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INFLUENCE OF THE CYCLIC BEHAVIOUR OF BEAM-TO-COLUMN CONNECTION ON THE SEISMIC RESPONSE OF REGULAR STEEL FRAMES

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SUMMARY: *The work presented is aimed at the investigation of the influence of beam-to-column connection typologies on the seismic response of MR-Frames designed according to the Theory of Plastic Mechanism Control (TPMC). The investigated typologies are four partial strength connections designed in order to obtain the same flexural resistance. The first three joints are partial-strength semi-rigid connections while the fourth one is a beam-to-column connection equipped with friction pads properly designed to assure the earthquake input energy dissipation. Beam-to-column joints are modelled by means of rotational inelastic spring elements located at the ends of the beams whose moment-rotation curve is characterized by a cyclic behaviour accounting for stiffness and strength degradation and pinching phenomenon. The parameters characterizing the joints cyclic hysteretic behaviour have been calibrated on the base of experimental results aiming to the best fitting. The prediction of the structural response has been carried out by means of IDA analyses.*

KEYWORDS: *Partial strength connections, dissipation, MRFs, friction dampers, beam-to-column connections*

1. INTRODUCTION

As it is known, modern seismic codes [CEN, 2005] promote the dissipation of seismic input energy by properly designed so-called dissipative zones that are some zones of structural members engaged in plastic range, properly detailed in order to assure wide and stable hysteresis loops [Formisano *et al*. 2006a]. To this scope various design methodology have been proposed [Mazzolani and Piluso, 1997, Castaldo, 2014, De Iuliis and Castaldo, 2012, Castaldo and De Iuliis, 2014, Ferraioli *et al.* 2014a, 2014b, Formisano *et al.* 2013] in order to design dissipative structures. However, it is important to promote the plastic engagement of the greatest number of dissipative zones by properly controlling the failure mode [Montuori *et al*., 2014a]. As the plastic engagement of columns can lead to non-dissipative collapse mechanisms, modern seismic codes, such as Eurocode 8, suggest the application of the beam-to-column hierarchy criterion which imposes that, at each joint, the flexural strength of connected columns has to be sufficiently greater than the flexural strength of the connected beams. However, it is important to underline that the fulfilment of this design criterion is not sufficient to assure the development of a collapse mechanism of global type, even if, generally, it allows to prevent the occurrence of soft-storey mechanisms.

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Depending on the beam-to-column joint typology, the dissipative zones can be located at the beam ends or in the connecting elements. In fact, beam-to-column connections can be designed either as full-strength joints, having sufficient overstrength with respect to the connected beam concentrating dissipative zones at the beams ends [Bruneau *et al.*, 1998; Faella *et al.*, 2000; Moore *et al.*, 1999], or as partial strength joints, so that the seismic input energy is dissipated by means of the plastic engagement of one or more joint components properly selected.
The use of rigid full-strength joints has been always considered the best way to dissipate the seismic input energy, that is why seismic codes provide specific design criteria for them, while there are no detailed recommendations dealing with partial-strength connections. However, in the last years there was a significant use of Reduced Beam Section (RBS) [Montuori, 2014, 2015, Kanyilmaz *et al.,* 2015, Castiglioni *et al*. 2012], that consists in the weakening of the terminal part of beams by properly cutting beam ends. Recently, Eurocode 8 has introduced the use of partial-strength joints for dissipating the seismic input energy in the connecting elements of beam-to-column joints because it has been recognized that semi-rigid partial-strength connections, if properly designed by means of an appropriate choice of the joint components where the dissipation has to occur, can lead to dissipation and ductility capacity compatible with the seismic demand.
The use of partial-strength joints, moreover, allows to avoid the plastic engagement of columns without their over-sizing, leading to convenient structural solutions particularly in case of long span MR-Frames [Faella *et al*., 1997].
However, beam-to-column connection equipped with friction dampers are full-strength joint whose dissipative capacity is demanded to the damping devices located at beam ends that can be equipped with friction pads made by simple steel or coated with other materials such as thermally sprayed aluminum [Latour *et al*., 2014; Latour *et al*., 2015]. In this work friction devices are not adopted to provide supplementary energy dissipation [Castaldo et al., 2016a, Castaldo et al., 2016b, Castaldo et al., 2016c, Palazzo *et al*., 2015, Castaldo and Tubaldi, 2015; Castaldo *et al*., 2015; Palazzo *et al*., 2014; Montuori *et al*., 2014b; Piluso *et al*., 2014, Longo *et al*., 2012a, 2012b, De Matteis *et al.,* 2011, Formisano *et al.*, 2006b] but they are rather used to properly substitute the traditional dissipative zones of MR-Frames.
Given the above, the main purposes of this paper are, on one end, the evaluation of the influence of beam-to-column connections on the seismic response of regular MR-Frames, and on the other end, the estimation of the performances of MRFs equipped with the investigated partial strength connections. After a proper calibration of the behaviour of the partial strength connections, the seismic performances of the investigated structures are carried out by means of IDA analyses using Seismostruct computer program.

