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A DISSIPATIVE FRICTION DEVICE FOR HYBRID STEEL-TRUSSED CONCRETE BEAM TO COLUMN CONNECTION

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Abstract

The most recent design strategies of multi-storey framed structures are increasingly welcoming the adoption of innovative techniques for the seismic energy mitigation, in order to guarantee a highly dissipative global behaviour able to prevent the structure from collapse with consequent loss of human lives. In particular, there is a large interest in the study of those devices able to absorb the whole seismic energy avoiding the damage of the primary load-bearing structural elements connected to the device, resulting in extremely high economic costs for their structural repair. In this context, steel frames are often equipped with dissipative beam to column connection, able to prevent any plasticization on the structure. In order to preclude interruption of use, the device include re-centering system, that prevent appreciable residual displacement at the end of the seismic event.
In the last thirty years, Hybrid Steel-Trussed-Concrete Beams (HSTCBs) have been widely used in civil and industrial building. HSTCBs are often designed to exploit the steel reinforcement made up of an encased steel truss in order to cover large spans with reduced depth. In such cases, a large amount of steel reinforcement is required within the panel zone which is often made using large diameter rebar. These features make both the end of the beam and the joint potentially vulnerable to the effects of cyclic actions induced by the earthquake and dramatically reduce the dissipation capacity of the entire structure. In reinforced concrete structure, only few dissipative devices for beam to column connections have been developed for precast element, while the use of such devices for steel concrete hybrid frames have not been yet investigated.
For this purpose, the present work investigates the introduction of friction dampers in the HSTCB-to-column joints of framed structures in seismic areas, adequately designed for their application on R.C. frames. The use of friction devices prevents the main structural elements from damage and limits the cracking of the panel zone thanks to the increase of the bending moment lever arm transmitted by the beams, which reduces the shear forces in the joint. The feasibility study is firstly conducted through the development of design criteria for the pre-dimensioning of the device and, successively, the proposed solution is validated through the generation of finite element models.

Keywords: friction dampers; hybrid steel-trussed-concrete beams; joint cyclic behavior; finite element models.

1. Introduction

The most recent design strategies of multi-storey framed structures are increasingly welcoming the adoption of innovative techniques for the seismic energy mitigation, in order to guarantee a highly dissipative global behaviour able to prevent the structure from collapse with consequent loss of human lives. In particular, there is a large interest in the study of those devices able to absorb the whole seismic energy avoiding the damage of the primary load-bearing structural elements [1-5]. Thus, irreversibly damage after a violent seismic event, resulting in extremely high economic costs for their structural repair is prevent. In the last thirty years, Hybrid Steel-Trussed-Concrete Beams (HSTCBs) have been widely used in civil and industrial buildings and, therefore, their mechanical performance was evaluated [6, 7] for ensuring the compliance with the capacity design criteria of framed structures, and achieving the adequate amount of seismic energy dissipation, particularly in the beam-to-column joints. HSTCBs are often designed to exploit the steel reinforcement made up of a steel truss completed by a concrete casting in order to cover large spans with reduced depth. In such cases, a large amount of steel reinforcement is required within the panel zone, which is often made using large diameter rebar. These features make both the end of the beam and the joint panel zone potentially vulnerable to the effects of cyclic actions induced by the earthquake. Large cracking of the panel zone and loss of rebar bond can dramatically reduce the dissipation capacity of the entire structure.

For this purpose, the present work investigates the use of friction dampers in the HSTCB-to-column joints of framed structures in seismic areas, adequately designed for their application on RC frames. The use of friction devices prevents the main structural elements from damage and limits the cracking of the panel zone thanks to the increase of the bending moment lever arm and the entity of the bond actions transmitted by the beam rebars, which reduces the shear forces in the joint. The proposed structural solutions are inspired from the systems increasingly adopted in steel frames for the same purpose. The feasibility study is firstly conducted through design of two different solutions. The first is characterized by a pinned connection of the top chord of the beam, and a friction device with two group of slotted holes, that allows the movement of the friction pad in any direction. A T stub top chord connection and a friction device with a single group of curved slotted holes characterize the second device. The monotonic and cyclic behaviour of two different proposed solutions is evaluated by FE models.

2. Friction dampers for low-damage beam-to-column joint

In the literature, the most adopted dissipative beam-to-column steel connections can be referred to four different categories depending on the adopted dissipation system: - asymmetric connections with two friction shims; -symmetric connections with one friction shims per side; -symmetric connections with additional steel element and two friction shims; -symmetric connections with two or more friction shims per side.

The connection developed at the University of Auckland and University of Canterbury belongs to the first group [1] (Fig. 1a). In this solution, the beam is connected to the column by means of two steel plates, which are welded to the column flange next to the beam. The connection between these steel plates and the beam is made with bolts, where the upper ones realize a standard friction bolted connection, while the lower ones are inserted through slotted holes to allow the rotation of the beam. Between the bottom beam flange and the lower plate is inserted a friction shim, which provides adequate dissipative capacity to the connection. A cap plate is bolted to the bottom side of the lower plate, putting between the latter ones another friction shim. The connection starts to rotate when the friction force provided by the friction shim inserted between the bottom flange and the lower plate is achieved. Increasing the rotation of the system, the total friction force grows to double when the friction shim inserted between the lower plate and the cap plate begins to slide.

