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DESIGN FOR ROBUSTNESS OF STEEL STRUCTURES WITH DISSIPATIVE FREEDAM JOINTS

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SUMMARY: *The high seismic performance of steel frame structures equipped with FREEDAM dissipative joints has been demonstrated through experimental and extensive numerical studies in the framework of a recent RFCS project. In particular, the performance of moment-resisting frames (MRFs) and dual concentrically braced frames (D-CBFs) was thoroughly investigated under seismic actions. Although the research has shown that such structures can dissipate the earthquake-induced energy with limited or even no damage, their robustness in case of accidental actions was not yet addressed. Hence, as required by the current version of Eurocodes, their ability to survive accidental events should be demonstrated. This paper presents the robustness assessment performed on 20 MRFs and D-CBFs under a Eurocode-compliant column loss scenario. In addition, analytical and numerical studies were conducted to demonstrate how the request for robustness influences the design of the joints initially tailored for seismic performances.*

KEYWORDS: *robustness, dissipative joints, steel frames, column loss, FE analysis*

1 Introduction

The design of structures for earthquake resistance is covered by EN 1998-1 [CEN, 2004], where specific focus is made on the ductility of the dissipative zones and the overstrength of the non-dissipative ones, so that their failure is prevented. This precondition may lead to the overdesign of structural joints or to uneconomical solutions deemed to meet the requirements for non-dissipative zones, since these imply that the design of joints accounts for the overstrength of the dissipative regions (here the beam ends). To avoid unpractical joint configurations due to the high strength/stiffness demands, Eurocode 8 enables the designer to resort to dissipative partial-strength joints in which plastic hinges develop before reaching the plastic resistance of the beam ends. In this situation, the joints must exhibit an adequate rotational capacity which must be explicitly quantified. However, EN 1993-1-8 [CEN, 2005] dealing with the design of steel joints does not cover the case of joints subjected to cyclic loading, and practical guidance on how to ensure/estimate their ductility is not provided. Hence, experimental testing is the only reliable assessment method to prove the ductility of a partial-strength joint under cyclic loading what is incompatible with daily design practice, thus making this type of joints a highly unpractical solution.
As an alternative to the use of classical partial-strength joints, multiple variants of connections with energy dissipators (i.e., dampers) were proposed in the last three decades [Grigorian *et al.*, 1993; Clifton *et al.*, 2004; Latour *et al.*, 2015]. These connections dissipate seismic energy

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through friction in damping devices or engaging plasticity in highly ductile plate components. Such innovative joints were designed and tested under various load types in a recent research project [Piluso *et al.*, 2020], demonstrating their efficiency in dissipating the earthquake induced energy with limited or even no damage, thus proving their applicability in seismic-resistant structures.

Extensive experimental campaigns and numerical studies demonstrated the high seismic performance of MRFs equipped with FREEDAM joints [Piluso *et al.*, 2020]. Despite this, according to EN 1991-1-7 [CEN, 2006], which deals with the design against accidental actions, any structure should be robust enough to survive unforeseeable events without triggering a progressive/disproportionate collapse or excessive damages that could cause human losses. The standard provides several design strategies aimed at fulfilling minimum requirements in terms of robustness, yet the extent of accepted damages and the accidental scenarios to be considered in the design are matters of interest to all parties involved in the design and exploitation of the building (e.g., designer, owner, and relevant authorities).

Frame structures with FREEDAM joints are considered highly resilient structures since the post-earthquake damage is concentrated in easily replaceable parts, thereby, their pre-event performance can be restored through local and well-targeted interventions. However, their robustness is also of great interest due to the abnormal resistance and displacement demands expected in case of severe accidental events. Under the standard-prescribed scenario of the loss of a bearing element (i.e., columns, beams supporting columns, walls), the structural robustness represents a last resort for the structure's survival through the activation of alternate load paths. Recent research conducted on the topic [Francavilla *et al*., 2018; Demonceau, 2008; Kuhlmann *et al*., 2008; Demonceau and Jaspart, 2010], identified the joints as critical elements for the development of catenary actions that enable the structure to reach a new equilibrium state in the large displacements-large deformations field.

This paper addresses the influence of dissipative FREEDAM joints on the robustness of MRFs and dual concentrically braced frames (D-CBFs). The structures initially designed for seismic actions were subjected to column loss scenarios in line with the recommendations of the current version of EN 1991-1-7 [CEN, 2006], and their robustness was assessed through analytical and numerical methods. The investigation results allow drawing conclusions regarding the impact of dissipative joints on the structural performance of frames under accidental actions. Furthermore, specific limitations of these joints were underlined, yet these can be overcome through simple detailing solutions as proved hereinafter.

2 Case study

2.1 Investigated structures

The robustness of twenty frame structures (10 MRFs and 10 D-CBFs) equipped with dissipative joints was assessed in compliance with the current European normative requirements. In addition, perimeter frames that serve as lateral load resisting systems were extracted from structures for office buildings with structural layouts depicted in Figure 1. According to their overall height, two structures were analysed: low rise (4 storeys) and medium-rise (8 storeys).

The design of the reference structures was conducted according to the new draft of EN 1998-1-2 [CEN, 2019], which defines three ductility classes for earthquake-resistant structures (DC1, DC2, and DC3). Accordingly, for each height category of frames, three structures were

designed to fulfil the normative requirements regarding ductility demands. Additionally, the Theory of Plastic Mechanism Control (TPMC) [Montouri *et al.*, 2015] was used to design structures falling into the DC2 and DC3 ductility classes. These structural systems with different ductility classes were achieved by varying the sectional characteristics of members — beams varying between IPE220 and IPE600, and columns ranging from HE200B to HE800B. However, the same S355 steel grade was used for the elements of all structures regardless of the latter's height category or ductility class.
These distinctive structural characteristics serve to identify the analysed structures hereinafter by means of individual labels such as "4St_DC1_MRF_EC8", where 4(8)St stands for the number of storeys, DC1 (to DC3) stands for the ductility class, MRF (or D-CBF) designates the frame type, and EC8 (or TPMC) defines the method used for the structural design.