2. EXAMINED BEAM-TO-COLUMN CONNECTIONS AND THEIR MODELLING

The first three beam-to-column typologies herein investigated are semi-rigid partial-strength connections whose structural detail has been designed by means of the component approach, aiming to obtain the same flexural resistance, but changing the weakest component. Therefore, they are characterized by different locations of the weakest joint component, leading to different values of the joint rotational stiffness and of the plastic rotation supply.
The fourth investigated typology is a beam-to-column connections equipped with friction pads. The use of beam-to-column joints equipped with friction dampers allows to obtain MR-Frames where the dissipative zones are constituted by damping devices located at the beam ends. The reason for investigating these beam-to-column joints is related to the availability of results

dealing with their cyclic rotational response, tested as structural sub-assemblages at the Materials and Structures Laboratory of Salerno University. The structural details of the connections are depicted in Figures 1-4. In order to point out how the cyclic behavior is governed by the location of the structural detail, the tested specimens have been designed aiming to obtain the same flexural strength, but different values of rotational stiffness and plastic rotation supply. The joint non-dimensional resistance $\overline{m}$, given by the ratio between the joint flexural resistance and the beam plastic moment is equal to 0.76 [Iannone *et al*., 2011].
Specimen EEP-CYC 02 (Figure 1) was designed aiming to obtain the aforementioned value of the non-dimensional resistance and relying on the ductility supply of the end-plate, by properly designing its thickness and bolt location [Piluso *et al*., 2001]. The first component to be designed is the weakest component, i.e. the end-plate, whose design resistance is obtained as the ratio between the desired joint flexural resistance and the lever arm. Successively, the other components are designed to have sufficient overstrength aiming to avoid their plastic engage.
Specimen EEP-DB-CYC 03 (Figure 2) is an extended end-plate connection, whose design is aimed at the investigation of the energy dissipation capacity of beams. However, aiming to obtain the same flexural resistance of the previous specimen, RBS (Reduced Beam Section) strategy, called also "dog-bone", has been adopted. The corresponding structural detail has been designed according to [Moore *et al*., 1999]. Specimen TS-CYC 04 (Figure 3) is a partial-strength joint with a couple of T-stubs bolted to the beam flanges and to the column flanges and designed to be the main source of plastic deformation capacity. The design goal is to avoid the plastic engage of the components related to the column web panel, the column web in compression/tension and the panel zone in shear. The main advantage of double split tee connections is due to their easy repair. In fact, if the panel zone is designed with adequate overstrength, it is possible to substitute only the end T-stubs after a seismic event. Also in this case the same flexural resistance of the other joints was imposed requiring, in addition, a plastic rotation supply of about 0.08 rad. The plastic deformation supply of the T-stub components has been predicted as suggested by [Piluso *et al*., 2001]. The last specimen, TS-M2-460-CYC 09, is a bolted double split tee beam-to-column connections equipped with friction dampers designed to slip before the yielding of the beam (Figure 4) where the energy is dissipated through the slippage between the stem of bolted tee stubs and the beam flange with an interposed friction pad [Latour *et al*., 2012; Latour *et al*., 2013] which are clamped by means of high strength bolts that allow to apply a constant force on the surfaces in contact by simply governing the value of the tightening torque and the number and diameter of the bolts. In particular, the structural detail depicted in Figure 4 has been subjected to experimental tests at the Material and Structure laboratory of Salerno University [Latour *et al*., 2012; Latour *et al*., 2013], by investigating the influence of different materials adopted as friction pads.
In order to evaluate the seismic performance of MRFs it is of preliminary importance to set up an appropriate model to accurately represent the cyclic rotational behavior of connections. In particular, hysteretic behavior of semi-rigid partial strength connections are affected by both the development of strength and stiffness degradation and pinching phenomena as far as the number of cycles increases. As the rules describing these phenomena cannot be deduced by means of theoretical approaches, a sufficient experimental data aiming to develop adequately accurate semi-analytical models are needed where the monotonic envelope is predicted by means of the use of mechanical models based on the component method, while the degradation rules are empirically derived by means of the available experimental results.

Table 1. *First class parameters (strength, stiffness and ductility parameters) of the SHM spring element adopted for modelling the four connection typologies in SeismoStruct*

CONNECTION	Initial Rotational Stiffness	First Yielding Moment	Plastic Moment	Yield Rotation	Ultimate Rotation	Post Yield stiffness ratio as % of elastic
	EI (kN/mm^2)	**PCP/PCN** (kN mm)	**PYP/PYN** (kN mm)	**UYP/UYN** (rad)	**UUP/UUN** (rad)	**EI3P/EI3N** (-)
EEP-CYC 02	41411466	116463/-130074	157000/-173000	0.01/-0.01	0.04/-0.04	0.0307/0.0352
EEP-DB-CYC 03	47238740	150000/-145000	201000/-207000	0.013/-0.017	0.06/-0.06	0.009/0.001
TS-CYC 04	20000000	185000/-190000	190000/-200000	0.022/-0.022	0.07/-0.07	0.025/0.055
TS-M2-460-CYC 09	30000000	90000/-100000	100000/-120000	0.013/-0.015	0.10/-0.10	0.03/0.03

Table 2. *Second class parameters (Hysteresis shape parameters) of the SHM spring element adopted for modelling the four connection typologies in SeismoStruct*

CONNECTION	HC	HBD	HBE	NTRANS	ETA	HSR	HSS	HSM	NGAP	PHIGAP	STIFFGAP
EEP-CYC 02	2	0.001	0.015	1.2	1.8	0.25	0.04	0.51	0.2	0.2	5
EEP-DB-CYC 03	12.5	0.01	0.1	0.75	0.5	0.10	10	1	0.2	0.2	0.2
TS-CYC 04	200	0.003	0.06	1	0.5	0.38	0.2	0.45	0.2	0.2	0.2
TS-M2-460-CYC 09	75	0.10	0.26	5	0.5	0.25	100	0.4	2	1000	2

