dowel (CC3 and CC4 in Fig. 10(a));

• 2 uniaxial strain gauges: on the column lateral side in the direction of the horizontal load (CC5 in Fig. 10(a)); on the frontal side in the orthogonal direction (CC6 in Fig. 10(a));

• 2 biaxial strain gauges on the beam bottom surface: in the frontal cover along the direction of the horizontal load with respect to the dowel (CB1 in Fig. 10(b)) and in the lateral cover, along the direction orthogonal to the load with respect to the dowel (CB2 in Fig. 10(b));

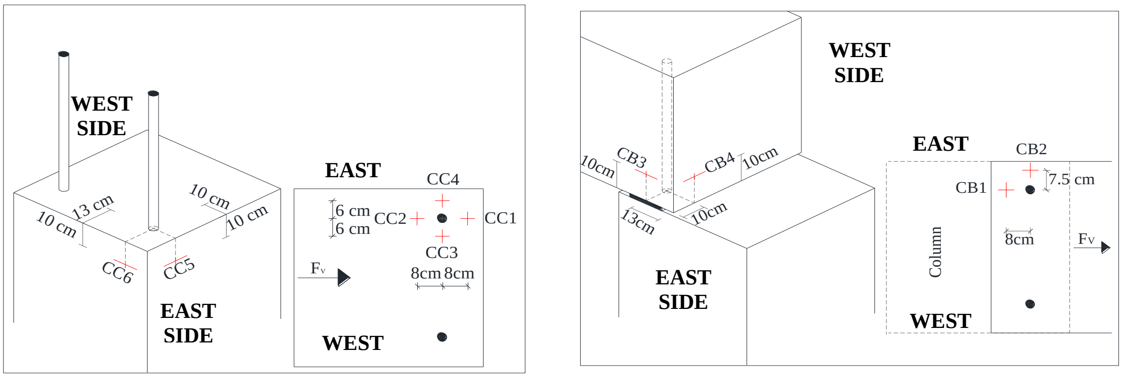
• 2 uniaxial strain gauges on the vertical surfaces of the beam: on the lateral cover in the direction of the horizontal load (CB3 in Fig. 10(b)); on the frontal cover in the direction orthogonal to the load (CB4 in Fig. 10(b)).

In order to evaluate the displacements of the beam with respect to the column, 2 LVDTs (L1 and L2 in Fig. 11) are placed at the beam-end cross section, at the same height, corresponding to the dowels in the load direction.



(a) Column stirrups (b) Dowel on the East side

Fig. 9 Geometrical layout of the stain gauges on the steel elements



(a) Column (b) Beam

Fig. 10 Geometrical layout of the strain gauges on the concrete elements located on the East side of the specimen

(a) Geometrical outline (b) Photo of the West side

Fig. 11 Geometrical layout of the two LVDTs installed to record the relative displacements between the beam and the column

3. Results

3.1 Damage pattern

In this section a detailed description of the damage recorded during the shear cyclic test is presented.

The first crack appears on the column lateral surface, at the dowels depth, and it develops towards the column frontal surface (Fig. 12 and Fig. 13). The same damage pattern is observed on the two sides of the column. The crack opening is confirmed by the records of the strain gauges on the column surface (Fig. 14): at time t=801 sec, the strain gauge on the column lateral cover (6 in Fig. 10(a)) reaches, in the load direction, the strain (ε_{ct}) corresponding to the maximum concrete tensile strength (blue curve in Fig. 14). The failure mode and the time are also confirmed by the records of the strain gauges (parallel to the load) on the steel stirrups in the column (Fig. 15): the steel strain of the second stirrup (green curve in Fig. 15) reaches the yielding value ($\varepsilon_{y,reinf}$) few seconds before the first crack occurrence; after the crack propagates the recording strain gauge (no. 23) is damaged. The steel strain of the third stirrup from the top (no. 33, blue curve in Fig. 15) also reaches the yielding value and it shows a stirrup deformation of 0,425%; it does not get damaged,

Fig. 12 First crack in the column lateral cover (West side)

Fig. 13 Splitting of the column lateral cover (West side)

