Seismic Performance of Earthquake-Resilient RC Frames Made with HSTC Beams and Friction Damper Devices

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Seismic behavior of RC Frames with Hybrid Steel-Trussed Concrete Beams is affected by panel zone damage due to a large amount of longitudinal reinforcement. Here the seismic efficiency of innovative frames characterized by Friction Damper Devices (FDDs) at beam-to-column connections is compared against traditional frame. Three configurations are investigated: FDDs alone; FDDs with column-to-foundation connections having preloaded threaded bars and disk springs; FDDs with self-centering friction devices. Non-linear analyses show that FDDs alone prevent plastic hinge formation at beam ends and beam-column joint damage. FDDs with self-centering friction devices effectively limit both peak and residual drifts, avoiding column base plasticization.

Keywords: friction dampers, hybrid steel-trussed concrete beams, cyclic behavior of beam-column joints, self-centering systems, low-damage structures

Introduction

Hybrid Steel Trussed Concrete (HSTC) beams are an effective tool for partial industrialization of the construction process of framed structures. Their use in seismic areas is hampered by the potential damage to the panel zone due to the large amount of longitudinal reinforcement that passes through it, and by the impossibility of placing an adequate amount of stirrups due to reinforcement congestion. In order to cope with this drawback, it is noteworthy that over the last few decades a new design philosophy has been gaining in popularity in earthquake-prone areas, aimed at guaranteeing that the structure experiences negligible damage even after large earthquakes (Takagi and Wada 2019). This behavior is achieved by appropriately placing structure energy-dissipating devices (e.g. yielding, friction, viscoelastic, and viscous ones) able to absorb the seismic energy, thus preserving the primary structural members from damage. This approach answers the need expressed by stakeholders to have buildings which experience little or no damage during the main shocks, and subsequent aftershocks, in order to be operative as soon as possible after a seismic event. As a matter of fact, the design of traditional Moment Resisting Frames (MRFs) is based on a “high damage” concept, namely the seismic energy is absorbed by plastic hinges at beam ends, leading to potentially highly damaged structural members after a strong earthquake. Thus the cost of repairing the structural (and non-structural) damage accrued and that related to the non-operability of the building are often higher than the cost of reconstruction. In light of this, traditional MRFs prove environmentally and economically unsustainable in areas with a high rate of seismic events.

In this context, several studies have been carried out in order to develop devices based on friction, able to dissipate seismic energy (Borzouie et al. 2015, 2016; Khoo et al. 2015; Latour, Piluso, and Rizzano 2015, 2018; Latour et al. 2018; Ramhormozian et al. 2018; Zhang et al. 2016). The main goal of these devices is to dissipate seismic energy by exploiting friction forces generated through sliding of plates made of several materials (e.g.: coated steel, polymer, composite) and clamped together by means of preloaded bolts (Khoo et al. 2012a; Latour, Piluso, and Rizzano 2014; Tsampras et al. 2016).

However, two main drawbacks affect the performance of MRFs having Beam-to-Column Connections (BCCs) realized with Friction Damper Devices (FDDs), namely the formation of plastic hinges at the column bases and the large increment of the average residual interstory drift.

To remedy these defects, MRF Column-to-Foundation Connections (CFCs) or braces which combine friction devices and self-centering systems (e.g. preloaded threaded bars with stacks of disk springs) designed to provide a flag-shaped hysteretic response are employed to reduce structural residual drift ratios (Belleri et al. 2017; Christopoulos et al. 2008; Fan et al. 2019; Freddi, Dimopoulos, and Karavasilis 2017, 2020; Hashemi et al. 2017; Hashemi et al. 2018; Hashemi et al. 2019; Khoo et al. 2012b; Khoo et al. 2013; Latour et al. 2019; Rojas, Ricles, and Sause 2005; Xu, Fan, and Li 2017). Several friction device configurations have been developed for steel structures, starting from the pioneering works of Grigorian and Popov (1994) and Yang and Popov (1995). Solutions have also been proposed for timber structures (Hashemi et al. 2017), while few studies concern friction devices employed in RC structures (Belleri et al. 2017; Morgen and Kurama 2004; Morgen and Kurama 2008; Song, Guo, and Chen 2014; Tsampras et al. 2018; Zhang et al. 2018).

In this context, a new solution for BCCs developed for cast-in-situ RC structures realized using HSTCBs has been proposed by Colajanni et al. (2020, 2021). They analyzed an HSTCB constituted by a spatial steel lattice built using inclined V-shaped rebars (Colajanni, La Mendola, and Monaco 2018a, 2018b; Colajanni et al. 2015). These rebars, which represent the transverse reinforcement of the beam, are also joined at the top to a variable number of bars constituting the upper chord by means of fillet welds, and at the bottom to a plate such as is usually employed in constructional steelwork. The above-described truss is made up in a factory and then the beam is completed with cast-in-situ concrete.

Thanks to reinforcement formed by a steel truss, HSTCBs are able to cover long spans with small section depths, often contained within the thickness of the slab. This characteristic requires the use of a large amount of reinforcement in the beam-column joints, often employing large diameter bars. Thus, both the beam ends and the joint panel become vulnerable to the effects of cyclic actions, like those induced by seismic excitation. Even if at the beam ends the presence of a properly-designed transverse reinforcement, which provides confinement to the concrete, is usually able to reduce the loss of both stiffness and strength due to cyclic actions, large diameter bars inside a smallsized joint panel cause concrete cracking and damage, inducing a loss of bond. This phenomenon causes degrading hysteresis cycles, which may lead to a reduced structural dissipative capacity (Colajanni et al. 2016).

To prevent this phenomenon, the use of a suitably designed friction BCC, characterized by an increased lever arm of the bending moment transferred from the beams to the joint, reduces the shear forces acting on the joint panel, preventing it from being damaged. This target is hampered by the uncertainties affecting friction devices due to variability in the application of the bolt preload and to the value of the friction coefficient of surfaces involved in sliding (Ferrante Cavallaro et al. 2015, 2018). Thus, with the aim of keeping the RC members in an almost elastic field, they have to be properly overstrengthened.

Recently, Pagnotta et al. (2019) compared the seismic behavior of RC frames realized with HSTCBs endowed or not with friction devices. However, since the latter were arranged at the BCC only, the column bases experienced severe plastic deformations, and the frame exhibited residual interstory drift, obtaining an overall performance of the innovative frame that was not completely satisfactory.