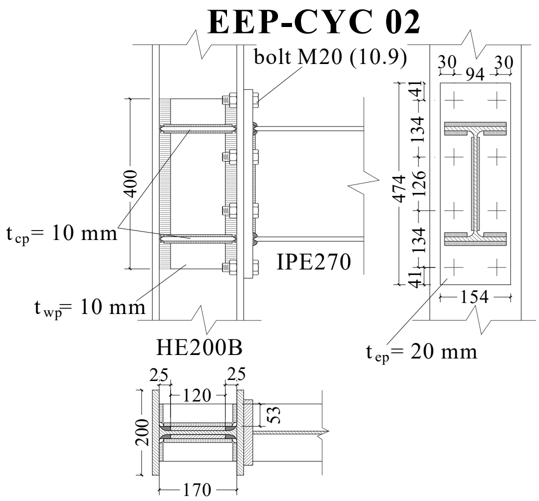

Figure 1. *Structural details of EEP-CYC 02 connection*

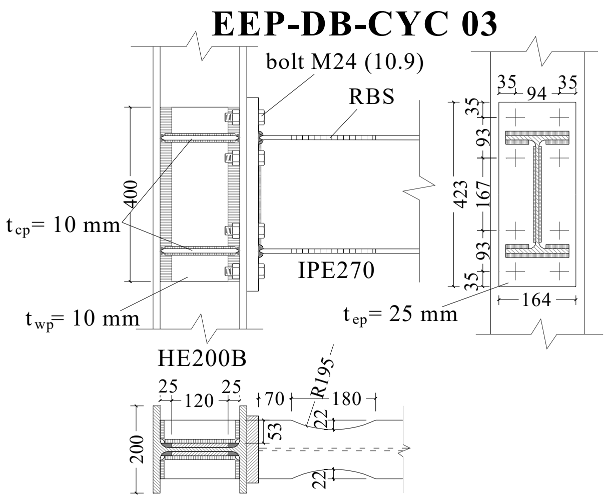

Figure 2. *Structural details of EEP-DB-CYC 03connection*

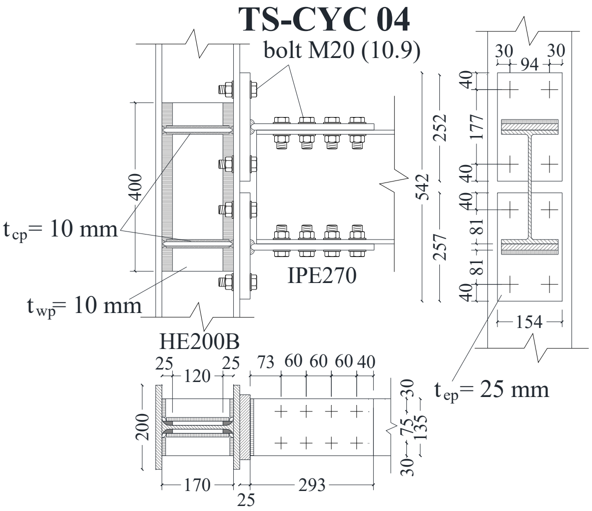

Figure 3. *Structural details of TS-CYC 04*

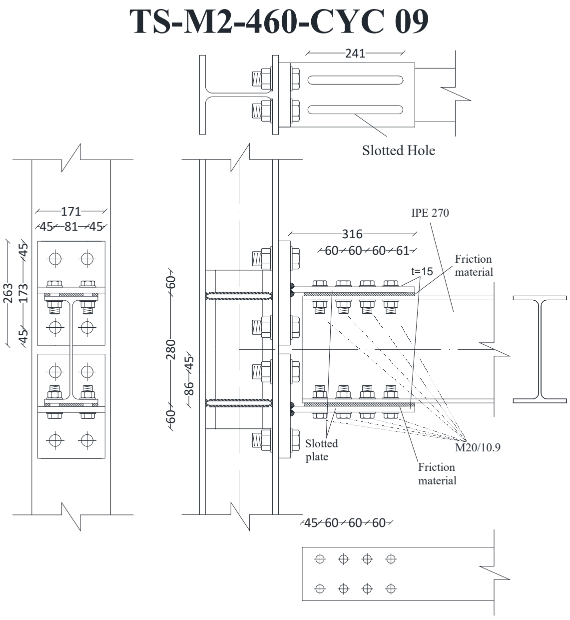

Figure 4. *Structural details of TS-M2-460-CYC 09*

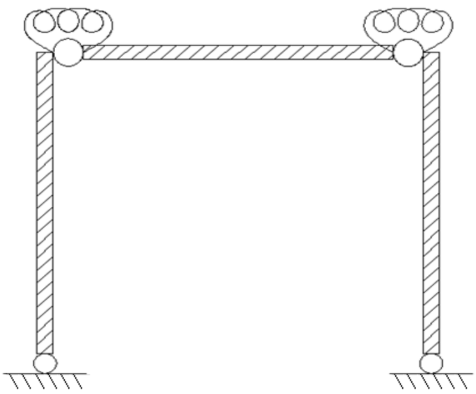

Figure 5. *Structural scheme adopted for spring elements calibration*

Therefore, it is possible to apply to such structural model a displacement time-history exactly reproducing that adopted in testing beam-to-column joint sub-assemblages and to compare the cyclic moment-rotation response of the spring element with the one obtained from experimental tests. By properly modifying the parameters modelling strength and stiffness degradation and pinching phenomena, it has been possible to select, for each tested specimen, the connection model leading to the best fitting between the analytical model and experimental test results. In particular, the adopted hysteretic model is the Smooth Hysteretic Model (SHM) [Bouc, 1967; Wen, 1976; Sivaselvan and Reinhorn, 2001; Sivaselvan and Reinhorn, 1999], available in the finite element library of SeismoStruct computer program. The hysteretic behaviour of the rotational spring depends on two class of parameters. The first class is related to the strength, stiffness and rotation parameters, while the second class is related to all the parameters influencing the shape of the hysteresis loops and also the pinching phenomenon.

