

## DESIGN AND ANALYSIS OF A STEEL SEISMIC RESILIENT FRAME EQUIPPED WITH SELF-CENTERING COLUMN BASES WITH FRICTION DEVICES

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**Abstract:** *In the last two decades many researchers focused on the development of innovative building structures with the aim of achieving seismic resilience. Among others, steel Moment Resisting Frames (MRFs) equipped with friction devices in beam-to-column joints have emerged as an effective solution able to dissipate the seismic input energy while also ensuring the damage-free behaviour of the system. However, to date, little attention has been paid to their column bases, which represent fundamental components in order to achieve resilience. In fact, column bases designed by current conventional approaches lead to significant seismic damage and residual drifts leading to difficult-to-repair structures. This work assesses the seismic performance of steel MRFs equipped with an innovative damage-free, self-centering, rocking column base joints, developed in accordance with the aims of the European project FREEDAM. The proposed column base consists of a rocking splice joint where the seismic behaviour is controlled by a combination of friction devices, providing energy dissipation capacity, and pre-loaded threaded bars with disk springs, introducing restoring forces in the joint. The design procedure of the column base is presented, a numerical OpenSees model is developed to simulate the seismic response of a perimeter seismic-resistant frame, including the hysteretic behaviour of the connection. Non-linear dynamic analyses have been carried out to investigate the effectiveness of the column base in protecting the first storey columns from yielding and reducing the residual storey drifts. The results show that the damage-free behaviour of the column bases is a key requirement when self-centering of MRFs is a design objective.*

### Introduction

In the last two decades, many research studies focused on the development of seismic design methods in order to improve the seismic performance of building structures. According to modern codes, structures must be designed to remain elastic or only slightly damaged in case of 'frequent' (low intensity) seismic events (*i.e.*, Damage Limit State). Differently, in case of 'rare' (high intensity) seismic events (*i.e.*, Ultimate Limit State) extensive damage is generally accepted. For this latter case, structures are typically designed to concentrate the seismic damage into specific zones, referred to as plastic hinges, whose ductility and energy dissipation capacity is properly designed through the adoption of specific detailing rules. At the same time, global ductility is achieved by capacity design rules with the aim of avoiding non ductile local failures and allowing the development of a global type collapse mechanism. In steel Moment Resisting Frames (MRFs), this strategy results in over-strengthened columns and connections leading to structures characterized by weak beams and column bases, with strong joints. This approach, from one hand, allows to reach the safety requirements specified in the seismic codes, on the other hand, has the drawback to lead to large economic losses. In fact, since dissipative zones belong to the main structural elements, after a destructive seismic event, the structure is often significantly damaged and characterized by large residual drifts. This implies high direct (*i.e.*, repair costs) and indirect (*i.e.*, business interruption) losses, which, in many cases, are not acceptable from both the social and economic prospective.

To overcome these drawbacks, in the last decades, many research works focused on the development of innovative structural systems, where the seismic damage is limited to easy to replace, or repair, dissipative fuses, promoting structural resilience. Within this framework, in steel

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MRFs, the conventional full-strength connections can be replaced by dissipative partial strength joints where yielding or friction devices (FDs) represents the weakest part of the connection. This approach allows to significantly improve the reparability of the structure while not affecting its seismic performance. Grigorian and Popov (1993) pioneered the first friction device to be included in beam-to-column connections. Successively, many theoretical and experimental works, as well as practical applications, were carried out developing the so-called Sliding Hinge Joint (SHJ) (*e.g.*, Clifton and Butterworth 2000). This is an asymmetric friction beam-to-column connection, including a supplemental energy dissipation system where the top flange of the beam is fixed to a plate from the column, while a bottom flange plate and a web plate with elongated holes permits sliding of a friction interface.

Recently, within the European project FREEDAM (FREE from DAMage Steel Connections, 2015-2018), a new low-damage joint typology based on Symmetric Friction Dampers has been proposed for application in steel MRFs (*e.g.*, Latour *et al.* 2018). This connection includes a slotted FD bolted to a steel haunch connected to the lower beam flange, slipping on friction shims pre-stressed with pre-loadable high strength bolts, thus ensuring a large energy dissipation capacity. This device, similarly to the SHJ, is designed to behave rigidly for low intensity earthquakes and to allow beam-to-column rotations for large seismic events.

Although the use of beam-to-column connections equipped with FDs can be an efficient solution to protect the frame components from damage, it does not allow to control the residual drifts. This issue has been tackled by several researchers by introducing elastic restoring forces able to control the rocking mechanism, in the form of post-tensioned (PT) bars. For example, Ricles *et al.* (2001) proposed an innovative lateral resisting system providing self-centering capabilities leading to minimal residual drifts under the design earthquake. In the proposed steel MRF, beams are post-tensioned to the columns by high strength PT strands parallel to the beams and anchored outside the connection, which is reinforced by steel angles where damage is expected.