A solution belonging to the second group has been developed at the University of Salerno, Italy [2]: the connection is made with two T stubs, which are bolted to the beam flanges on which slotted holes are realized to allow the sliding of the bolts (Fig. 1b). Between the T stub and the beam flange is inserted a friction shim, which provides the energy dissipation. This configuration does not have a defined centre of rotation. The same Authors, in collaboration with the University of Naples “Federico II”, Italy, propose two

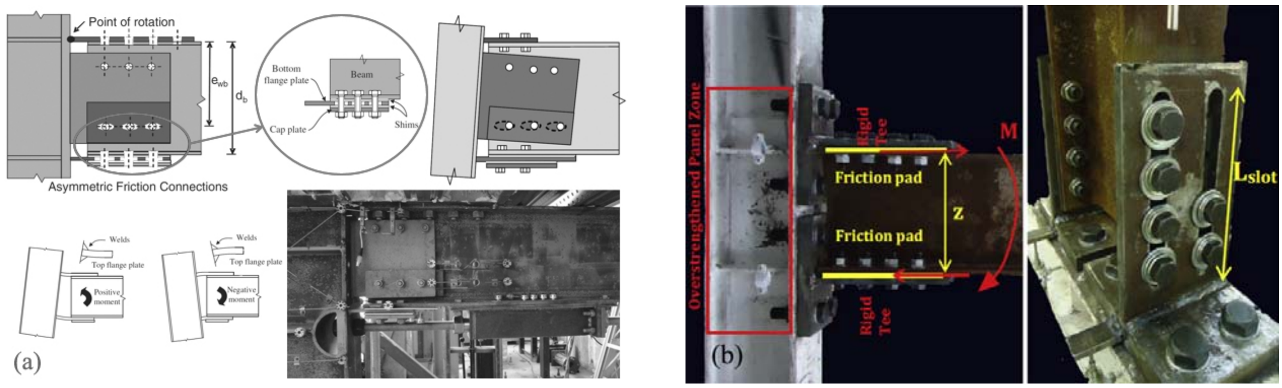

Fig. 1 – a) Two friction shims asymmetric connect. [1]; b) one friction shim per side symmetric connect. [2]

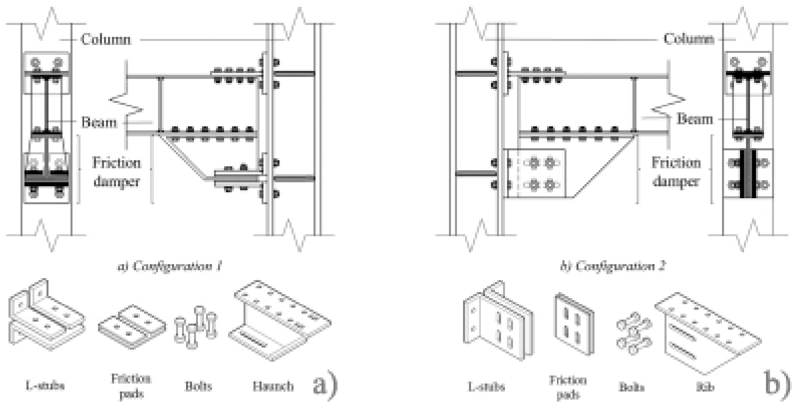

Fig. 2 – Symmetric connections with additional steel element and two friction shims: a) horizontal dissipative device; b) vertical dissipative device [3]

other solutions, one with the dissipative device in horizontal direction, the other one oriented in vertical direction, belonging to the third group [3]. Both of these solutions exploit an additional steel element which is bolted to the bottom beam flange with the aim of increasing the internal lever arm of the connection and, thus, reducing the forces acting to the panel zone. This additional steel element is an I-shaped profile in case of horizontal dissipative device, while is T-shaped in case of vertical one. The upper part of the beam is connected to the column flange by means of a T stub which is bolted to both the column and the beam. In both the solutions, the centre of rotation is expected to form at the base section of the T stub. The solution with the horizontal dissipative device is constituted by three steel angles which are bolted to the I-shaped profile, using one friction pad for each steel angle (Fig. 2a). On the contrary, the solution with the vertical dissipative device employs two steel angles, requiring the realization of two groups of slotted holes, one vertically-oriented on the steel angles, and the other one horizontally-oriented on the T-shaped profile. By doing so, bolts are able to move in any direction. In contrast, in the solution with horizontal dissipative device, the displacement component in vertical direction is absorbed by the deformation of the lower steel angles. Differently from the horizontal system, in which low damage of the lower steel angles is admitted, the vertical one (Fig. 2b) prevents any damage in the friction connection.

One of the first solutions proposed for friction devices applied at the beam-to-column joints, was reported in [4] and belongs to the fourth group: the system is constituted by T stubs, each of these bolted to the beam flanges and cover by an outer plate. At each interface T stub-beam flange and T stub-outer plate is inserted a friction shim. Beam web is bolted to the column flange via vertical steel angles, in which the central bolt acts as the centre of rotation, while the other bolts are inserted through slotted holes.

The solution proposed in [5], showed in Fig. 3, was developed for Precast Concrete (PC) structures and belongs to the fourth group as well. The connection between the column and the beam is made using only two bolts, both inserted through curved slotted holes. Each dissipative device is constituted by four

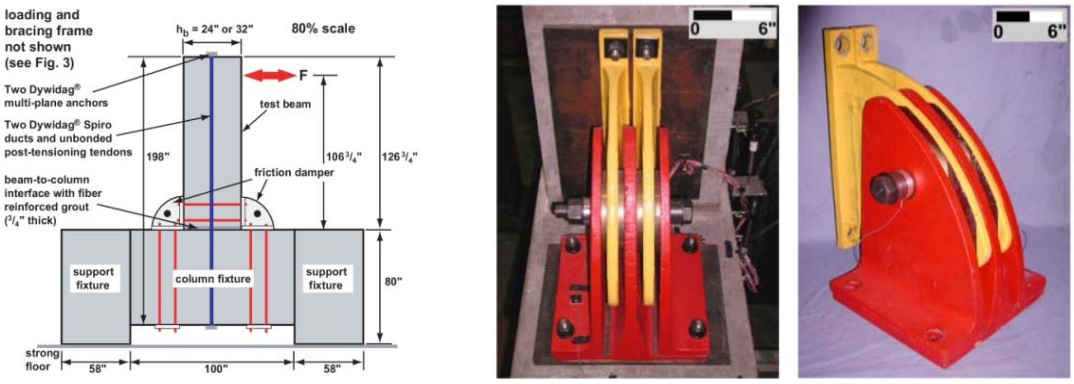

Fig. 3 – Symmetric connections with two or more friction shims per side for PC structures [5].