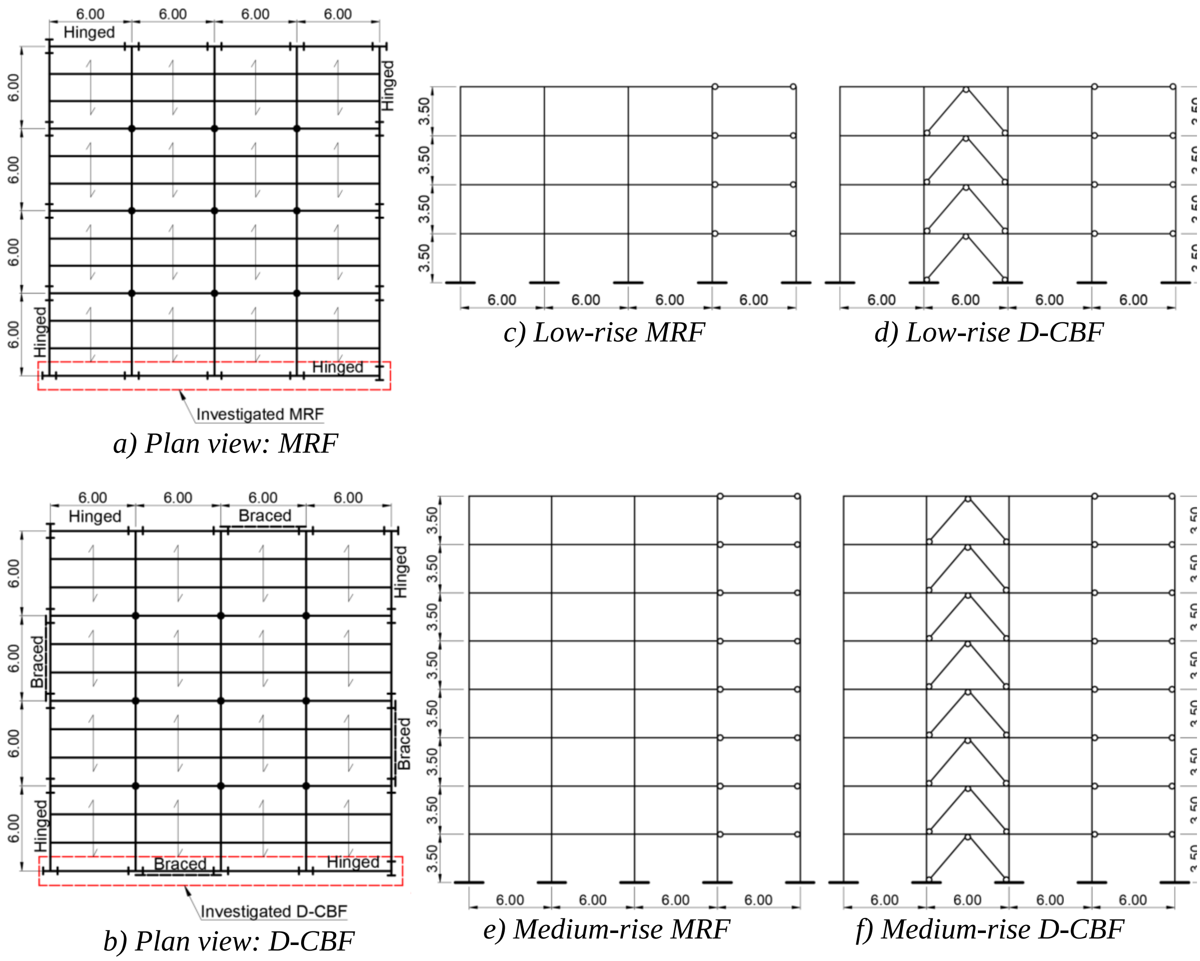


Figure 1 – *Structural layout of the analysed structures*

2.2 Characteristics of FREEDAM joints

Joints equipped with friction dampers allow for achieving high seismic performance in earthquake resistant structures by dissipating the seismic energy through friction in damping devices. The beam-to-column FREEDAM joints — recently prequalified for seismic applications in compliance with the European normative context [Piluso *et al.*, 2020] — are provided with symmetric friction connections comprising a damping device attached to the lower flange of the connected beam and a set of steel angles and T-stub elements connecting the beam to the column as shown in Figure 2.

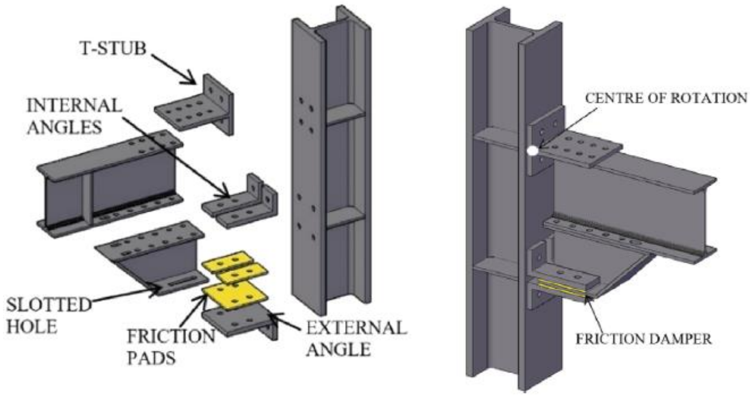


Figure 2 – *Detailed view of a FREDAM joint [Santos et al., 2019]*

The dissipative characteristics of these joints rely on the friction at the interfaces between the friction pads and the clamping force provided by high-strength bolts that fastens these pads and the steel plates of the device. Accordingly, the design bending capacity of the joints corresponds to the sliding onset in the friction device and may be controlled by adjusting the preloading force in the clamping bolts. Experimental tests and numerical analyses revealed that these joints exhibit a rigid pre-sliding behaviour. At the same time, the actual sliding resistance generally agrees with the design values [Latour *et al*., 2018]. Moreover, to ensure a local hierarchy, the design bending resistance of the joints is lower than the one of the connected beams. Hence, these joints may be classified according to EN 1993-1-8 [CEN, 2005] as rigid and partial-strength.

If properly calibrated, a FREEDAM joint may dissipate the seismic energy exclusively through friction in the damping device. However, in the case of high return period earthquakes, the dissipative friction capacity may be limited once the sliding between the friction pads is impeded by the so-called “stroke-end” limit condition where the bolts clamping the friction device reach the end of the slotted holes. Further to this moment, these bolts are subjected to shear, while other joint components (i.e., T-stubs, angles, and plates) are submitted to bending, bearing, and tension. Hence, the post-slippage behaviour of the FREEDAM joints is governed by the shear capacity of bolts and the ductile response of plate components. This specific range of behaviour is of great relevance to the structural robustness since, in column loss scenarios or lateral impacts, the likelihood of exceeding the damper’s sliding capacity is very high due to the significant demands in rotations and vertical/horizontal displacements typically met in these accidental situations.