Earthquake-resilient steel frames with beam-to-column connections including FDs have been extensively studied during the past decades, however, the application of FDs in column base connections is only a recent proposal. McRae *et al.* (2009) proposed two column base typologies based on the SHJ concept, able to provide energy dissipation capacity and, at the same time, prevent column yielding. Freddi *et al.* (2017) presented a rocking damage-free steel column base in which the dissipation of the seismic energy is provided by FDs and the rocking behaviour is controlled by high-strength steel PT bars. Simple analytical equations were used to describe the monotonic and cyclic moment-rotation behaviour while non-linear dynamic analyses were carried out to show the potential of the column base in preventing the first-floor column yielding and in eliminating the first storey residual drift. Similarly, Kamperidis *et al.* (2018) proposed a partial strength low-damage self-centering steel column base equipped with PT tendons to achieve self-centering behaviour and hourglass shape steel yielding devices, referred to as web hour-glass pins to dissipate the seismic energy.

Lately, within the context of the FREEDAM project, Latour *et al.* (2019) proposed and experimentally investigated, an innovative rocking splice connection where the seismic behaviour is controlled by a combination of friction devices, providing energy dissipation capacity, and pre-loaded threaded bars with disk springs, introducing restoring forces. The experimental tests demonstrated the damage-free and self-centering capabilities of this innovative column connection.

The present work numerically investigates and compares the seismic response of a traditional MRF with full strength beam-to-column and column base connections and an equivalent frame structure equipped with the innovative column base connection experimentally investigated by Latour *et al.* (2019). In both cases the beam-to-column joints are conventional full-strength welded joints and the design is performed in accordance with the Eurocodes. The main aim of the study is to assess the structural self-centering capabilities of the two systems and to evaluate the beneficial effect provided by the introduction of the innovative column base joint in reducing residual drifts after severe seismic events. In the following sections a case study structure is examined, addressing first the design of the column bases and then the seismic response of the two configurations. After introducing the main modelling hypotheses, the comparison is carried out in terms of residual and peak interstorey drifts (IDR) induced by the design and maximum credible earthquake (Ultimate Limit State and Collapse Limit State earthquakes according to the European definition).

### Case study frame

The considered case study structure is a 4-storey building with 5 bays in -x direction and 3 bays in -y direction. The considered layout has interstorey heights of 3,20 m except for the first level, whose height is equal to 3,50 m, while all the bays have spans of 6 m. Seismic resistant perimeter MRFs are located in the -x direction, while the -y direction is braced. Plan and elevation views of the case study frame are reported in Figure 1.

The case study frame is designed according to the Eurocode 8 and the Type 1 elastic response spectrum with a peak ground acceleration equal to 0,35g and soil type C is considered for the definition of the Design Based Earthquake (*i.e.*, DBE, probability of exceedance of 10% in 50 years). The building has non-structural elements fixed in a way so as not to interfere with structural deformations, and therefore, the interstorey drift limit for the damage limitation requirements (*i.e.*, under ground motions with 10% probability of exceedance in 10 years) is assumed as 1%. The maximum credible earthquake (MCE) is assumed to have intensity equal to 150% the DBE. The steel yield strength is equal to 355 MPa for the columns, 275 MPa for beams, and 900 MPa for the PT bars. The selected profiles are IPE 550 and IPE 500 for beams, while HE 600B and HE 500B for columns of the first and the last two stories, respectively. The study focuses on the assessment of the seismic resisting frame in the -x direction, where MRFs are employed.

The benchmark MRF is designed with rigid full-strength conventional column bases that promote the formation of plastic hinges in the bottom end of the first story columns. Differently, the MRF with rocking damage-free self-centering column bases (MRF-CB) fully protect the first storey column from yielding.

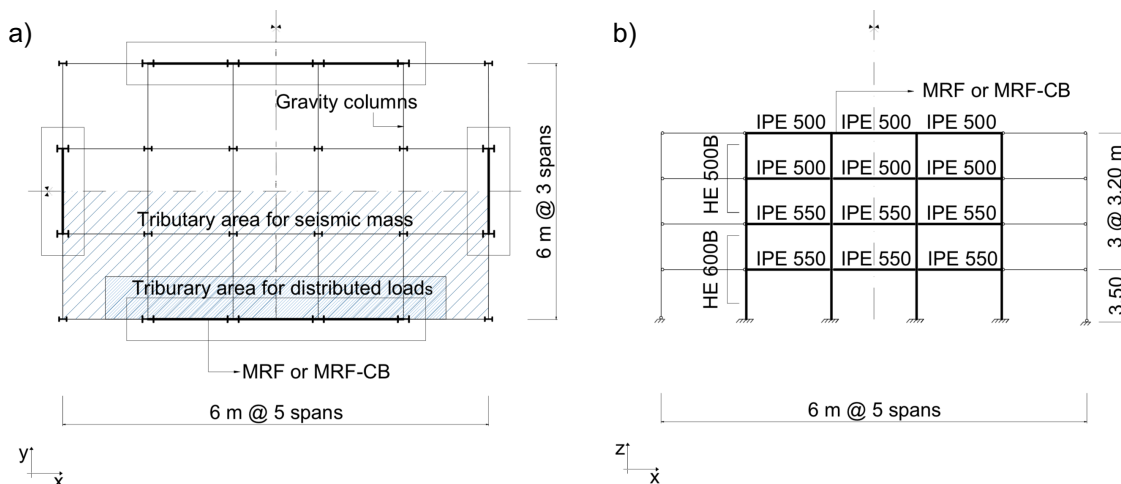


Figure 1. (a) Plan view and (b) Elevation view of the case study building.

