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RESEARCH ARTICLE

Comparison Between Different Design Strategies For Freedam Frames: Push-Overs and Ida Analyses

Rosario Montuori[1], Elide Nastri[1], Vincenzo Piluso[1], Simona Streppone[1,*], Mario D'Aniello[2], Mariana Zimbru[2] and Raffaele Landolfo[2]

[1]*University of Salerno, Department of Civil Engineering, Fisciano (SA), Italy*
[2]*University of Naples "Federico II", Department of Structures for Engineering and Architecture, Naples (NA), Italy*

Abstract:

Background:

Modern seismic code design rules are known to be based on capacity design principles. They try to assure the damage to occur in the ductile parts of the structure, such as beam ends while the other have to remain in elastic range. Therefore, in the aftermath of design earthquakes, plastic deformations at member or connection level will imply high repair costs. In the last decades, innovative structural solutions based on the so-called supplementary energy dissipation strategy allow increasing the dissipative capacity of structures through equipping it with special damping devices. In the case of substitution of dissipative zones with dissipative devices the strategy takes the name of substitutive strategy. This is the case of Moment Resisting Frames investigated in this paper, where traditional dissipa-tive zones, are equipped with innovative low damage frictional devices. However, the current version of codes does not provide any rules to design of MRFs equipped with this type of friction joints.

Methods:

Therefore, this paper reports two design approaches amply investigated and compared. The first one is based on the application of the Beam-to-Column Hierarchy Criterion (BCHC) while the second one exploits the Theory of Plastic Mechanism Control (TPMC). The comparison between them is herein discussed on the basis of the results of nonlinear static and dynamic analyses.

Conclusions:

Structures equipped with low damage frictional connections show larger drift demand than conventional Moment Resisting Frames. However, differently from traditional structures, the larger displacement demand of MRFs equipped with friction joints does not corre-spond to structural damage, thus allowing the reparability of the structure.

Keywords: Seismic Design, MRF, Free from damage structures, Push-over, IDA, TPMC.

1. INTRODUCTION

Steel Moment Resisting Frames (MRFs) designed in accordance with modern seismic design codes, *e.g.* Eurocode 8 [1] and the AISC 341-16 [2], need to resist seismic events by forming plastic hinges at the beam level [3 - 5]. In this way, a collapse mechanism of global type can be achieved also if first storey columns base sections are involved in plastic range, with the assurance of safety preservation but with the downside of high repair costs in the earthquake aftermath. In fact, dissipative zones,*i.e.* beam ends during a seismic event are widely involved in plastic range and are very hard to be substituted. For this reason, in last decades, research efforts were addressed to the practical development

* Address correspondence to this authors at the University of Salerno, Department of Civil Engineering, Via Giovanni Paolo II, 132, 84084, Fisciano (SA), Italy; Tel: 089963411; E-mail: sstreppone@unisa.it

of new strategies whose goal is the design of connections able to withstand, almost without any damage, not only frequent and occasional seismic events, but also destructive earthquakes such as those corresponding to rare and very rare events. The basic idea of these new researches [6 - 14] is inspired by the strategy of supplementary energy dissipation, but it is based on the use of damping devices under a new perspective. While passive control strategies have been commonly based on the integration of the energy dissipation capacity of the primary structure by means of a supplementary dissipation coming from damping devices; the new design strategy is based on the use of friction dampers conceived, in such a way, to substitute the traditional dissipative zones of MRFs,*i.e.* the beam ends. To this scope, beam-to-column connections can be equipped with friction dampers which can be located either at the level of the two flanges [6 - 10] or at the bottom flange level only [15]. Also, the beam web-to-column flange connection can be equipped with friction dampers. Moreover, symmetrical or asymmetrical friction devices can be exploited [16, 18]. In this work, MRFs are equipped with innovative frictional connection called FREEDAM dampers, where the dissipative zone, constituted by a layer of frictional material is located at the bottom flange of the beam ends.

Within the ongoing RFCS FREEDAM project, friction devices able to dissipate the seismic input by means of relative sliding at the beam ends are prequalified. Using this type of device, the MRF is designed for drift considerations just like an EC8 compliant frame, while the friction connection (damper) is designed as the dissipative element (substituting the beam plastic hinge), considering the seismic combination forces [17]. In this way, the traditional MRFs become structures with sufficient stiffness, less overdesigned and low post-earthquake damage, owing to the less demanding hierarchy rule and the dissipation mechanism.

