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ORIGINAL RESEARCH PAPER

Influence of different mechanical column-foundation connection devices on the seismic behaviour of precast structures

Bruno Dal Lago[1] · **Giandomenico Toniolo**[1] · **Marco Lamperti Tornaghi**[2]

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Abstract Dry-assembled precast concrete frame structures are typically made with dowel beam-to-column connections, which allow relative rotation along the beam direction. In the orthogonal direction the rotation of the beam is prevented but again the connections of the superimposed floor elements allow for relative rotation. All the ductility and energy dissipation demand in case of seismic action is therefore concentrated at the base of cantilever columns. Hence, the column-to-foundation connection plays a key role on the seismic performance of such structures. Mechanical connection devices, even if correctly designed for what concerns resistance, may affect the behaviour of the whole joint modifying the ductility capacity of the columns and their energy dissipation properties. An experimental campaign on different mechanical connection devices has been performed at Politecnico di Milano within the Safecast project (European programme FP7-SME-2007-2, Grant agreement No. 218417, 2009). The results of cyclic tests on full scale structural subassembly specimens are presented. Design rules are suggested for each of the tested connections on the basis of the experimental observations, and numerical analyses have been performed with hysteretic parameters calibrated on the experimental loops. The seismic performance of structures provided with those connections is investigated through a case study on a multi-storey precast building prototype, which has also been subject to full-scale pseudo-dynamic testing within the same research project at the European Laboratory of Structural Assessment of the Joint Research Centre of the European Commission. The comparison of the results from the structure provided with the different studied connections clearly highlights how some solutions may lead to both reduction of ductility capacity and dissipation of energy, increasing the expected structural damage and the seismic risk.

✉ Bruno Dal Lago
brunoalberto.dallago@polimi.it

[1] Department of Civil and Environmental Engineering, Politecnico di Milano, Piazza L. da Vinci, 32, 20133 Milan, Italy

[2] European Commission, Joint Research Centre (JRC), Directorate Space Security and Migration, Unit Safety and Security of Buildings, via Enrico Fermi, 2749, 21027 Ispra, Italy

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1 Introduction

A relevant part of the European prefabrication has developed through the last five decades towards the direction of dry and semi-dry assembled structures (mostly frames), identifying the maximum industrialisation of the construction process as the most economical and reliable choice. The typical structural arrangement for buildings with industrial and commercial destination is made of cantilever columns clamped at their base and hinged beam-to-column connections. In the orthogonal direction the rotation of the beam is prevented but again the connections of the superimposed floor elements allow for relative rotation. When precast frame structures are subject to horizontal actions, the ductility demand is hence concentrated at the base of the columns, therefore making it the most critical region of the structural assembly. Other types of dry or semi-dry assembled precast frame or wall structures have been facing the market in the last decades (Nakaki et al. 1999; Dal Lago and Dal Lago 2012), but the base of the columns still remains the critical region.

Pocket foundation is the traditional column-to-foundation connection, a connection where the column is inserted into a special pocket arising from the base footing. The temporary bearing is ensured by the crane and provisional props are fixed after the verticality regulation is made with wedges. The joint is finally filled with in-situ concrete pouring. Saisi and Toniolo (1998) presented an extensive experimental investigation on pocket foundation connections, showing their efficiency. To be noticed that for large size columns the pocket foundations become so big that their transportation turns out to be difficult. In addition, the pocket requires a large excavation. Recently, the market is demanding new technologies to perform quicker assemblages and to avoid in situ concreting. The answers are provided by different types of mechanical connections which are characterised by dry or semi-dry assemblage, temporary self-support of the column (to reduce the crane holding time) and mechanical regulation of verticality. In addition, in seismic areas pocket foundations shall be connected by transversal beams, whilst mechanical connections may be directly inserted within precast foundation grids. The innovative devices are mainly based on mechanical connection of the longitudinal reinforcing bars of the columns through bolts or splicing with mortar or resin pouring. Some of these solutions are described in the CEB-FIP bulletin 43 (2008). A solution based on the partial debonding of the reinforcing bars within the column-to-foundation connection has been tested by Belleri and Riva (2012). Mechanical connections are also easier to be dismounted in comparison with pocket connections, which might become in the near future a key point within the “design for deconstruction” included in the Ecodesign Working Plan 2015–2017 of the European Commission.

Among the other activities carried out by the Research team of Politecnico di Milano within the Safecast Project, oriented to study the seismic performance of precast structures provided with innovative connections (Toniolo 2012), a large experimental investigation has been performed regarding the column-to-foundation connections. The present paper summarises the experimental results of this investigation and provides comments regarding the design. Numerical simulations have been performed and show a good correspondence with the experimental results, assessing their design methodology and calibrating the hysteretic shape parameters.

The results of the experimentation, that covers four types of column-to-foundation connections with tests carried out on full scale sub-assemblies under cyclic loading, are presented in the following.

Figure 1 shows the four types of connections, ranging from a simple technology with post-screwed protruding bars connected by bonding to the foundation, to more complex mechanical solutions with bolted sockets and couplers. A modified socket solution is also proposed and tested. The connection devices are sketched in Fig. 2. Design rules are suggested for each tested technology on the basis of the experimental observation. Design suggestions are also reported in Negro and Toniolo (2012).

The effect of these connection on the seismic performance of full structural assemblies is investigated through dynamic non-linear analysis performed on a multi-storey precast building prototype, which has also been subject to full-scale pseudo-dynamic testing within

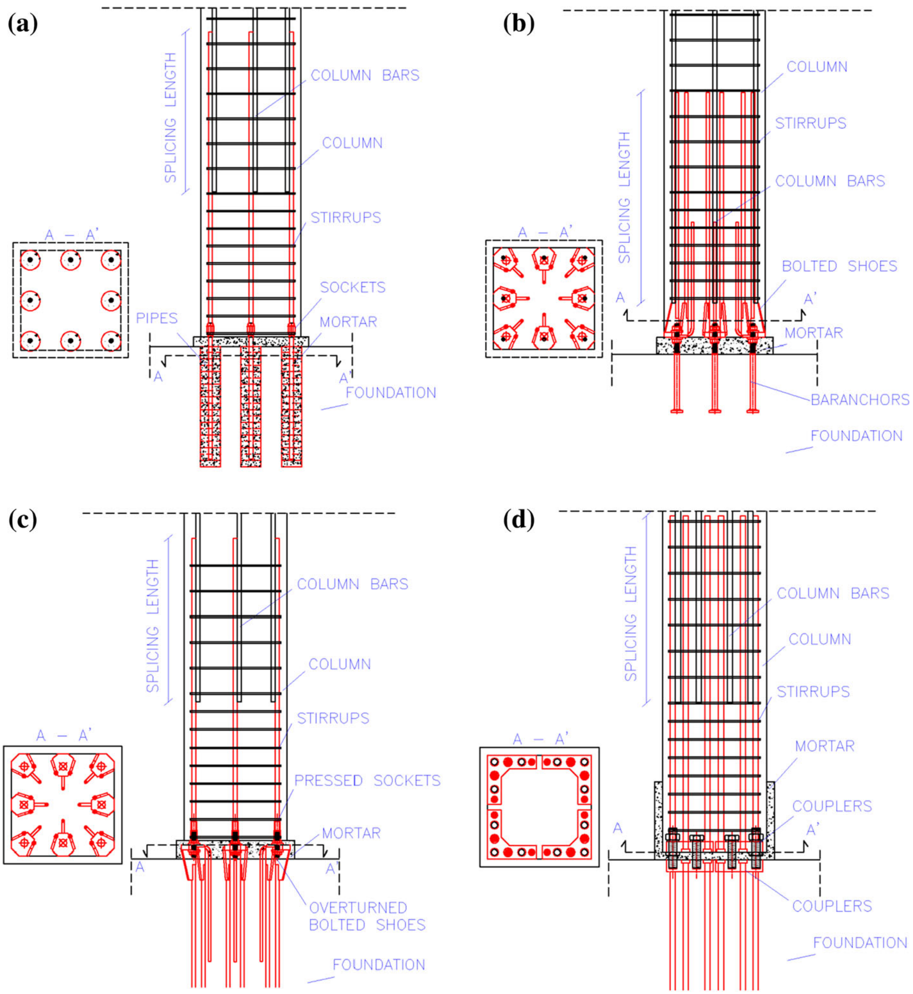

Fig. 1 Types of column-to-foundation connections tested: **a** post-screwed protruding bars, **b** standard bolted sockets, **c** overturned bolted sockets, **d** couplers