2.3 Robustness requirements

The basic requirement to design and built robust buildings is established in EN 1990 [CEN, 2002]. Furthermore, design strategies and details on how this requirement should be fulfilled are provided in EN 1991-1-7 [CEN, 2006] dealing with the structural integrity of buildings and civil engineering works under accidental actions.

Generally, the measures to be taken and the design strategies to be used rely on the potential consequences of failure of the structure in terms of human life losses and economic, social, or

environmental impact. Accordingly, EN 1991-1-7 [CEN, 2006] classifies the buildings based on the risk associated with their failure into three main consequences classes (CCs): CC1 (low consequences), CC2 (medium consequences), and CC3 (high consequences).
Judging by the number of floors and the type of occupancy, the frames analysed in this paper can be classified as CC2 structures for which the minimum robustness-related requirement is to ensure a continuity between the structural elements through the so-called "tying method" such that, under unforeseen accidental events, horizontal ties would provide sufficient resistance allowing for the development of catenary action. This requirement is assumed to be fulfilled if each continuous tie, including its end connections, can sustain a design tensile force equal to the following values:

- for internal ties: $T_i = 0.8(g_k + \psi q_k)sL$ or 75 kN (whichever is greater) (1)
- for perimeter ties: $T_p = 0.4(g_k + \psi q_k)sL$ or 75 kN (whichever is greater) (2)

where g_k is the characteristic value of the permanent load, q_k is the characteristic value of the variable load, s is the spacing of the ties, L is the span of the tie and ψ is the relevant load combination factor in the accidental design situation.
Although the tying method is prescribed for designing all structures with consequences of failure above CC1, the level of structural robustness achieved through it remains uncertain and its application can be seen as a necessary but insufficient measure. Moreover, a solid scientific background does not endorse the analytical formulae and the numerical values for the tying resistance to be ensured, which raises doubts about the method's applicability.
Therefore, the use of more complex procedures such as the standard-prescribed "notional removal of supporting elements" approach is justified for any structure with significant consequences of failure. The requirement to be met is to prove that upon the removal of any supporting column (or beam supporting a column), the stability of the structure is not affected, and the extent of local damages remains under specific limits. Since the loss of a supporting member can be caused by a multitude of accidental events, this approach enables the designer to assess the robustness of a structure regardless of the initiating cause, thus covering a wide range of unidentifiable accidental actions. When the loss of a member triggers a progressive collapse or the local damage exceeds the predefined limit, the design should turn towards methods of local enhancement of resistance and ductility of the member under consideration.

3 Robustness assessment of frames with FREEDAM joints

3.1 Tying method

As reflected in Figure 3, the analysed frames should fulfil the tying requirements corresponding to perimeter ties. Accordingly, the beams and their connections should be able to transfer a force calculated according to Eq. 2. Based on the loads considered in the design of the structures, this leads to a tensile force to be resisted of:

$$T_p = 0.4(4 + 0.5 \cdot 3.5)1 \cdot 6 = 13.8 \text{ kN} \quad (3)$$

which is considerably lower than the lower bound of 75 kN prescribed by EN 1991-1-7 [CEN, 2006]. Therefore, to fulfil the requirements of the tying method, the tensile resistance of the beams and the FREEDAM joints should be greater than 75 kN.
Since even for the smallest section (IPE220) used for the beams, the axial resistance exceeds by more than 10 times this demand ($N_{pl,Rd}$=1185 kN), it can be concluded that all the beams in the analysed frames fulfil the requirements for perimeter ties.
The tensile resistance of the FREEDAM joints can be conservatively estimated based on the sliding resistance of the friction damper. Throughout the seismic design, this sliding capacity was calibrated from joint to joint based on the bending moment distribution in the beams in the seismic load combination. Accordingly, the lowest sliding resistance of the damper was assigned to the joints at the 4th floor of the 4St_DC3_MRFs_TPMC frame ($M_{j,Rd}$=61.1 kNm), which leads to a tensile resistance of 260 kN. Once again, this is well above the minimum demand for perimeter ties and confirms that the investigated frames meet the standard requirements in terms of tying resistance.

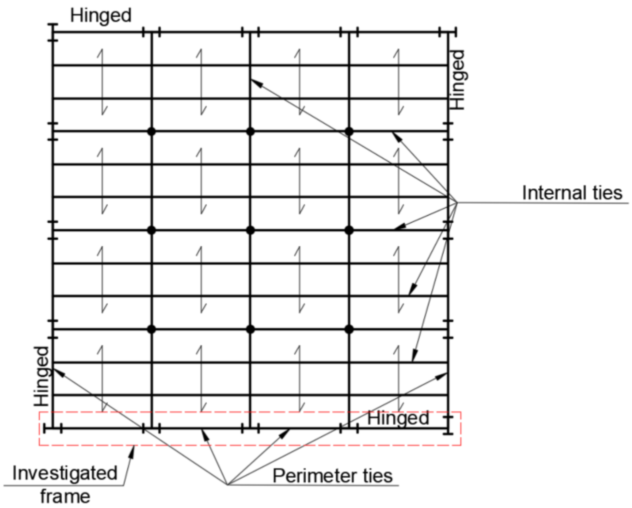
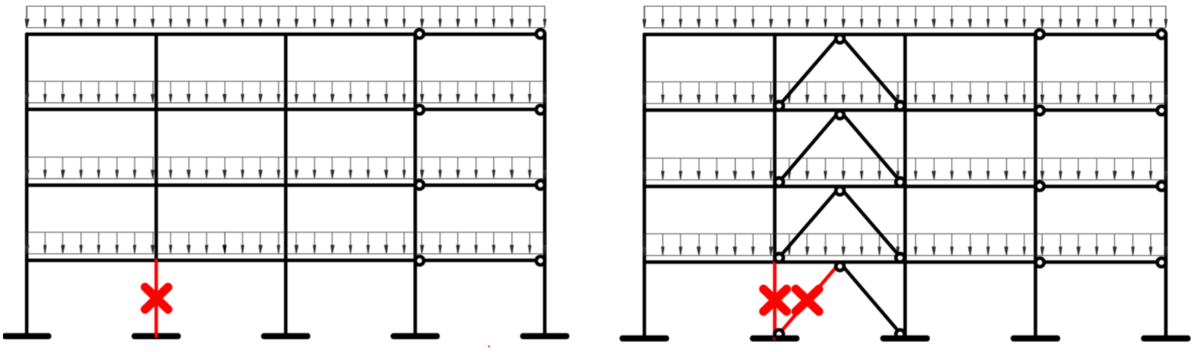


