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BEHAVIOUR OF JOINTS WITH FRICTION DAMPERS UNDER COLUMN LOSS SCENARIO

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Abstract

In recent years, several researches have been oriented towards the Low-Damage structures which are able to minimize the earthquake damage by localizing the demand in easily replaceable parts. In this framework, the European research project FREEDAM introduced a particular type of friction dampers for steel structure placed at beam ends of the moment resisting joints. These type of dampers have shown an excellent response under monotonic and cyclic loading, and they have been pre-qualified under seismic loads. The aim of this study is to investigate the performance of these type of joints subjected to a column loss scenario; to this aim a refined numerical model was properly developed and validated against the experimental tests present in literature. The results has highlighted that, when subjected to large rotation, the damage is concentrated in the bolts of the upper T-stub which could leads to a premature collapse of the joint. Therefore, a strengthening solution was properly designed and numerically investigated. The introduced strengthening solution does not interfere with the global joint performance under monotonic and cyclic loads, but allow to enhances the tensile resistance of the upper T-Stub providing a very ductile behaviour.

Keywords: Steel structures, Beam-to-column joints, friction damper, robustness, numerical analyses.

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1. INTRODUCTION

Steel structure are widely used in seismic area [1-12] since they can provide a ductile behaviour dissipating all the earthquake incoming energy within proper design elements.

To this end in the European building codes a capacity design approach is introduced to concentrate all the plastic deformations within specific structural elements (i.e. beams, brace and link). Despite this principle works very smooth, it let arise the issue of the economic cost of repairing the damaged elements and of the complete functionality recovery of the structure. Often the cheaper option is that to demolish the entire structure and rebuilt it, with great waste of money and natural resources.

An alternative solution could be the Low-Damage philosophy, that localizes the damage in easily replaceable parts; in this way the inertial energy derived from the earthquake can be dissipated by plasticization of a limited number of components that can be substituted quickly returning the structure to full functionality. This entire process results in saving money and optimizing the aforementioned hierarchy of resistances itself.

Friction dampers connection have been studied by several authors mainly for seismic design and can be useful be adopted in a Low-Damage structure; among them, FREEDAM device [13-14] are symmetric friction dampers (Symmetric Friction Connection (SFC)) that can be placed at beam ends of a steel moment resisting frame (MRF).

FREEDAM dampers have been seismically prequalified in Europe thanks to previous studies [13-14]. These connections are similar to a bolted double split Tee connection, with the friction damper placed at the bottom of the connection where the T-stub is substituted by two L-stubs that enclose two friction shims and a gusset plate connected to the lower flange of the beam by means of a plate (see Figure 1). The gusset plate is welded to the aforementioned plate and this component is generally called "rib". Slotted holes are made on these component, namely L-stubs web, friction shims, gusset plate, to allow relative displacements and energy dissipation by friction. Friction shims are properly manufactured to have a precise friction coefficient. The connection is designed as partial strength, since the design bending moment resistance is lower than the bending moment resistance of the connected beam. The entire connection behaves as fully rigid up to the Design Level Earthquake, beyond which the rib can slide inside the friction damper giving a design bending moment resistance [12-15]. As shown by Latour et al. [14], this design resistance value can be imposed by choosing the corresponding value of bolt preload of the damper bolts.