Within this framework, the aim of this paper is to show that the use of FDDs at the BCC is a suitable solution to prevent degrading performance due to cracking and bond losses, caused by the high percentage of reinforcement that characterized the panel zone of MRFs with HSTC beams. Moreover, it will be shown that excellent seismic performance can be obtained only when FDDs at the BCC are used in conjunction with a self-centering dissipative connection at the column bases. To this aim, comparison of the seismic performance of traditional and innovative RC frames built using HSTCBs is performed. The comparison focuses on the different level of damage experienced by RC beams, columns and panel zones belonging to traditional and innovative frames. The latter are characterized by the use of the FDD reported in Colajanni et al. (2020, 2021) specifically developed for RC beams realized with HSTCBs. Three different configurations of the Innovative Frame (IF) are investigated. The first one (IF) only has FDDs at the BCCs. The second one (IF-TB), along with the FDDs, is equipped with preloaded threaded bars and disk springs at the CFC. The third one (IF-FD), beside the FDDs at the BCCs, is characterized by a CFC equipped with an adapted version of the self-centering friction connection used in steel structures and proposed by Latour et al. (2019).

The seismic response of RC frames is assessed by means of both pushover analyses and Non-Linear Time History Analyses (NLTHAs). The former are carried out to prove that the use of innovative systems in MRFs is effective in preventing damage to RC members for drift values up to attainment of the ultimate rotation of these systems. NLTHAs are carried out to assess the global and local responses of the different frames subjected to seismic excitation. To highlight the advantage in using the innovative systems, the different amount of damage undergone by the RC members belonging to the four types of frames is calculated by means of the Park and Ang Index (Park and Ang 1985) for beam ends, column bases and joints.

The outline of the paper consists of five sections in which firstly the different components constituting the RC frames are described and the RC frame models characterized and, secondly, the analysis of the results is discussed. More precisely, in the first section the macro-modeling approach that reproduces the cyclic behavior of RC beam-column joints is discussed. Then the simulation of a previously carried out experimental test (Colajanni et al. 2016) is performed using the above approach. After that, the innovative systems, RC frames, and seismic input are described. Then the parameters characterizing the cyclic response of FDD and CFCs are calculated and the validation of the numerical models used is carried out. In the last section, the results of the seismic response analyses performed for the four types of RC frames are commented on.

Macro-modeling of the cyclic behavior of RC beam-column joints

The first step to carry out the comparison between RC frames with HSTCBs endowed or not with friction devices is to properly model the cyclic behavior of HSTCBs and RC beam-column joints.

With regard to HSTCBs, several studies have been carried out focusing on their mechanics over the last decade. In detail, shear and flexural behavior were analyzed in Colajanni, La Mendola, and Monaco (2015), Monaco (2016) and Ballarini et al. (2017), the phenomenon of buckling of the inclined bars constituting the steel truss was studied in Colajanni, La Mendola and Monaco (2014) and Colajanni et al. (2015), and modeling of the bonding between concrete and steel was analyzed in Colajanni et al. (2015) and Colajanni et al. (2018a, 2018b). In Colajanni, La Mendola, and Monaco (2015) the effectiveness of a fiber beam-column element in reproducing the cyclic hysteretic flexural behavior of HSTCBs is shown.

Reliable models of seismic response of MRFs have to take into account that beam-column joints can experience severe shear distortions, often manifesting stiffness and strength loss, significantly contributing to interstory drifts (Pan, Guner, and Vecchio 2017). In order to validate the FEM model reproducing the cyclic behavior of beam-column joints of traditional RC frames made with HSTCBs, the results of a previously carried out experimental campaign are used, reported in Colajanni et al. (2016).

Experimental test

The subassemblies tested were representative of four-way beam-column joints, constituted as follows (Figure 1): two half-columns having cross-section 300 mm width and 400 mm depth, reinforced with 10 rebars of 20 mm diameter; two HSTCBs made with a truss having a 5-mm-thick lower steel plate, 3ϕ16 bars constituting the upper chord and inclined transverse ϕ12 bars positioned at a 300 mm spacing. The beam cross-section is equal to 300 × 250 mm. The bending moment resistance of the beam end section is obtained ignoring the truss contribution and considering only the added top and bottom longitudinal reinforcement, namely 4ϕ24 and 2ϕ24 bars, respectively.

The results showed degrading behavior in terms of strength, stiffness and pinching of hysteretic cycles. Among the three subassemblies tested, specimen no. 2, whose column was subjected to an axial load equal to 800 kN, was selected in order to calibrate the cyclic behavior of the joint. The scheme of the constraints, load condition and geometrical characteristics of specimen no. 2 are reported in Figure 1.

Mechanical model

With the aim of taking into account the above-mentioned degrading phenomena in the analysis of RC frames, according to the suggestions of Pan, Guner, and Vecchio (2017), the mechanical model proposed by Lowes and Altoontash (2003) was adopted. This model, represented in Figure 2, was constituted by a four-node, 12-DOF super-element which comprises the following:

- A 2D element subjected to shear action only which mimics the stiffness and strength deterioration exhibited by the concrete core of the beam-column joint;
- Eight linear springs reproducing the strength and stiffness degradation caused by slippage of the longitudinal reinforcement within the concrete core due to bond stress reduction;

- Four linear springs emulating the decreased ability to transfer shear actions at the joint perimeter because of crack opening.

This model employs the Modified Compression Field Theory (MCFT) (Vecchio and Collins 1986) to evaluate the monotonic curve of the shear-stress vs. shear-strain relationship of the concrete core within the beam-column joint. As illustrated in Figure 2, the 2D element transfers all loads via shear stresses. Consequently, the MCFT is employed setting to zero the normal stresses acting on the concrete shell, and neglecting the normal strains. In the model proposed by Lowes and Altoontash, the cyclic response of the joint core is calibrated using the experimental study carried out by Stevens, Uzumeri, and Collins (1991). In the present paper, with the aim of simplifying the study, the parameters proposed by Sivaselvan and Reinhorn (2000) were employed, as will be discussed below.

The monotonic curve of the bar-stress vs. the bar-slip history is defined assuming several simplifications regarding anchorage behavior (Lowes and Altoontash, 2003). First of all, the bond stress throughout the anchorage zone of the longitudinal reinforcement was supposed to be constant if the bar was loaded below the elastic limit, or piecewise constant for bars experiencing yielding. Secondly, the bar slip determines the relative displacement between the bar and concrete core perimeter and is related to the strain state experienced by the bar. Thirdly, no slip is assumed to occur at the section of zero normal stresses. In Figure 3 bond stresses and stress state along the anchorage zone of a longitudinal reinforcement anchored within the concrete core of a beam-column joint loaded over the elastic limit are schematically illustrated. The symbols used in Figure 3 are defined as follows: - f_s: bar stress at the concrete core perimeter; - f_y: the steel yield stress; - τ_E: bond stress in case of elastic steel; - τ_Y: bond stress in case of yielded steel; -

A_b: rebar cross-sectional area. Moreover, l_e and l_y, are the rebar segments whose stress is below or beyond the yield value.