Figure 3 – *Horizontal tying system in the analysed structures*

3.2 Column loss scenario

Given the variety of accidental actions that can lead to it (e.g., fire, explosions, impacts), the column loss can be treated as a dynamic or quasi-static event. Generally, the dynamic effects induced in the structural response by transient actions are accounted for through Dynamic Increase Factors (DIFs). However, the lack of normative guidance on the use of appropriate DIFs and the few and only rough rules suggested for dynamic applications in the current version of Eurocodes does not allow for consistent and reliable robustness assessment in the case of dynamic column removals. Despite this, accidental events such as a localised fire could cause a quasi-static column loss, thereby excluding any dynamic effects.
In this paper, a static column loss is assumed to occur at the base floor of the frames, as illustrated in Figure 4. The same scenario was considered for both low-rise and medium-rise frames. For the D-CBFs, the brace connected to the base of the lost column had to be removed for consistency reasons.

The chosen location for the column removal ensures the activation of FREEDAM joints in both spans adjacent to the lost column, thus being relevant for the investigation of the influence of these joints on the overall robustness of the frames.



a) MRFs b) D-CBFs

Figure 4 – *Considered column loss scenario*

3.3 Preliminary analysis

The robustness of buildings under accidental scenarios is typically investigated through nonlinear numerical analyses, although their use is not justified for the full range of possible investigations for which rather simple yet effective alternatives are available.

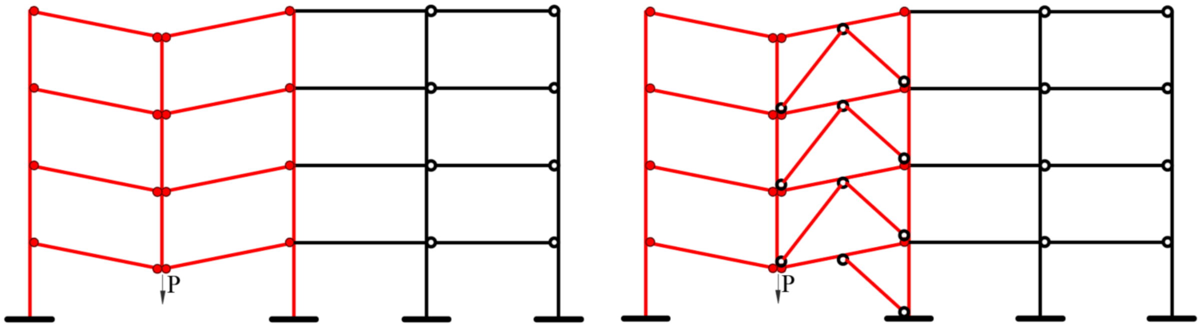
Provided that the plastic bending capacity of the joints (or beam ends in case of full-strength joints) is known, a plastic mechanism analysis can provide a good guess on the structural response further to a static column loss. Two possible outcomes are envisaged: either the structure enters the plastic range of behaviour, or it remains elastic (i.e., it survives the column loss without permanent deformations).

The bending resistance of the dissipative joints used in the analysed structures has been fine-tuned throughout the seismic design so that it matches the design bending moment (seismic load combination) at the column face. Since this bending resistance corresponds to the onset of sliding in the friction damper, it will remain constant until the sliding capacity of the device is reached (i.e., the stroke end). This allows defining a “pseudo-plastic” resistance of the joints associated to the sliding initiation in the damping device. Furthermore, the damper's response is identical for both loading directions which allows assuming a similar joint behaviour for both sagging and hogging moments.

Upon the static column loss, a plastic mechanism will be reached in a frame equipped with such joints once the pseudo-plastic resistance of all beam-to-column connections in the directly affected part (DAP) of the structure bridging above the lost column is attained (see Figure 5). Based on this assumption, a vertical force P corresponding to attainment of a full plastic mechanism can be easily computed. By comparing the plastic resistance P with the design axial force N_d acting in the column before its removal (accidental load combination), two possible situations are envisaged:

- $P > N_d$, meaning that further to the column removal, the sliding resistance of all joints in the DAP is not reached, and the structure may be considered as robust enough without performing any further nonlinear analyses;

- $P \leq N_d$, corresponding to the attainment of the sliding resistance in all joints of the DAP, which enters in a post-slippage phase (nonlinear response), a nonlinear analysis being required to assess the full-range behaviour of the structure.



a) MRFs *b) D-CBFs*

Figure 5 – *Plastic mechanism formed upon the column loss*

The preliminary results summarised in Table 1 indicate that only 3 out of 10 analysed MRFs exhibit a sufficient pseudo-plastic resistance to prevent the slippage in the joints after the column loss, while 6 out of 10 D-CBFs would respond elastically to the event. Accordingly, the 11 frames (7 MRFs and 4 D-CBFs) can be identified as "critical", and numerical analysis is required to assess their robustness in the assumed column loss scenario.

Table 1 – *Preliminary investigation results*

Structure ID	Axial force, N_d (kN)	Plastic resistance, P (kN)	P/N_d ratio
4St_DC1_MRF_EC8	484	329	0.68
4St_DC2_MRF_EC8	484	395	0.82
4St_DC3_MRF_EC8	484	349	0.72
4St_DC2_MRF_TPMC	484	318	0.66
4St_DC3_MRF_TPMC	484	318	0.66
8St_DC1_MRF_EC8	977	926	0.95
8St_DC2_MRF_EC8	977	1254	1.28
8St_DC3_MRF_EC8	977	1324	1.35
8St_DC2_MRF_TPMC	977	750	0.77
8St_DC3_MRF_TPMC	977	1149	1.18
4St_DC1_D-CBF_EC8	473	982	2.08
4St_DC2_ D-CBF_EC8	473	522	1.10
4St_DC3_ D-CBF_EC8	473	770	1.63
4St_DC2_ D-CBF_TPMC	473	376	0.79
4St_DC3_ D-CBF_TPMC	473	385	0.81
8St_DC1_ D-CBF_EC8	971	1124	1.16
8St_DC2_ D-CBF_EC8	971	1004	2.35
8St_DC3_ CBF_EC8	971	1035	1.07
8St_DC2_ CBF_TPMC	971	854	0.88
8St_DC3_ CBF_TPMC	971	1000	1.03

3.4 Numerical analysis

3.4.1 Modelling assumptions

The column loss was numerically simulated through nonlinear static analyses in the homemade finite element software Finelg [2019] for the MRFs and in SAP2000 [2019] for D-CBFs, respectively. Both software allows performing different types of analyses (e.g., elastic, nonlinear, static/dynamic) with account for geometric and material nonlinearities.
Full numerical models of the analysed frames were built using classical 3D and linear beam elements with material behaviour laws accounting for the yielding plateau and the strain hardening of steel material. The provisions of the new draft of prEN 1993-1-14 [CEN, 2020] were used to define the nonlinear behaviour law for the S355 steel as illustrated in Figure 6.