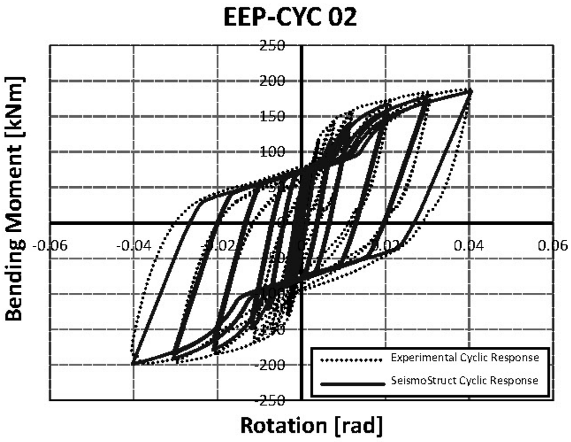

Figure 6. *Comparison between the SHM cyclic moment-rotation response of the spring elements with the experimental tests for EEP-CYC 02 connection*

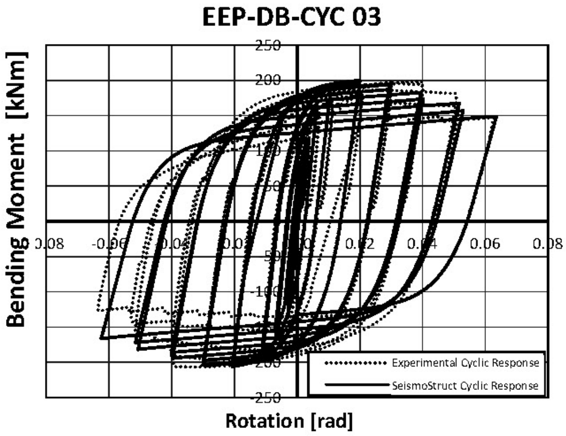

Figure 7. *Comparison between the SHM cyclic moment-rotation response of the spring elements with the experimental tests for EEP-CYC 03 connection*

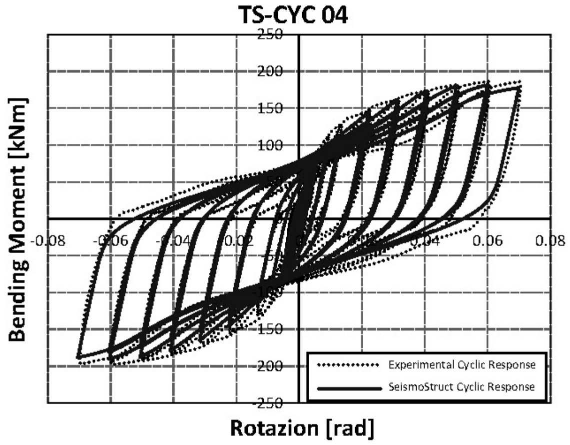

Figure 8. *Comparison between the SHM cyclic moment-rotation response of the spring elements with the experimental tests for TS-CYC 04 connection*

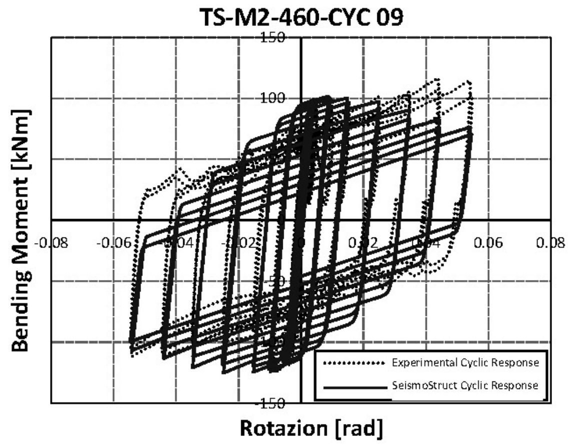

Figure 9. *Comparison between the SHM cyclic moment-rotation response of the spring elements with the experimental tests for TS-M2-460-CYC 09 connection*

In Table 1 all this first class parameters are reported. In particular they are EI (Initial Stiffness), PCP (first yield moment positive), PCN (first yield moment negative), PYP (plastic moment positive), PYN (Plastic Moment Negative), UYP (yield rotation positive), UYN (yield rotation negative), UUP (ultimate rotation positive), UUN (Ultimate Rotation Negative), EI3P (post yield stiffness ratio as a percentage of elastic stiffness positive) and EI3N (post yield stiffness ratio as a percentage of elastic stiffness negative). This class of parameters are also suitable for the Polygonal Hysteretic Model.

As testified by the developed experimental tests, the cyclic response of connections in terms of shape of the hysteresis loops, stiffness and strength degradation and resulting dissipation capacity are directly related to the components involved in plastic range, mainly the weakest component.

For this reason, the non-linear cyclic rotational response of beam-to-column joints has been modelled by means of the spring elements included SeismoStruct (version 5.2) software by properly calibrating parameters on the bases of available experimental results to account for both stiffness and strength degradation and for pinching phenomenon. In order to derive the parameters governing the cyclic response of the spring elements, a cyclic push-over analysis under displacement control has been carried out with reference to the structural scheme depicted in Figure 5, having infinitely rigid beam and column elements, whose feature is that its structural response is dependent on the cyclic response of the spring elements only.
Regarding the second class of parameters, HC locates the pivot point, HBD and HBE represent the measure of strength degradation related to ductility and to energy, respectively. In addition, the SHM model is characterized by smoothing parameters which describe the transition from the elastic branch to the plastic branch of the hysteretic cycle. They are: NTRANS which is the parameter governing the elasto-plastic transition (NTRANS=20 corresponds to a bilinear behaviour), ETA which governs the unloading shapes and HSR, HSS, HSM which are the parameters influencing the pinching phenomenon that are the slip length, the slip sharpness and the parameter for the mean moment level of slip. Finally, NGAP, PHIGAP and STIFFGAP take into account the strength increase for high deformation levels. The parameters describing the hysteretic behaviour of the spring element, adopted in the SeismoStruct model, are delivered in Table 2. In addition, the damage phenomena occurring under cyclic loading conditions have been accounted for by calibrating the corresponding parameters by minimizing the scatter in terms of dissipated energy between experimental test results and numerical results obtained with SeismoStruct spring element. The comparison between the cyclic moment-rotation curve of the tested connections, depicted in Figures 1 to 4 and the corresponding rotational response, predicted by SeismoStruct with the spring element modelling parameters given in Table 1 and Table 2 is provided in Figures 6 to 9. From these figures, it can be observed that SeismoStruct provides a satisfactory degree of accuracy in the modelling of the beam-to-column joint cyclic behaviour.