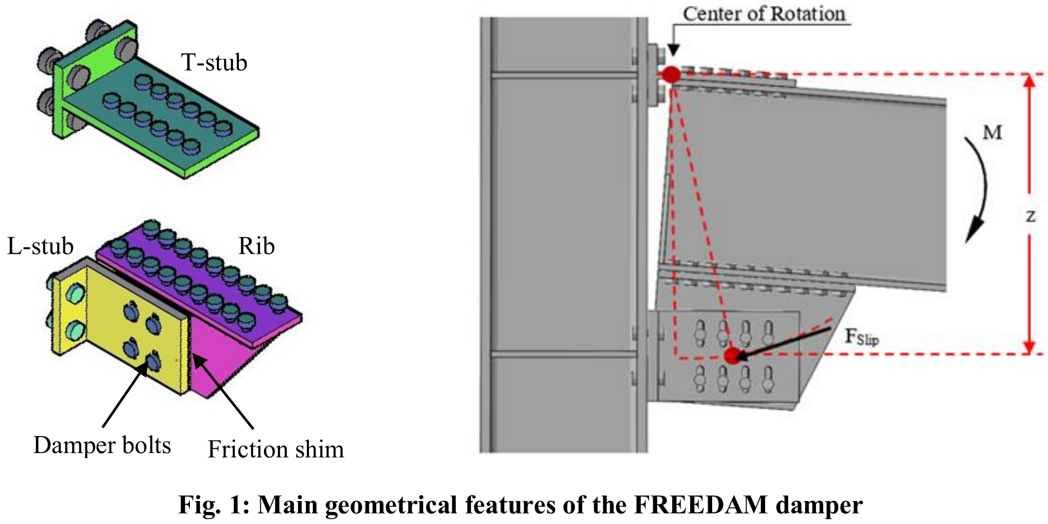


Fig. 1: Main geometrical features of the FREEDAM damper

FREEDAM connections were designed and pre-qualified to resist seismic events, without taking into account their performance in case of exceptional events (i.e. column loss, fire and explosions). Thus, the present study aim to investigate the effectiveness of this type of connection when subjected to a column loss scenario and in particular imagine the loss of the column adjacent to the investigated joints.

The paper is organised in three steps: in the first, the assessment of two beam-to-column assemblies was numerically conducted by means of FE analyses. Then a strengthening intervention was properly designed to increase the upper T-Stub capacity and finally its effectiveness was numerically investigated.

FREEDAM CONNECTION BEHAVIOUR

FREEDAM connection are designed to ensure the required strength and ductility supply, by complying with the design capacity principles. The friction damper should be the weakest component of the connection, while all the other components should be designed to resist the maximum force transmitted by the damper.

Design bending moment of the connection is given by Eq.(1):

$$M_{Rd,j} = \mu_{dyn} F_{p,C,d}\, n_b\, n_s\, h_s \tag{1}$$

Where:

- μ_{dyn} : dynamic friction coefficient of the friction pads;
- $F_{p,C,d}$: preload force for damper bolt, at long-term situation;
- n_b : number of damper bolts;
- n_s : number of shear surfaces;
- h_s : lever arm of the connection, that is to say the distance between the centre of the damping bolts and the upper flange of the beam.

The proper FREEDAM dampers (D1-D5) should be selected as prescribed by [22] in function of the beam-to-column assembly; in particular:

(i) Each FREEDAM device (D1, D2, D3, D4, D5) is suitable for a certain range of beam and column profiles. Bending capacity level $m = M_{Rd,j} / M_{pl,Rd,beam}$ of the connection (i.e. the ratio between the design bending resistance of the connection and the plastic bending resistance of the connected beam), is chosen according to the structural analysis of the beam-to-column assembly. This factor must be between 0.3 and 0.6 to guarantee the connection to be partial strength and a good behaviour of the friction damper [22]. The design preload force, that should be applied to the dampers bolts, is given by Eq.(2):

$$F_{p,C,d} = \frac{m\, M_{pl,Rd,beam}}{\mu_{dyn}\, n_b\, n_s\, h_s} \tag{2}$$

(ii) The value of $F_{p,C,d}$ must be between 0.4 and 1.0 of the preload force prescribed by EN1993:1-8 [16] for an optimal functioning of the device;

(iii) Rotational capacity depends by the length of the slotted holes on the rib, and the centre of rotation of the connection is located on the T-stub web close to the flange.