### Column base connection design

#### Concept

The investigated column base connection, already tested by Latour *et al.* (2019), consists in a slotted column splice equipped with FDs assuring the seismic input energy dissipation capacity and PT bars with disk springs to introduce restoring forces in the joint, granting the self-centering behaviour. The symmetrical FDs are realized slotting the upper part of the column above the splice, adding cover plates and friction pads pre-stressed with high strength pre-loadable bolts on both web and flanges. In this way, the alternate slippage of the surfaces in contact, on which a transversal force is applied by means of high strength bolts, dissipates the seismic energy. The cyclic behaviour of FDs is characterized by a rigid-plastic hysteretic model, which depends on the clamping force and on the friction coefficient  $\mu$  of the contact interfaces. The self-centering behaviour is granted by a system composed of PT bars and disk springs, increasing the structure tendency to return towards the initial straight position at the end of the seismic event. Having the advantage of being slender, light and highly resistant, PT bars are a promising solution to create self-centering systems if properly combined with disk springs. In fact, a system of disk springs arranged in series and parallel forms a macro-spring element able to ensure sufficient deformability to the connection. The threaded bars alone typically do not possess a stiffness adequate to allow the gap opening without plastic deformations. To hold these bars, plates,

designed to withstand the self-centering force in elastic range, are welded to the column. In order to allow the gap opening, the holes are designed to accommodate a minimum rotation of 40 mrad which is the benchmark rotation established by AISC 341-16, for Special Moment Frames.

**Structural details**

According to design requirements of the Eurocodes, under both gravity and seismic loads, the first storey columns have sections HEB600 made of S355 steel. The column base is constituted by two parts, connected using S355 steel plates, fastened by HV M30 10.9 class bolts to the web and flanges, applied in both outer and inner parts of the column. Friction pads consist of 8 mm thermally sprayed friction metal steel shims placed between the steel plates and the column. Figure 2 shows a 3D view of the proposed column base connection and the required components.

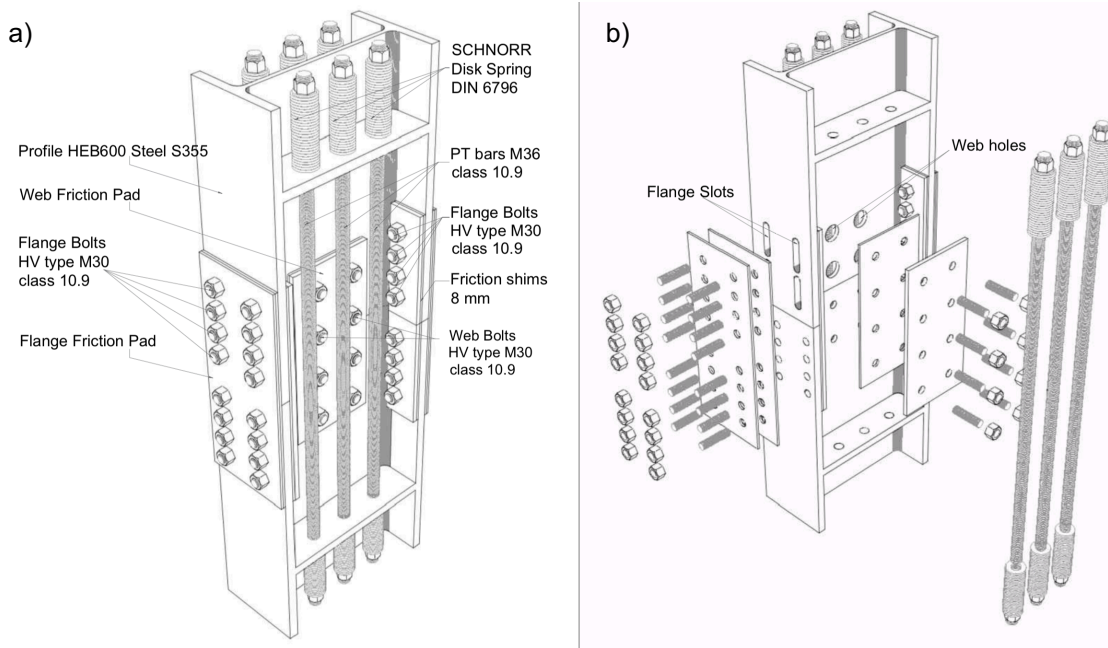


Figure 2. Details of the proposed column base connection: (a) 3D view; (b) Exploded 3D view.

**Moment-rotation behaviour**

The expected forces in each component during the rocking behaviour can be represented by imposing static equilibrium at the centre of rotation (COR) as reported in Figure 3(a).  $F_w$  and  $F_f$  represent the forces in the friction pads on the column web and flange respectively, while  $F_{TB}$  represents the sum of the forces provided by the threaded bars with disk springs.  $F_{TB,0}$  represents the post-tensioning forces while  $\Delta F_{TB}$  represents the additional forces as consequence of the gap opening. The moment-rotation behaviour of the column base connection is function of the response of each component and can be represented by the flag-shape curve of Figure 3 (b).