3. ANALYSED MR-FRAME AND ITS STRUCTURAL MODELLING

In this paper, the influence of the constructional detail of beam-to-column joints on the seismic response of a three bays - six storeys MR-Frames with partial-strength connections is investigated. Regarding the design loads, a uniform dead load (G_k) equal to 12.00 kN/m and a uniform live load (Q_k) equal to 6.00 kN/m are applied. The spans of the longitudinal frames are equal to 6.00 m, while the interstorey heights are equal to 3.20 m with the exception of the first storey whose height is equal to 3.50 m. The uniform vertical load adopted for beam design is $q = 1.3G_k + 1.5Q_k = 25.20\ kN/m$. A design value of the beam plastic moment approximately equal to $qL^2/8$ has been chosen and IPE270 profiles made of S275 steel grade have been adopted for the beams. The size of the columns of both regular and irregular structures are selected by adopting a rigorous design procedure assuring a collapse mechanism of global type [Montuori *et al*., 2014].

Table 1. *Mechanical properties of members*

	$f_{y,f}$ [N/mm^2]	$f_{u,f}$ [N/mm^2]	$f_{y,w}$ [N/mm^2]	$f_{u,w}$ [N/mm^2]
Column	430	523	382.5	522
Beam	405	546	387	534

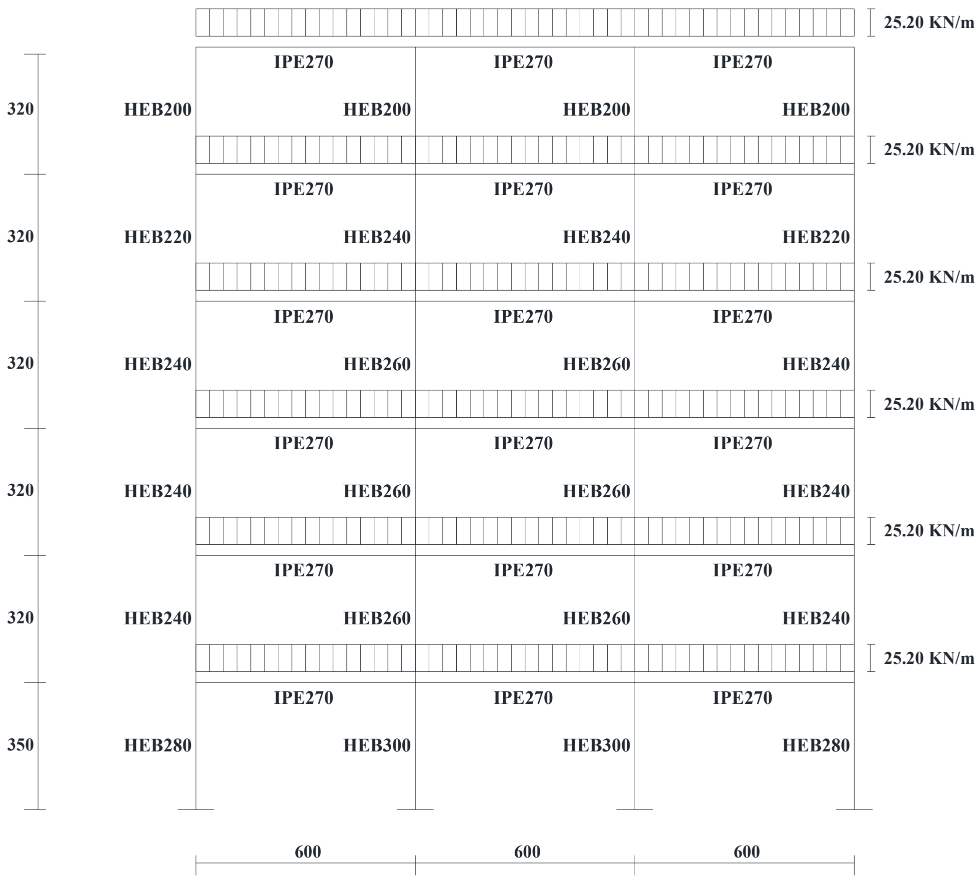

Figure 10. *Analysed MR-Frame structural scheme*

The whole design procedure has been carried out with reference to S275 steel grade. However, in order to assure a frame structural response consistent with the joint rotational behavior obtained from experimental tests and modeled as preliminarily described, the values of column and beam material mechanical properties to be adopted in non-linear dynamic analyses are assumed to be equal to those measured in testing beam-to-column joint sub-assemblages and reported in Table 1.

4. INFLUENCE OF BEAM-TO-COLUMN JOINTS ON SEISMIC RESPONSE

Regarding beam and column elements, a bilinear model characterized by a hysteretic behavior with no stiffness degradation, no ductility-based strength decay, no hysteretic energy-based decay and no slip has been considered. However, it is useful to underline that this issue is not significant, because the use of partial-strength connections leads to the concentration of yielding at connections, so that only the connection modeling is of primary importance.
The seismic performances of the examined MR-Frame with the four structural details of beam-to-column connections are investigated by means of non-linear dynamic analyses, carried out by means of SeismoStruct computer program, for increasing levels of the seismic intensity measure. Record-to-record variability is accounted for performing Incremental Dynamic Analyses considering ten earthquake records selected from PEER database. Mass and stiffness proportional damping, 3% of critical value, has been assumed.