MODELLING ASSUMPTIONS

Numerical simulations were carried out by means of ABAQUS 2014 software [17]. Eight-node brick element with reduced integration (C3D8R) was adopted as *element type* to mesh all parts of the model. Sensitivity analysis was conducted to choose the more suitable mesh size [18-20]. Three steps were provided: an initial one for the definition of the boundary conditions, a second one for the application of the preload on the bolts, a third one to impose a controlled displacement at beam end. From Step Module, geometrical non linearities were accounted and a "Quasi-static" loading application was selected. True stress-true strain curves were used for all the materials, obtained from coupon tensile tests. Steel parts were made by S355 steel grade with an Elastic Modulus of 210000 MPa and Poisson's Ratio of 0.3, while the bolts were modelled as defined by Authors in a previous works [20-21].

The Material non linearity was modelled by using a Von Mises yield criterion with combined isotropic hardening. In the Interaction Module all contacts between parts were modelled explicitly as surface-to-surface interactions, with Penalty friction formulation for the tangential behaviour and Hard Contact for the normal behaviour, allowing the "separation after contact". Bolt preload was applied as Bolt Load in the Load Module, with a magnitude calculated according to EN1993:1-8, with the exception of the damper bolts, that were preloaded with the corresponding design value.

Boundary condition were applied to the Reference Point (RP) positioned in the central point of the corresponding cross-section, while rigid body constraints were imposed between this Reference Point and the remaining points of the same cross-section. Full penetration welds were modelled by means of Tie constraints between the two surfaces in contact. The joints performance were investigated under both monotonic loading (both hogging and sagging moment) and under column loss scenario, modelling the loss of the column adjacent to the investigated joints.

The boundary conditions are applied in the RPs and in particular: the column base is pinned, the column top has a slider that allows displacements along the column axis, and lateral-torsional restraints placed along the beam length. Beam tip is left free in case of monotonic hogging and sagging moment, while its axial elongation was prevented in case of column loss scenario. In all the investigated cases, displacement histories are applied at the reference point located at the beam tip cross-section.

INVESTIGATED JOINTS

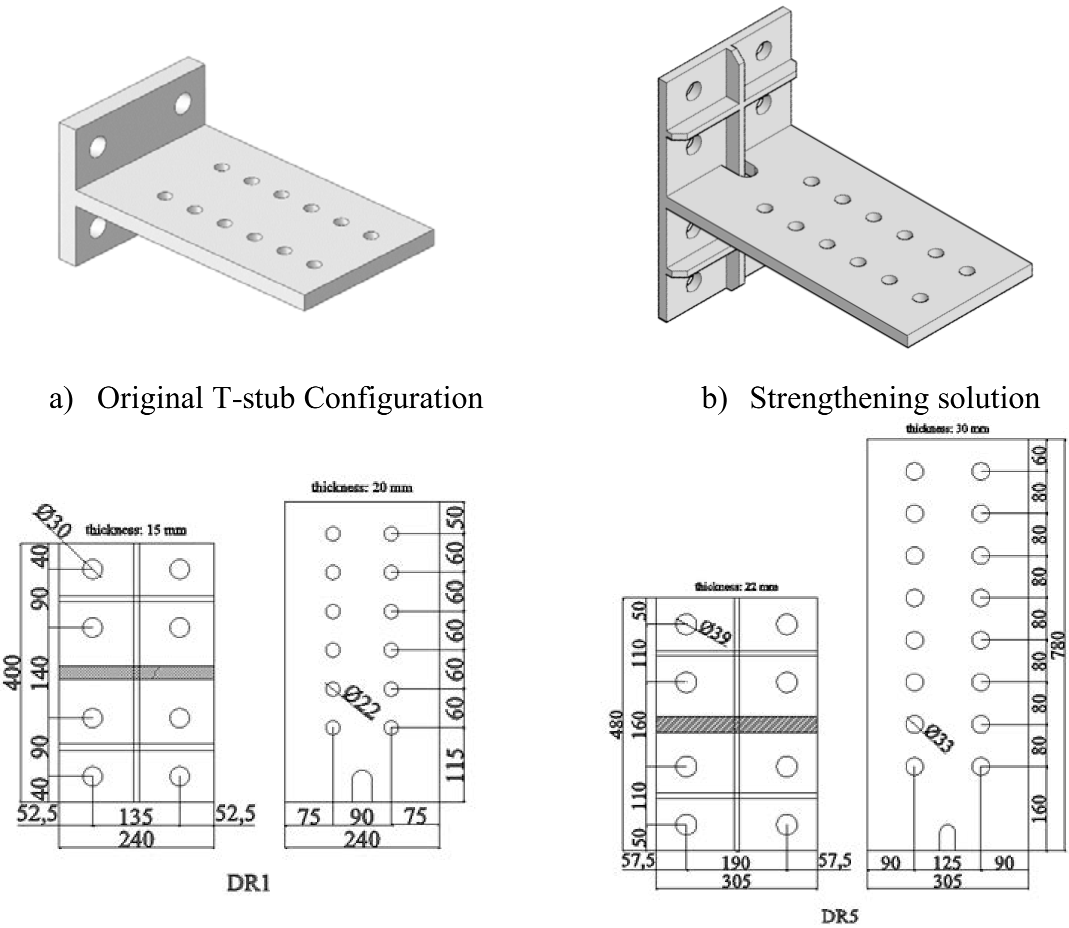
The two investigated beam-to-column assemblies were selected from a previous study [22], namely D1-A, with shallow beam and D5-C with deep beam; the assemblies main geometrical features are summarised in Table 1.