friction shims. Moreover, beam and column are connected with an unbounded post-tensioned tendon, which is positioned at the mid height of the beam cross-section, providing self-centring behaviour to the connection. Experimental results have shown that concrete has been damaged at the interface between beam and column, being the latter ones in contact. The above-described solutions are highly influenced by two aspects: material of the friction shims and bolt preload [8, 9]. Therefore, from the analysis of the solutions found in literature, the following considerations are developed, which will be used to define a design solution to be used for the aim of the present paper:

- symmetric dissipative system ensures a stable response in terms of bending moment resistance, at the same time reducing the stresses experienced by the bolts;
- steel angles connected to the vertical dissipative device remain within the elastic range, simplifying the constructional system with respect to the horizontal dissipative device;
- the dissipative system used only at the lower part of the beam does not interfere with the other layers at the upper part of the beam;
- the cinematic behaviour of the beam-to-column connection is predictable only if it is known a priori the position of the centre of rotation;
- separating the beam end section and the column face prevents that damage can occur at this interface;
- loss of bolt preload due to plastic deformations has to be properly reduced inserting spring washers;
- loss of bolt preload due to long-term effects can be reduced employing a bolt preload comprised between 30 and 60% of the maximum bolt preload allowed by Eurocode 3 [10].

3. Design process of the dissipative friction connection and evaluation via 3D FEMs

In order to design the friction device to be used in connection between HSTC beams and cast-in-situ RC columns, the solution with the dissipative device in vertical position has been adopted, being the most efficient among the above-mentioned solutions. Moreover, with the aim of avoiding interferences with the slab in the upper part of the beam, the solution with friction devices at both the upper and lower part of the beam has been excluded. At the same time, within the beam-to-column connection, a system behaving like a perfect hinge has to be defined. Lastly, transfer of the stresses from the HSTC beam to the steel elements to which is connected has to avoid damage in the concrete. In the following section, two solutions are presented, the first one characterized by a pin connection and vertical and horizontal slotted holes, while the second one characterized by a T stub connection and curved slotted holes.

3.1 Solution with vertical and horizontal slotted holes

The first solution is illustrated in Fig. 4, consisting in the adaptation to the HSTC beam of the vertical dissipative device with symmetric connection. It includes a vertical steel plate (here called rib plate) placed on vertical plane including the beam axis, two steel angles, and two friction pads placed at the interface of the aforementioned elements. Two groups of slotted holes, one vertically-oriented on the steel angles, and the other one horizontally-oriented on the rib plate, allow the bolts to move in any direction. By doing so,

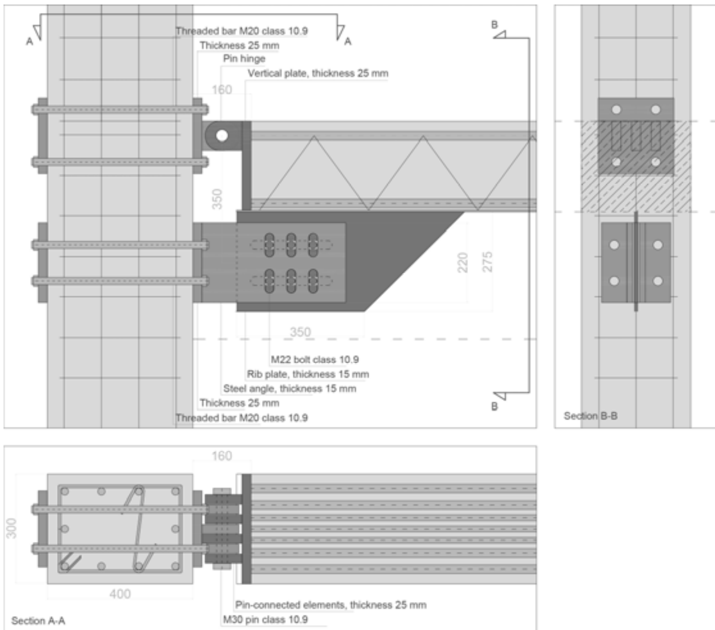

Fig. 4 – First solution with vertical and horizontal slotted holes and pin hinge

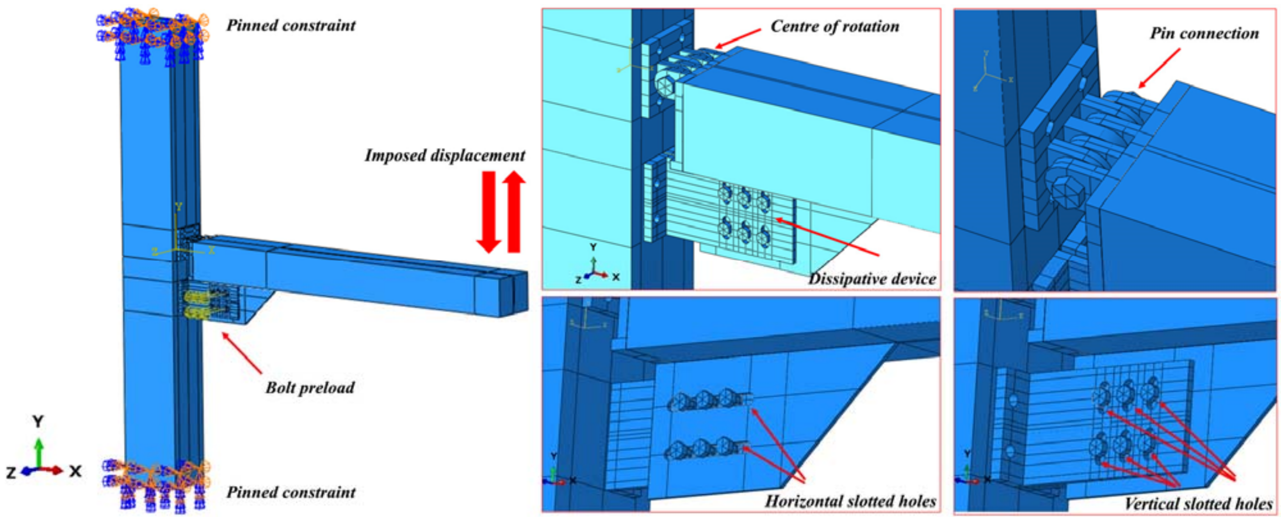

Fig. 5 – 3D model of the first solution of dissipative system on FEM software ABAQUS CAE