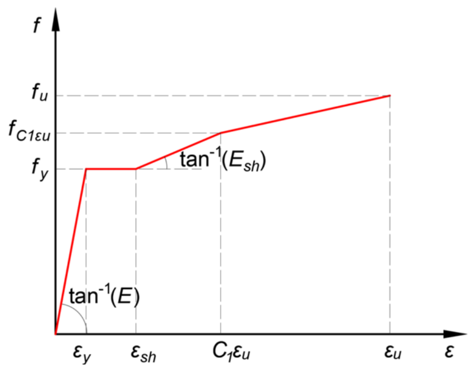


Figure 6 – *Material constitutive law (adapted from [CEN, 2020])*

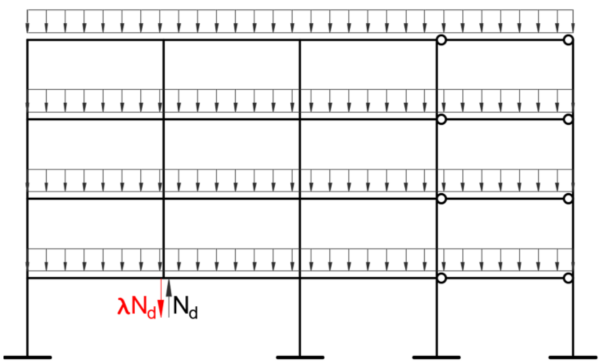


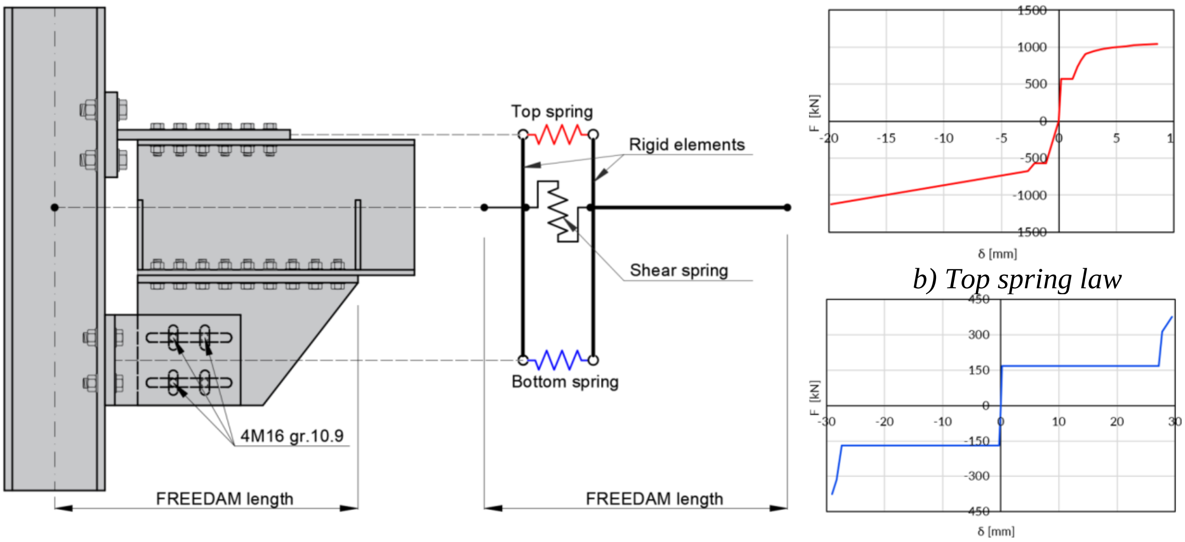
Figure 7 – *Column loss loading sequence*

Figure 7 illustrates the loading sequence used in the numerical analyses to simulate the static column loss. The two-step sequence consists of fictively replacing the lost column by applying a vertical reaction at the upper end of the column to be removed equal to the design axial force N_d in the column (in accidental load combination). Then, a nonlinear pushdown analysis is performed by applying at the same node an incremental downward force $F = \lambda N_d$. By plotting the vertical displacements against the applied downward force λN_d, the complete nonlinear response of the structure further to the column loss can be expressed through force-vertical displacement (F-$\delta_{vertical}$) curves. The frame may be considered sufficiently robust in the assumed scenario if a load factor $\lambda \geq 1$ is reached.

3.4.2 Modelling of FREEDAM joints

Depending on their characteristics (e.g., stiffness, strength, ductility), the joints may significantly influence the distribution of forces and displacements in frame structures. Given the dissipative feature of FREEDAM joints, the latter could affect the overall structural response even more than the conventional partial-strength joints.
Typically, the behaviour of joints is integrated in structural analyses through rotational springs simulating the flexural characteristics of joints. However, this modelling approach does not allow for a proper consideration of the moment-axial force (M-N) interaction in the joints, which makes it unsuitable for simulations of column losses since, in this situation, the joints

may experience a gradual shift from a bending-predominant to an axial-predominant loading along with the development of catenary actions in the DAP.
Multi-layer spring models derived with the component method introduced in EN 1993-1-8 [CEN, 2005] could potentially accommodate the *M-N* interaction, provided that the translational springs simulating the joint components are appropriately characterised. However, such sophisticated models pose significant challenges in their implementation in frame analyses and moreover, their use does not necessarily guarantee a satisfactory outcome.
Based on the component method, a simplified two-spring model for FREEDAM joints was developed and validated against experimental evidence in [D'Antimo, 2020]. The model consists of two extensional springs (top and bottom) interconnected by rigid elements as represented in Figure 8a. A rigid shear spring ensures the transfer of shear forces at the beam ends. The so-built model accommodates the *M-N* interaction and accounts for the behaviour of basic joint components characterised by extensional springs with nonlinear behaviour laws (see Figure 8b-c) derived from the component method. As demonstrated in [Santos *et al.*, 2020], the component method can be extended to effectively characterise both pre- and post-sliding behaviour of the FREEDAM joints. The method's application falls outside this paper's scope, yet relevant information regarding its applicability for FREEDAM joints may be found in the aforementioned references.