The friction connections dissipate energy by relative sliding of plates held in contact by preloaded bolts. The friction properties are given by the friction pads,*i.e.* special plates that undergo surface treatments, while the slipping part Fig. (**1**) is made out of stainless steel, due to its higher superficial hardness. The hysteretic response of this joint typologies is strictly related with the material of the friction pads, but typically they exhibit stable cyclic behaviour with minimal resistance, stiffness and ductility degradation [19 - 20]. Therefore, the damage in the structural elements is highly mitigated and localized in easily substitutable parts, at the friction connection level, meaning that frames equipped with friction joints could be called free from damage moment resisting frames (FD-MRFs).
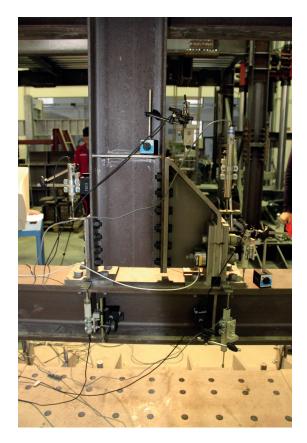

Fig. (1). Beam-to-column connection equipped with friction dampers.

The design of FD-MRFs can be assumed to be similar to one of the traditional MRFs, although, the former frames have particularities that require attention. In particular, starting from EC8 compliant frames [21], the dissipation mechanism is different (the plastic deformation is replaced by relative sliding). Therefore, the overstrength coefficient to be taken into consideration for the design of non-dissipative elements needs to be evaluated differently.

Indeed, the material overstrength factor γ_{ov} is no longer relevant, as well as the material uncertainty coefficient 1,1. Furthermore, the design overstrength, in the way it is typically defined (the ratio between the beam flexural strength M_{Rd} and the design bending moment M_{Ed}) is no longer valid due to the change of the dissipative mechanism.

The work hereby presented focuses on two procedures for the design of FD-MRFs: the first one is compliant with the design rules based on the so-called beam-column hierarchy criterion, while the second one exploits the Theory of Plastic Mechanism Control (TPMC). The two methodologies are denominated as follows: (i) FD-BCHC where the structure is designed according to beam-column hierarchy criterion; (ii) FD-TPMC where the structures are designed according to TPMC. In particular, two sets of frames were designed and analysed considering the proposed methodologies: the first one with full strength connections and the second one with FREEDAM connections. Finally, the response of the designed structures is investigated by means of push-over and IDA analyses, and differences between both the design approaches are pointed out.

2. DESIGN PROCEDURES

2.1. EC8 compliant design methodology

The MRFs equipped with friction devices can be designed following the recommendations of Eurocode 8, with minor alterations to the capacity design procedure. The hereby proposed procedure uses the same algorithm for the initial design of the structure *i.e.* once the geometrical configuration is established and the forces are applied (considering the EC1 recommendations) the SLS limitations are verified. This will be the condition that primarily dictates the beam and column member profiles. The following step consists in the design of the dissipative element (the friction connection) for the design forces coming from the seismic combination. In particular, the slip force (F_{slip}) is calibrated for a level of moment compatible with the bending moment at the end of the beam from the seismic combination (M_{Ed}). Equation 1 depicts the way to determine the design force for the friction connection, considering the notations previously described and assuming z as the lever arm between the compression and tension forces.

$$F_{slip} = \frac{M_{Ed}}{z} \tag{1}$$

Subsequently, the non-dissipative elements (beams, columns, connections) are designed considering an overstrength with respect to the dissipative element. The overstrength factor Ω is essential for this step, but it is evaluated based on different considerations compared with the typical plastic mechanism. Indeed, the yield strength variability, the hardening of the steel or the beam overdesign are no longer relevant for the capacity design. Instead, the variability in terms of dynamic and static friction coefficient (μ) and the clamping force applied at the damper level, are both determinants for the sliding phenomena, contribute to the definition of Ω_μ (Eq. (2)).

$$\Omega_\mu = \frac{\mu_{st.95\%}}{\mu_{dyn.5\%}} \frac{N_{b.95\%}}{N_{b.5\%}} \tag{2}$$

where:

- $\mu_{dyn.5\%}$, $N_{b.5.\%}$ lower-bound values of dynamic friction coefficient and tightening force
- $\mu_{dst.5\%}$, $N_{b.95.\%}$ upper-bound values of static friction coefficient and tightening force

The design forces for the non-dissipative elements are evaluated based on Eq. (3).

$$E_d = E_{d.G} + \Omega_\mu E_{d.E} \tag{3}$$

where E_d is the design value of the internal forces for non-dissipative elements, $E_{d.G}$ is the effect coming from gravitational loads and $E_{d.E}$ is the effect coming from the seismic action.