bolts can slide with a curved trajectory limiting the damage of the structural elements during the sliding step. The first difference with the device reported in Fig. 2 is constituted by the rib plate of the friction device, which is welded, rather than bolted, to the steel plate at the lower part of the HSTC beam. Regarding the upper connection, differently from the solutions described in the introduction, a pin connection has been selected, being able both to determine the centre of rotation, and to prevent the damage of the structural elements constituting the upper connection. The connection to the column is realized by means of threaded bars embedded through the height of the cross-section of the column. The above-described beam-to-column connection lengthens the internal lever arm of the bending moment acting on the beam-column joint, reducing the stresses experienced by the panel joint, thus enhancing its cyclic performance. Once the geometry of the first solution has been defined, 3D FEM analysis on the equivalent structural model has been carried out (Fig. 5). A subassembly representing an exterior connection has been modelled using the FEM software ABAQUS CAE, considering the column pinned at both the top and the bottom sections. Cross-section dimensions and length of both half-column and half-beam are equal to 300x400 mm and 3 m,

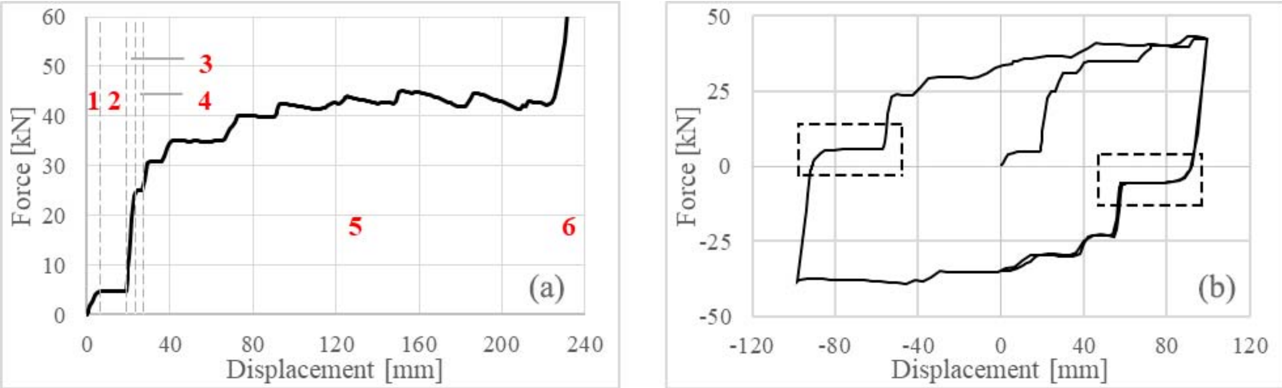

Fig. 6 – Force-displacement curve of the first solution: monotonic test (a), cyclic test (b)

250x300 mm and 2.5 m, respectively. The structural model has the following characteristics: -beam and column with a linear elastic behaviour, without any reinforcement; -steel plate of the upper pin connection and lower steel angles are connected to the column with an idealized constraint; -realistic model of the contacts between the elements constituting the friction device (penalty in the tangential direction and hard contact in the normal one); - initial configuration of the pin in an ideal centred position with respect to the hole in which is inserted, the latter being not calibrated and, therefore, not in contact with the pin shank.

As regards materials, the steel elements are modelled using elasto-plastic behaviours. In particular, steel elements and bolts were modelled using the same elastic modulus (E_s) and ultimate strain (ε_u), equal to 210 GPa and 0.3 %, respectively. Moreover, the yield stress of the steel elements and bolts were set 355 MPa and 900 MPa, respectively. Two displacement control tests have been simulated, by imposing to the free-end of the beam a vertical displacement. Namely the first one was monotonic up to 240 mm and the other one was cyclic with displacement in the range ±100 mm. Before application of the displacement history, the preloading force is applied to the bolts of the damping device. In Fig. 6, the force-displacement curves of both the monotonic and cyclic tests are reported, emphasizing six different phases during the monotonic test:

1. In the linear elastic branch the friction stresses at the interface of the dissipative device are able to avoid the sliding of the surfaces. Because the upper pin is not in contact with the holes of the steel elements in which is inserted due to the clearance hole, the bending moment is totally absorbed by the friction device;
2. In the second branch, the resistance provided by the friction stresses is achieved, structural elements slide mutually, and the beam rotates around a point identified within the plates constituting the dissipative device, until the pin is in contact with the holes of the steel elements (Fig. 7a);
3. The system assumes a new configuration in terms of stiffness, the distribution of the friction stresses on the dissipative device is modified, because the bending moment resistance of the beam-to-column connection is provided mainly from the horizontal reaction of the pin and the friction forces at the dissipative device. The system behaves elastically again. In this step, the shear force experienced by the connection is totally absorbed by the friction device;
4. The sliding system activates and the beams rotates around the centre of rotation in the contact area between the pin and the steel elements. In this branch the system response is perfectly plastic, namely the force value for which the device slides is constant while the beam rotation increases. The friction device withstands the whole shear force in this step as well. The structural elements constituting the dissipative device behave elastically, reproducing the behaviour expected during the design procedure, which will be explained in the next section;
5. Bolt shanks, which during previous steps were not in contact with the horizontal slotted holes of the rib plate, due to the hole clearance, go in contact with the latter ones. In order to keep rotating the system, bolts of the dissipative device have to be drawn below by the rib plate. To do so, the friction forces between the bolt heads and the external faces of the steel angles have to be overcome. These friction forces, which usually are not considered in steel structures, cause the sawtooth-shaped progressive increment of the load, as showed in Fig. 6a. This increment is due to the contact between horizontal slotted holes and bolt shanks which gradually happen while the beam rotation increases (Fig. 7b);