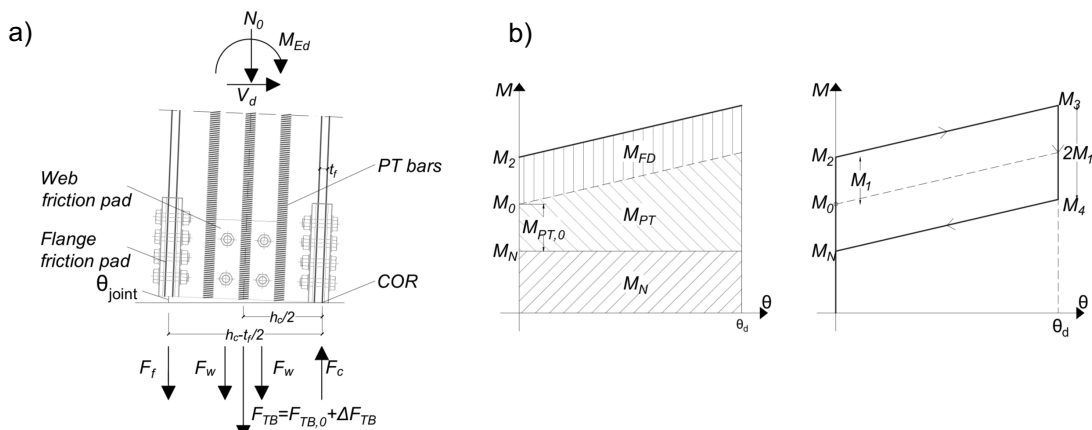


Figure 3. Behaviour of the connection. (a) Expected force interaction among the components during the gap-opening phase; (b) Theoretical moment-rotation relationship of the column base: contribution of the components and hysteretic behaviour.

As given in Figure 3(b),  $M_0$  represents the decompression moment due to the sum of the moment contribution due to gravity loads directly applied on the structure ( $M_N$ ) and to the moment provided by the PT bars at zero rotation ( $M_{PT,0}$ ).  $M_0$  can be calculated as follows:

$$M_0 = (N_0 + F_{TB,0}) \frac{h_c}{2} \quad (1)$$

where  $N_0$  is the axial load applied to the joint section,  $F_{TB,0}$  is the sum initial post-tensioning forces of the PT bars and  $h_c$  is the height of the column's section.  $M_1 = M_{FD}$  represents the contribution to the bending moment due to FDs and is equal to:

$$M_1 = F_f \left( h_c - \frac{t_{fc}}{2} \right) + 2F_w \frac{h_c}{2} \quad (2)$$

where  $F_f$  and  $F_w$  represent the sliding force in the friction pads of the column flanges and web.  $M_2$  is the moment that initiate the gap opening and is given by the sum of  $M_0$  and  $M_1$  while  $M_3$  is the maximum moment achieved at the design rotation  $\theta_d = 0,04$  rad. Its value is determined by accounting for the additional forces in the PT bars as consequence of the gap opening.

$$M_3 = M_0 + M_1 + \Delta M_{PT} \quad (3)$$

$$\Delta M_{PT} = \Delta F_{TB} \cdot \frac{h_c}{2} \quad (4)$$

$$\Delta F_{TB} = K_{eq} \theta_{joint} \cdot \frac{h_c}{2} \quad (5)$$

where  $K_{eq}$  is the rotational stiffness of the whole system, which can be determined as follows:

$$K_{eq} = \frac{K_{TB} \cdot K_{ds}}{K_{TB} + K_{ds}} \quad (6)$$

The stiffness provided by the threaded bars ( $K_{TB}$ ) and the disk springs system ( $K_{ds}$ ) are given respectively by the following relationships:

$$K_{TB} = \frac{n_b E_{tb} A_{tb}}{l_{tb} + l_{ds}} \quad (7)$$

$$K_{ds} = \frac{n_{par}}{n_{ser}} K_{ds1} \quad (8)$$

where  $n_b$  is the number of bars employed in the connection symmetrically placed with respect to the column centre,  $l_{tb} + l_{ds}$  is the bars length considering the total length of the disk spring system,  $n_{par}$  is the number of disk springs in parallel,  $n_{ser}$  is the number of disk springs in series and  $K_{ds1}$  is the stiffness of the single disk spring. It is important to stress that the maximum moment  $M_3$ , must be lower than the yielding moment of the column to avoid damage. Based on these equations, the self-centering behaviour of the connection is achieved if the moment generated by the axial components ( $M_0$ ), is higher than the moment provided by the FDs ( $M_1$ ).

$$M_0 \geq M_1 \Rightarrow F_{TB} \geq F_f \left( 2 - \frac{t_{fc}}{h_c} \right) + 2F_w - N_0 \quad (9)$$