The resistance and stability checks for the members (beams and columns) and the design of connections, must be

done according to EN 1993 1-1 and 1-8, respectively. In the end, the local hierarchy at the node must be checked, again, as recommended by EC8.

2.2. TPMC Compliant Design Methodology

TPMC is based on a rigorous theoretical approach assuring a collapse mechanism of global type [22], which exploits the kinematic theorem of plastic collapse extended to the concept of mechanism equilibrium curve:

$$\alpha = \alpha_0 - \gamma \delta \tag{4}$$

where, following the theory of rigid-plastic analysis, α_0 is the first order collapse multiplier of horizontal forces, γ is the slope of the linearized mechanism equilibrium curve due to second order effects and δ is the plastic top sway displacement. TPMC states that the mechanism equilibrium curve corresponding to the global mechanism has to be located below those corresponding to all the undesired mechanisms until a design displacement δ_u compatible with the local ductility supply [22], Fig. (**2**). The column sections at each storey that guarantee a global collapse mechanism are the unknowns of the design problem, while the dissipative zones are preliminarily designed according to the first principle of capacity design. More details about this design procedure applied can be found in [22 - 28] for steel MRFs and MRFs equipped with friction dampers.
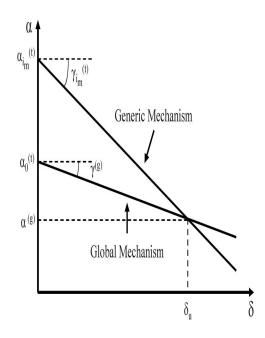

Fig. (2). TPMC statement.

3. ANALYZED SCHEMES

In order to assess the performance of MRFs equipped with friction devices, 2 frames alternatively designed according to both design approaches have been analyzed, namely: (i) MRFs with full strength connections and (ii) MRFs with FREEDAM dampers. The study cases herein investigated refers to a building whose plan configuration is depicted in Fig. (**3**). The structure is a 4 storey perimetral frame with 3 spans of 5m in both directions. In particular, MRFs are located in the longitudinal direction (x), while concentrically braced frames are considered in the transverse direction (y). Fig. (**3**) shows also the leaning column adopted in structural modelling to account for second order effects due to the internal gravity load resisting system. In addition, permanent G_k and live Q_k loads considered for the structural analysis and evaluation of masses corresponding to the gravitational frames, chosen within a practical range, are reported in (Table **1**).
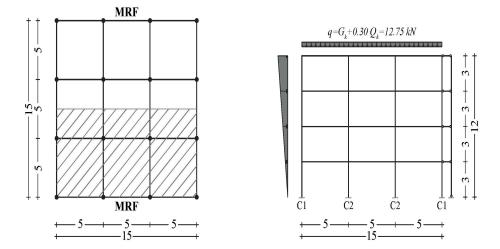
Fig. (3). Reference frame layout.

Table 1. Characteristic values of loads and combination factors.

Location	Type	Load [kN/m²]	$\psi_{2,i}$	$\psi_{E,i}$
Roof	Permanent	4.5	0.3	0.24
	Variable	2		
Intermediate Stories	Permanent	4.5	0.3	0.15
	Variable	2		

S275 steel grade (f_{yk} = *275 MPa*) was used for all members of the frames. The beams of the MRF were designed to withstand vertical loads accounting also for serviceability requirements. The design horizontal forces were determined according to EC8, assuming a peak ground acceleration equal to 0.35g, a seismic response factor equal to 2.5, a behaviour factor equal to 6.5 for the design spectrum with a 2% damping, a response spectrum Type 1 and soil Type C. The effects of second order (P-Δ) effects are accounted for in the design as well as the accidental torsional effects. The interstorey drift for serviceability conditions was limited to 1.0% of the storey height. Global and local ductility requirements according to EN 1998-1 [1] were checked. Fig. (**4**) reports the shapes of the members of the designed frames.
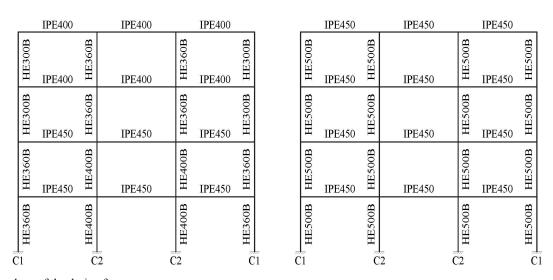

Fig. (4). Members of the design frames.