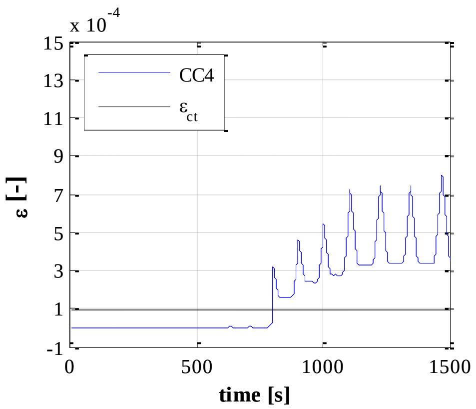


Fig. 14 Records of the strain gauge (CC4 in Fig. 10(a)) on the column top surface (blue curve) in the load direction, along with the concrete limit tensile strain (black line)

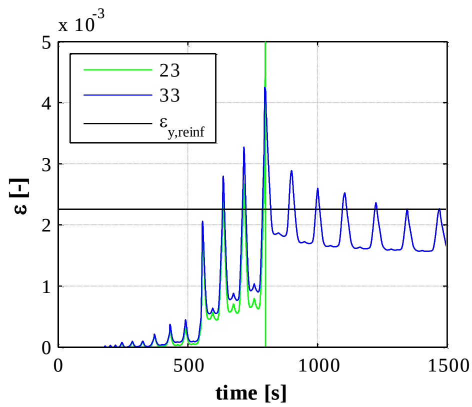


Fig. 15 Records of the strain gauges on the column stirrups (Fig. 9) along with the yielding strain of the reinforcement bar steel (black line)

since it is placed at 410 mm from the column top surface, while the crack has a length of about 300 mm.The first stirrup strains are not considered in this study since their records are highly influenced by the column damage during the first steps of the test.

At larger values of horizontal displacement, the lateral crack width increases and two other inclined cracks form at the column frontal surface (Fig. 16). An inclined crack also appears on the beam frontal surface (Fig. 17), with an increasing width up to the end of the experiment (Fig. 18). Also other cracks open in the column: they are parallel to the first crack and provide the expulsion of other concrete portions from the column (Figs. 19(a)-(b)). The test continues up to the complete concrete expulsion is evidenced (Fig. 20).

Fig. 16 Crack opening in the column frontal cover

Fig. 17 Crack opening in the beam frontal cover

Fig. 18 Crack in the beam at the end of the test

(a) West side (b) East side

Fig. 1 Final damage state of the specimen at the end of the cyclic test

Fig. 20 Splitting of the cover at the end of the test

3.2 Force -displacement curve

The force-displacement curve of the performed cyclic test is reported in Fig. 21. This curve shows the mean value of beam-to-column relative displacements recorded by the LVDTs L1 and L2 (Fig. 11(a)), along with the horizontal force recorded by the actuator load cell. The force-displacement curve is plotted till a 20% strength degradation is recorded, i.e., up to the 6th step of the loading history. The positive values of both the displacements and the forces correspond to the load against the column concrete core, while the negative results correspond to the load against the concrete cover. The behavior of the connection is unsymmetrical, also for low values of displacements. In the positive direction both the shear force and the horizontal displacement increase up to the end of the test with a limited stiffness degradation. Conversely, when the horizontal load is applied against the column frontal cover (negative values), the connection exhibits a significant degrading behavior: after the attainment of the shear strength, the response of the connection quickly degrades in terms of both strength and stiffness. The beginning of the degradation in the negative direction corresponds to the first crack formation in the lateral cover (red circle in Fig. 21). Consequently, according to the described curve and damage, the concrete splitting in the lateral cover of the column corresponds to the failure mechanism of the connection, and the maximum strength attained before it corresponds to the connection characteristic strength.

Since the stated forces also include the additional strengths of the setup (i.e., frictional strength of the neoprene pad, frictional strength of the teflon sheets and strength of the setup components), a cyclic test is performed in order to evaluate these additional contributions. This additional test is performed on the same specimen with the same load protocol without the dowel connection, i.e., before connecting the two dowels to the beam by the mortar. The results of this additional test are reported with a gray solid curve in Fig. 21: up to the 6th step of the loading history, the frictional strength can be completely related to the friction strength due to the concrete-neoprene contact, evaluated according to Magliulo *et al*. (2011).