The design actions for the design of the proposed column base connection are derived based on the seismic analysis of the equivalent frame with rigid full-strength conventional column bases. The column axial load  $N_0$  is assumed equal to the axial load considering the seismic combination for gravity loads and is equal to  $N_{0,int} = 400$  kN,  $N_{0,ext} = 200$  kN for the inner and outer column respectively. The columns' bending moment is calculated considering by the most unfavourable combination of axial forces and bending moments as required by the Eurocode 8 (§6.6.3) (*i.e.*,  $M_{Ed} = M_{Ed,G} + 1.1 \gamma_{ov} \Omega M_{Ed,E}$ ) where  $\gamma_{ov}$  is the overstrength factor and is assumed equal to 1.25 while the  $\Omega$  factor is the minimum value of  $\Omega_i = M_{Pl,Rd,i} / M_{Ed,i}$ , where  $M_{Ed,i}$  is the design value of the bending moment in beam  $i$  in the seismic design situation and  $M_{Pl,Rd,i}$  is the corresponding plastic

moment. The design bending moments are defined considering the proper location of the column splices and have been set respectively equal to  $M_{Ed,int} = 1985$  kNm and  $M_{Ed,ext} = 1875$  kNm for the inner and outer columns. Finally, the shear force is computed as  $V_d = M_{Ed}/L_o$ , where  $L_o$  is the shear length. Hence, the shear force is equal to  $V_{d,int} = 894$  kN and  $V_{d,ext} = 845$  kN for the inner and outer column respectively.

## Structural modelling

### *Frame modelling*

Two-dimensional finite element models of the frames are developed in OpenSees (e.g., Mazzoni et al. 2010). The structural models are able to describe the non-linear response of the system by detailed modelling of the components. Beams are modelled by a lumped plasticity approach where the internal part of the beams is modelled with elastic elements, while plastic hinges are modelled by zero-length non-linear rotational springs at beams' ends. The rotational behaviour of these non-linear springs follows a bilinear hysteretic rule based on the modified Ibarra-Krawinkler deterioration rule implemented as suggested by Lignos and Krawinkler (2011).

Differently, in order to capture the axial force-bending moment interaction, columns are modelled with non-linear beam-column elements and each section is discretized into 4 fibres along the depth. The non-linear behaviour is represented using the distributed plasticity concept where the plastic behaviour occurs over a finite length. The 'Steel01' material of OpenSees for 355 MPa yield strength and 0.002 post-yield stiffness ratio is employed. At the beam-column connections, the panel zone is modelled with the 'Scissor' model (e.g., Charney and Downs, 2004) where two rigid links are connected to a single hinge at the centre. The relative rotation of the two rigid links is controlled by two rotational springs representing the shear behaviour of the panel zone and bending behaviour of the column flanges, as with the 'Krawinkler' model (Krawinkler et al. 1987). The panel area of the column is stiffened with doubler plates, to promote the plastic engagement of the beams only.

The rigid-floor diaphragm is modelled by assigning a high value to the axial stiffness of the beams. Gravity loads are applied on the beams by considering the seismic combination of the Eurocode 8, while the masses are concentrated at the beam-column connections. Damping sources other than the hysteretic energy dissipation are modelled through the Rayleigh damping matrix where the values of the mass-related and stiffness-related damping coefficients are considered for a damping factor of 3% for the first two vibration modes.

### *Column base modelling*

A two-dimensional finite element model of the proposed column base is developed in OpenSees (e.g., Mazzoni et al. 2010) as reported in Figure 4. The column is modelled with non-linear force-based fibre elements associated with the 'Steel01' material of OpenSees for 355 MPa yield strength and 0.002 post-yield stiffness ratio. The rigid elements of the rocking interface are modelled with 'Elastic beam-column' elements with very high flexural stiffness and are used to connect the lower and the upper part of the column through non-linear springs. These springs are represented by four bilinear 'zero-length' elements in parallel with gap elements to simulate the bilinear hysteretic response of the FDs and the contact behaviour of the column interfaces. FDs for both flanges and web are modelled by the 'Steel01' material in OpenSees considering a very high initial stiffness and very low post-elastic stiffness in order to model the rigid plastic behaviour of the FDs. Differently, the contacts elements are by the 'Elastic compression-no tension' (ENT) material of OpenSees, which exhibits an elastic compression-no tension force-displacement behaviour. The compression stiffness of the contact spring is assumed equal to 10 times the axial stiffness of the column.

Additionally, a central 'Zero-length' translational springs with bilinear elastic-plastic behaviour is used to model the force provided by PT bars with disk springs. In this case, being symmetrically placed, the six PT bars can be modelled by a single central spring with the stiffness of the whole system. The initial post-tensioning force is modelled by imposing an initial strain equal to  $F_{PT}/A_{PT}E_{PT}$  by using the 'Initial strain material' along with the elastoplastic material 'Steel01'.