4. PUSHOVER ANALYSES

Non-linear static analyses (push-over) were carried out by means of SAP 2000 [29]. The primary aim of this analysis is the assessment of the collapse mechanism typology and a first preliminary comparison between the performances of the structures designed with the examined design approaches.

Beams and columns were modelled by means of beam-column elements, whose non-linearities are concentrated in plastic hinges ("P-hinge" elements) located at their ends. In particular, plastic hinges accounting for the interaction between axial force and bending moment were defined for columns, while the rotational behaviour resulting from the friction dampers located at the bottom flange level of the beam ends was modelled in pure rigid-plastic bending. Therefore, the plastic moment of the corresponding "P-hinge" element is equal to the product between the slippage resistance of the friction dampers and the lever arm. All plastic elements were modelled with a rigid-plastic curve whose plastic threshold, corresponding to the slippage resistance of friction dampers, accounts for the over-strength due to the random variability of the friction coefficient and the uncertainties of the bolt preloading level. The elastic component of the structural behaviour is directly taken into account by the elasticity of the beam-column element.

In Figs. (**5-8**), the validation of the nonlinear static analysis is reported with reference to MRFs with full strength connections and MRFs equipped at beam-to-column level with FREEDAM connections designed with both the proposed design approaches. The analysis was carried out under displacement control taking into account both geometrical and mechanical non-linearities. In addition, out-of-plane stability checks of compressed members were performed at each step of the non-linear analysis.

The hinge pattern of push-over curves is reported in Figs. (**9-12**), where it can be recognized that the structures designed by TPMC show the formation of a global mechanism at a roof displacement equal to 0.48 m. The same happens for the structure designed by beam-column hierarchy criterion but equipped with FREEDAM dampers. Conversely, the structure with full strength connections shows a partial collapse mechanism. This is also testified by the collapse mechanism equilibrium curves of Fig. (**5-8**) where only in Figs. (**6-8**) the softening branch of push-over curve perfectly coincides with the aforementioned curve. Finally, it was also checked that friction dampers do not slip under the action of the bending moment corresponding to the vertical load combination at the ultimate limit state, *i.e.* $1.3G_k + 1.5 Q_k$.
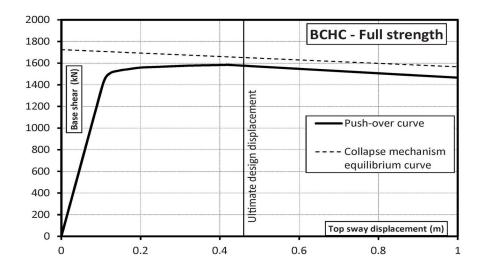

Fig. (5). Push-over curve for the structure designed according to beam-column hierarchy criterion with full strength connections.
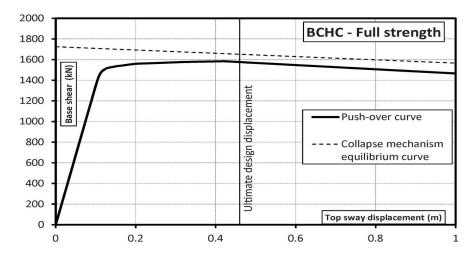
Fig. (6). Push-over curve for the structure designed according to TPMC with full strength connections.
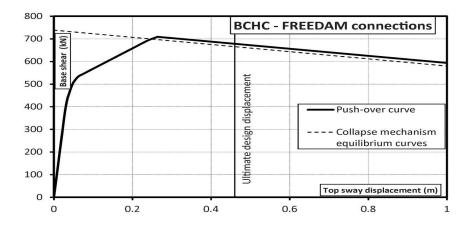

Fig. (7). Push-over curve for the structure designed according to beam-column hierarchy criterion with FREEDAM connections.
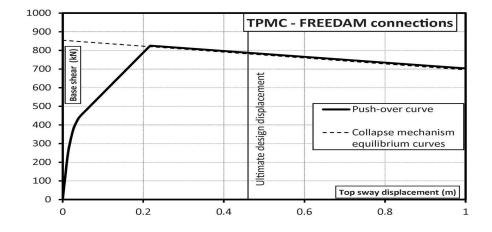

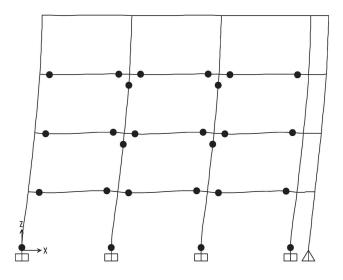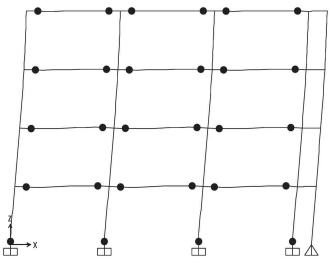**Fig. (8).** Push-over curve for the structure designed according to TPMC with FREEDAM connections.