If the maximum recorded displacements at each step are reported with the corresponding experimental force, the envelopes of the cyclic behavior are obtained: Fig. 22 shows the three envelopes, corresponding to the three cycles of each step up to the 6th step. When the horizontal load acts against the column concrete core, at the last step the first cycle achieves a force of 193.8 kN at a horizontal displacement of 1.04 mm; the force slightly decreases to 185.8 kN and to 177.3 kN at the second and third cycle, respectively. This trend is observed during the whole test, i.e., the maximum shear strength in the first cycle is always greater than the values achieved in the following cycles at the same displacement. However, this strength degradation is not significant.

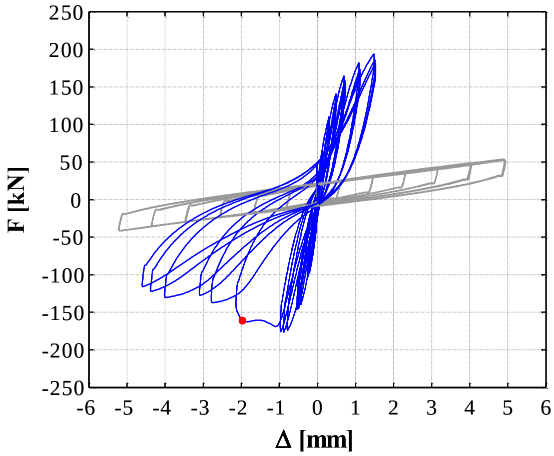


Fig. 21 Force-displacement curve of the whole cyclic test and frictional resistance (gray line) up to 6th step. The red point indicates the first crack formation in the lateral concrete cover

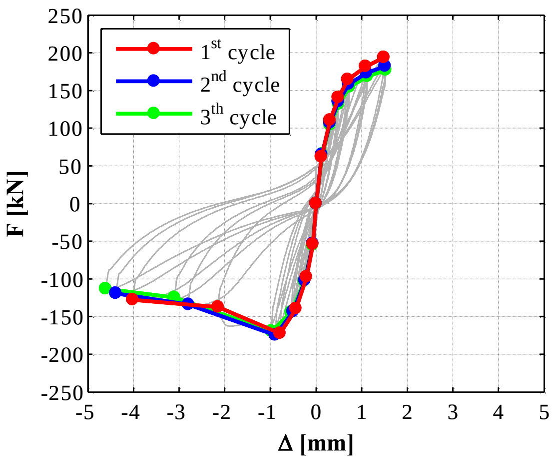


Fig. 22 Force-displacement curve (gray curve) and envelope of all the steps (circle markers) for each cycle: 1st cycle (red line), 2nd cycle (blue line) and 3rd cycle (green line)

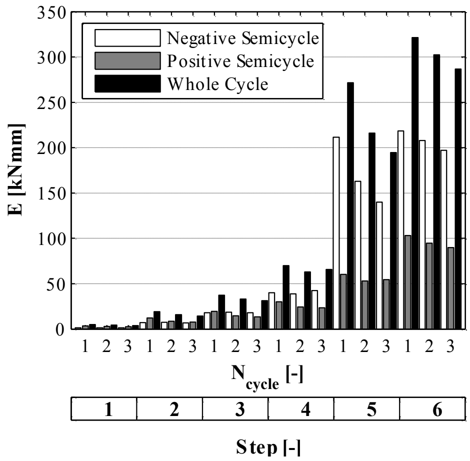


Fig. 23 Dissipated energy evaluation during the cyclic test, for the positive and negative semi-cycles and for the complete cycles, up to 6th step

When the horizontal load acts against the column concrete cover, the maximum shear strength, equal to 176.6 kN, is reached at second cycle of the fourth step.