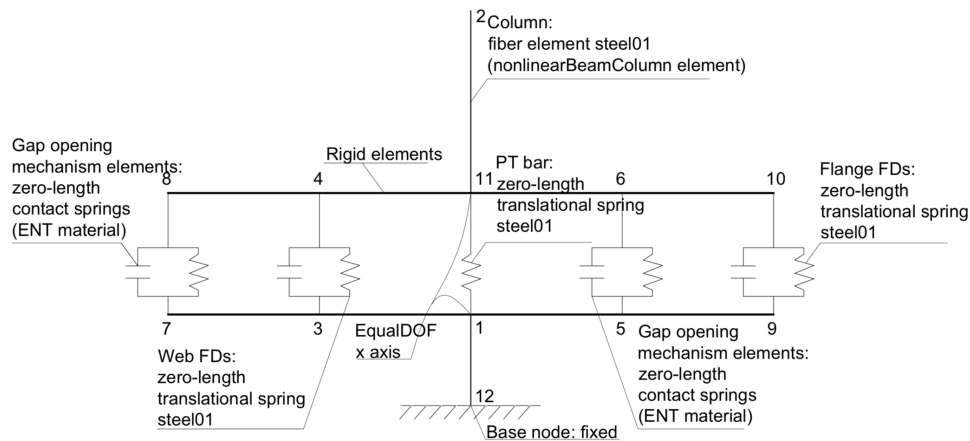


Figure 4. Two dimensional OpenSees model of the column base connection.

*OpenSees model validation*

In order to validate the finite element model in OpenSees (e.g., Mazzoni et al. 2010) a comparison of the numerical and experimental results is carried out. Several full-scale cyclic tests on the proposed damage-free self-centering rocking column base with FDs were performed at the Laboratory of Materials and Structures of the University of Salerno (Salerno, Italy) as part of the experimental investigation of the FREEDAM project (2015-2018). The experimental campaign was performed on a HEB 240 specimen for the column and further information are provided in Latour et al. (2019). Numerical and experimental results are compared for 5 cyclic tests, varying the axial load in the column, the pre-loading force in the bolts of the FDs and with and without the self-centering PT bars. In order to account for the bolts' pre-loading loss during the experimental test the friction coefficient has been fictitiously reduced of the 20% with respect to the experimental value. For the sake of brevity, the comparison for two tests is reported in Figure 5 showing a good correspondence between the two hysteretic curves.

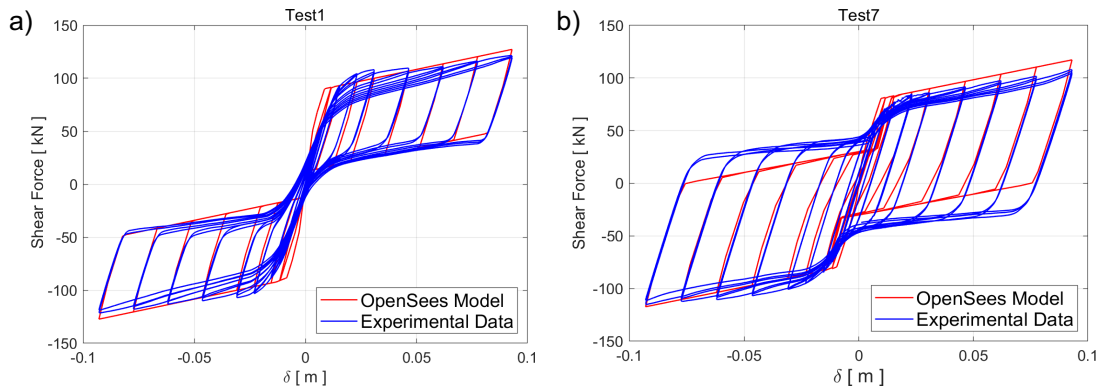


Figure 5.  $F$ - $\delta$  behaviour of the column base connection. Comparison between OpenSees and Experimental results for the TEST 1 (a) and TEST 7 (b).

**Non-linear Incremental Dynamic Analysis**

Non-linear dynamic analyses are performed in order to assess how the proposed column base influence the seismic response of the frame. The MRF with conventional columns bases and the same frame including the proposed column bases are analysed and the results compared. The fundamental period of vibration  $T_1 = 0,74$  sec is the same for both the structures and is used as intensity measure (IM). A set of 30 natural ground motions are selected from the SIMBAD Database (e.g., Iervolino et al. 2010) and scaled to increasing values of IM to cover the whole range from elastic to non-linear seismic response up to collapse in order to perform Incremental Dynamic Analyses (IDAs) (e.g., Vamvatsikos and Cornell, 2002). Global and local parameters are recorded allowing the comparison of the seismic performance of the two systems. Figures 6 (a) and (b) show the comparison of the first storey displacements of the two frames for a single ground motion record respectively for the DBE and MCE intensities demonstrating the



effectiveness of the proposed column base in reducing the residual drifts of the first story. For the same ground motion, Figure 7 (a) and (b) show the distribution of residual drifts at all the storeys respectively for the DBE and MCE intensities. It can be observed that, despite a self-centering system is present only at the first storey through the introduction of the proposed column bases, they allow a reduction of the residual drifts on the whole structure. In this case, for the DBE and MCE, the MRF-CB experiences values of residual drifts lower than 0,5 % often considered the threshold beyond which repair of the building may not be economically viable (e.g., McCormick et al. 2008). This limit is not satisfied at MCE for the structure with full-strength column bases. Figure 7 (c) and (d) report the peak interstorey drifts for the DBE and MCE intensities showing that the introduction of the proposed column bases does not affect the maximum response parameters of the structure.