ID	Beam	Column	L_{beam} / 2 [mm]	L_{column} [mm]	Bolts [n_b x d]
D1-A	IPE270	HE 240 B	3000	3500	4xM16
D5-C	IPE 750 X 196	HE 650 M	4000	4000	8xM24

Tab 1: Investigated beam-to-column assemblies

The assessment of the T-stub behaviour is made according to the Equivalent T-stub theory, for each component of the bolt rows as indicated in EN1993:1-8 [16]. The ultimate resistance of each component *i* is given by the minimum resistance corresponding to three failure mode: (i) failure Mode 1 that implies the activation of four plastic hinges within the plates (very ductile), (ii) failure Mode 3 that implies the failure of the bolts (not very ductile) and (iii) failure Mode 2, where the plastic deformation are concentrated on both the plate and within the bolts (medium ductility). From the component methos it results that under column loss scenario all the investigated T-stubs (belonging to D1-A and D5-C) show a brittle failure (i.e. mode 3 or 2 very close to mode 3).

Therefore, in the present work a strengthening intervention was designed to increase the upper T-Stub resistance providing at the same time a ductile failure mode. The strengthening solution foresees the introduction of two additional bolt rows, reducing at the same time the upper T-Stub endplate thickness; moreover, both horizontal and vertical stiffeners were placed between all the bolts rows. The main geometrical features of the strengthened joints (i.e. D1-A-S and D5-C-S) are depicted in Figure 2.



a) Original T-stub Configuration b) Strengthening solution

c) D1-A-S d) D5-C-S

Fig. 2: Geometrical description of the adopted strengthening solution

NUMERICAL RESULTS

Assessment under monotonic loads

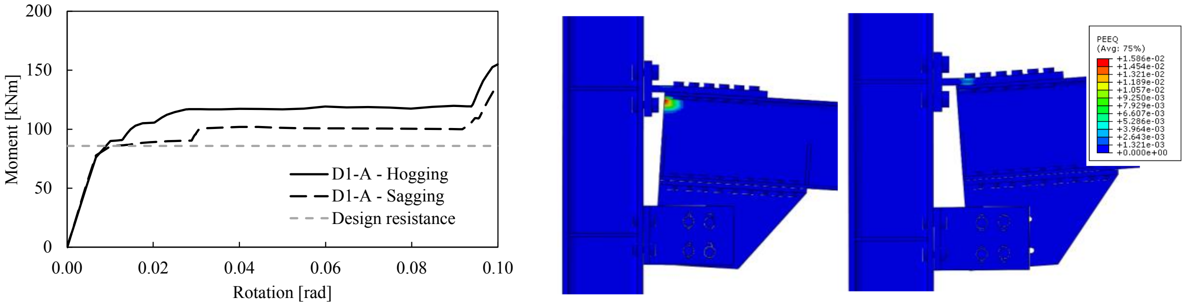
Figures 3 depicts the response of both D1-A and D5-C under monotonic action in terms of moment rotation curves and the distribution of the equivalent plastic deformations (PEEQ) at 8% of rotation. It can be observed that in line with the results provided by Tartaglia et al. [22], independently from the joint configurations and the dimension of the beam-to column assemblies, for both the joints under monotonic loads the same behaviour can be recognised; in particular the bending performance of the joints can be defined by three segment:

(i) an initial linear branch, where the connection behaves full rigid joint, up to the design bending moment resistance (dashed line);

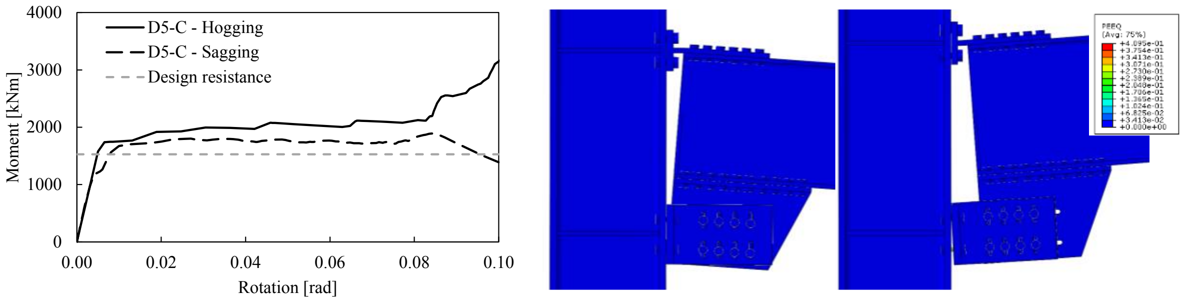
(ii) a constant branch, with a value close to the design bending moment resistance of the connection, up to a value of 0.08 rad of rotation;

(iii) a crescent non-linear branch, where the damper bolts have extinguished their stroke and they are in contact with the edge of the slotted holes of the rib.

It should be observed that, the monotonic response changes depending on the direction of the bending moment: the bending resistance under hogging moment is slightly higher with respect the response curve in sagging, due to the higher deformability of the L-stubs in tension [22].



a) b)



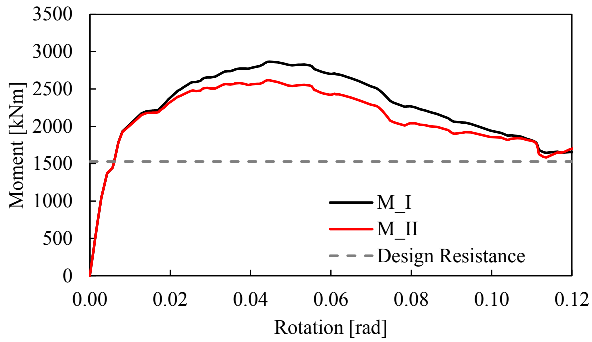
c) d)

Fig. 3: Numerical response in terms of moment rotation and PEEQ distribution of: D1-A (a and c) and D5-C (b and d)