Fig. 23 shows the dissipated energy during the cyclic test, evaluated as the area under the force-displacement curve. For each cycle of each step, gray bars represent the dissipated energy during the compressive loading phase (positive semi-cycle); white bars represent the dissipated energy during the tensile loading phase (negative semi-cycle). Black bars represent the dissipated energy of a complete cycle. The positive dissipated energy is quite similar to the negative one up to the third step, confirmed by the initial symmetric response of the connection, shown in Fig. 22. On the contrary, after the fourth step the negative dissipated energy is always much greater than the positive one, since the degrading behavior of the connection in the cover direction leads to higher relative displacements in the connection at the same imposed absolute displacement with respect to the core direction. Dissipated energy of a whole cycle at each step is always greater for the first cycle due to strength reduction which occurs in each successive cycle within the same step. The low dissipated energy demonstrates the negligible influence of this kind of connection on the overall structure dissipative properties.

3.3 Comparison with a monotonic shear test

The described test belongs to an experimental campaign that also provides a monotonic test on a beam-to-column connection (Fig. 24), that has the same geometrical features of the specimen presented herein. As in the cyclic test, the vertical load of 450 kN is applied by means of a vertical jack on the beam. During the monotonic test a monotonically increasing displacements history is applied to the specimen, pulling the beam, i.e., applying the load against the connection concrete cover.

The results of this test are reported in Fig. 25 (red curve) in terms of actuator horizontal force and beam-to-column relative displacement. The monotonic curve, reported up to the cyclic test collapse displacement, shows a high strength degradation after the peak strength, similarly to the negative part of the cyclic curve. Moreover, the experimental evidence shows a crack at the lateral cover in the column (Fig. 26), whose appearance corresponds to the maximum value of the horizontal force of the envelope curve.

By the comparison between the monotonic force-displacement curve and the negative semi-cycles of the cyclic test (Fig. 25), it is evident that the elastic stiffness of the monotonic test is equal to the cyclic one and the connection maximum strengths are very close.

It is found that no larger degradation occurs to this kind of connection when cyclic loads are applied with respect to monotonic loads. An interesting comparison can be drawn between this evidence and some literature experimental results, as the ones reported in Zoubek *et al*. (2013). In this work, the cyclic strength values of the dowel beam-to-column connections are significantly lower than the values recorded during the corresponding monotonic tests. This is due to the connection damage and failure mechanism, which involve the yielding of the steel dowel and the crushing of the surrounding compressed concrete; on the contrary, in the presented paper, the reference tests (cyclic and monotonic) experienced a failure mechanism that involves the concrete cover under tensile stresses (Fig. 19 for the cyclic test and Fig. 26 for the monotonic test), with low deformation in the steel dowel (Fig. 27 for the cyclic test and Fig. 28 for the monotonic test) and in the compressed concrete core. Just at the end of the two tests it was possible to detect the plastic hinge on the steel dowel (Fig. 29 for the cyclic test and Fig. 30 for the monotonic test). So it can be stated that in this case both the kind of damage and the failure mechanism justify the absence of degradation phenomenon.

Fig. 25 shows that also the degrading branches after the failure mechanism of the cyclic and monotonic curves are very similar, even though, at this stage, the formation in the cover of the first crack and the following tensile concrete damage already occurred and the horizontal loads should be borne by the steel dowels and by the surrounding compressed concrete. This can be justified demonstrating that up to the assumed ultimate displacement value (~5 mm, which corresponds to the dash dot line in Fig. 28 for the monotonic test and in Fig. 27 for the cyclic test), the dowels are still in the elastic field of behavior and, then, their cyclic behavior does not show larger degradation with respect to the monotonic one. Indeed, during the cyclic test, the records of the strain gauge placed on the column dowel (blue curve in Fig. 27) show that the yielding steel strain (black curve in Fig. 27) is overpassed at a time equal to 2579s, corresponding to the 9th cycle at a displacement (Fig. 8) of ~8 mm (much larger than 5 mm). The considered strain gauge, located at 100 mm from the top column surface (D1 in Fig. 9(b)), is very close to the position of the plastic hinge, located at 80 mm from the bottom surface of the beam (Fig. 29).