Once these simplifications have been defined, the equations which relate bar-stress and bar-slip in the monotonic field are equal to:

$$
\begin{aligned}
s &= \frac{d_b\, f_s^{\,2}}{8E\,\tau_E} & f_s \le f_y \\
s &= \frac{d_b}{4}\left\{\frac{f_y^{\,2}}{2E\,\tau_E} + \frac{f_s - f_y}{\tau_Y}\left[\frac{f_y}{E} + \frac{\left(f_s - f_y\right)}{2E_h}\right]\right\} & f_s > f_y
\end{aligned}
\qquad (1)
$$

in which E is the steel elastic modulus; E_h the strain hardening modulus; and d_b the nominal rebar diameter.

It is well known that bond resistance drops when reinforcing bars experience a certain value of slippage (Eligehausen, Popov, and Bertero 1983). As suggested in Lowes and Altoontash (2003), the slip threshold after which the bond strength decays is assumed equal to 3 mm.

With respect to the bond strength values to be adopted in Eq. (1), an experimental campaign focusing on anchorage behavior of reinforcing bars (e.g. Eligehausen, Popov, and Bertero 1983) pointed out that the bond strength is related to both the concrete condition and the state of stress surrounding the anchored-length bar and transversal reinforcement around the anchored bar. In this paper, consistently with Lowes and Altoontash (2003), values of 1.8 $f_c^{0.5}$ and 0.2 $f_c^{0.5}$ were adopted for τ_E and τ_Y, respectively, f_c being the compressive strength of concrete, and linear springs modeling the capacity to transfer shear actions at the joint perimeter are assumed to be elastic.

Subassembly modeling and validation

The macro-model described in the previous section is implemented in FE software, and it is validated by comparison of numerical and experimental cyclic curves. In this study, both the benchmark test on the subassembly and the analyses in the following on RC frames were carried out using the SeismoStruct software (SeismoSoft, 2020). Structural members were modeled using distributed plasticity fiber-section elements with force-based formulation, while 2D shear-panel component and bar-slip linear springs were modeled by means of link elements. The latter are 3D elements, having uncoupled axial, shear and flexural behaviors, linking two coincident structural nodes defining a force-displacement or moment-rotation response relationship which is independent for each of its 6 DOFs. In Figure 4 the structural model employed to adapt the aforementioned beam-column joint model to the software is illustrated.

It is constituted by the following:

- Four rotational springs at the beam-to-joint and column-to-joint interfaces to simulate the relative rotation of the structural member due to slippage of the longitudinal reinforcement within the concrete core;
- One rotational spring at the center of the beam-column joint to simulate the relative rotation between columns and beams due to shear deformation of the joint region.

Moreover, four rigid links were introduced between central nodes and nodes on the beam-to-joint and column-to-joint interfaces, assuming that the cyclic behavior of the joint is totally represented by the central rotational spring. The rotational spring of the panel zone was modeled by means of a link element in which a moment-rotation behavior is defined using the “multi-lin” constitutive law. The latter is based on the Polygonal Hysteretic Model (PHM) introduced by Sivaselvan and Reinhorn (1999), which is able to simulate both stiffness and strength deterioration and the pinching phenomenon. The parameters able to model the cyclic behavior are calibrated using the values suggested in Sivaselvan and Reinhorn (2000). With regard to the bar-slip mechanism, the "gen-hyst" constitutive law is adopted. The parameters governing the cyclic behavior of the bar-slip mechanism are tuned on the basis of the experimental results. For both the constitutive laws, parameters through which the backbone curve is defined are computed using the procedures described in the former section. The parameters used for the panel zone and the bar-slip mechanism are reported in Table 1 and Table 2, respectively.

The concrete model adopted is the one proposed by Mander et al. (1988) modified with the cyclic rules reported in Martinez-Rueda and Elnashai (1997) (SeismoSoft, 2020).

Due to the high level of preload applied on the column and the high ratio between the bending moment resistance of column and beam, preliminary analyses showed that partialization of the column cross-section does not occur. Therefore, for the sake of simplicity, the rotational spring representative of the bar-slip mechanism at the column-to-joint interface is not modeled.

A comparison between experimental and numerical curves is reported in Figure 5. It can be seen that the structural model is able to reproduce stiffness and strength deterioration and pinched hysteretic cycles. The difference between the curve with and without damage to the panel zone highlights the contribution of the joint to the overall response of the subassembly.

Moreover, the energy dissipated by the subassembly during the test for each loading cycle for both experimental and numerical results is compared in Figure 6. Regarding the capability of the model in reproducing the dissipated energy by the subassembly, the numerical results are in good agreement with the experimental ones, although some discrepancies can be observed for each loading cycle. The maximum value of the ratio between the difference of the dissipated energy during the i^{th} cycle calculated from the experimental and the numerical results, and the total energy dissipated during the experimental test, is observed at the 6th cycle, and is about 6%. The trend of the energy dissipated by the subassembly during the test versus cumulative displacement is shown in Figure 7. It can be seen that, despite some differences, the overall trend is similar and the total amount of energy dissipated at the end of the test is almost equal.

Friction device and self-centering systems

A thorough analysis of the FDD developed for HSTCBs can be found in Colajanni et al. (2020, 2021). Here, a brief description concerning the configuration of the device is reported. The dissipative BCC represented in Figure 8 is constituted by the following components: the friction connection on the lower part of the beam, constituted by a vertical central steel plate with curved slotted holes and steel angles, connected by high strength bolts properly preloaded according to the slip force required; the upper T-stub connection anchored to the column and bolted to a C-shaped steel profile, which is welded to the upper longitudinal rebars of the steel truss of the HSTCB. Once the slip friction force is achieved, the dissipative system rotates around a center of rotation located at the base of the T-stub, in which a reduced section able to favor formation of the plastic hinge has been created. To enhance the stiffness between the vertical central plate, steel truss top chord, and the concrete block, rebars with varying inclinations are welded to the upper part of the lower steel plate and to the top chord of the truss. T-stub and steel angles are anchored to the column by means of threaded bars, which run throughout the section height of the column and are connected on the opposite side of the column using steel counterplates. Using the proposed friction device at the BCC lengthens the internal lever arm of the moment acting on the joint, thus reducing the diameter of the threaded bars d_{tb} connecting the FDD to the column. In this way the detrimental effect of the bar-slip mechanism on the overall cyclic performance of the joints is reduced. Indeed, a lower ratio (e.g. < 0.2 as suggested by ACI 318) between d_{tb} and the height of the column cross-section leads to improved cyclic response of joints.