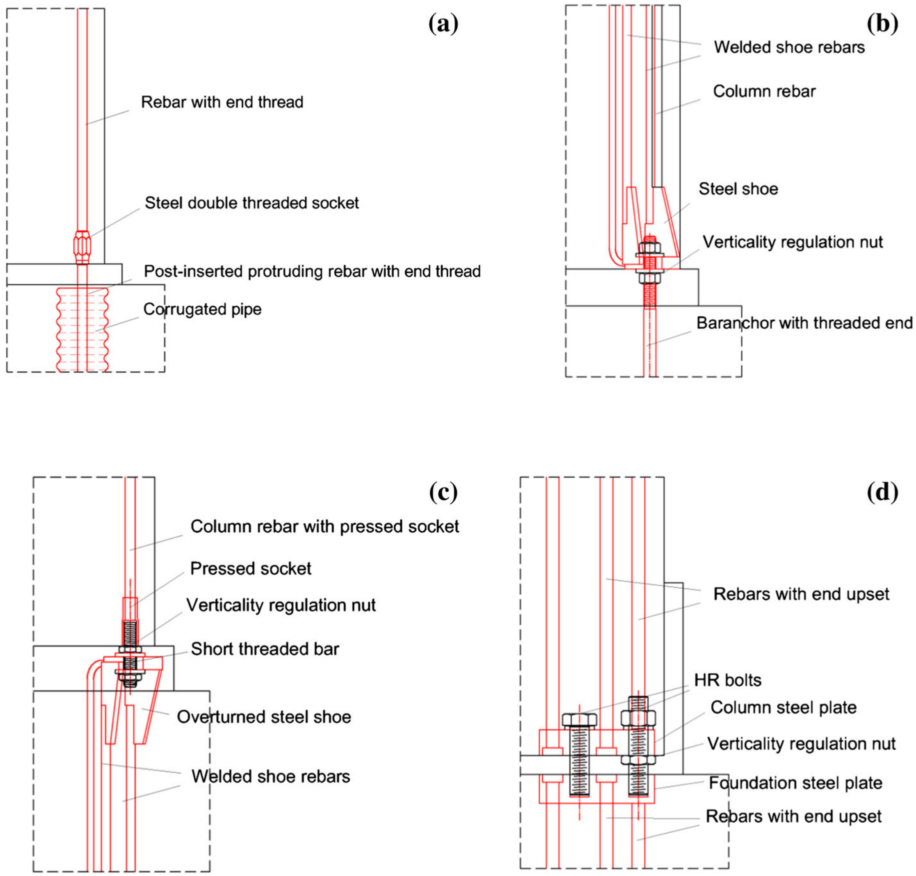

Fig. 2 Connection devices: **a** post-screwed protruding bars, **b** standard bolted sockets, **c** overturned bolted sockets, **d** couplers

the Safecast project at the European Laboratory of Structural Assessment (ELSA) of the Joint Research Centre (JRC) of the European Commission in Ispra, Italy. The non-linear macro-models with calibrated hysteretic parameters are used to model the connections. The comparison of the results allows to derive important considerations about the modification of ductility and energy dissipation capacity of the structural system due to the introduction of the concerned devices.

2 Mechanical column-to-foundation connections

2.1 Protruding bars

Protruding bars are a common solution for the connection of a column to its foundation. With this solution the longitudinal bars of the column protrude from its bottom and are introduced in situ into large diameter corrugated pipes pre-inserted in the foundation. After its verticality regulation, the column is fixed with provisional props and the pipes are filled with non-shrinking high resistance mortar. This solution, if compared to a traditional pocket foundation, avoids concrete in-situ pouring, requiring only a small quantity of mortar for completion, besides reducing sensibly the overall size of the connection. Several experimental results are available in literature, presented by Buratti et al. (2012) and Popa et al. (2015) among the others, showing that this connection can provide a reliable full base support of the column. This type of connection is also treated in CEB-FIP bulletin 43 (2008) in the overturned version with corrugated pipes inserted into the column. The particular connection examined in the present work has a technologic variation which is diffused in practice: the reinforcing bars are not directly protruding from the column and are provided with an end pressed socket, to which short bars are screwed in situ, avoiding in this way the transportation problems and possible distortion or damaging of the protruding bars (Fig. 2a). This discontinuity creates a weak point in the longitudinal reinforcement due to the reduced resisting core area of the threaded part. Special truncated cone threaded terminals are used to reduce the loss of tensile resistance of the bars and the consequent ductility loss. Figure 3a shows the in-situ screwing of the protruding parts of the bars and Fig. 3b shows the final positioning of the column on the foundation.

2.2 Standard bolted sockets

The standard bolted sockets are metallic shoes made of steel plates welded together, to which couples of bars are directly welded for the splicing with the reinforcing bars of the column. The splice length is over-dimensioned by capacity design to avoid a bond failure. A hole in the middle of the horizontal base plate allows the threaded baranchor inserted in the foundation to go through and to be tightened with an upper nut. It must be pointed out that the tested version is threaded for the portion protruding from the foundation only. A lower nut provides the temporary bearing and allows the verticality regulation of the column (Fig. 2b). The joint of the assembled connection is filled with non-shrinking high resistance mortar. Since the welded bars cannot develop large ductility due to the thermal shock applied to the steel and the consequent brittle tensile failures that may occur behind the weld, the B450C steel baranchor is designed, according to the data declared by the producer, so to reach its maximum resistance with the upper bars remaining in elastic field. A similar version of this connection has been previously tested by Fagà et al. (2010). Figure 4a shows a picture of the foundation with the baranchors threaded in the protruding

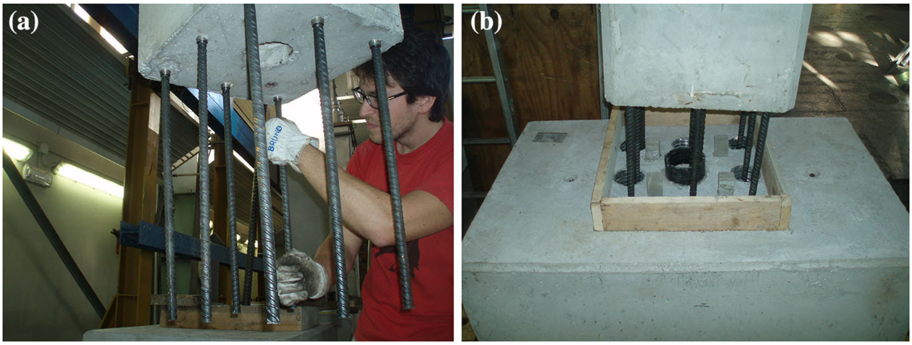

Fig. 3 Post-screwed protruding bars: **a** screwing of the protruding bar crop in embedded and **b** insertion of the protruding bars in corrugated sleeves embedded in the foundation

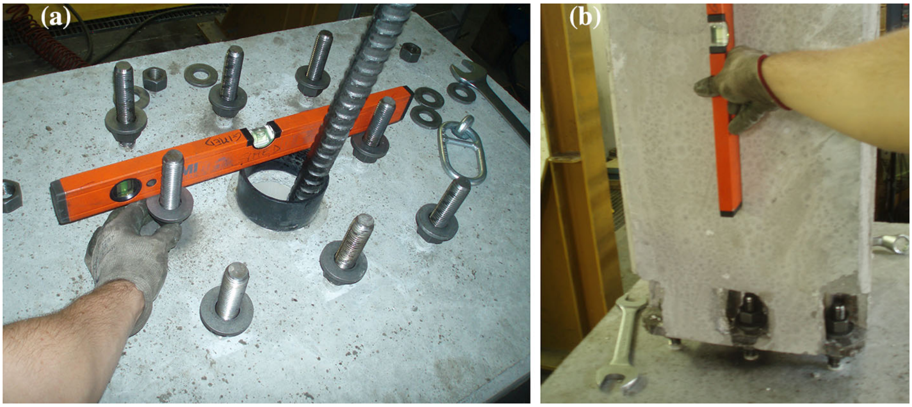

Fig. 4 Standard bolted sockets: **a** protruding baranchors embedded in the foundation, **b** bolting of the baranchors in the sockets and mechanical verticality regulation

portion ready for temporary support of the column. Figure 4b shows the assembled connection during the verticality regulation before the mortar pouring.