Fig. (9). Push-over hinge pattern for the structure designed according to beam-column hierarchy criterion with full strength connections.

Fig. (10). Push-over hinge pattern for the structure designed according to TPMC with full strength connections.
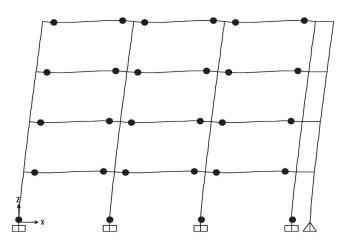

Fig. (11). Push-over hinge pattern for the structure designed according to beam-column hierarchy criterion with FREEDAM connections.
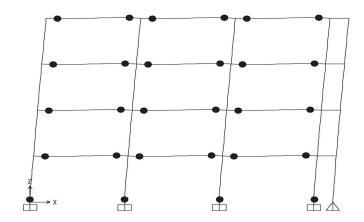
Fig. (12). Push-over hinge pattern for the structure designed according to TPMC with FREEDAM connections.

5. INCREMENTAL DYNAMIC ANALYSES

In order to provide a more robust evaluation of seismic performance of investigated structures, Incremental Dynamic Analyses (IDA) were performed. The leaning column has been modelled in order to take into account the vertical loads acting on the frame (including masses) and also the second order effects. In addition, 14 natural records were selected from RESORCE database [30] and properly scaled to match the design spectrum for the given seismicity level as reported in [26 - 29].The basic data of the selected ground motions are reported in (Table **3**).

The IDA analyses were carried out by increasing the $S_a(T_1)/g$ value until the achievement of a target drift equal to 0.04 rad corresponding to the final stroke of the designed FREEDAM dampers. However, it is important to point out that this limit state does not correspond to the structural collapse because the structures owns additional dissipative capacity due to the shear mechanisms in bolts.

In addition, Rayleigh formulation for a 2% damping has been assumed with the proportional factors computed with reference to the first and second mode of vibration. The first and second period of vibration T_1 and T_2 are reported, for each structural scheme, in (Table **2**).

Table 2. First and second period of vibration of considered structures.

	T_1 [s]	T_2 [s]
FD-BCHC	0.65	0.21
FD-TPMC	0.54	0.16

Fig. (13). Comparison between natural signals and EC8 design spectrum [26 - 29].

Table 3. Basic data of the selected ground motions.

Earthquake Name	Station Name	Country	Date	Magnitude Mw	PGA [m/s²]	Length [s]
Alkion	Xylokastro-O.T.E.	Greece	24.02.1981	6.6	6.69	17.49
Montenegro	Bar-Skupstina Opstine	Montenegro	24.05.1979	6.2	6.95	41.87
Izmit	Yarimca (Eri)	Turkey	13.09.1999	5.8	4.46	40.03
Izmit	Usgs Golden Station Kor	Turkey	13.09.1999	5.8	7.07	55.31
Faial	Horta	Portugal	09.07.1998	6.1	9.58	105.30
L'Aquila	L'Aquila - V. Aterno - Aquila Park In	Italy	06.04.2009	6.3	3.36	114.39
Aigion	Aigio-OTE	Greece	15.06.1995	6.5	3.22	124.01
Alkion	Korinthos-OTE Building	Greece	24.02.1981	6.6	4.47	40.01
Umbria-Marche	Castelnuovo-Assisi	Italy	26.09.1997	6.0	7.46	30.02
Izmit	Heybeliada-Senatoryum	Turkey	17.08.1999	7.4	5.29	47.34
Izmit	Istanbul-Zeytinburnu	Turkey	17.08.1999	7.4	7.46	48.20
Ishakli	Afyon-Bayindirlik ve Iskan	Turkey	03.02.2002	5.8	6.05	9.34
Olfus	Ljosafoss-Hydroelectric Power	Iceland	29.05.2008	6.3	5.63	93.13
Olfus	Selfoss-City Hall	Iceland	29.05.2008	6.3	6.26	40.03