Aiming to perform an IDA, all the records have been properly scaled to provide increasing values of the spectral acceleration $S_a(T_1)$ corresponding to the fundamental period of vibration of the structure, equal to T_1=1.6 sec for connections EEP-CYC 02 and EEP-DB-CYC 03 and equal to T_1=1.7 sec for connection TS-CYC 04 and TS-M2-460-CYC-09. In particular, the analyses have been repeated increasing the $S_a(T_1)/g$ until the target collapse value corresponding to the attainment of the experimental ultimate value of the plastic rotation supply of EEP-CYC 02, EEP-DB-CYC 03 and TS-CYC 04 connections and the target interstorey drift for TS-M2-460-CYC-09 connection (Table 2).

Table 2. *Ultimate rotation of EEP-CYC 02, EEP-DB-CYC 03, TS-CYC 04 connections and ultimate interstorey drift of TS-M2-460-CYC 09 connection*

	Ultimate Rotation (rad)
EEP-CYC 02	0.04
EEP-DB-CYC 03	0.06
TS-CYC 04	0.07
	Ultimate interstorey drift (rad)
TS-M2-460-CYC 09	0.10

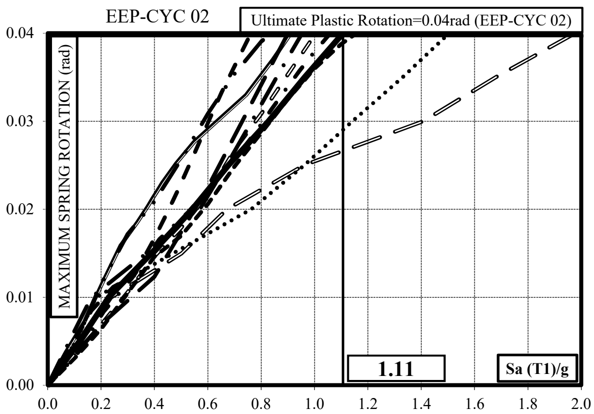

Figure 11. *Maximum Spring Rotation vs Spectral Acceleration for EEP-CYC 02 connection*

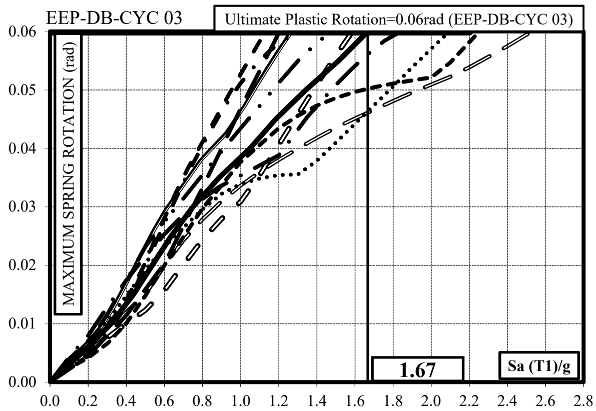

Figure 12. *Maximum Spring Rotation vs Spectral Acceleration for EEP-CYC 03 connection*

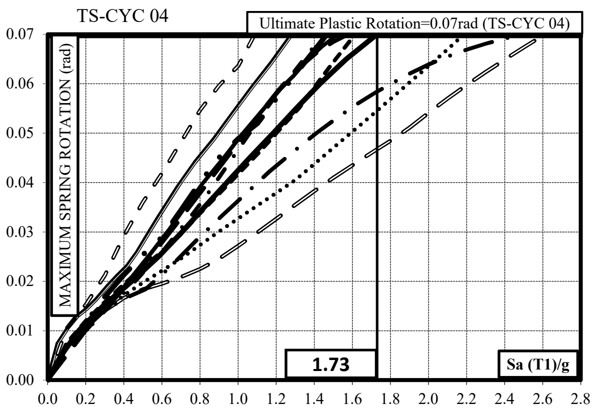

Figure 13. *Maximum Spring Rotation vs Spectral Acceleration for TS-CYC 04 connection*

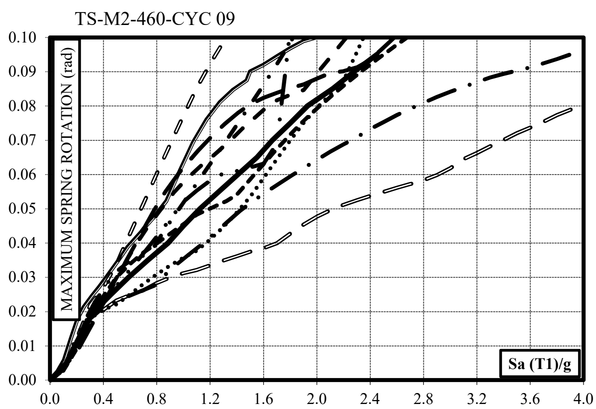

Figure 14. *Maximum Spring Rotation vs Spectral Acceleration for TS-M2-460-CYC 09 connection*