2.3 Overturned bolted sockets

This connection, proposed by the authors, is a modification of the standard bolted socket one. In the proposed version, the sockets are overturned and inserted in the foundation, with the last 50 mm protruding from it. The longitudinal reinforcing bars of the column are provided with end pressed sockets. The connection is assembled with the insertion of short over-dimensioned threaded bars provided with two nuts (Fig. 2c), for the temporary bearing and the mechanical regulation of the verticality. This configuration has been suggested to move the plastic hinge from the short threaded length of the baranchor to the base of the column with its longitudinal reinforcement and to improve in this way the cyclic response in terms of energy dissipation. In this way, according the designer's perspective, the capacity design is respected, hence the connection does not affect the mechanical properties of the column, which remains the critical area for strength, ductility and energy dissipation. Figure 5a shows a view of the foundation with the overturned sockets and Fig. 5b shows the insertion of the short threaded bars into the sockets of the column.

2.4 Couplers

Couplers are in general over-dimensioned devices that ensure the continuity of the longitudinal column reinforcement with the foundation reinforcement. Several solutions have been developed, based on mechanical or bond connections with in-situ pouring of special mortar or resin. The tested device is fully mechanical, made of two thick steel plates, one inserted into the column and the other inserted into the foundation, to which the reinforcement is connected by an upset end enlargement. The connection is assembled with high resistance bolts tightened in between the plates, with a double nut configuration for the vertical regulation (Fig. 2d). Their short free length is then grouted with non-shrinking high strength mortar. The mortar filling can include the concrete recesses made for the bolt insertion, and used also to include additional stirrups for confinement. Figure 6a shows the

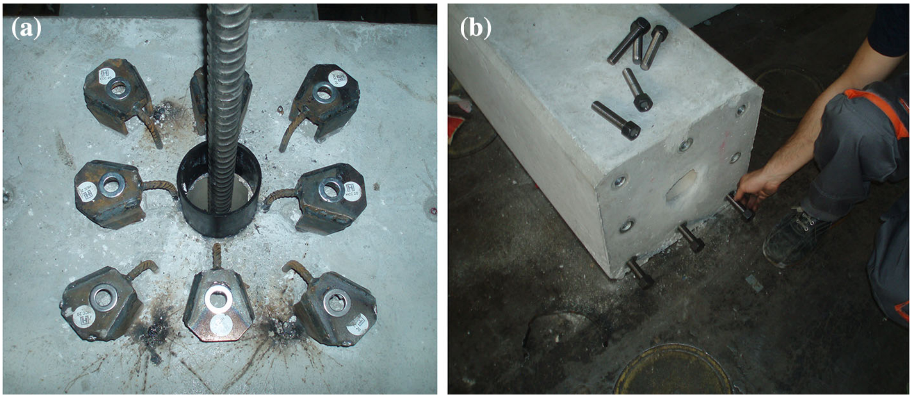

Fig. 5 Overturned bolted sockets: **a** overturned sockets embedded in the foundation, **b** screwing of the threaded bar segments in sockets embedded in the column

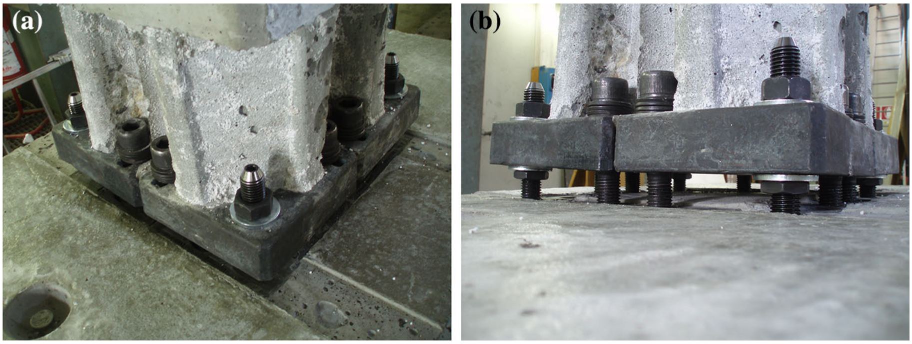

Fig. 6 Couplers: **a** thick steel plates at the column base already connected to the foundation counter-plates with bolts, **b** particular view of the mechanical regulation system

column base before the completion with mortar grouting and Fig. 6b shows a detail of the bolts that connect the column and foundation plates.

3 Cyclic behaviour of connections

The cyclic behaviour of the connections has been mechanically characterised through load tests on full-scale structural column-foundation sub-assemblies. All columns have a square cross section of 400 mm side and a central Φ100 mm hole, with 2.5 m height. All foundations are properly reinforced and have sides of 1.1 and 0.7 and a height of 0.5 m. The concrete class is C55/45 and the reinforcing steel is B450C grade. The tests have been conceived with a mono-axial setup with the application of constant axial load. Figure 7 shows the testing setup.

The foundation is bolted to the strong steel beam of the laboratory through anchoring sockets. An hydraulic jack fixed to the strong steel frame of the laboratory applies the displacement history to the top of the specimen. The cyclic displacement history is transferred through a bolted connection that encloses the column top. The vertical load is

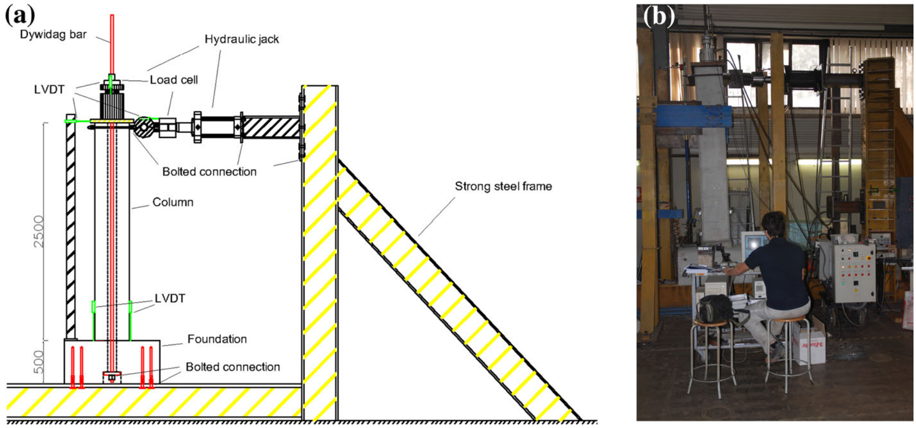

Fig. 7 Testing setup: **a** scheme, **b** picture

applied by a large diameter bar running through all the specimen in the central hole anchored at its base and pre-tensioned by a vertical jack that constantly applies an axial load equal to 380 kN. Since the axial load follows the displacement, the bending moment at the base of the columns can be simply calculated by multiplying the applied top force as recorded by the horizontal jack load cell by the height of the column, since second order amplification is not acting. The shear force on the column is calculated deducing from the applied top force the horizontal component of the axial load.

The tested columns are reinforced with the minimum longitudinal reinforcement ratio suggested by CEN-EN 1998-1:2004 (1 % of reinforcement ratio), consisting in 8 Φ16 mm placed at the corners and at mid-sides (the current cross section is shown in Fig. 8a), with the exception of the specimen with couplers, which is reinforced with the 2.3 % of reinforcement ratio, consisting of 8 Φ24 mm (the current cross section is shown in Fig. 8b). All columns are provided with double Φ8/70 mm square well-anchored stirrups. The latter choice (with larger reinforcement ratio) is due to the fact that the particular coupling device subject to testing is traditionally used with large diameter bars. More details about the experimentation are available in Dal Lago et al. (2013). The results are presented in the

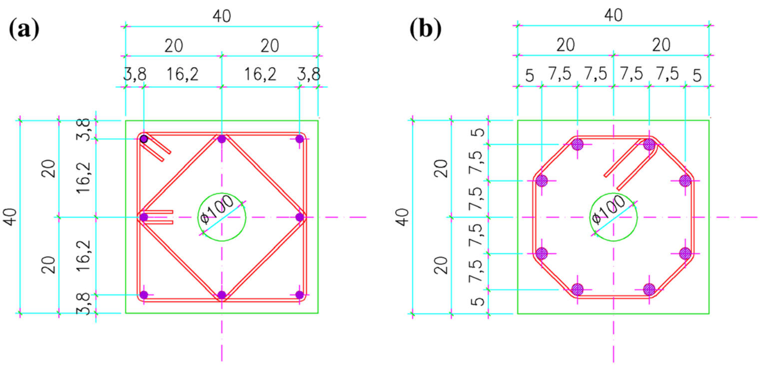

Fig. 8 Cross section of the current part of the column (measures in cm) reinforced with **a** 8φ16 and **b** 8φ24 bars

following. In order to allow a better comparison, the bending moment has been adimensionalised with respect to the characteristic resisting moment of the column current section.