In Figs. (**14-17**) the MIDR curves for both full strength and FREEDAM connections designed according to both BCHC and TPMC are reported. From these figures, it is observed that at the same seismic intensity both structures with conventional MRF joints are characterized by lower displacement demand than the corresponding structures equipped with FREEDAM devices. However, in these latter cases, the larger deformation demand does not correspond to any structural damage, thus allowing easy reparability after the earthquake that is not possible in the case of conventional MRF. The comparison between BCHC and TPMC frames confirms that EC8-compliant structures do not exhibit a global mechanism.
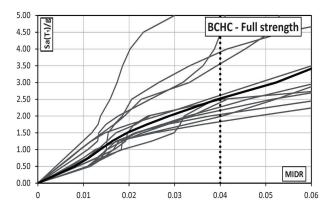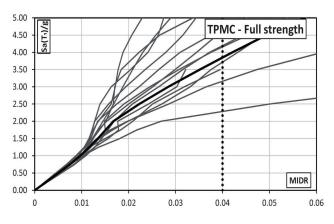

Fig. (14). Maximum inter-storey drift for BCHC MRF.

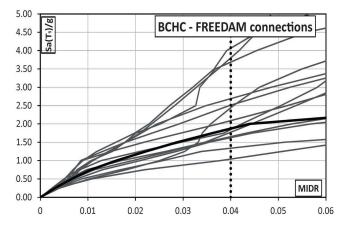

Fig. (15). Maximum inter-storey drift for TPMC MRF.

Fig. (16). Maximum inter-storey drift for FD-BCHC.
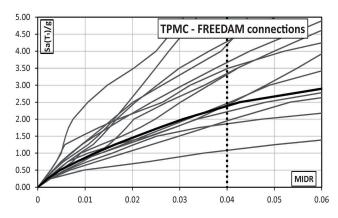

Fig. (17). Maximum inter-storey drift for FD-TPMC.

From Figs. (**18-21**) hinge patterns of the analyzed schemes are depicted. Plastic hinge patterns correspond to a MIDR value equal to 0.04. In particular, the performance of the MRFs with conventional full strength joints is highly influenced by the design approach, namely for the given target MIDR, BCHC structure shows a partial collapse mechanism while that designed according to TPMC shows a full global mechanism. Conversely, both structures equipped with FREEDAM joints show the global mechanism. This finding confirms the effectiveness of using FREEDAM joints.
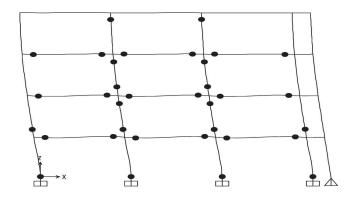

Fig. (18). Hinge pattern of MRF with full strength joints designed according to BCHC, evaluated at MIDR 0.04 corresponding to Sa/g=1.43.
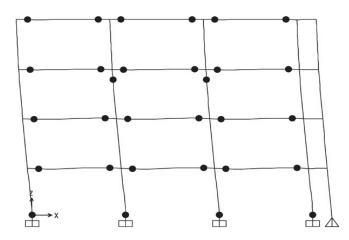
Fig. (19). Hinge pattern of MRF with full strength connections designed according to TPMC, evaluated at MIDR 0.04 corresponding to Sa/g=2.76.
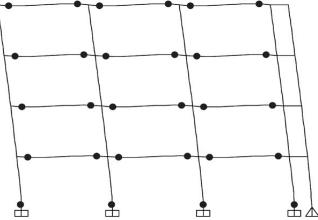

Fig. (20). Hinge pattern of MRF with FREEDAM connections designed according to BCHC, evaluated at MIDR 0.04 corresponding to Sa/g=3.35.
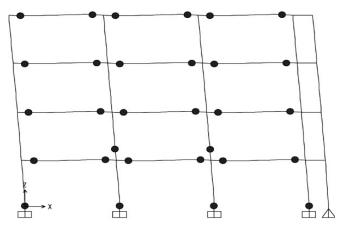

Fig. 21. Hinge pattern of MRF with FREEDAM connections designed according to TPMC, evaluated at MIDR 0.04 corresponding to Sa/g=2.97.