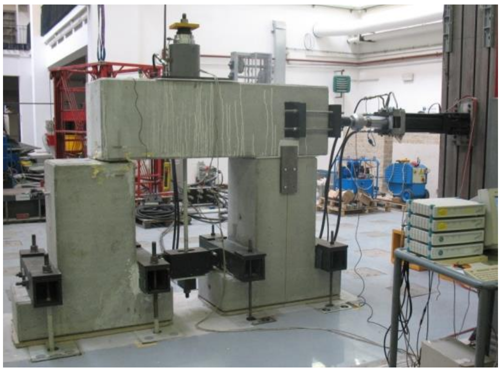


Fig. 24 Experimental setup of the monotonic shear test on a beam-to-column dowel connection

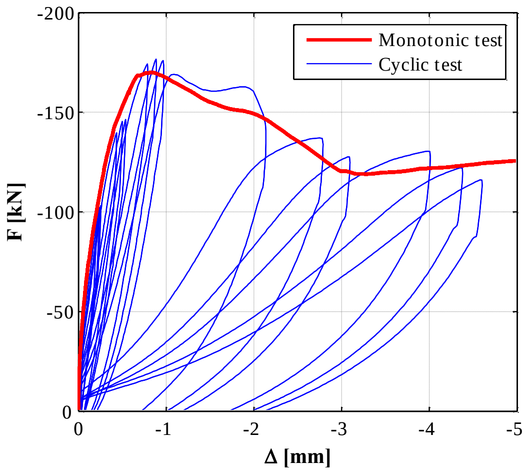


Fig. 25 Comparison between force-displacement curves: monotonic test (red line) vs cyclic test (blue line)

Fig. 26 Final step of the monotonic test: splitting of the concrete

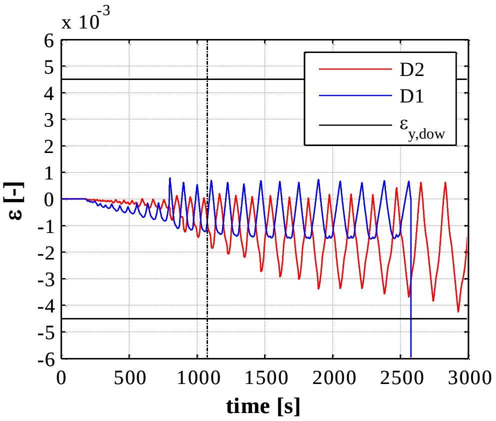


Fig. 27 Records of strain gauges on the steel dowels in the beam (red curve) and in the column (blue curve) along with the yielding strain of the dowel (black line) during the cyclic test

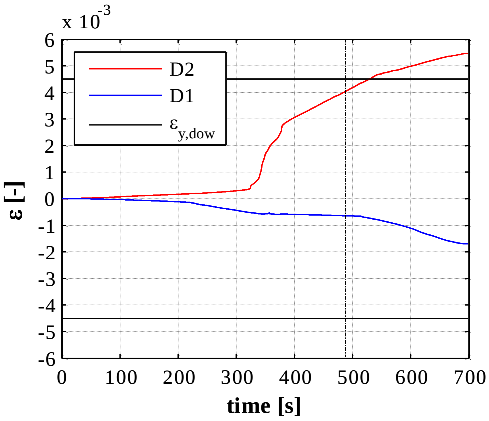


Fig. 28 Records of strain gauges on the steel dowels in the beam (red curve) and in the column (blue curve) along with the yielding strain of the dowel (black line) during the monotonic test

Fig. 29 Detection of the plastic hinge in one of the two steel dowels in the column at the end of cyclic test

Fig. 30 Detection of the plastic hinge in one of the two steel dowels in the column at the end of monotonic test

3.4 Comparison with the results of other experimental tests

In order to support the presented experimental results (in the following "presented tests"), in this section they are compared with the results of other similar experimental campaigns (in the following "reference tests"). As reported in the introduction of this paper, several research studies have been developed on RC precast connections. However, only few studies are referred to either modern dowel connections in precast structures or specimen exhibiting failure mode 2 (concrete splitting). As a consequence, the comparison is only related to two experimental works, described in Vintzeleou and Tassios (1987) and in Psycharis and Mouzakis (2012b).