The hogging yield moment of the aforementioned HSTCB is equal to M_{Rd} = 165 kNm. Therefore, the design value of the bending moment to activate slippage of the friction device is set equal to $M_d = M_{Rd} / \Omega_\mu = 110$ kNm, where Ω_μ is the overstrength coefficient equal to 1.5, introduced to take into account the variability of both bolt preload and static and dynamic friction coefficients. The overstrength coefficient should be obtained through experimental tests carried out on FDDs. In the absence of the latter, the aforementioned value is selected on the basis of the values proposed by other Authors (Khoo et al. 2015; Latour et al. 2018). To ensure the same yield moment in the case of a sagging bending moment, 2ϕ14 rebars are added at the bottom chord of the HSTCB.

As reported in Colajanni et al. (2020, 2021), the cyclic response of the HSTCB endowed with the proposed friction device calculated by means of a FEM analysis satisfies the design requirements, i.e. it exhibits a symmetric response for a hogging and sagging bending moment and does not evidence any damage in the loading-unloading phases.

Within the framework of the design of earthquake-resilient structures, frames have to be coupled to self-centering systems in order to limit the residual drift within a maximum allowable threshold (e.g. 0.5%, as suggested by McCormick et al. (2008)) for which the structures can be considered functional. For this reason, two different self-centering solutions, both acting at the column bases, are investigated in this paper. They are both adaptations of one of the solutions developed in the last few years for steel structures to RC ones. The first solution is constituted by a column splice with cover plates on both flanges and web, having the holes on the column slotted to ensure the rotation of the connection (Figure 9). Bearing type connections transfer shear forces between a standard base joint and the column. By means of threaded bars and disk springs, this CFC has the advantage of providing an elastic bilinear moment-rotation response, avoiding formation of a plastic hinge at the column base. However, since all the elements involved behave in an ideal elastic manner, the connection has no dissipative capacity. The numerical analyses performed will show that this solution is not efficient enough to ensure limited maximum interstory drift and residual displacements.

Thus, in the second solution the bearing type connections of the above CFC with preloaded threaded bars coupled with disk springs are substituted by friction pads with preloaded bolts. Preloaded threaded bars and disk springs ensure self-centering behavior of the whole connection, while friction pads provide the required dissipative capacity. In the present paper, the solution reported in Latour et al. (2019) is adapted to an RC column. Since the detailed design of each element of the connections falls outside the scope of the present paper, only the design of the main elements constituting the connection will be performed, and the macroscopic behavior of the above-mentioned connections, namely their moment-rotation behavior, will be studied in the following section.

RC frame and seismic input

In order to perform a comparison between the seismic response of RC frames endowed or not with friction devices, a generic RC frame having two stories 3 m high and two spans 5 m long is considered. This frame is the 1-y (or the equivalent 4-y) frame of the spatial structure having three bays in the y-direction and two in the x-direction whose structural model is represented in Figure 10. The column longitudinal and transverse reinforcement is that of the subassembly described in Section 2, except for columns of 2/3-y frames, whose cross-sectional dimensions are 400×500 mm. All the columns have the strong axis oriented along the x-direction, excepting the four external columns of the six that belong to the 2/3-y frames. The structure is located in Reggio Calabria (Italy), which is a highly seismic area, whose elastic and design spectra are reported in Figure 11. The behavior factor *q* used is 3.

The dead load of the floor and the live load acting on it in the seismic load combination is equal to 14.5 kN/m^2. The one-way slabs are supported by the beams oriented along the y-direction. Therefore, the top and bottom reinforcement of the beams oriented along the x-direction are the same and equal to 4ϕ24. The column bases are fixed and a Rayleigh damping equal to 5%, calculated on the basis of the 1^{st} and 2^{nd} periods of vibration of the structure, is used. The tributary area for seismic masses belonging to the investigated RC frame leads to a distributed mass of 3.8 ton/m added on each beam, in addition to the dead load. Four types of frame are considered: the first one is the Traditional Frame (TF), the second one is the Innovative Frame with FDDs at the BCCs (IF), the third one is the Innovative Frame + preloaded Threaded Bars and disk springs as self-centering system (IF-TB), and the fourth one is the Innovative Frame + the system constituted by preloaded threaded bars, disk springs and Friction Devices (IF-FD). The fundamental period of the TF is 0.5 sec, while that of the IFs, due to the lengthened panel zone and, consequently, shortened column, is 0.48 sec.

The seismic input is modeled by 30 artificial accelerograms, generated using the procedure developed by Vanmarcke and Gasparini (1977) in the recursive form proposed by Cacciola, Colajanni and Muscolino (2004). The modulating function in the amplitude is the one proposed by Jennings, Housner and Tsai (1969). Each accelerogram has a duration of 30 seconds, with a strong motion phase of 20 seconds. With the aim of assessing the residual drift of the frame, 5 seconds are added at the end of the seismic action in which the structure is free to oscillate. Preliminary analyses showed that, when subjecting the frames to generated accelerograms spectrum-compatible with the code elastic spectrum, limited damage was registered in the structural members. In order to investigate the system response to destructive seismic events, able to produce drift similar to that attained during the experimental test, which is about 2.5%, thus also causing, extensive cracking of panel zones and reinforcement bond losses, in addition to formation of plastic hinges, the accelerograms were scaled to 170% of the design intensity. In Figure 11 the response spectra and the average one of the 30 accelerograms generated are shown, while in Figure 12 one of these acceleration time histories is reported.

Innovative Frame model (IF)

In Figure 13 the node modeling scheme in the case of an RC frame with friction device is shown. The panel zone is modeled as illustrated in Figure 4, with a non-linear rotational spring representative of joint shear deformation, while the friction device is represented by means of a bilinear kinetic rotational spring. The parameters of the former are reported in Table 1, calculated consistently with the procedure described in the previous sections, while those of the latter are reported in Table 3, defined in order to simulate the behavior of the friction device, i.e. rigid in the elastic branch and with very low stiffness in the post-yielding branch. As already discussed, the rotational springs representative of the bar-slip behavior at the column-to-joint interface are not modeled.

Innovative Frame model with self-centering Friction Device (IF-FD)

For brevity's sake, the design and modeling of an IF-FD are now reported, and those related to an IF-TB are presented below as a particular case of IF-FD.