CONCLUSION

Two different design approaches for MRFs with or without FREEDAM connections are described and compared in this paper. The first approach is based on the application of the Beam-to-Column Hierarchy Criterion (BCHC), while the second one exploits the Theory of Plastic Mechanism Control (TPMC). A comparison between both the design approaches and the results of the new rules are herein examined by means of nonlinear static and dynamic analyses. Both designs approach applied to structures equipped with FREEDAM devices guarantee satisfactory seismic performance with damage distribution in agreement with the expected global mechanism. Conversely, the structure with

full strength connections designed according to beam-column hierarchy criterion exhibits a partial collapse mechanism, while the frame designed according to TPMC is characterized by overall ductile and dissipative failure mode. Results of IDA analyses point out that, at the same seismic intensity level, FREEDAM structures show larger drift demand than conventional MRFs [31 - 34]. However, differently, from this latter case, the larger displacement demand of MRFs equipped with FREEDAM joints does not correspond to structural damage, thus allowing the reparability of the structure after a seismic event.

CONSENT FOR PUBLICATION

Not applicable.

CONFLICT OF INTEREST

The authors declare no conflict of interest, financial or otherwise.

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References

[1]   CEN (European Communities for Standardization), EN 1998 Eurocode 8: design of structures for earthquake resistance —part 1: General rules, seismic actions and rules for buildings, 2005

[2]   "NSI/AISC 341-10 Seismic Provisions for Structural Steel Buildings", *American Institute of Steel Construction.,* 2010.

[3]   A. Tenchini, M. D'Aniello, C. Rebelo, R. Landolfo, L.S. da Silva, and L. Lima, "Seismic performance of dual-steel moment resisting frames", *J. Construct. Steel Res.,* vol. 101, pp. 437-454, 2014.

[4]   D. Cassiano, M. D'Aniello, C. Rebelo, R. Landolfo, and L. da Silva, "Influence of seismic design rules on the robustness of steel moment resisting frames. Steel and Composite Structures", *Int. J.,* vol. 21, no. 3, pp. 479-500, 2016.

[5]   M. D'Aniello, R. Tartaglia, S. Costanzo, and R. Landolfo, "Seismic design of extended stiffened end-plate joints in the framework of Eurocodes", *J. Construct. Steel Res.,* vol. 128, pp. 512-527, 2017.

[6]   "Experimental behaviour of friction T-stub beam-to-column joints under cyclic loads", *Steel Construt.,* vol. 6, pp. 11-18, 2013.

[7]   H. Khoo, C. Clifton, J. Butterworth, G. MacRae, S. Gledhill, and G. Sidwell, "Development of the self-centering Sliding Hinge Joint with friction ring springs", *J. Construct. Steel Res.,* vol. 78, pp. 201-211, 2012.

[8]   H. Khoo, J. Clifton, J. Butterworth, and G. Macrae, "Experimental study of full-scale self-centering sliding hinge joint connections with friction ring springs", *J. Earthquake Eng.,* pp. 972-997, 2013.

[9]   J. Borzouie, G. Macrae, J.G. Chase, G. Rodgers, R. Xie, J.C. Golondrino, and C. Clifton, "Cyclic performance of asymmetric friction connections", *Bridge Struct. Eng.,* vol. 45, p. 1, 2015.

[10]   M. Latour, V. Piluso, and G. Rizzano, "Experimental analysis of innovative dissipative bolted double split tee beam-to-column connections", *Steel Construct.,* vol. 4, pp. 53-64, 2011.

[11]   M. Titirla, K. Katakalos, G. Zuccaro, and F. Fabbroccino, "On the mechanical modeling of an innovative energy dissipation device", *Ing. Sism.,* vol. 34, no. 2, pp. 126-137, 2017.

[12]   P. Castaldo, B. Palazzo, and F. Perri, "Fem simulations of a new hysteretic damper: The dissipative column", *Ing. Sism.,* vol. 33, no. 1, pp. 34-45, 2015.

[13]   F. Mazza, and A. Vulcano, "Design of hysteretic damped braces to improve the seismic performance of steel and r.c. framed structures", *Ing. Sism.,* vol. 31, no. 1, pp. 5-16, 2014.

[14]   T.S. Yan, and E.P. Popov, Experimental and analytical studies of steel connections and energy dissipators.*UCB/EERC-95/13.,* Earthquake Engineering Research Center: Berkeley, 1995.

[15]   H. Khoo, C. Clifton, J. Butterworth, G. MacRae, S. Gledhill, and G. Sidwell, "Development of the self-centering Sliding Hinge Joint with friction ring springs", *J. Construct. Steel Res.,* vol. 78, pp. 201-211, 2012.

[16]   H. Khoo, G. Clifton, G. Macrae, and S. Ramhormozian, "Proposed design models for the asymmetric friction connection", *Earthquake Eng. Struct. Dynam.,* vol. 44, pp. 1309-1324, 2014.