The resisting moment corresponding to an axial compression load of 380 kN is calculated on the basis of a classic bi-linear elastic-plastic stress–strain relationship for steel and of a parabola-rectangle stress–strain relationship for concrete. The characteristic resistance of materials is used. The resisting moments calculated in this way are of about 190 kNm for the column reinforced at 1 % and of about 270 kNm for the column reinforced at 2.3 %.

To be noticed that in all cases the hysteretic response of the specimens is inclusive of the column behaviour and the considerations on the obtained diagrams refer to the overall specimen as more or less affected by the local behaviour of the base connection. For a more meaningful definition of the ductility and energy dissipation capacities of the tested specimens, the obtained diagrams of moment-drift cycles and corresponding dissipated energy histograms shall be compared to those of the solution with protruding bars that, if bond slips within the foundation are prevented, is assumed to save the full capacities of the column without any detrimental influence of the base connection (except for the reduction of ductility due to the post-screwed technology). The dissipated energy is expressed with the dimensionless value as the ratio with the maximum possible dissipation of the corresponding perfect elastic-plastic cycle. The area included within the perfect elastic-plastic cycle has been deduced by identifying a reference moment plateau value corresponding to the full yielding of the section, considering the design mechanical properties of materials. The elastic stiffness is obtained by connecting the origin of the moment vs curvature diagram with the point obtained by crossing the first yield curvature with 75 % of the resisting bending moment, following the suggestions of Paulay and Priestley (1992).

The slenderness of the columns used in the test campaign is comparable to the common high values used in precast constructions. Considering that, the failure modes were related to flexure, and no indications came from the experimental data with reference to shear behaviour, except that no relevant effects due to shear were observed.

3.1 Post-screwed protruding bars

The hysteretic response of one specimen provided with post-screwed protruding bars is shown in Fig. 9a in terms of dimensionless moment-drift cycles. The cyclic behaviour is stable through the cycles. A good overall ductility of the specimen is shown by the envelope curve that should correspond to the monotonic behaviour. The histogram of the specific dissipated energy (Fig. 9b) shows dimensionless values that range from 0.30 to 0.45 for drifts between 3.0 and 4.5 %, which is close to the values obtained by Saisi and Toniolo (1998) for the pocket foundation connection and corresponds to the value expected for a well-detailed reinforced concrete monolithic connection. The failure is due to the progressive breaking of the bars embedded in the foundation in correspondence of the threaded part (Fig. 10a), and started at 4.5 % of drift. Figure 11a shows a picture of the column base with its cracking pattern just before failure. The foundation showed some small cracks, but it did not significantly contribute to the overall deformation. The bars fixed by bond in the ribbed pipes and the ribbed pipes embedded in the foundation did not show any pull-out tendency. The socket coupler of the bars did not show any damage. Up to failure, the plastic hinge extended in the column base with a diffused cracking. At the end of the test, all the six reinforcing bars at both sides of the column have been found broken. Some bars have been subjected to early buckling. The moment attained in the test is about 1.3 times higher than the value calculated for the current section of the column and

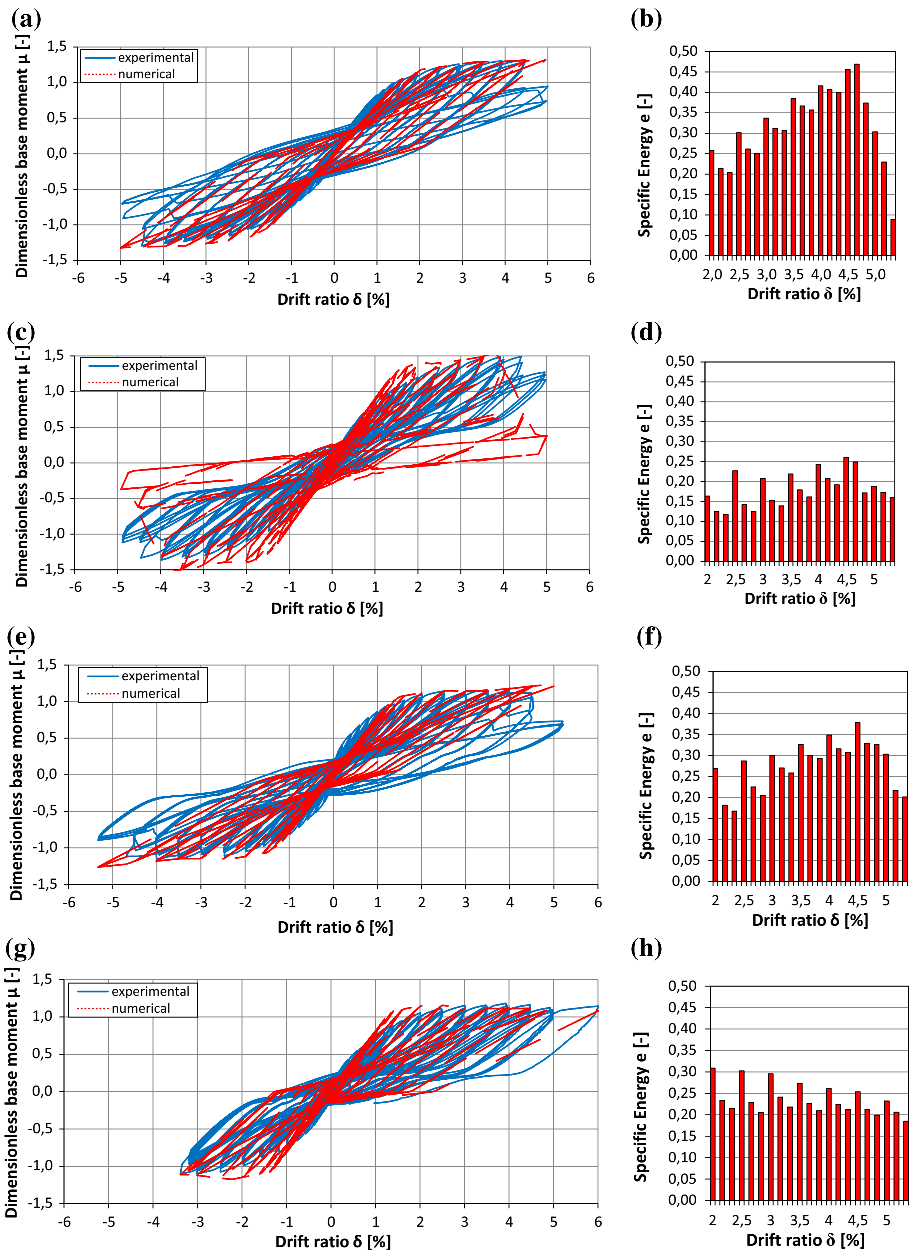

Fig. 9 Moment-drift cycles (*left*) and dissipated energy (right): **a**, **b** post-screwed protruding bars, **c**, **d** standard bolted sockets, **e**, **f** overturned bolted sockets and **g**, **h** couplers

this is probably due to the difference between the actual strength value of the steel present in the column and the standard characteristic strength value of the steel B450C used in calculation.