a) Two-spring model

b) Top spring law

c) Bottom spring law

Figure 8 – *FREEDAM joint modelling*

4 Results and discussion

To assess the reliability of the preliminary analytical investigation's reliability, the plastic mechanism analyses results are confronted with the numerically obtained nonlinear F-$\delta_{vertical}$ curves. Figure 11 depicts the behaviour of the low-rise MRFs further to the assumed column loss. A good agreement between the results of the two investigations is observed (the estimated force P corresponds to the "pseudo-plastic" capacity of the frames), which validates the assumptions made in the preliminary analytical study. Nonetheless, regarding robustness, the 6 MRFs show an inadequate response to the column loss (continuous red line in Figure 8), the structures collapsing under an applied load F smaller than N_d.

An in-depth analysis of the results and the failure modes associated with the collapse of the frames allowed concluding that the initial detailing solution for the friction damper leads to unsatisfactory structural performances in the column loss scenario for MRFs. More specifically, providing 2 M16 bolts, which slide within the same slotted hole in the haunch, halves the shear capacity of the whole friction damper. This is caused by the fact that at the stroke end, a single bolt per slotted hole leans on the slot end and thereby is engaged in resisting the shear. Since the shear resistance of 2 M16 bolts is considerably lower than the plastic resistance of any plate component of the device, a brittle failure by bolt rupture occurs. This significantly reduces the ductility of the whole frame; the collapse being triggered shortly after the dampers' sliding capacity is reached.

A solution to enhance the robustness of the frames is to ensure that at the end of the slippage phase, all the damper high-strength bolts are engaged in resisting the shear. This can be achieved by simply replacing the long-slotted holes with shorter slots for each bolt as illustrated in Figure 10. The proposed modification does not limit the sliding capacity of the device as long as the shorter slots provide the same horizontal displacement possibility. Moreover, the sliding resistance of the joint is not affected since the bolt preloading force remains unchanged.

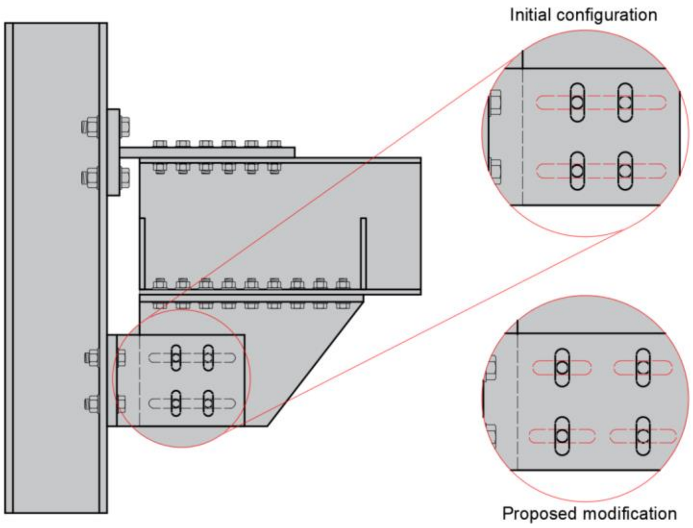
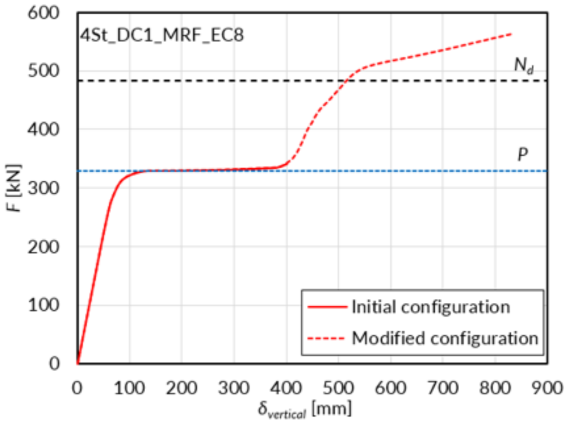


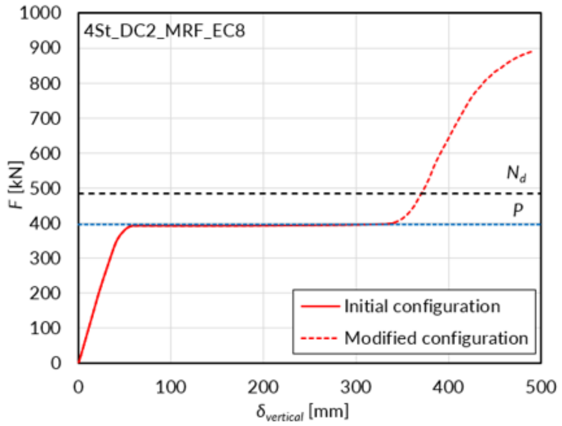
Figure 10 – *Proposed detailing of the FREEDAM joint for robustness enhancement*

As shown in Figure 11, the proposed detailing leads to a tremendous enhancement of the robustness of low-rise MRFs. By increasing the shear resistance of the friction device, the overall ductility of the joints increases, and the ultimate bending capacity of the joints increases, allowing all the frames to survive the event. Only the DC1 low-rise MRF exhibits a failure mode associated with the attainment of the ultimate strength in the beam sections close to the joints. This may be attributed to the rather low bending capacity of the beams (here IPE300) concerning the ultimate resistance of the joints.

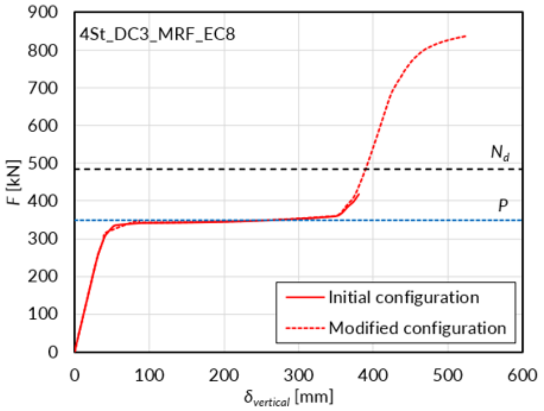
The collapse in the other low-rise MRFs still occurs as a result of the failure in shear of the high-strength bolts that fasten the friction damper, although significant yielding develops and propagates in the end sections of the beams as well.