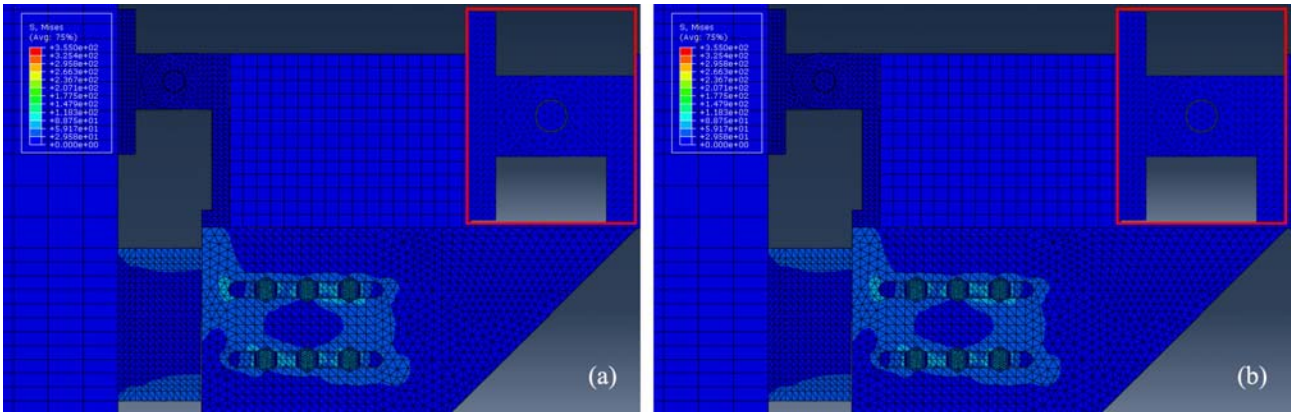

Fig. 7 – Stress state of the system during: (a) step 2, (b) step 5

6. The bolts of the friction device go in contact with the end of slotted holes, thus the system shows a new stiffness. The displacement value for which this phenomenon is achieved is well beyond the design rotation capacity with which the beam-to-column connection has been designed.

Once the results of the monotonic test have been discussed, confirming what has been already faced in literature, the cyclic test has been carried out, providing the force-displacement response reported in Fig. 6b. From the analysis of this figure, some considerations can be carried out:

- The phenomenon faced in the previous test and described as “step 2”, namely the step during which the pin is not in contrast with the steel elements, is become critical when the bending moment to which the connection is subjected is inverted (in Fig. 6 are highlighted with the dashed rectangles). As a matter of fact, the two highlighted horizontal branches represent the resistance provided by the friction device when the pin moves inside the holes of the steel elements. As a consequence, not only the stiffness, but also the dissipative capacity of the connection are highly influenced by this phenomenon;
- Wide and stable hysteretic cycles are obtained because the 3D FEM model does not include both degrading phenomena regarding the friction coefficient and loss of bolt preload.

3.2 Solution with curved slotted holes

Based on the above-discussed results, the second device scheme, illustrated in Fig. 8, aims to solve the weaknesses of the first one, i.e. the pinching of the connection response which depends on the position of the pin, and the additional resistance of the dissipative device when the bolts are drawn above and below. The first problem is solved by substituting the pin connection with a T stub and a C-shaped steel plate bolted connection. The cross-sectional shape of the latter makes the steel element much more stiff than the T stub using the same cross-sectional area. At the same time, the C-shaped profile width is less than that of the beam to which is connected, in order not to obstacle the concrete pouring operation. As already known in literature, the centre of rotation of the connection is supposed to form near the base section of the T stub. In the adopted solution, the cross-sectional area of the horizontal plate of the T stub is reduced by means of two holes, in order to force the system to form the plastic hinge at the reduced section, ensuring the definition of a centre of rotation. As for the second problem, namely the additional resistance due to the secondary friction forces, the horizontal and vertical slotted holes were substituted with curved slotted holes realized on the rib plate. By doing so, the dissipative device is able to rotate and the bolts remain in their initial position. On the basis of the above-defined centre of rotation, length and width of the curved slotted holes have been determined with the aim of ensuring the design rotation capacity (50 mrad), avoiding, once the sliding step is achieved, any contact between bolt shanks and slotted holes.

Preliminary FEM analyses showed that the system exhibited no sufficient stiffness in the in-plane direction. Therefore, 12 mm diameter inclined rebars have been welded on both the lower plate of the HSTCB and the top chord, as depicted in Fig. 8. These web bars have variable inclination and allow the activation of the stress transfer mechanism between concrete, steel top chord and slotted-hole central plate.