Fig. 10 Failure modes: **a** post-screwed protruding bars, **b** standard sockets, **c** overturned sockets, **d** couplers (failure not attained)

3.2 Standard bolted sockets

The hysteretic cycles of the test performed on the column-to-foundation standard bolted socket connection are shown in Fig. 9c in terms of dimensionless moment-drift diagram. The cyclic behaviour is stable through the cycles. A good overall ductility of the specimen is shown by the envelope curve that should correspond to the monotonic behaviour. The histogram of the dissipated energy (Fig. 9d) shows dimensionless values that are much lower than those of the post-screwed protruding bars solution and this indicates that this connection affects negatively the energy dissipation capacity of the structural assembly. This result doesn't mean that this solution cannot be used in seismic zones, but only that its effects shall be properly taken into account in calculation. The failure corresponds to the progressive rupture of the threaded baranchors embedded in the foundation, the first of which failed at 4.5 % of drift. A high pinching effect of cycles is recorded, especially at large drifts, due to the emerging plays at the nut-shoe interface: once the threaded bar elongates enough to compensate the deformation given by the manual tightening of the bolts, the connection starts rocking without stiffness following the increasing gap either in compression or in tension. Most of the damage is concentrated at the joint between column

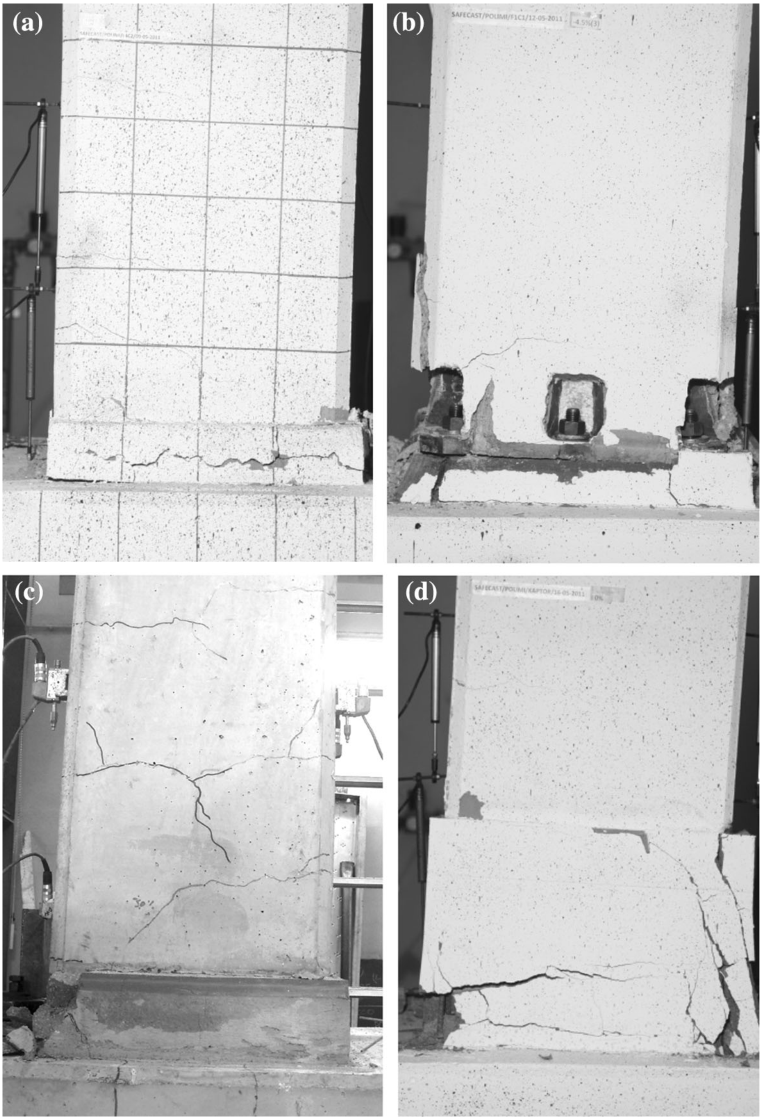

Fig. 11 Pictures of the column bases before failure: **a** post-screwed protruding bars at 4.5 % of drift, **b** standard bolted sockets at 4.5 % of drift, **c** overturned bolted sockets at 4.5 % of drift, **d** couplers at 6.0 % of drift (failure not attained)

and foundation, with spalling of the unconfined mortar. In Fig. 10b all broken baranchors are visible. Figure 11b shows the picture of the column base with the cracking pattern just before failure. Since the failure occurred in correspondence of the interface section in the baranchors that have an enlarged section with respect to that of the single longitudinal bars placed in the current part of the column and its reinforcement in the first bottom segment with spliced bars is bigger, the maximum dimensionless bending moment attained in the test is relevantly larger (about 1.5 times) than the one calculated.

3.3 Overturned bolted sockets

Figure 9e shows the experimental moment-drift loops of the specimen provided with overturned bolted sockets. The cyclic behaviour is stable through the cycles. A good overall ductility of the specimen is shown by the envelope curve that should correspond to the monotonic behaviour. The histogram of the dissipated energy (Fig. 9f) shows dimensionless values that are a little lower than those of the protruding bars solution and this indicates that this connection does not relevantly lower the energy dissipation capacity of the structural assembly. The observed failure occurred within the base joint and it started at 4.5 % of drift, with the unexpected quasi-brittle failure of few pressed sockets. In Fig. 10c a broken socket is visible. Figure 11c shows the picture of the column base with the cracking pattern just before failure, with a distributed plastic hinge extended in the base of the column. The attained moment is a little higher (about 1.2 times) than the one calculated due to the actual steel strength value, which is a little higher than the characteristic one assumed in calculation. For the rest the same considerations presented for the standard bolted solution can be confirmed. The improvement of the cyclic behaviour expected from the overturned arrangement of the connecting devices has been confirmed by the test only in a small measure, due to the early unexpected failure of the connecting devices.

3.4 Couplers

The experimental hysteretic response of the specimen with couplers is shown in Fig. 9g. It is asymmetrical due to a not well centred positioning of the specimen, which did not allow to explore larger drift in one direction. The imposed cyclic displacement history has been modified in order to have asymmetrical cycles with a reduced maximum drift in the quoted direction. After having attained the 5 % of drift, failure or resistance decrease were not observed. A further half-cycle up to the 6 % did not provoke moment losses. A full series of cycles at 6 % has not been done because of setup limitations. The cyclic behaviour is stable through the cycles. A pinched behaviour is observed, due to the specific technology of the connection. The upset end enlargement allows to fix the bar to the plate in tension, and in compression the bar bears on the mortar filling. However, at large drift the mortar can get damaged and a gap develops leading to the progressive pinching behaviour. The selected device has been modified by the producer as a consequence of the results by threading the upset enlargement, directly screwing it to the plate without weakening the bar. A good overall ductility of the specimen is shown by the envelope curve that should correspond to the monotonic behaviour. The histogram of the dissipated energy (Fig. 9h) shows dimensionless values that are sensibly lower than those of the post-screwed protruding bars solution and this indicates that this connection affects negatively the seismic behaviour of the structural assembly. This shall be taken into account in the seismic design. The foundation did not show visible cracking. Damage developed along the height of the column (Figs. 10d, 11d), with plastic deformation beyond the mechanical device, which could be easily unscrewed for dismounting.

3.5 Length of plastic hinge

Different lengths of plastic hinges and cyclic shapes have been observed in the experimental results. While the different lengths of plastic hinge affect the deformation profile at the base of the column and can concentrate the plasticity in a short length, reducing the displacement ductility of the column, the pinching behaviour reported for some connections strongly affects the energy dissipation.

Since the introduction of mechanical column-to-foundation connectors in precast structures may relevantly modify the energy dissipation capacity of the joint, traditional design approaches that refer to well-detailed cast-in-situ concrete joints, like the force-based simplified approach adopted in CEN-EN 1998-1:2004 or more refined performance-based methods (Fajfar and Gaspersic 1996), based on the force reduction factor q, need to be redefined. The displacement ductility capacity of a cantilever column is directly related to the length of the plastic hinge that can develop at its base. Proper formulations have been derived by literature classical formulae (Priestley et al. 2007), adjusted on the basis of a deeper understanding of the connection behaviour, assessed by the experimental observations. The classical formulation for well-detailed cast-in-situ solutions considers the effective length of plastic hinge L_p as the sum of a portion of the shear span L_c from the base to contra-flexure point and the length of strain penetration L_{sp}. The length of strain penetration and the coefficient k, reducing L_c taking into account the over-resistance of the reinforcement, have been evaluated from Priestley et al. (2007), with

$$L_{sp} = 0.022\ f_y \Phi \quad \text{and} \quad k = 0.2\left(f_u/f_y - 1\right) \le 0.08$$

where f_y is the yield resistance of the steel, f_u is the ultimate resistance of the steel, Φ is the diameter of the longitudinal bar.