The self-centering friction connection applied to the column bases of the IF-FD is modeled by the non-linear rotational spring employing the "scb" constitutive law; its parameters were computed referring to Figure 14. The self-centering friction connection was designed according to the procedure reported in Latour et al. (2019), accepting the simplifications proposed there. The design moment strength of the connection M_d^{FD} was set equal to 200 kNm, which corresponds to 75% of the bending moment resistance of the column. In order to ensure the self-centering behavior of the connection, the moment resistance value provided by the threaded bars and the axial force (decompression moment M_0) must be greater than or equal to that provided by the friction pads, M_1. In this case, the former is set to 60% of the total moment resistance value, i.e. M_0 = 120 kNm. The design shear resistance value V_d^{FD} is equal to $2\Omega\, M_d^{FD} / L$ = 200 kN, where Ω is an overstrength factor equal to 1.5, and L is the column height equal to 3 m. The shear force must be absorbed by the web friction pads, which are designed with four class 10.9 M16 bolts per side, and two friction interfaces, the configuration being symmetrical. Consistently with what is suggested by Ferrante Cavallaro et al. (2015, 2018), the clamping force should be in the range 30-60% of the clamping force calculated according to EN 1993:1-8 (2005), in order to reduce the preload loss. For this reason, the ratio between the effective and maximum clamping force acting on the bolts employed in the web friction pads is computed as follows:

$$r_w = \frac{V_d^{FD}}{\mu\, n_{ws} n_{wb} F_p} = 0.57 \quad (2)$$

μ being the friction coefficient equal to 0.4, n_{ws} the number of friction interfaces, n_{wb} the number of bolts, and F_p the code-consistent clamping force equal to 110 kN. The moment strength provided by the web friction pads was:

$$M_{1,w} = r_w \mu\, n_{ws} n_{wb} F_p \frac{h}{2} = 40\, kNm \quad (3)$$

where the lever arm of the friction force is approximated to h/2. The moment resistance provided by both flange and web friction pads being M_1 = 80 kNm, the contribution due to the flange friction pads is thus equal to $M_{1,f}$ = 40 kNm. This value is obtained by using four class 10.9 M16 bolts per side, with two friction interfaces. The ratio between the effective and maximum clamping force of the latter is obtained as follows:

$$r_f = \frac{M_{1,f}}{\mu n_{fs} n_{fb} F_p h} = 0.29 \quad (4)$$

in which n_{fs} is the number of interfaces and n_{fb} the number of bolts both referred to the flange. The moment strength associated to the threaded bars $M_{0,tb}$ is the difference between the decompression moment M_0 and the bending moment associated to the axial force acting on the column, $M_{0,af}$. For the lateral column N_l = 200 kN; thus approximating the lever arm of the axial force to h/2, the values of $M_{0,af}$ = 40 kNm and $M_{0,tb}$ = 80 kNm are obtained. Therefore, using four M30 threaded bars class 10.9, the ratio between effective and maximum preload force to be applied to each bar is given by:

$$r_{tb} = \frac{M_0 - M_{0,af}}{n_{tb} F_{tb} \frac{h}{2}} = 0.26 \quad (5)$$

where n_{tb} is the number of bars and F_{tb} = 393 kN the maximum preloading force. Hence, the effective preload force is $F_{t,eff} = r_{tb}\ F_{tb}$ = 100 kN. With the aim of ensuring the elastic behavior of the threaded bars, a group of disk springs has to be designed and coupled to the bars. The ultimate rotation of the system (θ_u = 40 mrad) leads to a gap-opening Δ of 8 mm. A standard disk spring with internal diameter 31 mm, external diameter 76 mm, overall height 8.3 mm and thickness 6.6 mm is selected, having a stiffness of about 60 kN/mm. In order to satisfy the abovementioned strength and displacement requirements, 12 groups in series of 3 disk springs in parallel are coupled to each bar, leading to global stiffness K_{ds} of the disk springs of 15 kN/mm. The last step to define the moment-rotation behavior of the connection is to calculate the stiffness of the branch after attainment of M_d^{FD}. Firstly, the stiffness of the threaded bar is computed as follows:

$$K_{tb} = \frac{E_{tb}A_{tb}}{l_{tb}} = 214\,kN/mm \quad (6)$$

Then, the rotational stiffness of the connection considering both the stiffness of the threaded bars and the group of disk springs is expressed as:

$$K_{\theta,2}^{FD} = \frac{1}{\frac{4}{h^2}\left(\frac{1}{n_{tb}K_{tb}} + \frac{1}{n_{tb}K_{ds}}\right)} = 2261\,kNm/rad \quad (7)$$

Lastly, the bending moment value at the ultimate rotation is M_u^{FD} = 290 kNm. To prevent the formation of a plastic hinge at the column base up to ultimate rotation, 2ϕ20 are added to the column in order to increase the moment strength along the strong-axis direction.

The parameters characterizing the "scb" constitutive law employed in the rotational spring elements used to model the above-described connection are summed up in Table 4.

Innovative Frame model with preloaded threaded bars (IF-TB)

The parameters characterizing the connection are the same as the above-mentioned self-centering friction connection, except for the contribution to the moment strength provided by the friction devices, which in this connection are replaced by bearing type connections with un-preloaded bolts. Therefore, the design moment strength of the connection M_d^{TB} is equal to 120 kNm, given by the preload acting on the threaded bars and the axial force acting on the column. The stiffness of the post-yielding branch is $K_{\theta,2}^{FD}$, and thus the ultimate moment strength of the connection M_u^{TB} is 210 kNm.

Validation of numerical models

To validate the design procedures presented above, a comparison between FEM results/ experimental tests and numerical models of the friction device and the self-centering friction connection is carried out. More precisely, as for the friction device, the cyclic behavior of the FEM model presented in Colajanni et al. (2021) and that of the numerical model employed in this paper are compared and reported in Figure 15a. Regarding the self-centering friction connection, the cyclic behavior of experimental test no.1 presented in Latour et al. (2019) and that of the numerical model used in this paper on the basis of the parameters suggested in Elettore et al. (2019) are compared and reported in Figure 15b. Both the comparisons confirm the reliability of the adopted design procedures.

Analysis of results

The performance of the four types of frames are assessed by means of pushover analyses and NLTHAs.

Pushover analyses calculate the yielding and ultimate rotations of plastic hinges by means of the equations reported in EN 1998 (2004). Generally speaking, the group of frames endowed with innovative devices provides capacity curves with stiffer elastic branches, if compared to that of the traditional frame, because of the shorter columns due to the lengthened panel zones (Figure 16). Moreover, the innovative frames show a lower maximum base shear due to the lower design moment strength of the friction devices, chosen in order to ensure the capacity design criterion. This means that innovative frames are designed assuming a higher behavior factor q than that of the traditional frame. Different behavior of the curve can also be noticed once yielding of the CFC is achieved. In fact, the IF-TB and the IF-FD show hardening behavior in the post-yielding branch of the capacity curve, ensured by the stiffness of the post-yielding branch of the CFC employed, able to overcome the P-Δ second-order effect. By contrast, the IF shows softening behavior due to the degrading strength of the plastic hinge at the column base and the Elastic-Perfectly Plastic (EPP) behavior of the BCC. As regards the TF, the hardening behavior of the HSTCB plastic hinge compensates for the P-Δ second-order effect, exhibiting an EPP capacity curve. None of the RC members belonging to the IF-TB and IF-FD undergoes plastic hinge formation for a global drift higher than 5% (top displacement = 0.30 m). Above this drift value, consistently with design provisions, ultimate rotation of the friction devices is achieved.