The target Maximum Interstorey Drift (MIDR) has been assumed as 0.1 according to FEMA [FEMA 351, 2000, FEMA 352, 2000] for the Near Collapse Limit State. Scaling the records at the same value of S_a gives the possibility to reduce the variability of structural seismic response. In Fig. 11 to Fig. 14, Maximum Spring Rotation for increasing values of the spectral acceleration are reported. IDA analyses have been stopped for EEP-CYC 02, EEP-DB-CYC 03 and TS-CYC 04 connections when the first dissipative element achieve the ultimate available rotation reported in Table 2. This operation cannot be performed for the TS-M2-460-CYC 09 for which the collapse condition is achieved for the target MIDR equal to 0.1. With reference to the first three connections, IDA analyses in term of maximum spring rotation show the dependence from the earthquake record and, first of all, from the available ductility supply of the connection. It can also be observed that such results are a combination of the effects due, on one hand, to the plastic rotation supply of connections and, on the other hand, to the quality and stability of the hysteresis loops. In addition in Figures 15 to 18 the maximum interstorey drift ratio (MIDR) versus spectral acceleration curves are reported for the different earthquake records. The behavior of the structures is similar. This is probably due to the design method adopted for the analyzed MR-Frame which is devoted to assure a collapse mechanism of global type. This type of collapse mechanism leads to a structural damage well distributed among the different storeys. Finally, the seismic performances of the analyzed structure in term of maximum spectral acceleration for the achievement of the available ductility of the connections are reported in Table 4 where the average value of the spectral acceleration for each connection typology are considered.
Generally speaking, TS-CYC 04 and EEP-CYC 03 connections leads to good seismic performances as testified by Fig. 19 and Fig. 20. This result is justified because TS-CYC 04 connection assured the higher ductility capacity in term of plastic rotation (Table 2) which is able to redeem the significant pinching effect. From its side EEP-DB-CYC 03 connection show a lower plastic ductility supply if compared with the TS-CYC 04 connection but more stable hysteresis loops. However, the average best behavior is assured by the connection namely TS-M2-460-CYC 09 showing the best performances both in term of maximum plastic rotation than MIDR. However, taking into account the results of each selected ground motion, it is possible to observe that the structures equipped with friction dampers show the best performances with the only exception of Santa Barbara earthquake (terms underlined in Table 4) being assumed an ultimate interstorey drift equal to 0.1 at CP limit state. In addition, it is important to observe that after the achievement of the maximum device stroke the collapse did not occur because of the activation of new resistant mechanism as the shear resistance of bolts.
Conversely, the worst seismic performances are always obtained in case of extended end plate connections (EEP-CYC02). As the motivation making EEP-CYC02 the worst, it is useful to remember that, during the experimental test, being the displacement amplitude increased, the plastic engagement of the end-plate at the welded flange-to-end plate connection zone increases, leading to the formation of a crack along the whole width of the end-plate starting from the heat affected zone which progressively propagated along the thickness up to the complete fracture of the end-plate [Iannone *et al.*, 2011]. Even though this failure mode is consistent with the design purposes of type-1 collapse mechanism for the end-plate in bending, it provides a reduction of the plastic rotation supply under cyclic loads.

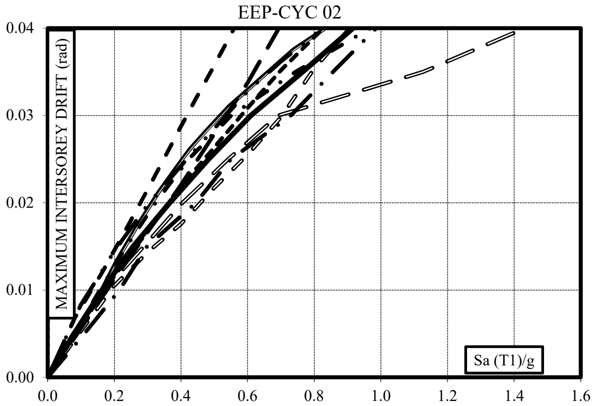

Figure 15. *Maximum Interstorey Drift Ratio vs Spectral Acceleration for TS-CYC 04 connection*

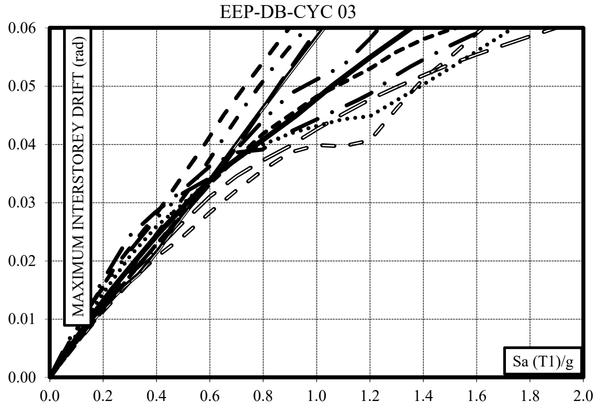

Figure 16. *Maximum Interstorey Drift Ratio vs Spectral Acceleration for EEP-CYC 03 connection*

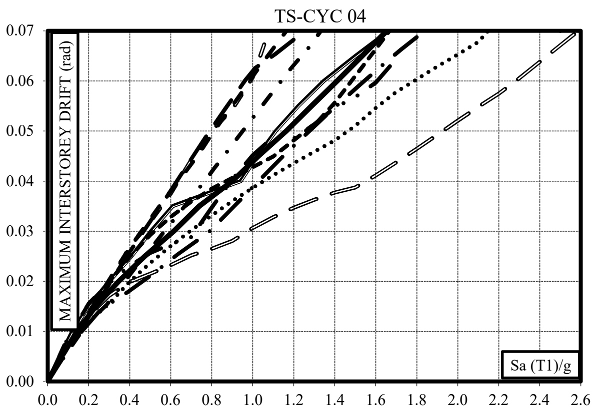

Figure 17. *Maximum Interstorey Drift Ratio vs Spectral Acceleration for EEP-CYC 02 connection*

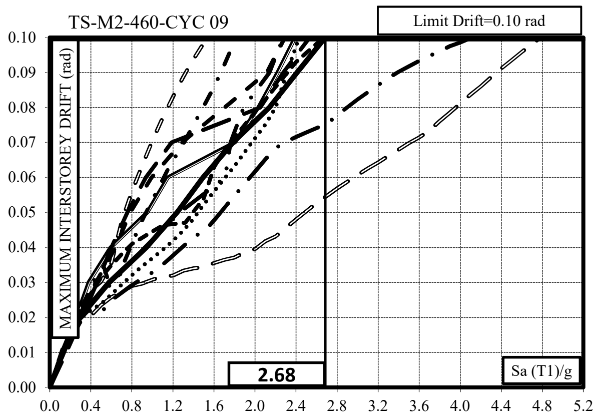

Figure 18. *Maximum Interstorey Drift Ratio vs Spectral Acceleration for TS-M2-460-CYC 09 connection*