The behaviour of the mechanical column-to-foundation devices considered in the present paper differ from that of a well-detailed cast-in-situ solution. In particular, post-screwed protruding bars normally have a weakening of the bars, which may concentrate the plastic demand in that small length region or lower the diffusion of plasticity along the bars, depending on the thread depth. Standard bolted sockets are conceived not to develop plasticity along the shear span, concentrating the plastic demand in correspondence of the foundation baranchor. A distinction is made between baranchors fully threaded and threaded only for a portion. If the baranchor is fully threaded, the length of strain penetration may be activated, while if only the outer part is threaded, the diffusion of plasticity depends again on the thread depth. Overturned bolted sockets develop plasticity along the shear span, but the foundation hardware is over-dimensioned with respect to the column one, thus not allowing the length of strain penetration to develop. Couplers make the reinforcement behave like a cast-in-situ one, with the exception of the over-dimensioned height of the device, which does not plastically elongate.

The above-mentioned observations are taken into account in formulating the following proposals for the evaluation of the length of plastic hinge of column-to-foundation precast joints provided with the considered mechanical connection devices:

Protruding bars	$L_p = kL_c + L_{sp}$	$\ge 2L_{sp}$
Post-screwed protruding bars	$L_p = \rho\left(kL_c + L_{sp}\right)$	$\ge \rho 2L_{sp}$
Standard bolted sockets	$L_p = d + L_{sp}$	if all threaded, else
	$L_p = (1 + \rho)d$	$\le d + L_{sp}$
Overturned bolted sockets	$L_p = kL_c$	$\ge L_{sp}$
Couplers	$L_p = kL_c + L_{sp} - s$	$\ge 2L_{sp}$

where d is the free threaded length of the baranchors below the upper nut and above the foundation, s is the over-dimensioned length of the device that remains in elastic field, ρ is the stress reduction factor.

The stress reduction factor ρ, varying between 1 and 0, is defined as follows, assuming a linear hardening branch for steel:

$$\begin{array}{ll} \rho = \left[f_u(A_{core}/A_0) - f_y\right] / \left(f_u - f_y\right) & \text{if } (A_{core}/A_0) \geq \left(f_u - f_y\right) \\ \rho = 0 & \text{if } (A_{core}/A_0) < \left(f_u - f_y\right) \end{array}$$

where A_{core} is the minimum area of the thread core out of the nut and A_0 is the area of the non-threaded bar.

3.6 Numerical modelling

Several non-linear numerical analyses have been carried out to simulate the experimental results and to calibrate the models. Peculiar features of joint devices and in general discontinuities in reinforced concrete structures are suitable to be treated with lumped plasticity models if using traditional beam elements. Macro-models have been chosen with the aim is to derive designer-friendly tools that may be used in commercial codes. The column-to-foundation sub-assemblies have been analysed in SAP2000 (CSI 2010) ambient. A zero-length non-linear link placed at the base is provided with the characteristics of the plastic hinge. A non-linear moment-curvature diagram has been calculated for the critical section, considering the mean resistance values of concrete and steel parameters. A Sargin model (Sargin 1971) has been used for unconfined concrete, while for confined concrete the same model has been modified with a constant stress after peak up to the maximum deformation calculated in accordance with the Model Code (CEB 1985) to take into account the confinement provided by the stirrups. A tri-linear behaviour, considering a linear hardening after yielding and a linear softening after peak has been used for the steel. For post-screwed protruding bars, a direct tensile test on one rebar has been carried out and the experimental resistance has been used to calibrate the model. The yield stress has been taken equal to 510 MPa, the ultimate resistance equal to 640 MPa. For the other specimens there has not been the possibility to test the resistance of the steel of each component. The mean resistance stresses are calculated multiplying the characteristic values (f_y = 450 MPa, f_u = 540 MPa) by 1.08 and used in the analyses for standard bolted sockets, overturned bolted sockets and for couplers.

Figure 12a provides the diagram calculated for the current cross sections of the columns with 8Φ16 and 8Φ24 mm bars subject to an axial compression of 380 kN.

Figure 12b shows the scheme of the numerical model of the column with its support, where the experimental cyclic displacement history is applied to the upper support. The zero-length link at the base of the column has been provided with a non-linear moment-rotation diagram, where the rotation is obtained by multiplying the curvature by the length of the effective plastic hinge. The column has been modelled with an elastic beam element, to which the cracked stiffness has been attributed for the cracked height and the gross stiffness for the upper uncracked portion. For the non-linear hysteretic behaviour, a pivot model (Dowell et al. 1998) has been used within the zero-length element, which hysteretic parameters have been calibrated on the base of the experimental diagrams. Table 1 shows the calibrated hysteretic parameters adopted in the analyses, together with the plastic hinge effective lengths calculated in accordance with the proposed design guidelines formulae. The bending moment vs rotation diagrams assigned to each zero-length non-linear element are shown in Fig. 13. It can be noticed that the maximum rotations significantly differ among the different connection technologies.

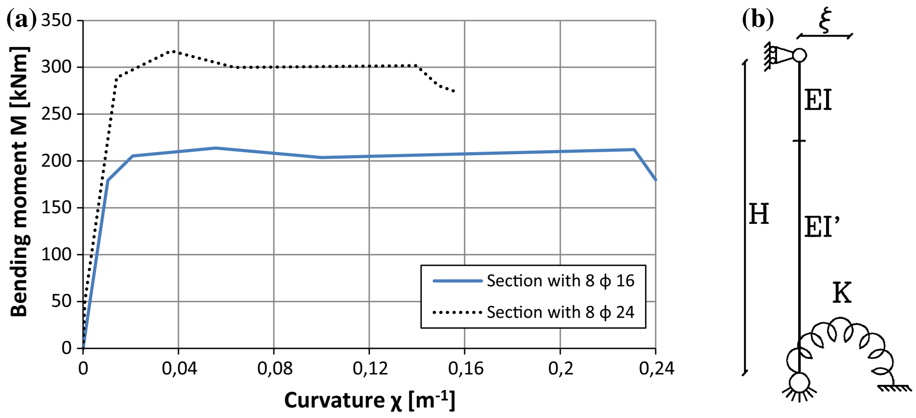

Fig. 12 **a** Moment versus curvature diagrams for the sections reinforced with 8Φ16 mm and 8Φ24 bars, **b** numerical model scheme

Table 1 Features of the zero-length elements in the cyclic non-linear analyses

Connection	Reinforcement	Resisting moment M_{Rd} (kNm)	Lp (mm)	Cyclic parameter α	Cyclic parameter β
Protruding bars	8ϕ16	229.87	359	1.20	0.30
Standard sockets	8ϕ20 (threaded)	248.35	99	1.55	0.40
Overturned sockets	8ϕ16	229.87	179	2.00	0.45
Couplers	8ϕ24	339.00	337	6.00	0.60

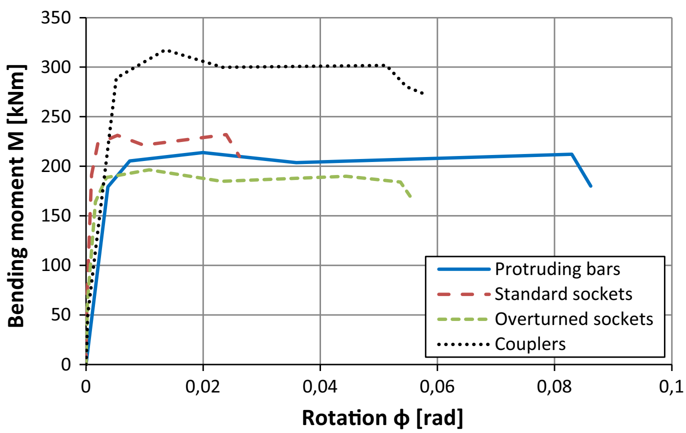

Fig. 13 Moment versus rotation diagrams assigned to each lumped plasticity zero-length non-linear element located at the base of the columns

In Fig. 9 the experimental and the analytical cycles are compared for the different configurations. The numerical results show a good agreement with the experimental ones, with some peculiar differences. In the case of post-screwed protruding separated bars, the extension of the plastic hinge would lead to a larger displacement ductility, while failure did occur at the 4.5 % of drift in the test, due to the early breaking of the separated bars in correspondence of the thread. The effective plastic hinge length applied to the numerical model has not been provided with limitation induced by the ρ factor due to the limited knowledge of the truncated-conic geometry of the thread. Therefore, the numerical analysis would have computed a larger displacement capacity, but the analysis has been limited to the application of the experimental displacement history. This indicates that the use of pressed socket and threaded protruding short bar negatively affects the ductility of the structural assembly. In the case of standard bolted sockets, the behaviour is well predicted by the analysis, including the failure. To be noticed that this connection also reduces ductility with respect to a monolithic connection (for instance protruding bars). For the case of overturned bolted sockets, the behaviour is well predicted with the exception of the premature failure, which is not caught by the numerical model, the main cause of which is the unexpected failure in the socket, that led to a smaller ductility. For the case of couplers, the behaviour is satisfactorily well predicted, and this connection therefore does not affect ductility.