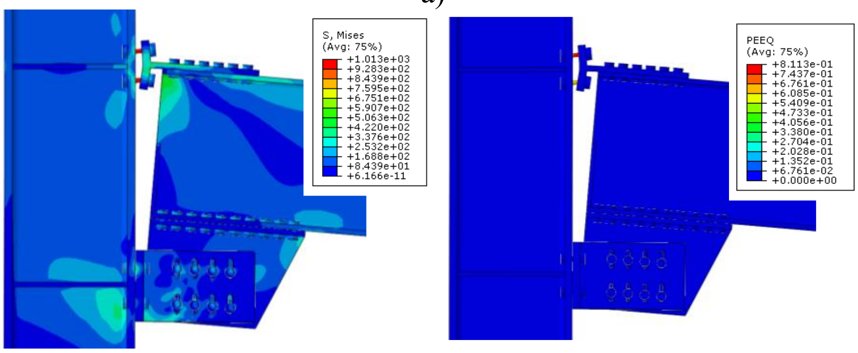
Assessment under column loss action

Figure 4 depicts the results of D5-C joint under column loss scenario, and in particular modelling the loss of the column adjacent to the investigated joint. Due to the large rotation reached and to the born of the catenary action within the connected beam, in the moment-rotation diagram both the first order moment (M_I) and the second order moment (M_{II}), evaluated accounting for the contribution of the catenary action to the bending capacity, were pointed out.

For small rotation, the presence of the catenary action is negligeable, while increasing the rotation an enhance of the joint resistance can be observed, with respect to the joint seismic performance. The joint show a good behaviour up to 6% of rotation, value above which a large concentration of plastic deformation develops within the upper T-Stub of the joint and a failure mode 3 can be observed.



a)



b)

Fig. 4: Column loss behaviour of D5-C in terms of: moment rotation curve, Von Misses stress and PEEQ deformation

Behaviour of strengthened solution

The performance of D5-C strengthened configuration (D5-C-S) is depicted in Figure 5 in terms of moment rotation curves, Von Misses stress distribution and PEEQ deformations.

As expected, and in line with the design purpose, the strengthening intervention does not influence the joint behaviour under pure bending moment; indeed increasing the T-Stub resistance and ductility does not influence the performance of the whole joints under both hogging and sagging moment providing almost the same behaviour in terms of elastic stiffness and design resistance (see Figure 5a and b).

Contrarywise, when subjected to column loss scenario, an important difference can be pointed out between the original and the strengthened joint configurations. In particular, it can be observed that the introduction of the strengthening intervention on the upper T-Stub does not influence the joint elastic stiffness and its maximum resistance, but it allow to reach up to 12% of rotation avoiding any brittle failure. Indeed, the new T-Stub configuration, implies a reduction of the tensile action within the bolts, concentrating the most of the plastic energy within the T-Stub flange. As expected indeed, despite the most of the internal action is concentred within the internal bolts, the presence the additional bolt rows and the small stiffeners welded on the T-Stub plate allow an internal redistribution among all the bolt rows.