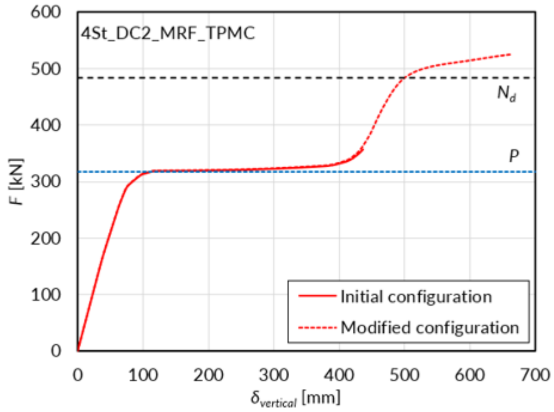
a) 4St_DC1_MRF_EC8



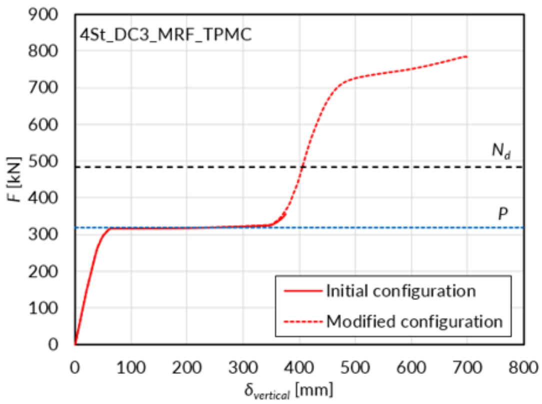
b) 4St_DC2_MRF_EC8



c) 4St_DC3_MRF_EC8



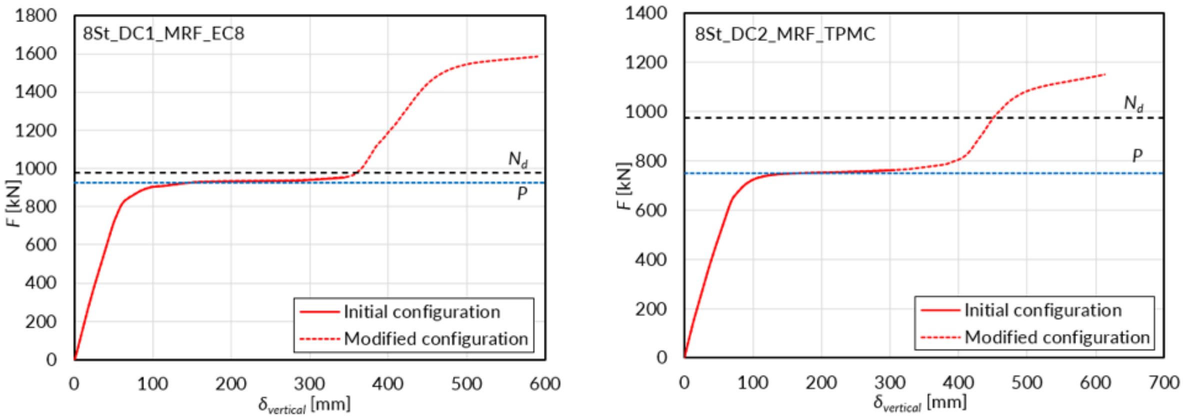
d) 4St_DC2_MRF_TPMC



e) 4St_DC3_MRF_TPMC

Figure 11 – *Structural response of the low-rise MRFs*

Similar observations can be made for the response of the medium-rise MRFs represented in Figure 12. Once again, the initial detailing of the dissipative device prevents the development of catenary actions in the DAP of the MRFs, the latter being unable to survive the column loss. The proposed modification overcomes this limitation and leads to satisfactory structural performances under the assumed scenario.

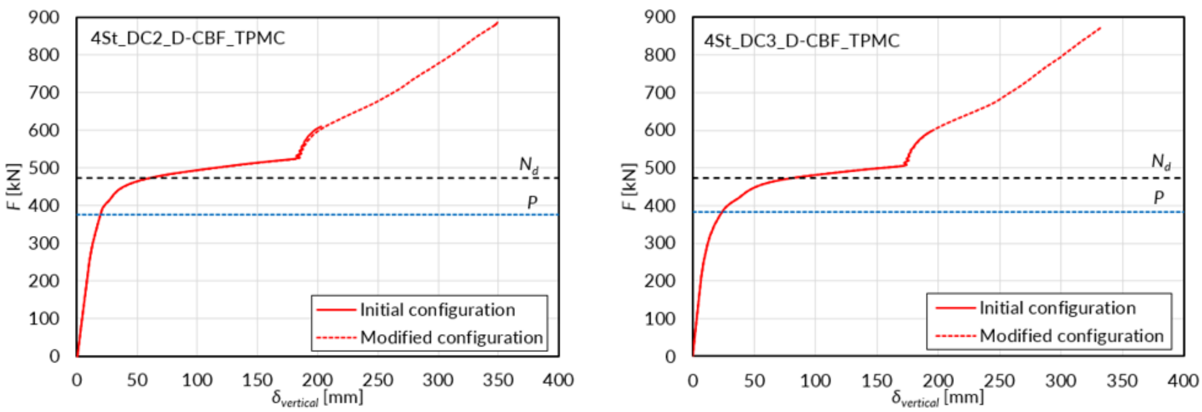


a) 8St_DC1_MRF_EC8 *b) 8St_DC2_MRF_TPMC*

Figure 12 – *Structural response of the medium-rise MRFs*

The F-$\delta_{vertical}$ curves reported in Figures 13 show a good agreement between the predicted plastic force P and the stiffness shift in the behaviour of the numerical models of the low-rise D-CBFs. It is worth noting that the response of the D-CBFs is slightly different than the one of MRFs due to the variance in the structural configuration. The results show that the D-CBFs are sufficiently robust to survive the column loss even with the initial detailing of the damper (i.e., with two slotted holes for 4 M16 bolts). The collapse of the D-CBFs is again initiated by the failure in shear of the high-strength bolts that clamp the friction device. However, by comparing the F-$\delta_{vertical}$ curves reported for the D-CBFs with the two joint detailing choices, it becomes clear that providing a slotted hole for each bolt represents a practical solution for both ductility and robustness enhancement.