4 Structural performance under seismic action

From the previous considerations on the cyclic behaviour of the column-to-foundation connections it has been shown how the ductility and energy dissipation properties may be sensibly altered if using mechanical devices. From the suggested design rules, it may be noticed how the effective length of plastic hinge does not depend on the height of the column for the solution with standard bolted sockets only. Therefore, the comparison between the experimental results described in the previous chapters is affected by the dimension of the specimens, especially by the limited height of the column. In order to understand how the seismic performance of a full structural assembly is modified by the use of different column-to-foundation connections, the dynamic non-linear behaviour of a structural assembly under seismic accelerogram is studied through numerical analysis. The prototype structure tested at full scale in the ELSA laboratory of the JRC within the Safecast project has been chosen as a reference for the numerical campaign. The building is a regular three stories frame for office use, made with 2 by 2 bays with 7 by 7 m span and inter-story height of 3.4–3.2–3.2 m. More details about the structure (a picture of which is reported in Fig. 14) can be found in Negro et al. (2013), together with a full report of the experimental results. The Tolmezzo earthquake (Fig. 15), with a duration of 12 s and modified to fit the response spectrum given by CEN-EN 1998-1:2004 for subsoil class B has been applied to the frame structure. It has been scaled to different peak ground accelerations, the largest equal to 0.30 g, that corresponds to the structure design PGA for ULS.

A three-dimensional model of the structure has been prepared with the code SAP2000 (CSI 2010). Roof, beam and column members have been modelled with elastic beam elements, providing the rotational degrees of freedom corresponding to the dowel connections. Zero-length non-linear elements have been applied at the base of the columns and provided with non-linear moment-rotation diagrams, similarly to what has been done with the single column cyclic analyses described in the previous clause. The mass is attributed to the horizontal members only, properly enlarged in order to get the global storey mass

Fig. 14 Precast structure full scale 3-storey prototype assembled at the ELSA laboratory of the JRC of the EU in Ispra (Italy)

values (including non-structural dead and live loads in seismic combination) used as input in the pseudo-dynamic tests.

The results obtained for the frame configuration with column-to-foundation protruding bars connections are given in Fig. 16 in comparison with the experimental ones. With the low elastic stiffness of the structure the effect of higher modes is predominant on the response, as it can be noticed by the confused peaks of the base shear-displacement plots. The high structural deformability leads to large displacements. The comparison between numerical and experimental vibratory curves shows that the displacement course has been correctly caught up to about 10 s, after which some stiffening and damping effects lowered the response of the structure diverging from the numerical one. This may be due to stiffening of the dowel beam-to-column connections at large rotation, as also reported in Biondini et al. (2012).

Dynamic non-linear analyses have been further performed on the model with standard sockets, overturned sockets and couplers column base connections. Table 2 lists the cyclic parameters attributed to the pivot model (Dowell et al. 1998) for the different connections and the main results from the analysis. The results are plotted in Fig. 17 with reference to the top displacement history. A very similar displacement history is shown, with small differences between the maximum displacements attained. Much larger differences are noticed in the bending moment-rotation loops of the zero-length elements attributed to the column bases, plotted in Fig. 18 with reference to one corner column.

In particular, it may be observed that very similar loops with respect to the protruding bars solution are obtained with the coupler connections, while larger plastic rotation, to which corresponds damage, may be identified with the use of overturned sockets and still more with the use of standard sockets with baranchors threaded for the protruding part only. The main results of the analysis are reported in Table 3. Very similar maximum bending moments are obtained, with an increase with the use of the standard sockets of

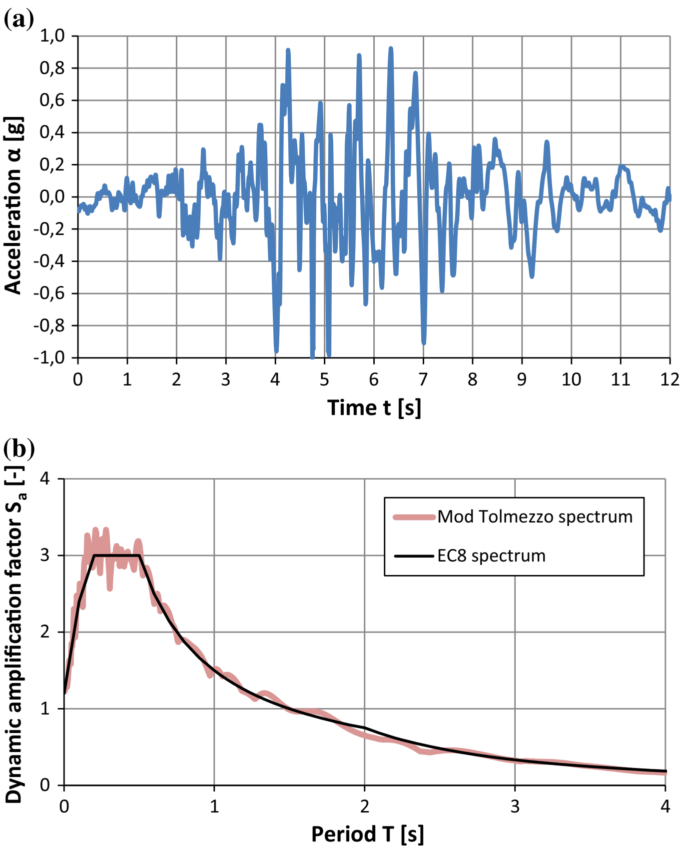

Fig. 15 Modified Tolmezzo earthquake: (a) accelerogram, (b) response spectrum in comparison with the one suggested by Eurocode 8 for subsoil class B

about the 10 % due to the larger net diameter of the threaded baranchor with respect to the longitudinal rebars. Also similar values are obtained for the maximum rotation and the corresponding displacement ductility, with a slightly lower value for the use of standard sockets due to their higher resistance. Very large differences are however found in the rotational ductility demand, with values increased with respect to those obtained with the protruding bars solution equal to 61 % with the use of overturned bolted sockets, due to the reduction of ductility caused by the absence of the plastic strain penetration in foundation, and equal to 491 % with the use of standard sockets, due to the large reduction of plastic strain and to the poorer dissipation of energy.

The global seismic performance of the building is therefore very different, since the same event causes low structural damage to the structure provided with protruding bars and couplers connections, slightly larger damage with overturned bolted sockets connections, and severe damage to that provided with standard bolted sockets connections.