[17]   CEN (European Communities for Standardization), *EN 1993-1-8 Eurocode 3: Part 1-8: Design of joints. 2005.*.

[18]   J. Borzouie, G.A. MacRae, J.G. Chase, G.W. Rodgers, and G.C. Clifton, "Experimental studies on cyclic performance of column base weak axis aligned asymmetric friction connection", *J. Construct. Steel Res.,* vol. 112, pp. 252-262, 2015.

[19]   M. Latour, V. Piluso, and G. Rizzano, "Experimental analysis on friction materials for supplemental damping devices", *Constr. Build. Mater.,*

vol. 65, pp. 159-176, 2014.

[20]   M. Latour, V. Piluso, and G. Rizzano, "Free from damage beam-to-column joints: Testing and design of DST connections with friction pads", *Eng. Struct.,* vol. 85, pp. 219-233, 2015.

[21]   EN 1998-1, Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.

[22]   R. Montuori, E. Nastri, and V. Piluso, "Advances in theory of plastic mechanism control: Closed form solution for MR-Frames", *Earthquake Eng. Struct. Dynam.,* vol. 44, pp. 1035-1054, 2015.

[23]   R. Montuori, E. Nastri, V. Piluso, and M. Troisi, "Influence of the cyclic behaviour of beam-to-column connection on the seismic response of regular steel frames", *Ing. Sism.,* vol. 33, no. 1-2, pp. 91-105, 2015.

[24]   R. Montuori, E. Nastri, V. Piluso, and M. Troisi, "Influence of connection typology on seismic response of MR-Frames with and without 'set-backs'", *Earthquake Eng. Struct. Dynam.,* vol. 46, pp. 5-25, 2017.

[25]   R. Montuori, E. Nastri, and V. Piluso, "Preliminary analysis on the influence of the link configuration on seismic performances of MRF-EBF dual systems designed by TPMC", *Ing. Sism.,* vol. 33, no. 3, pp. 52-64, 2016.

[26]   E. Nastri, "Document Eccentrically braced frames designed for the energy dissipation optimization", In: *ECCOMAS Congress 2016 - Proceedings of the 7th European Congress on Computational Methods in Applied Sciences and Engineering*, 2016, pp. 8476-8491.

[27]   E. Nastri, "An example of energy dissipation optimization for steel MRFs with pin-jointed column bases", In: *ECCOMAS Congress 2016 - Proceedings of the 7th European Congress on Computational Methods in Applied Sciences and Engineering*, 2016, pp. 8460-8475.

[28]   R. Montuori, E. Nastri, and V. Piluso, "Theory of plastic mechanism control for the seismic design of braced frames equipped with friction dampers", *Mech. Res. Commun.,* vol. 58, pp. 112-123, 2014.

[29]   *SAP 2000: Integrated Finite Element Analysis and Design of Structures. Analysis Reference. Computer and Structure Inc.,* University of California: Berkeley, 2007.

[30]   S. Akkar, M.A. Sandıkkaya, M. Şenyurt, A. Azari Sisi, B.Ö. Ay, P. Traversa, J. Douglas, F. Cotton, L. Luzi, B. Hernandez, and S. Godey, "Reference database for seismic ground-motion in Europe (RESORCE)", *Bull. Earthquake Eng.*.http://link.springer.com/article/10.1007/s10518-013-9506-8 in press
[http://dx.doi.org/10.1007/s10518-013-9506-8]

[31]   M. D'Aniello, S. Costanzo, and R. Landolfo, "The influence of beam stiffness on seismic response of chevron concentric bracings", *J. Construct. Steel Res.,* vol. 112, pp. 305-324, 2015.

[32]   A. Tenchini, M. D'Aniello, C. Rebelo, R. Landolfo, L. da Silva, and L. Lima, "High strength steel in chevron concentrically braced frames designed according to Eurocode 8", *Eng. Struct.,* vol. 124, pp. 167-185, 2016.

[33]   S. Costanzo, M. D'Aniello, and R. Landolfo, "Seismic design criteria for chevron CBFs: European vs North American codes (Part-1)", *J. Construct. Steel Res.,* vol. 135, pp. 83-96, 2017.

[34]   S. Costanzo, M. D'Aniello, and R. Landolfo, "Seismic design criteria for chevron CBFs: Proposals for the next EC8 (Part-2)", *J. Construct. Steel Res.,* vol. 138C, pp. 17-37, 2017.