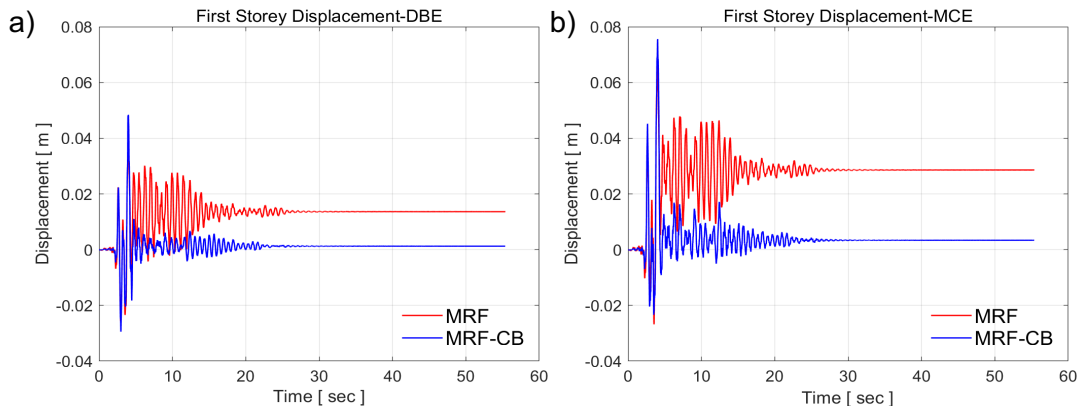


Figure 6. 1<sup>st</sup> storey displacement time history for (a) DBE and (b) MCE intensities

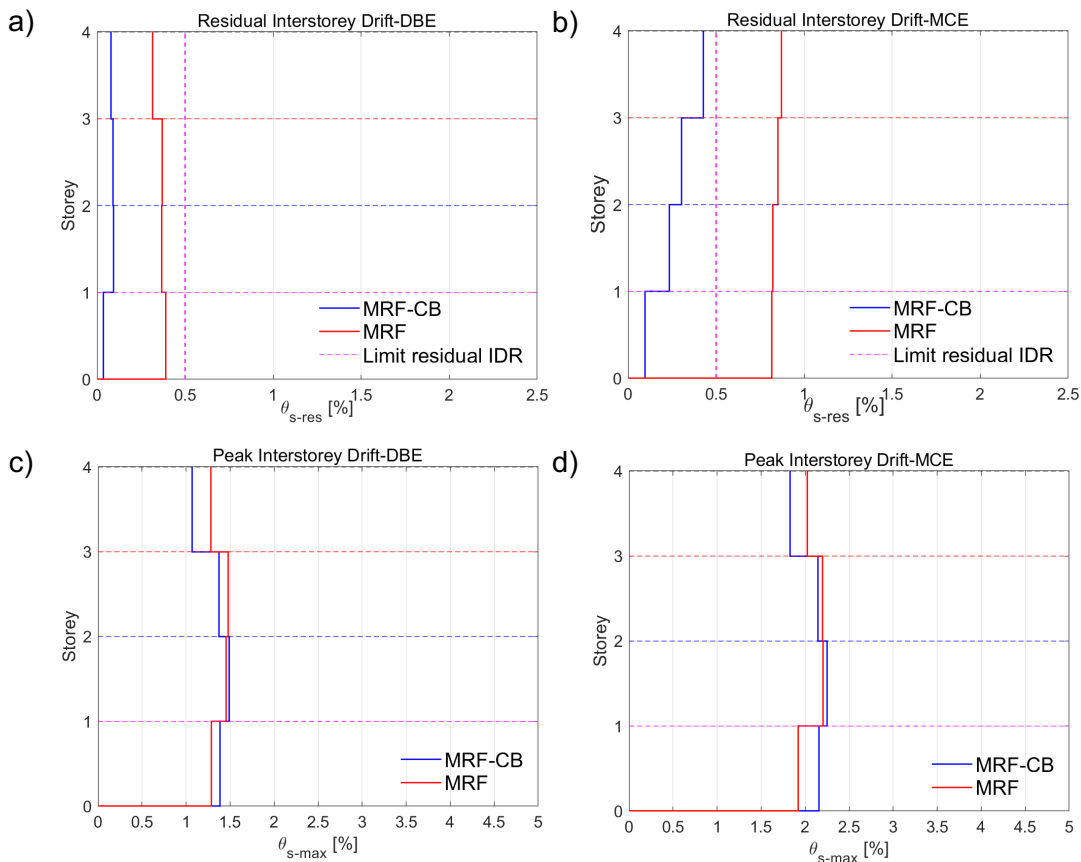


Figure 7. Residual drifts for the (a) DBE and (b) MCE intensities; Peak interstorey drifts for the (c) DBE and (d) MCE intensities.



For the same ground motion, results highlight that in both structures, beams develop plastic hinges, while bottom sections of the first storey columns remain in elastic range for the MRF-CB, since the column base connections protect them from yielding, as expected. Moreover, webs and flanges of the panel zones remain in elastic range for both the two structures. Figure 8 compares the moment–curvature hysteresis in the bottom sections of the two structures, showing that the MRF with conventional column bases experiences plastic deformations and damage, thus leading to the need to be repaired after strong earthquakes, while the MRF-CB fully protects the columns from yielding under DBE and MCE.