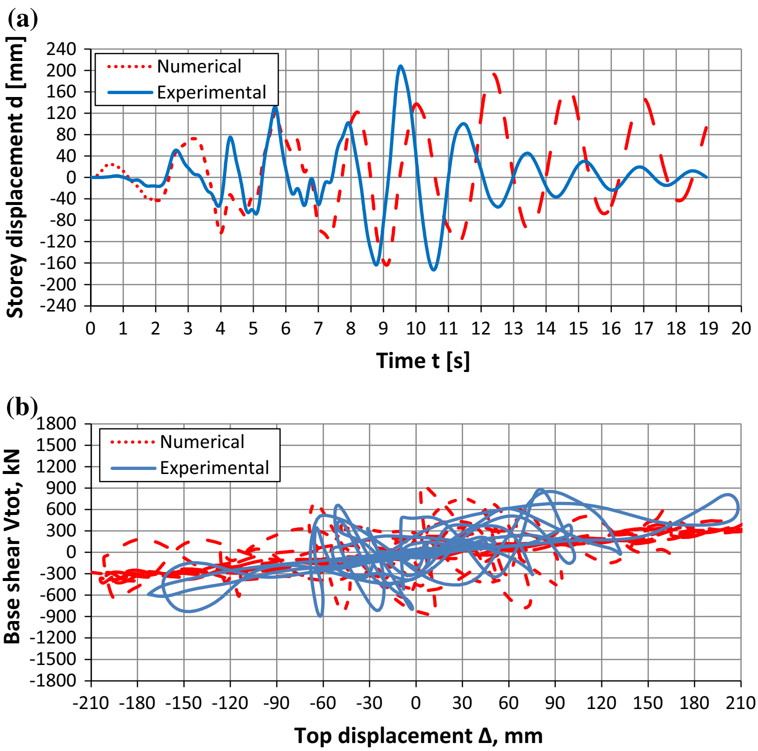

Fig. 16 Comparison between numerical and experimental results for the application of the modified Tolmezzo accelerogram scaled at a PGA equal to 0.30 g: **a** top displacement history and **b** base shear versus top displacement

Table 2 Main results from the dynamic non-linear analyses

Connection	Reinforcement	Lp (mm)	Cyclic parameter α	Cyclic parameter β	Max displacement d (mm)	Max drift ratio δ (%)	Max base shear V (kN)
Protruding bars	8ϕ20	605	1.20	0.30	179	1.94	859
Standard sockets	8ϕ26 (threaded)	105	1.55	0.40	156	1.69	939
Overturned sockets	8ϕ20	380	2.00	0.45	170	1.85	842
Couplers	8ϕ20	575	6.00	0.60	172	1.87	853

5 Conclusions

Several innovative column-to-foundation connection technologies have been experimentally studied with tests on structural sub-assemblies with the aim to characterise their mechanical behaviour under cyclic load, identifying their backbone curve, failure mode, plastic hinge location and hysteretic shape. For all the tested connections the resistance is

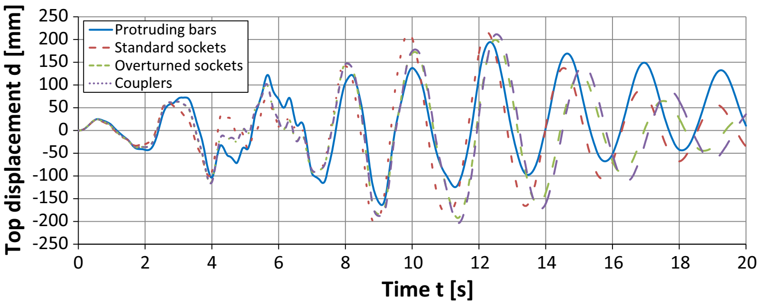

Fig. 17 Comparison of the top displacement responses of the structures provided with different column-to-foundation connections and subjected to the modified Tolmezzo accelerogram scaled at a PGA equal to 0.30 g

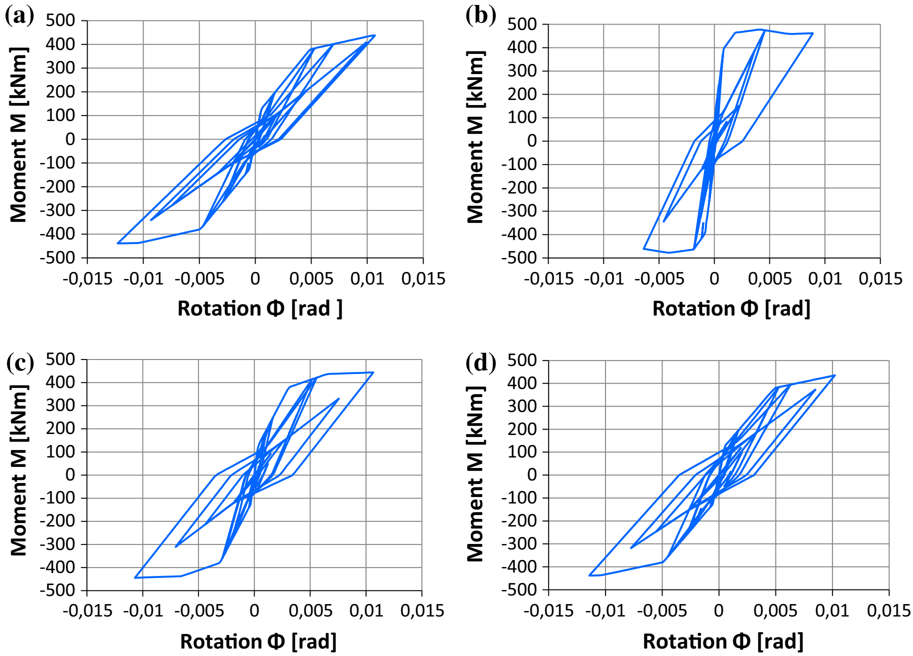

Fig. 18 Local responses at the base of one corner column in terms of moment versus rotation diagrams: **a** protruding bars, **b** standard bolted sockets, **c** overturned bolted sockets, **d** couplers

similar to the expected values. Relevant differences are however recorded on displacement ductility and energy dissipation capacity, mainly because of their effective length of plastic hinge and their cyclic shape, both affecting the energy dissipation properties of the joints.

Some formulae have been suggested for the calculation of the effective length of plastic hinge of a column provided with the different connections. Hysteretic shape parameters have been calibrated on the basis of the experimental results in order to be applied with designer-friendly codes.

In comparison with monolithic solutions, post-screwed protruding bars connections may lead to severe decrease of ductility, unless special care is provided in the manufacturing of

Table 3 Demand on one corner column from the dynamic non-linear analyses

Connection	Max column moment M (kNm)	Max rotation φ (rad)	Rotational ductility demand μ_φ (−)	Displacement ductility demand μ_δ (−)
Protruding bars	343.36	0.011	2.26	1.68
Standard sockets	375.38	0.0091	11.09	1.5
Overturned sockets	348.73	0.011	3.66	1.61
Couplers	342.9	0.0105	2.14	1.71

the thread, like the truncated-cone shaped thread tested, which allows the development of a certain degree of ductility.

Standard bolted sockets connections provide a strong limitation of effective plastic hinge length due to either its concentration in the foundation baranchor and a low hysteretic dissipation capacity, affected by pronounced pinching effects. This solution, over-reinforcing the column base and driving failure to baranchors that are slightly over-dimensioned with respect to the current section bars, also provides a non-negligible increase of resistance with respect to the current column section, which shall be taken into account in the capacity design, for instance of the upper dowel connections.

The proposed solution of overturned bolted sockets improves the response in terms of both ductility and dissipation, even if the absence of strain penetration in foundation leads to a moderate reduction of effective plastic hinge length. The experimental ductility is less than that what expected due to the early unexpected failure of the connecting devices.

Couplers connections provide practically the same effective plastic hinge length of the monolithic solution, given that the device itself is short enough, while a strong reduction in hysteretic energy dissipation properties due to pinching has been observed.

These results can be summarised saying that, due to their small dimensions, the mechanical connections of concern cannot display an overall significant dissipation of energy with respect to the dissipation that the column by itself can display with its flexural behaviour. An adequate length of plastic strain is necessary to transform the local ductility into an overall displacement ductility. For this reason, the mechanical devices at the base joint shall be over-proportioned by capacity design to move the plastic hinge into the column.

The effect of the use of such devices to connect columns and foundations on the seismic performance of real structural assemblies has also been investigated, using the suggested formulae of the plastic length. The results from dynamic non-linear analyses performed on a 3-storey precast structural assembly refer to a single case study not sufficient for a general conclusion. Anyhow, as a first indication, they show that the use of different connections does not influence much the vibratory curves and the maximum drift, but on the other hand the level of local damage attained at the column-to-foundation connection is very different, from low to severe, depending on which connection is used. In particular, the bolted socket solution brought to a large increment of local damage. An important size effect has been observed between the results from the column-foundation joints sub-assembly tests, with column slenderness (length over side ratio) equal to 6.25, and those of the analysis on the structure prototype with much more slender columns, having a ratio equal to 19.2. A wider analysis campaign would be required to broaden the above considerations. In general, tests performed on specimens with low slenderness may lead to the observation of a relatively large ductility, which may be influenced by the base contributions of the connections themselves, which remain the same for all column heights.

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