In Vintzeleou and Tassios (1987) some shear tests on dowel connections were described. Among the others, two shear tests (one monotonic and one cyclic) with the failure mode in the concrete cover are considered for the comparison. Both the experimental tests showed that the reduced covers in the connected elements caused the splitting of the concrete before the dowel yielding. Furthermore, an asymmetric response in the two loading directions was recorded: in the direction of the failed cover the shear strength was significantly smaller than in the opposite direction. Both these phenomena agree with the result of the cyclic test presented in this paper.

The other reference study of the comparison was performed by Psycharis and Mouzakis (2012b) in the framework of the SAFECAST project. It consisted of several monotonic and cyclic tests on dowel connections, with different geometrical features (e.g., concrete cover width, dowel diameter). Since in the presented tests the failure mode occurred in the concrete cover, the comparison is provided only with two reference tests (a monotonic one and a cyclic one), which showed the same failure mode (splitting of the concrete cover). By comparing the results in terms of shear strength, the following conclusion can be drawn.

- In the reference cyclic test the shear strength of the connection has the same order of magnitude of the strength found in the presented study; the slightly lower shear strength shown in the reference test can be justified by some differences with respect to the presented test, i.e., the absence of the axial load and the failure of the frontal cover instead of the lateral one.

- Negligible strength and stiffness degradation is found in the reference cyclic test with respect to the reference monotonic test up to the concrete splitting in the beam cover; this experimental evidence agrees with the results of the presented tests. Furthermore, the reference cyclic test, as the presented one, shows a strength degradation after the concrete cover failure.

3.5 Comparison with literature formulas

In this section the experimental results of the cyclic test are compared with some literature formulas, proposed to evaluate the cyclic shear strength of dowel connections. The considered relationships are listed and presented below.

• The CNR 10025/98 (2000) formula evaluates the strength of a dowel beam-to-column connection according to Eq. (1). This formula is valid if the eccentricity of the shear force is less than the halved diameter (d_b) of the dowel. Since the eccentricity is defined as the half of the thickness of the support between the beam and the column, i.e., the halved neoprene pad thickness (10 mm), the Eq. (1) is valid in the analyzed case

$$V_{Rd,CNR} = c \cdot d_b^2 \cdot \sqrt{f_{yd} \cdot f_{cd}} \tag{1}$$

where c is equal to 1.6 if the concrete of the connection is confined and equal to 1.2 if it is not confined; f_{cd} is the concrete design compressive strength and f_{yd} is the dowel design yielding strength. The CNR relationship does not consider the influence of the concrete cover depth on the connection shear strength and the case of cyclic loads.

• According to Vintzeleou and Tassios (1986), when concrete covers are greater than 6-7 times the dowel diameter, failure mode 1 (simultaneous dowel yielding and concrete crushing) occurs and the dowel connection shear strength can be evaluated by Eq. (2), if eccentricity is equal to zero

$$V_{Rd,V\&T^1} = k \cdot d_b^2 \cdot \sqrt{f_{ys} \cdot f_{cc}} = 1.3 \cdot d_b^2 \cdot \sqrt{f_{ys} \cdot f_{cc}} \tag{2}$$

where f_{cc} is the concrete compressive strength and f_{ys} is the dowel yielding strength. As anticipated in the Introduction, Vintzeleou and Tassios (1987) stated that the design dowel force under cyclic loads in case of failure mode 1 can be obtained by multiplying the design strength values by 0.5

$$V_{Rd,V\&T^{I}_{cyclic}} = 0.5 \cdot V_{Rd,V\&T^{I}} \tag{3}$$

If the dowel is pushed towards the concrete cover and the concrete cover is lower than 6-7 times the dowel diameter, the strength of the connection is related to concrete failure (concrete splitting) rather than to dowel yielding. Depending on the ratio between depth of the frontal cover and of the lateral cover, a side splitting or a bottom splitting could occur.

If the lateral cover is smaller than the frontal one, a side splitting occurs and the connection shear strength can be calculated by Eq. (4)

$$V_{Rd,V\&T^{II}_{side}} = 2 \cdot d_b \cdot b_{ct} \cdot f_{ct} \tag{4}$$

where b_{ct} is the net width of the concrete section, f_{ct} is the concrete tensile strength and d_b is the dowel diameter.