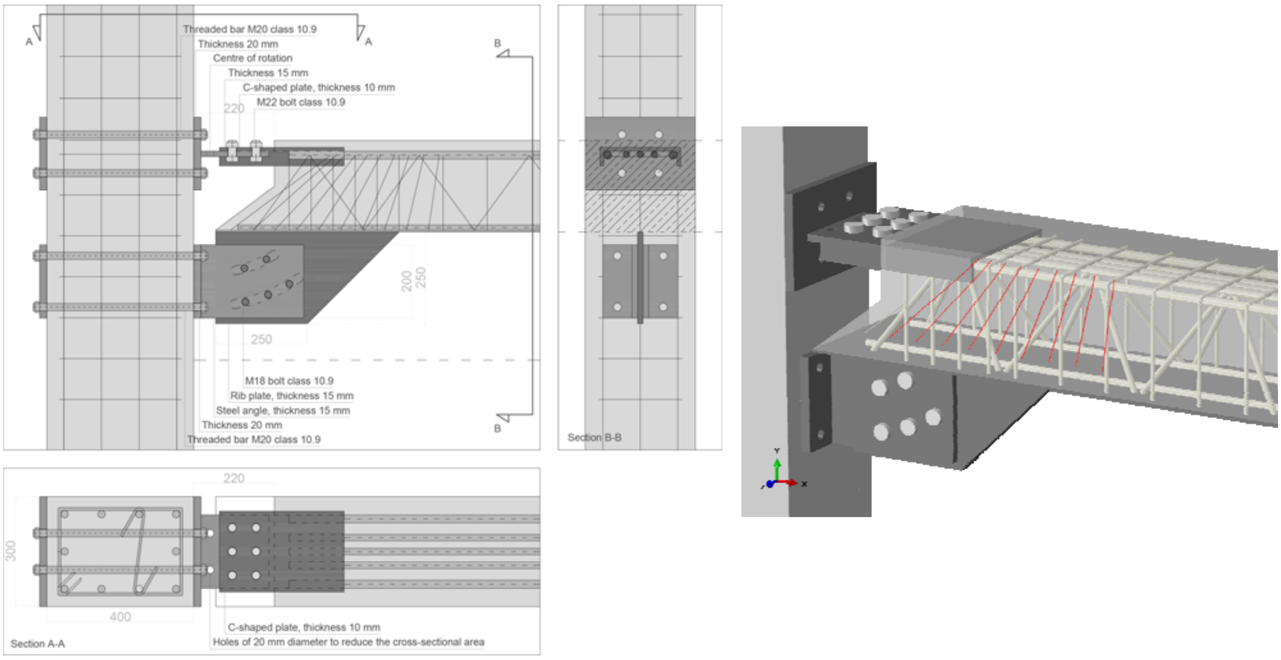

Fig. 8 – Second solution with curved slotted holes and T stub

3.2.1 Design method

In order to develop the theoretical feasibility study, the structural details of the construction are not taken into account. Therefore, a design value of the bending moment is arbitrarily assumed M_d = 110 kNm. Moreover, with the aim of providing overstrength to all components of the friction connection with respect to the load able to activate the slippage, an overstrength coefficient Ω_μ is adopted. In the absence of experimental campaigns for the assessment of the characteristic values of the static and dynamic friction coefficient, in this study the value $\Omega_\mu = 1.5$ is assumed. Hence, the overstrengthened design bending moment is equal to $M_{Rd} = \Omega_\mu M_d = 1.5\text{x}110 = 165$ kNm and, assuming a length L of 5 m for the beam, the required shear resistance results $V_{Rd} = 2\ M_{Rd} / L = 66$ kN in the absence of distributed loads.

The friction damper is designed to withstand a tensile slip force F_d equal to the design bending moment M_d divided by the lever arm z which is equal to 380 mm according to the conception of the geometry of the device (Fig.9):

$$F_d = \frac{M_d}{z} = \frac{110}{0.38} = 289.5\ kN \tag{1}$$

It is assumed to use five M18 bolts 10.9 class, whose area is A_{res} = 192 mm^2 and yield and ultimate strength are f_{yb} = 900 MPa and f_{ub} = 1000 MPa. Thus, the preloading force F_{pc} of each bolt is equal to:

$$F_{pc,M18} = 0.7\, f_{ub} A_{res} = 134.4\, kN \tag{2}$$

According to Eurocode 3 [10], the sliding force $F_{s,Rd}$ is calculated through the following expression:

$$F_{s,Rd} = \frac{k_s n_b n_s \mu}{\gamma_{M3}} F_{pc} \tag{3}$$

being: - k_s coefficient that depends on the shape of the slotted hole (assumed equal to 1); - n_b the number of bolts (5 in this case); - n_s the number of surfaces in contact (2 in this case); - μ the friction coefficient (herein assumed equal to 0.4); - γ_{M3} safety factor (assumed equal to 1). Furthermore, regarding the preloading force,

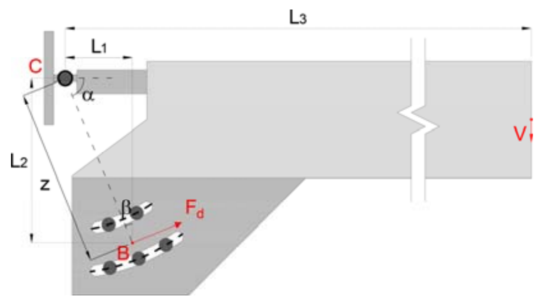

Fig. 9 – Geometric scheme of the friction device

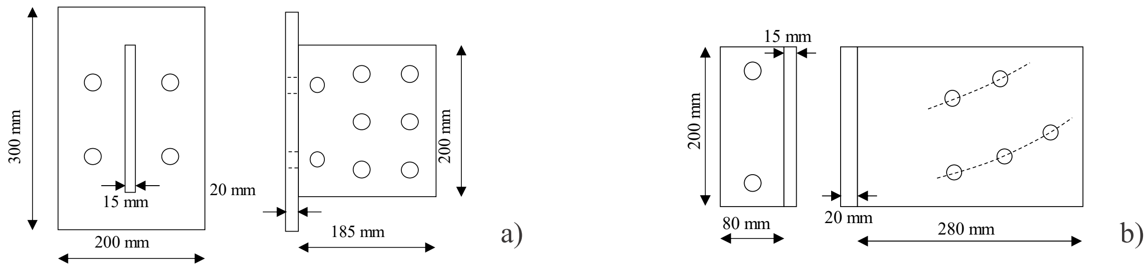

Fig. 10 – Dimensions of the: a) T stub, b) steel angle

several studies in the literature show that the value of F_{pc} decreases progressively due to creep phenomena which are affected to the high amount of preloading force applied to the bolt. With the aim of limiting these effects, some Authors [8, 9] propose to contain the bolt preload within the range 30-60% of the maximum load suggested by the Eurocode 3 [10]. Therefore, in the present study, the design sliding force $F_{s,d}$ is:

$$F_{s,d} = t_s n_b n_s \mu F_{pc} \tag{4}$$

where the parameter t_s is introduced for representing the stress level of the bolt, i.e. the aliquot of the maximum preload that is applied to the bolt and which is set between 0.3 and 0.6 as mentioned before. In particular, by equating Eq. (1) and Eq. (4) a stress level t_s = 0.538 is obtained. Successively, the design preloading force $F_{pc,d}$ to be applied to each bolt results:

$$F_{pc,d} = t_s F_{pc,M18} = 0.538 \times 134.4 = 72.3\,kN \tag{5}$$

Based on these calculations and on the geometrical requirements provided in Eurocode 3, the conception of the damping device is depicted in Fig. 9. Five bolts collocated on two rows are used and curved slotted holes are designed, with centre of rotation C indicated in the same figure. The dimensions indicated are: L_1 = 158 mm; L_2 = 342 mm; L_3 = 2.5 m; α = 68° and β = 22°. The device is connected to the beam and the column through bolted steel plates, T-stub and angles which are depicted in Fig. 10.

The connection between T-stub and beam is calculated as a classical friction bolted connection where the design sliding force $F_{d,h}$ is evaluated as the horizontal component of the force defined in Eq. (1), i.e.:

$$F_{d,h,upper\,connection} = \Omega_\mu F_d \sin(\alpha) = 402.6\ kN \tag{6}$$

α being the angle between the beam longitudinal axis and the axis connecting the centre of rotation C and the application point of the sliding force F_d indicated in Fig. 13. For this connection, six M22 bolts of 10.9 class are used. The number of bolts required has been calculated through the following equation:

$$n_b = \frac{F_{d,h}\,\gamma_{M3}}{k_s n_s \mu F_{pc,M22}} = \frac{402.6 \cdot 1.25}{1 \cdot 1 \cdot 0.4 \cdot 212.1} = 5.93 \Rightarrow 6 \tag{7}$$

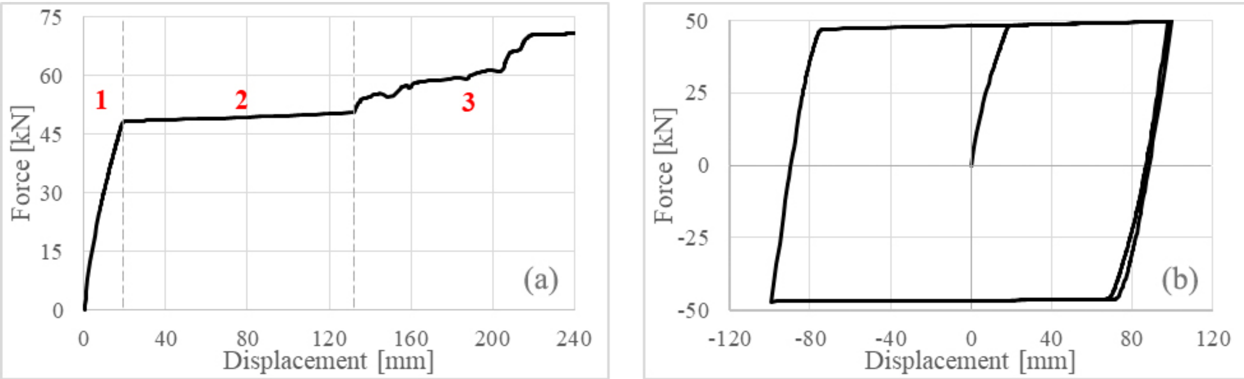

Fig. 11 – Force-displacement curve of the second solution: monotonic test (a), cyclic test (b)

The thickness of the T-stub web is dimensioned assuming that it must absorb only the traction expressed by Eq. (6) while the capacity to withstand the shear force is demanded to the steel angles.

The two lower steel angles are bolted to the slotted-hole rib plate. Each angle is subjected to one half of the force expressed by Eq. (1) according to the horizontal and vertical components which can be calculated through the following expressions:

$$F_{d,h,lower\,connection} = 0.5\Omega_\mu F_d \sin\left(\alpha\right) = 201.3\ kN \tag{8}$$

$$F_{d,v,lower\,connection} = 0.5\Omega_\mu F_d \cos\left(\alpha\right) = 81.4\ kN \tag{9}$$

Therefore, the web of the steel angle is dimensioned considering the bending moment in the presence of axial force while the flange is checked according to the plastic failure mechanisms of Eurocode 3.

3.2.2 FEM analysis

The FEM model has the same characteristic of the model developed for the first solution. FEM were implemented by using first order tetrahedral for all element components, with the exception of column and C-shaped profile for which linear bricks are selected. For the concrete material, a compressive strength of 25 MPa and an elastic modulus of 28960 MPa have been adopted. The plastic behaviour of concrete is modelled by using the Concrete Damaged Plasticity model based on the theory of plastic continuous damage of quasi-brittle materials. Fig. 11 reports the monotonic and cyclic load-displacement curves that characterizes the device response. It can be observed that the system is able to provide the full dissipative expected cyclic response. In particular, in the monotonic test, three phases can be individuated:

- Phase 1: the friction device does not slip and the system behaves elastically;
- Phase 2: the sliding is activated and the behaviour of the system turns into an almost perfectly-plastic behaviour, exhibiting a slight hardening probably due to the plasticization of the upper connection. The deformations of the beam slightly move the centre of rotation assumed during the design;
- Phase 3: the design displacement limit is reached; a progressive increment in load is caused by the contact between the bolt shank and the slotted hole internal surfaces.

Fig. 12 shows the stress state in the steel elements in the transition between phase 2 and 3. It can be noticed that all steel elements constituting the connection behave elastically, except for the T stub.