Concerning NLTHAs, several parameters were inspected with the aim of understanding the local and global behavior of the above-described structures. The structural parameters investigated are the following: mean and CoV of Maximum Interstory Drift Ratios (MIDRs) and Residual Interstory Drift Ratios (RIDRs); moment-rotation curves of beam end sections, column-base sections, link elements representing panel zone, bar-slip mechanism, friction device and self-centering systems. Moreover, the Park and Ang damage Index (PAI) (Park and Ang, 1985) was evaluated for the main elements experiencing cyclic actions.

In Table 5 average and CoV of MIDRs and RIDRs of the four types of RC frames analyzed are reported. As can be seen, similar average values of MIDRs are provided by TF and IF, with a slight decrement of the values in the case of frames endowed with friction devices. However, the average RIDRs obtained with the IF are between three- and four-fold higher than those provided by the TF, with almost half of the analyses providing RIDR values higher than the permissible residual drift considered, i.e. 0.5%.

Looking at the results of the IF-TB, it can be noticed that the adoption of a self-centering system with a much lower yield moment led to an increase of the average

MIDR. Nevertheless, the RIDR average values are considerably lower if compared to those of the IF, proving the efficiency of the self-centering system. On the other hand, comparison with the results of the TF highlights behavior which is not ideal, with a reduction of the RIDR average value at the first floor, and an increment of that at the second floor. Indeed, 3 analyses give RIDR values of the second floor beyond the permissible threshold. Among the four configuration of frames investigated, the most satisfactory results are provided by the IF-FD. As a matter of fact, the IF-FD gives practically the lowest mean MIDRs and similar mean RIDRs to those of the TF, and only one of the RIDR values is greater than the permissible value of 0.5%. These results highlight that the combined use of innovative systems both at the BCC and CFC, with yielding strength lower than the strength of the connected members, is able to limit MIDR and RIDR. It must be stressed that the main goal of the adoption of friction devices and self-centering systems is not to mandatorily reduce MIDR and RIDR, compared to those obtained with traditional frames, but to mitigate damage in structures.

In order to stress achievement of this goal, the different sources of energy dissipation for the four RC frame configurations are highlighted by comparison of the hysteresis cycles of dissipative elements for one of the accelerograms. For the sake of brevity, only the moment-rotation curves of the following elements are reported:

- Right end of the first-story second-span beam (Figure 17a for TF, Figure 17b for IF);
- Link element representing the beam-column joint shear distortion of the first-story internal joint (Figure 17c for TF, Figure 17d for IF);
- Link element connected to the right end of the aforementioned beam representing the bar-slip mechanism (in the case of the TF) (Figure 17e);

- Link element connected to the right end of the aforementioned beam representing the friction device (in the case of the IF) (Figure 17f);
- Link element connected to the base of the central column representing the preloaded threaded bars and disk springs (in the case of the IF-TB) (Figure 18a) and the self-centering friction connection used in steel structures (in the case of the IF-FD) (Figure 18b);
- Base of the first-story central column (Figure 18c for TF, Figure 18d for IF, Figure 18e for IF-TB, Figure 18f for IF-FD).

The moment-rotation curves are selected considering the same time-history analysis. The investigated beam end of the TF (Figure 17a) underwent slight plastic deformations due to yielding of the bottom longitudinal bars. By contrast, consistently with design provisions, the investigated beam end of the IF (Figure 17b) showed an elastic moment-rotation curve.

Regarding the link elements representing the cyclic behavior of the beam-column joint investigated, it can be seen that the panel zone of the TF (Figure 17c) experienced loss of strength and stiffness, achieving a significant level of damage. By contrast, the panel zone of the innovative frame (Figure 17d) did not reach the yield moment; thus no degrading phenomena affecting the strength and stiffness were registered. As a result, the damage undergone by the panel zone was negligible, being limited to crack formation.

In the TF, the link element representing the bar-slip phenomenon (Figure 17e) showed the first appearance of a degrading cyclic behavior, contributing to the global level of damage experienced by the beam-column joint of the TF.

With respect to the link element representing the friction device (Figure 17f), it can be noticed that the seismic energy previously absorbed by the panel zone and the plastic hinges of the beams is dissipated, in the IF, by the friction devices. The hysteretic cycles of the FDD are wide and stable, having assumed that the cyclic performance of the device is not dependent on the cumulative displacement experienced.

A comparison between Figure 18a and Figure 18b highlights that the connection with preloaded threaded bars and disk springs used in IF-TB undergoes lower maximum moment and larger rotation, if compared to those experienced by the self-centering friction connection used in IF-FD. Thus, a reduced overstrength factor could be used in the design of the column base. However, the lower yield moment leads to a weaker frame and, as already seen in Table 5, higher average MIDR values. Moreover, the system constituted by threaded bars and disk springs only is not ideally able to dissipate energy, unlike the self-centering friction connection, which dissipates a considerable amount of energy thanks to its flag-shaped moment-rotation curve.

As for the moment-rotation curves of the column bases illustrated in Figure 18c-f, it can be stated that the use of the FDD at the BCC alone in the IF does not prevent formation of a plastic hinge. By contrast, the column bases behave elastically when threaded bars and disk springs or self-centering friction connections are used in the IF-TB and IF-FD, respectively.

To shed light on the different level of damage to RC members in the four types of frames investigated, the Park and Ang damage Index (PAI) (Park and Ang, 1985) in the modified version proposed by Kunnath et al. (1992) is calculated for all the elements included in the above list, also considering the left end of the first-story first-span beam. The PAI is given by the sum of two terms, concisely representing the kinematic and hysteretic behavior showed by an RC member during a time-history analysis, respectively. The improved version of the PAI is given as:

$$PAI = \theta + E = \frac{\theta_{\max} - \theta_y}{\theta_u - \theta_y} + \frac{\beta}{M_y \theta_u} \int dE \qquad (8)$$

in which $\theta_{\max}$ is the maximum rotation experienced by the RC member, θ_y is the yielding rotation, θ_u is the ultimate rotation, M_y is the yielding moment, $\int dE$ is the dissipated energy, and β is the model parameter, set equal to 0.15 for beams and columns as proposed by Cosenza et al. (1993), and 0.25 for joint panels as suggested by Altoontash (2004). θ_y and θ_u of the beams and columns are obtained by using the equations proposed by EN 1998 (2004). Concerning the joint panels, θ_y is obtained via MCFT, while θ_u is selected as suggested by De Risi, Ricci and Verderame (2017).