On the contrary, when the frontal cover is smaller than the lateral one, the bottom splitting failure occurs and the force strength is equal to

$$V_{Rd,V\&T^{II}_{bot}} = 5 \cdot f_{ct} \cdot c \cdot d_b \cdot \frac{c}{0.66 \cdot c + d_b} \tag{5}$$

where c is the value of the frontal cover.

• In the final report of the SAFECAST project (Negro and Toniolo 2012), the guidelines for the design of the mechanical connections in precast elements under seismic actions are reported. Concerning the dowel beam-to-column connection, some numerical formulas are proposed in order to evaluate the strength for different failure mechanisms. The failure mode 1 strength is evaluated as

$$V_{\mathrm{Rd},Safecast^{I}} = 0.9 \cdot n \cdot d_b^2 \cdot \sqrt{\left[f_{yd} \cdot f_{cd} \cdot \left(1 - \alpha^2\right)\right]} \tag{6}$$

where α is the ratio between the normal tensile stress due to other possible contemporary effects on the dowel and the characteristic yielding strength of the dowel, and n is the number of dowels.

The strength of the connection related to the splitting of the column concrete cover is given by

$$V_{Rd,Safecast^{II}} = 1.4 \cdot k \cdot d_b^{\alpha} \cdot h^{\beta} \cdot \sqrt{f_{ck,cube} \cdot c^3} \cdot \psi_{re} \tag{7}$$

where

$$\alpha = 0.1 \cdot \left(h / c\right)^{0.5} \tag{8}$$

$$\beta = 0.1 \cdot \left(d_b / c\right)^{0.2} \tag{9}$$

$$k = b / \left(3 \cdot c\right) \le n \tag{10}$$

$f_{ck,cube}$ is the cubic concrete compressive strength in N/mm^2, h is the effective length of the dowel, assumed equal to $8d_b$, $\psi_{re} = 1.4$ in presence of edge reinforcement and $\psi_{re} = 1.0$ in all the other cases, and b is the column cross section dimension orthogonal to the load. Such a formula derives from the relationships proposed in literature for predicting the strength of anchorage in concrete

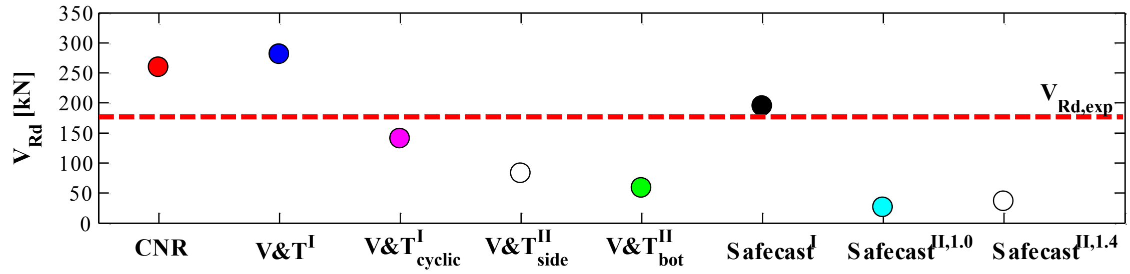


Fig. 31 Comparison between the cyclic experimental strength of the dowel connection ($V_{Rd,exp}$) and the literature formulas

elements (Eligehausen *et al*. 2006).

In Fig. 31 the comparison between the cyclic experimental strength of the dowel connection ($V_{Rd,exp}$) and the result of the cited literature formulas is reported. All the strengths are evaluated with the material strength mean value and with the dowel effective diameter.

The presented literature and code formulas can be distinguished in three groups: 1) formulas of the shear strength for failure mode I (dowel yielding and crushing of the surrounding concrete), i.e., Eqs. (1),(2),(3),(6); 2) formulas of the shear strength for failure mode 2 (concrete frontal or lateral cover splitting), i.e., Eqs. (4)-(5); 3) formulas of the shear strength, based on the behavior of anchorage systems in concrete elements (Eq. (7)).