Fig. 13 shows the cracked concrete in the transition between phase 2 and 3. Despite of its high stiffness, the flexural deformation of the C-shaped profile induces a progressive degradation of the concrete cover around the steel profile. The output of Phase 3 is not analysed because it refers to the behaviour of the device beyond the design working condition. From the load-displacement curve of the cyclic test reported in Fig. 11b, it can be observed that the system behaves according to the design requirements, i.e. it exhibits a symmetric response for hogging and sagging bending moment and does not evidence any damage in the

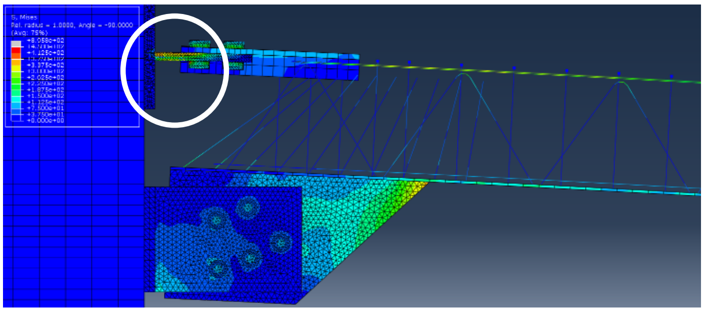

Fig. 12 – Stress state in the device between phase 2 and 3

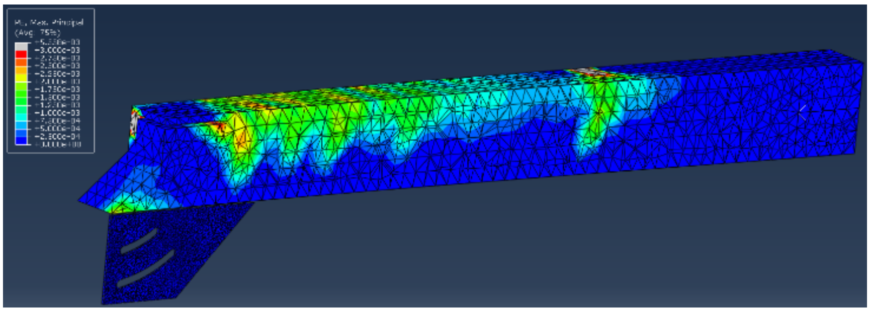

Fig. 13 – Maximum principal strains in the concrete between phase 2 and 3

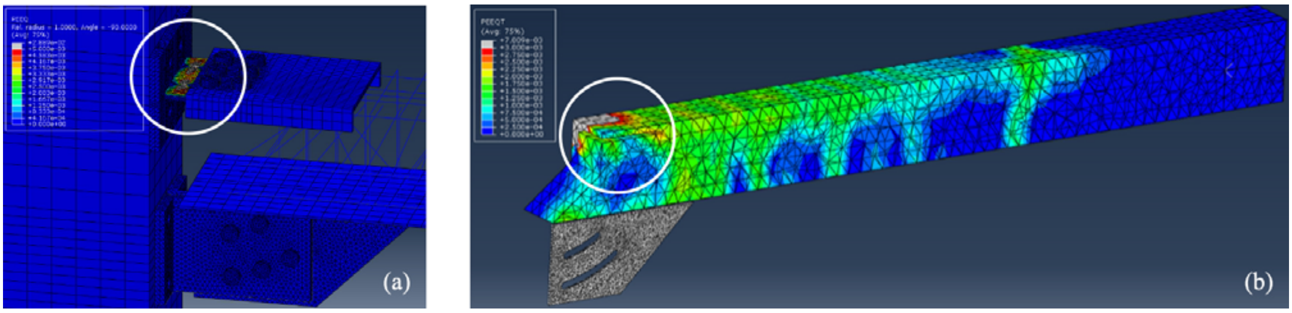

Fig. 14 –a) Plastic strain distributions in the steel elements at the end of the cyclic test; b) maximum principal strains in the concrete at the end of the cyclic test

loading-unloading phases. The analysis of the stress state is the same already described for the monotonic test. In Fig. 14 the plastic strains cumulated on the components at the end of the cyclic test are illustrated. In particular, Fig. 14a reports the distribution of the equivalent plastic strains: it can be observed that all steel elements are in the elastic range, expect the horizontal flange of the T-stub which experiences plastic deformations according to the design requirements. As to the concrete block, it can be useful to assess the cracking state of the material based of the equivalent tensile plastic strains showed in Fig. 14b. As expected, in the inner rim of the beam, at its intrados, the concrete cracks when the beam is subjected to positive bending moment. In the same time, the cyclic action produces a greater deformation of the C-shaped profile of the upper connection with respect to the behaviour observed in the monotonic simulation, increasing the plastic strains of the surrounding concrete cover.

4. Conclusions

The introduction of friction dampers in the HSTCB-to-column joints of framed structures has been investigated. The feasibility study has been firstly conducted through the development of design criteria for the pre-dimensioning of the device and, successively, the proposed solution has been validated through the generation of finite element models.

The calculation has been conducted according to the current prescriptions for buildings in seismic areas and the damping device has been adequately designed for its application on RC frames. The friction device components and the connections to beam and column have been dimensioned and verified according to Eurocode 3 prescriptions. Conversely, the structural details of the column and the connection within the latter have been neglected, in the meaning that the column behaviour has been assumed linear elastic and the efficacy of the anchors perfect. The device has been conceived for a working level of the preloaded bolts of 0.538 adopting an overstrength coefficient of 1.5.

Monotonic and cyclic tests have been simulated, showing that the device behaves according to the design requirements. In particular, the steel elements remain elastic with the exception of the T-stub components in which the rotation centre of the system was theoretically assumed during the design process. The cyclic behaviour has shown a symmetric response under hogging and sagging bending moment without degradation during the unloading and reloading phases. Validation of the FEM model by comparisons of results between finite element analysis and loading test are the topic of ongoing researches.

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