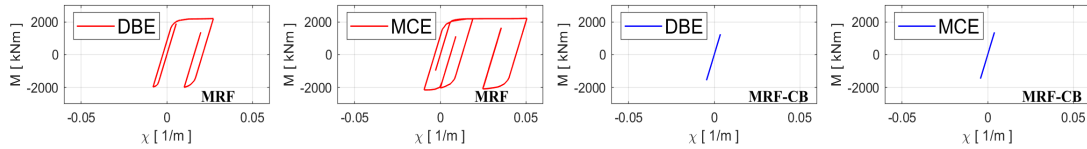


Figure 8. Moment-curvature relationship in the bottom section of one of the first storey columns of the MRF and MRF-CB for a specific ground motion scaled at DBE and MCE.

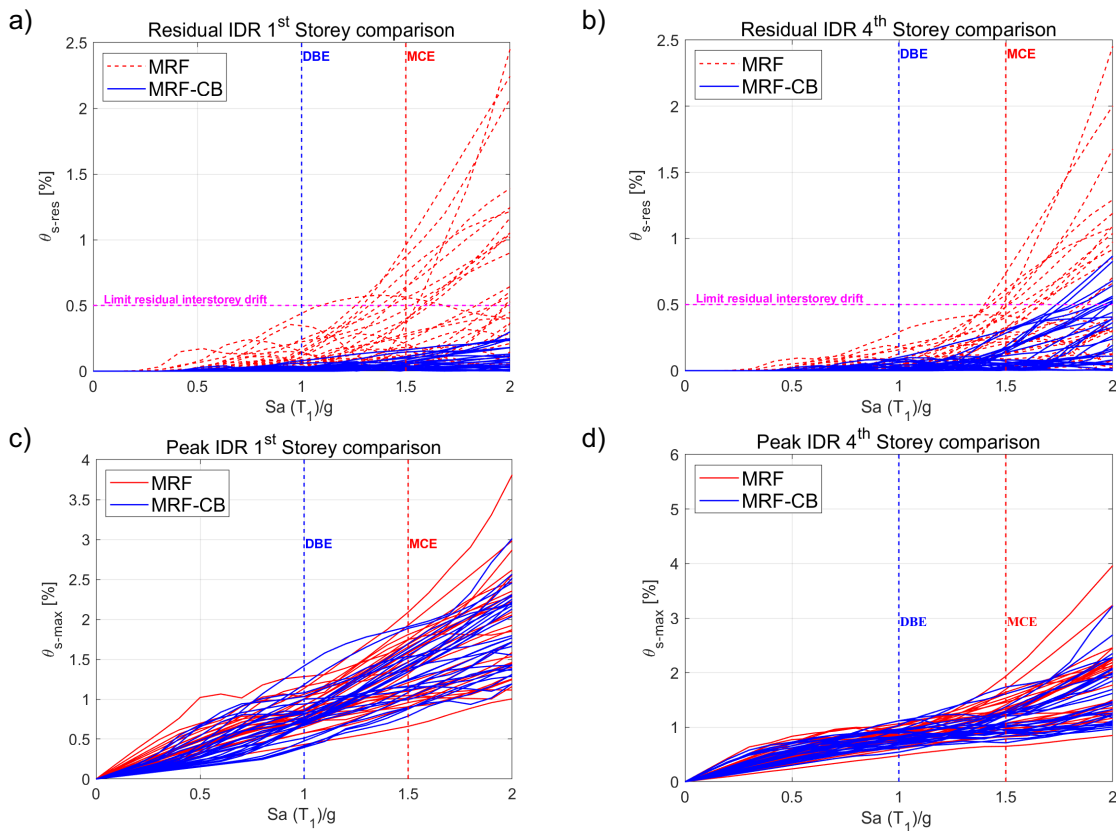


Figure 9. IDA comparison of the results MRF – MRF-CB.

Figure 9 shows the results of the IDA for the residual and peak interstorey drifts (IDR) for the 1<sup>st</sup> and 4<sup>th</sup> storey. The results confirm what observed in Figures 6 and 7 while considering a larger number of ground motions. Despite a large record-to-record variability can be observed, especially while looking at the residual drifts, the results highlight that the inclusion of the proposed column base is always beneficial.

### Summary and conclusions

This work proposes a column base connection composed of a rocking splice joint where the seismic behaviour is controlled by a combination of friction devices, developed in accordance with the aims of the European project FREEDAM. The column base is implemented on numerical OpenSees model to predict the seismic behaviour of a perimeter Moment Resisting Frame belonging to a case-study building. Non-linear dynamic analyses are performed to validate the effectiveness of the proposed column bases connections. The following conclusions are drawn:

- The global behaviour of the structure is significantly enhanced by the self-centering capability of the column bases in limiting residual deformations, under both basis and maximum credible earthquake intensities.
- The column bases fully protect the first storey column from yielding, thus avoiding non-reparable damage, even under strong ground motion events.

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