In the first group, Eqs. (1) and (2) provide larger values than the experimental strength; this result can be justified by the different failure mechanism of the experimental test with respect to the one assumed by the two considered formulas. However, if the reduction factor of 0.5 for cyclic load is applied, Eq. (3) provides a value quite close to the experimental one. A very close matching to the experimental results is also provided by the SAFECAST formula (Eq. (6)) for the failure mode 1.

If the occurred failure mechanism is considered, the comparison should be performed with the formula of the second group concerning the lateral concrete splitting (Eq. (4)); however, the resulting value is much smaller than the recorded strength. The same result is found if the formula concerning the bottom splitting failure is taken into account (Eq. (5)). These large differences between the results of the experimental test and the values of the literature formulas can be justified by the presence of transversal steel bars in the tested concrete elements, which the analyzed formulas do not take into account.

The values provided by Eq. (7) are much lower than the experimental one. Even if in the report a correction factor (equal to 1.4) is suggested in order to take into account the reinforcement details in the concrete elements, the evaluated strength is still too small with respect to the experimental result. In the case of this formula the main issue is related to the reference mechanism from which the relationship was obtained, i.e., the shear strength of steel anchorage in concrete elements, described in Eligehausen *et al*. (2006). Such an evidence was also underlined in Belletti *et al*. (2013): in an analytical study the authors found out that the strength of some dowel roof-to-beam connections is always larger than the predicted values by the SAFECAST formulas if side spitting failure occurs.

4. Conclusions

An extensive experimental campaign is performed in order to investigate the seismic behavior of the dowel beam-to-column connections in precast structures. The presented paper describes the results of one cyclic test on a dowel connection between a column and a roof beam. The setup is designed according to the current European and Italian building codes. It is loaded with a vertical load and with a predefined displacement history up to the failure.

During the test the horizontal load mainly causes damage to the column, while no relevant cracks affect the beam until the end of the test. The damage pattern and the instrumentation records show that the first damage in the connection is the cracking in the column concrete lateral cover. According to the force-displacement curve, this damage is achieved when the load is applied against the column cover and it corresponds to the maximum strength of the connection in this direction. For this loading condition the behavior shows a high degradation of stiffness and strength after the maximum strength is achieved. When the horizontal load acts against the column concrete core, the force-displacement curve presents higher strength since the connection failure is related to the mechanism that involves the steel dowels yielding and the crushing of the surrounding confined concrete.

The evaluated dissipated energy of the connection is very low, demonstrating that this connection cannot influence the dissipative properties of the whole structure under dynamic actions.

The cyclic force-displacement curve, recorded when the load is applied against the column cover, is compared with the results of a monotonic shear test, performed on a dowel beam-to-column connection with the same geometrical features. By the comparison the following conclusions can be drawn:

• the elastic stiffness of the two specimens as well as the failure mechanism are equal in the two experiments;

• the strengths that correspond to the concrete lateral cover splitting in the column are very close in the two tests and represent the maximum strength of the connection;

• the brittle nature of the failure mechanism is confirmed in both the tests by the highly degrading behavior after the peak strength in the force-displacement curve;

• no more degradation in the connection behavior in terms of strength and stiffness is recorded if cyclic, instead of monotonic, loads are applied to the specimen, since the achieved failure mechanism is related to the concrete response under tensile stresses.

The reliability of the presented experimental tests should be supported by means of additional experimental tests on identical specimens, in order to take into account the variations in the experimental results due to imperfections. However, the results of similar tests, found in the recent scientific literature, confirm the conclusions of this paper.

In the final part of the paper the experimental results are compared to the existing literature formulas for the calculation of the shear strength of dowel connections, showing that these formulas underestimate such a strength if the observed failure mode (splitting of the concrete cover) is taken into account. The reason of this underestimation can be found in the presence of stirrups in the tested elements, which is not taken into account by the analyzed formulas. However, the SAFECAST formula also gives too small values, even if a correction factor, taking into account the presence of stirrups, is considered. In any case, the observed underestimation can be reasonable if the analyzed formulas are used for the design of the connection, considering that a brittle failure mode is observed.

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