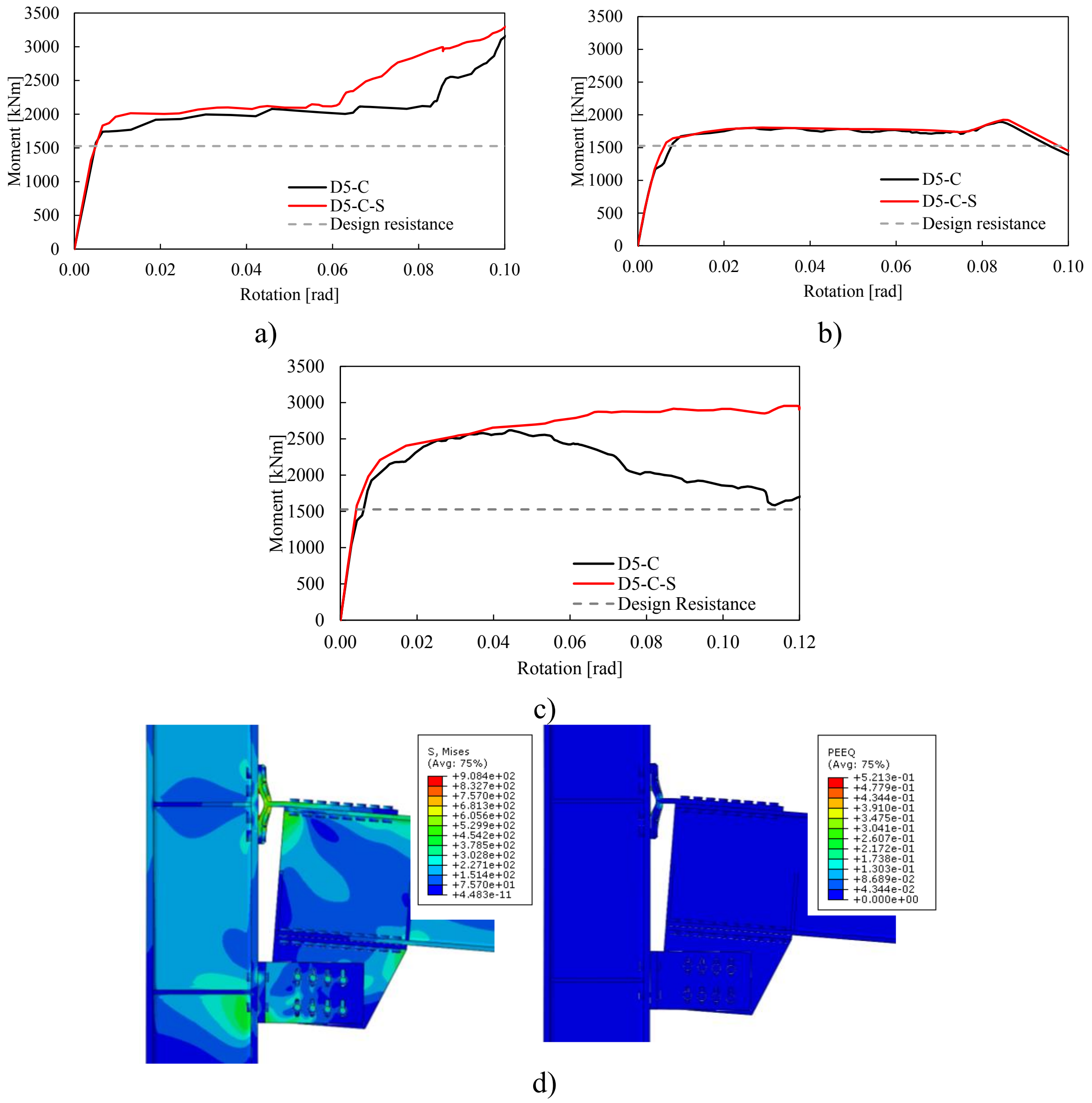


Fig. 5: Behaviour of strengthened joints under both hogging and sagging moment.

CONCLUSIONS

Several studies have shown that FREEDAM friction dampers are a valuable option for designing Low-Damage structures; in particular these kind of connections can be positioned at beams ends of a moment resisting frame, providing stable and predictable response dissipating the seismic incoming energy. However, the performance of this type of joints under column loss scenario, is still not investigated in literature.

Starting from the numerical results observed within the present work, the following conclusions can be pointed out:

- The monotonic response (i.e. both under sagging and hogging moment) of the FREEDAM joints show a stable behaviour up to 8% of rotation; an increase of the joint capacity, due to the contact between the plate and the dampers bolts, can be observed only for large rotational level (i.e. 10%).

- In case of column loss scenario, the activation of the catenary action within the connected beam, imply an increase of the joint resistance and at the same time a large concentration of the tensile action within the upper T-Stub that could exhibit a less ductile behaviour with the activation of failure mode 3.
- The introduction of two additional bolt rows with a thin flange of the T-Stub (i.e. with respect to the traditional FREEDAM joint) imply an increase the of the T-Stub resistance under tensile action allowing at the same time to obtain a very ductile failure mode (i.e. mode 1 or mode 2 close to mode 1).
- Despite the increase of local resistance of the T-Stub, when subjected to the seismic action, the strengthened joint response remains almost the same of the one observed by FREEDAM joints, avoiding any interference with the FREEDAM dampers behaviour.

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