In order to prove the efficiency of the Park and Ang index, the latter is calculated for the results of the test on the subassembly investigated in Section 2, considering both the experimental and the numerical results. The equation used is the original one proposed by Park and Ang (1985) reported below:

$$PAI = \frac{d_{\max} - d_y}{d_u - d_y} + \frac{\beta}{F_y d_u} \int dE \tag{9}$$

in which:

- d_{max}: maximum displacement experienced by the RC member;
- d_u: ultimate displacement of the RC member. The adopted value is obtained by means of the numerical model, as the subassembly displacement when the joint panel attains the assumed ultimate rotation (since it occurs before attainment of the end section ultimate curvature or ultimate slippage of the reinforcing bars), and it is 331 mm;
- F_y and d_y: force and displacement at yielding of the RC member, respectively. In the absence of an experimental result of the subassembly subjected to monotonic load, these parameters were also obtained by means of the monotonic force-displacement curve of the numerical model. The actual curve was converted into

an equivalent bilinear elastic-perfectly plastic curve by using the energy balance criterion, setting the initial stiffness according to the suggestion of ACI 374.2R-13 (2013) as the secant stiffness at the force level 0.7 F_y. The adopted values for F_y and d_y are 98.4 kN and 62 mm, respectively.

β and $\int dE$ assume the above-mentioned meaning. The PAI values for both experimental test and numerical model versus the cumulative displacement are shown in Figure 19. The results are in good agreement with one another and prove the accuracy of the numerical model adopted.

It is worth calculating the PAI value assuming failure of the experimental test when a loss of 20% of the strength is observed (as usually assumed in several test programs as described in ACI 374.2R-13 (2013)); thus, the experimental result provides a maximum displacement of $d_{max,exp}$ = 207.5 mm and a cumulative displacement of $d_{cumulative,exp}$ = 1732 mm, with a dissipated energy of $E_{h,exp}$ = 57576 kNm, for which the calculated PAI value is 0.983. This proves the effectiveness of the PAI in representing the damage undergone by the system, as well as the effectiveness of the procedure for evaluating the parameters that characterize the PAI. Alternatively, experimental calibration of PAI, i.e. the search of the β value for which the PAI attains the value 1 for $d_{max} = d_{max,exp}$ = 207.5 mm, would have provided β = 0.26.

Thus, as β calibration on a single test may not be sufficiently accurate, since it is not able to take into account the natural dispersion of the results, the β value suggested in the literature was maintained. This is consistent with the suggestion of Williams et al. (1997).

In Table 6 there are reported θ_y, θ_u, M_y, average and CoV values of kinematic and hysteretic terms θ and E, and PAIs for the elements of the list above, considering the group of 30 analyses. Generally speaking, the results prove that use of innovative systems is effective in limiting or preventing damage to RC members. More precisely, with regard to beams, small average PAI values for TF confirm that slight damage occurs at the beam ends, mainly due to hysteretic behavior. By contrast, average kinematic and hysteretic terms equal to 0 for beam ends belonging to innovative frames mean that these RC members experience no plastic rotation and do not dissipate energy in any analysis. Concerning column bases, large average and small CoV PAI values for TF mean that the bulk of the central columns are near to the failure condition. Conversely, in the case of IF, even if the damage indexes of the columns are reduced by about 20% compared to those of the TF, their values are well beyond the limit of repairability, assumed equal to 0.4 (Park and Ang, 1985). The efficiency of using specific devices at the CFC is confirmed by the average PAI values of the column of IF-TB and IF-FD, which are about 0.1 in both cases. Moreover, their reliability is proven by the small CoV PAI values. Kinematic and hysteretic terms of panel joints demonstrate the effectiveness of FDDs in mitigation of structural damage. With regard to the TF, average PAI value near to 1 and low scatter imply that the investigated panel joint is near collapse in all the analyses, and it achieves failure condition in almost half of them. By contrast, none of the panel joints in innovative frames reaches the yield moment, leading to kinematic terms equal to 0, while the hysteretic terms are drastically reduced if compared to that of the TF. Lastly, a relatively small average PAI value characterizes the bar-slip mechanism. However, since the CoV value is large, a small number of analyses are characterized by a bar-slip mechanism with PAI values above the limit of repairability.

Conclusions

Hybrid Steel-Trussed Concrete Beams (HSTCBs) are able to cover long spans with small section depths, requiring a large amount of reinforcement in the beam-to-column joints, often employing large diameter bars. Therefore, both the beam ends and the beam-column panel zone become vulnerable to the effects of seismic actions. To deal with these drawbacks, the use of Friction Damper Devices (FDDs) in HSTCB frames is a very promising technique that has only recently begun to be studied in the literature.

Thus, a comparison among the seismic response of RC frames realized with HSTCBs endowed or not with FDDs was carried out. Besides the traditional frame (TF), three different configurations of the innovative frame (IF) were investigated: FDDs at the beam-to-column connections (BCCs) alone (IF), FDDs and column-to-foundation connections (CFC) endowed with preloaded threaded bars coupled with disk springs (IF-TB) or self-centering friction devices (IF-FD). The model for joint shear distortion and slippage of the longitudinal bars within the panel zone was validated by experimental test on beam-column subassembly realized by means of HSTCBs.

The seismic response of RC frames was assessed by means of both pushover analyses and Non-Linear Time History Analyses (NLTHAs), monitoring maximum and residual story drift ratios, element hysteretic responses and damage indexes.

Analysis of TF seismic responses proves that the panel zone and first-story column base section are the elements prone to the greatest damage which, even if adequately designed, in the presence of high intensity earthquakes easily exceed the repairability threshold, up to the failure condition.

Conversely, the IF provides structural performance consistent with design forecasts, i.e. beam end sections with nearly-elastic behavior and panel zones which experience negligible level of damage. Nevertheless, the column bases of the IF still experience significant damage. Moreover, the average RIDR obtained with the IF is much higher than that provided by the TF, with almost half of the analyses providing RIDR values higher than the permissible residual drift assumed to consider the structure functional, i.e. 0.5%. The use of self-centering systems in the IF-TB and IF-FD is able to remedy the aforementioned drawbacks, leading to column bases behaving elastically and an average RIDR similar to that given by the TF. However, a higher average MIDR is provided by the IF-TB, due to the limited dissipative capacity of the system constituted by a column base connection made of preloaded threaded bars and disk springs only. By contrast, the IF-FD provides complete prevention of any damage and interruption of use, ensuring an average MIDR comparable to that of the TF, and smaller RIDR.

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Disclosure statement

The authors declare that they have no conflict of interest.