Table 3. *Spectral acceleration corresponding to the achievement of the connections ultimate plastic rotation of each considered earthquake record*

Earthquake	EEP-CYC 02	EEP-DB-CYC 03	TS-CYC 04	TS-M2-460-CYC 09
Coalinga	1.16	2.25	1.61	2.67
Imperial Valley	0.76	1.13	1.52	2.29
Northridge	0.91	1.27	1.29	2.39
Spitak Armenia	1.06	1.30	1.45	1.82
Victoria Mexico	0.90	1.20	1.58	2.53
Kobe	0.95	1.84	2.50	4.11
Friuli	1.98	2.53	2.61	4.78
Helena	0.84	1.48	1.47	2.28
Santa Barbara	1.00	1.59	1.09	1.54
Irpinia	1.50	2.09	2.19	2.43
Average Sa(T_1/g)	**1.11**	**1.67**	**1.73**	**2.68**

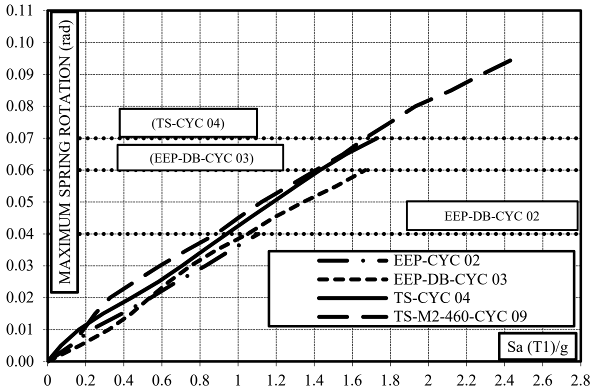

Figure 19. *Comparison between Maximum Spring Rotation vs Spectral Acceleration for the four considered connections*

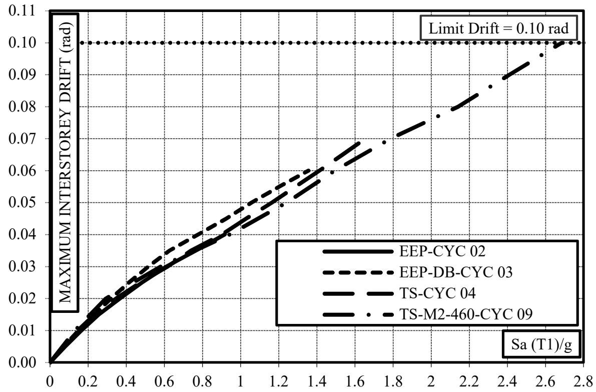

Figure 20. *Comparison between Maximum Interstorey Drift vs Spectral Acceleration for the four considered connections*

5. CONCLUSIONS

Partial-strength connections offer a number of advantages if compared with full strength connections because the yield point may be controlled in the design process leading to lighter beams and columns. From their side, semi-rigid partial-strength connections if well designed, own enough ductility and dissipation capacity in order to satisfy the seismic demand leading to hysteresis loops properly wide and stable.

In this paper the influence of partial strength beam-to-column connections on the seismic response of regular MR-Frames has been studied. Starting from the knowledge of the cyclic rotational behaviour of beam-to-column joints, four different MR-Frames have been considered. The first three are semi-rigid connections while the last one is a full rigid connection equipped with friction dampers. Each structure is characterized by a different structural detail of beam-to-column connections. The cyclic behaviour of each joint has been modelled by means of the spring element of Seismostruct computer program with the smooth hysteretic model whose parameters have been calibrated on the base of available experimental tests obtaining a good agreement between experimental and modelled behaviour. The observation of the results obtained from IDA performed by means of Seismostruct shows that the behaviour of the analysed MR-Frames equipped with TS-CYC04 connections, i.e. bolted double split tee connections, is comparable to the one of the MR-Frame equipped with EEP-DB-CYC03 connections, i.e. bolted extended end-plate connection with RBS. The worst performances are achieved by EEP-CYC02 connection while it is possible to conclude that MRF equipped with friction dampers constitute the more suitable solution for destructive earthquake exhibiting the best performances as testified by the spring rotation and MIDR results. Additional comparisons between the behavior of MR-Frames regular and with "set-backs" investigating the same connection typologies herein presented are reported in a previous work [Montuori *et al.*, 2016].

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INFLUENZA DEL COMPORTAMENTO CICLICO DEI COLLEGAMENTI TRAVE-COLONNA SULLA RISPOSTA SISMICA DI TELAI IN ACCIAIO REGOLARI

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SOMMARIO*: Il presente articolo ha lo scopo di investigare l'influenza dei collegamenti trave-colonna a parziale ripristino di resistenza inseriti all'interno di telai in acciaio regolari, progettati mediante la Teoria del Controllo del Meccanismo di Collasso (TPMC). I quattro collegamenti utilizzati sono progettati al fine di ottenere la medesima resistenza flessionale. I primi tre sono progettati sfruttando il metodo delle componenti considerando una differente disposizione dell'elemento debole che porta ad una rigidezza e ad una rotazione ultima differente. Il quarto collegamento è invece un dissipatore ad attrito opportunamente progettato per garantire la dissipazione dell'energia in ingresso.*
Le strutture investigate sono state modellate al fine di rappresentare accuratamente sia la rigidezza che la resistenza nonché le capacità deformative, specialmente con riferimento alle connessioni semi-rigide. I collegamenti trave-colonna sono modellati per mezzo di molle rotazionali collocate alle estremità delle travi per le quali i cicli di isteresi sono caratterizzati sia da un degrado di rigidezza che da fenomeni di pinching.
I parametri che caratterizzano il comportamento ciclico del nodo sono stati calibrati sulla base delle prove sperimentali. A partire da tale calibrazione, si sono condotte analisi dinamiche incrementali con lo scopo di predire il comportamento di telai regolari in acciaio equipaggiati con tali dispositivi.

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