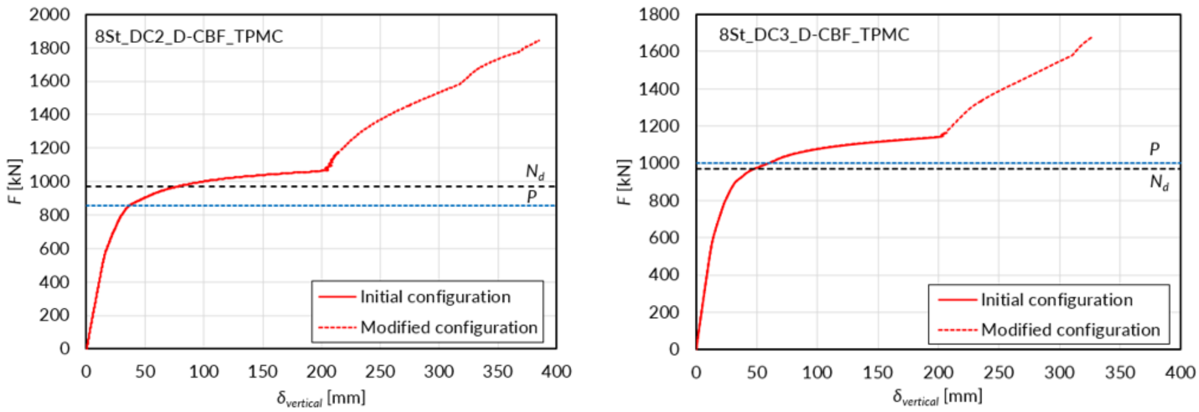
A slight reduction of the ductility with respect to the structures' height is observed. The higher number of floors above the lost column leads to the premature failure of the joints at the lower storeys of the medium-rise D-CBFs. In contrast, the joints at the upper floors are subjected to a relatively moderate combination of moments and axial forces.



a) 4St_DC2_D-CBF_TPMC *b) 4St_DC3_D-CBF_TPMC*

Figure 13 – *Structural response of the low-rise D-CBFs*

In the medium-rise D-CBFs with modified dampers, yielding occurs in the beam end sections in the unbraced bay after the slippage of the connections. This limited yielding can be prevented by reducing the ultimate resistance of the FREEDAM joints since, as shown in Figure 14, the structures exhibit a significant resistance reserve after the column removal. The onset of the frames' collapse corresponds to the shear failure of the bolts connecting the bracings to the beams above the 2nd floor.



a) 8St_DC2_D-CBF_TPMC *b) 8St_DC3_D-CBF_TPMC*

Figure 14 – *Structural response of the medium-rise D-CBFs*

5 Conclusions

This paper presents the robustness assessment performed on 20 MRFs and D-CBFs equipped with dissipative FREEDAM joints. Analytical and numerical analyses were employed to determine the structural performances of such structures subjected to a static column loss scenario. At first, a preliminary analytical investigation was conducted to identify the frames in which a plastic mechanism is likely to occur further to the column removal. Afterwards, nonlinear numerical analyses were conducted on the so-identified critical frames to investigate the influence of smart joint solutions on structural robustness.

Several key conclusions can be drawn based on the results of the case study presented in this paper, and more specifically:

- Although all the investigated frames fulfil the requirements of the tying method prescribed by the current version of Eurocodes, they are not sufficiently robust to survive an accidental column loss. This emphasises that the simple approach of ensuring a minimum continuity within the structures does not guarantee an adequate response in case of accidental events, and that this method should be considered as a necessary but insufficient measure for the design of robust structures.
- The agreement between the results of the preliminary analysis and the numerical analyses proves that, for specific cases, a simple plastic mechanism analysis is sufficient to verify if the structures exhibit adequate robustness to prevent the collapse after a static column loss.
- Seven out of ten MRFs equipped with FREEDAM joints are not sufficiently robust to survive the column loss. The collapse is initiated by the premature shear failure of the bolts that fasten the friction device.
- A practical and cost-effective solution to prevent the brittle failure of bolts in shear was identified in this study. The proposed modification implies a simple reconfiguration of the slotted holes provided on the haunch connected to the beam's lower flange, which ensures that at the end of the slippage in the friction device, four bolts are engaged in resisting the shear instead of two. The conducted numerical analyses validate the effectiveness of this

detailing choice; all the frames equipped with the modified friction device surviving the assumed scenario.

- All the analysed D-CBFs exhibited adequate levels of robustness to withstand the column removal even with the initial configuration of the FREEDAM joints. However, the adjustment proposed in this paper brings tremendous implications in both resistance reserve and ductility of the frames.
- Specific emphasis is required on the detailing in a robustness-oriented design of structures. The detailing optimisation proposed in this paper demonstrates how an increase attention to small details in the conception phase can enhance the overall structural performances over the lifespan of a structure.

6 Acknowledgements

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PROGETTAZIONE PER ROBUSTEZZA DI STRUTTURE IN ACCIAIO CON GIUNTI DISSIPATIVI FREEDAM

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SUMMARY: *Le elevate prestazioni sismiche delle strutture a telaio in acciaio dotate di giunti dissipativi FREEDAM sono state dimostrate attraverso studi numerici sperimentali e approfonditi nell'ambito di un recente progetto RFCS. In particolare, le prestazioni dei telai (MRF) e dei telai accoppiati con controventi (D-CBF) sono state accuratamente studiate sotto le azioni sismiche. Sebbene la ricerca abbia dimostrato che tali strutture possono dissipare l'energia indotta dal terremoto con danni limitati o addirittura nulli, la loro robustezza in caso di azioni accidentali non è stata ancora affrontata. Pertanto, come richiesto dall'attuale versione degli Eurocodici, dovrebbe essere dimostrata la loro capacità di sopravvivere a eventi accidentali. Questo lavoro presenta la valutazione della robustezza eseguita su 20 MRF e D-CBF in uno scenario di perdita di colonna conforme all'Eurocodice. Inoltre, sono stati condotti studi analitici e numerici per dimostrare come la richiesta di robustezza influenzi la progettazione dei giunti inizialmente pensati per le prestazioni sismiche.*

KEYWORDS: *robustness, dissipative joints, steel frames, column loss, FE analysis*

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