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Table 1. Parameters of the rotational spring element for modeling the cyclic behavior of the Panel Zone in the Subassembly (PZ-S), Traditional Frame (PZ-TF) and Innovative one (PZ-IF)

		PZ-S	**PZ-TF**	**PZ-IF**
First-class parameters (backbone curve)	Initial rotational stiffness (kNm/rad)	235000	245000	330000
	Cracking moment (kNm)	47	49	66
	Yield moment (kNm)	290	298	406
	Yield rotation (rad)	0.006	0.006	0.006
	Ultimate rotation (rad)	0.2	0.2	0.2
	Post-yield stiffness ratio as % of elastic	0.001	0.001	0.001
Second-class parameters (hysteresis shape)	Stiffness degradation	4	4	4
	Ductility based strength decay	0.6	0.6	0.6
	Hysteretic energy based strength decay	0.6	0.6	0.6
	Slip parameter	0.5	0.5	0.5

Table 2. Parameters of the rotational spring element for modeling the cyclic behavior of the Bar-Slip mechanism in the Subassembly (BS-S) and Traditional Frame (BS-TF)

		BS-S	**BS-TF**
First-class parameters (backbone curve)	Elastic stiffness (kNm/rad)	52000	65000
	Yield moment (kNm)	168/-94	174/-169
	Rotation at peak moment strength (rad)	0.02/-0.02	0.02/-0.02
	Peak moment strength (kNm)	176/-96	184/-179
	Residual moment strength (kNm)	1.74/-0.93	1.86/-1.73
Second-class parameters (hysteresis shape)	Pinching factor	0.5	0.5
	Deterioration factor	0.6	0.6

Table 3. Parameters of the bilinear kinetic rotational spring for modeling the cyclic behavior of the friction device

Initial rotational stiffness (kNm/rad)	10^6
Yield moment (kNm)	110
Post-yield stiffness ratio as % of elastic	10^{-4}

Table 4. Parameters of the rotational spring element for modeling the cyclic behavior of the self-centering systems used in IF-TB and IF-FD

Self-centering systems	**IF-TB**	**IF-FD**
Initial rotational stiffness $K_{\theta,1}$ (kNm/rad)	10^6	10^6
Yield moment (kNm)	120	200
Post-yield stiffness $K_{\theta,2}$ (kNm/rad)	2261	2261
Ratio of forward to reverse yield moment	10^{-4}	0.8

Table 5. Average and CoV of maximum interstory drift ratios of the four types of RC frames.

Story		**Maximum IDR**				**Residual IDR**			
		TF	**IF**	**IF-TB**	**IF-FD**	**TF**	**IF**	**IF-TB**	**IF-FD**
1	Mean	2.72%	2.56%	3.17%	2.57%	0.17%	0.47%	0.10%	0.09%
	CoV	13.27%	18.14%	12.61%	13.86%	77.99%	79.74%	86.60%	70.89%
2	Mean	2.91%	2.84%	3.10%	2.65%	0.14%	0.59%	0.24%	0.19%
	CoV	11.16%	16.58%	17.04%	17.91%	78.71%	78.15%	79.57%	76.96%

Table 6. Average and CoV values of Park and Ang damage Index (PAI), kinematic (θ) and hysteretic (E) terms for the left and right ends of the first-story beams, column base, first-story internal joint, bar-slip mechanism of first-story beam.

Parameters				**θ**	**E**	**PAI**
Left end of the first-story first-span beam	θ_y = 0.021 rad θ_u = 0.08 rad M_y^- = 169 kNm	TF	Avg	0.012	0.084	0.095
			CoV	182.98%	19.75%	36.07%
		IF/ IF-TB/ IF-FD	Avg	0	0	0
			CoV	-	-	-
Right end of the first-story second span beam	θ_y = 0.021 rad θ_u = 0.08 rad M_y^+ = 174 kNm	TF	Avg	0.018	0.083	0.101
			CoV	218.39%	17.69%	50.75%
		IF/ IF-TB/ IF-FD	Avg	0	0	0
			CoV	-	-	-
Base section of the central column	θ_y = 0.013 rad θ_u = 0.056 rad M_y = 278 kNm (for TF, IF, IF-TB) M_y = 326 kNm (for IF-FD)	TF	Avg	0.370	0.502	0.872
			CoV	26.72%	8.35%	13.42%
		IF	Avg	0.287	0.413	0.701
			CoV	38.59%	10.43%	19.50%
		IF-TB	Avg	0	0.118	0.118
			CoV	-	8.36%	8.36%
		IF-FD	Avg	0	0.080	0.080
			CoV	-	6.96%	6.96%
First-story internal joint	θ_y = 0.006 rad θ_u = 0.065 rad M_y = 298 kNm (for TF) M_y = 406 kNm (for IF, IF-TB, IF-FD)	TF	Avg	0.331	0.651	0.982
			CoV	25.41%	7.21%	10.63%
		IF	Avg	0	0.179	0.179
			CoV	-	6.56%	6.56%
		IF-TB	Avg	0	0.178	0.178
			CoV	-	7.03%	7.03%
		IF-FD	Avg	0	0.184	0.184
			CoV	-	7.48%	7.48%
Bar-slip mechanism	θ_y = 0.002 rad θ_u = 0.022 rad M_y = 169 kNm	TF	Avg	0.222	0.119	0.342
			CoV	63.07%	76.25%	65.67%

Figure 1. Scheme of constraints and load condition of the specimens

Figure 2. RC beam-column joint model

Figure 3. Bond stress and bar stress distribution for a bar anchored in a beam-column joint

Figure 4. Structural model implemented in SeismoStruct

Figure 5. Experimental and numerical curves of the subassembly with and without damage of the panel zone

Figure 6. Energy dissipated by the subassembly for each loading cycle for both experimental and numerical results

Figure 7. Trend of the energy dissipated by the subassembly versus cumulative displacement for both experimental and numerical results

Figure 8. Structural solution adopted for the beam-to-column connection

Figure 9. Self-centering connections at RC column base (a), exploded 3D view (b)

Figure 10. Structural model of the RC structure considered

Figure 11. Elastic and design spectra (red and blue), response spectra and the average one of the group of artificial accelerograms used (grey and black)

Figure 12. Acceleration time history of one of the artificial accelerograms used

Figure 13. Node modeling scheme for RC frame with friction device

Figure 14. Theoretical moment-rotation curve of the self-centering friction connection used in IF-FD

Figure 15. FEM result taken from Colajanni et al. (2021) and numerical model of the

friction device (a), experimental test no.1 in Latour et al. (2019) and numerical model of the self-centering friction connection (b)

Figure 16. Capacity curves of the four types of frames investigated

Figure 17. Moment rotation curve of: beam end section of TF (a), IF (b); beam-column joint shear deformation of TF (c), IF (d); bar-slip mechanism of TF (e); friction device of IF (f)

Figure 18. Moment-rotation curve of: the connection with preloaded threaded bars + disk springs of IF-TB (a); the self-centering friction connection of IF-FD (b); the column base in case of: TF (c); IF (d); IF-TB (e); IF-FD (f)

Figure 19. Trend of the Park and Ang Index of the subassembly versus cumulative displacement for both experimental and numerical results