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SEISMIC ASSESSMENT OF BEAM-TO-COLUMN JOINTS FOR A NON-CONFORMING MRF EXISTING STRUCTURE

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Abstract

As widely investigated in literature, the local behavior of the joints strongly influences the whole performance of the steel structures; thus, both in the design and assessment of the moment resisting frame (MRF) the joint performance should be properly accounted for. The present work is focused on local assessment and retrofit the beam-to-column joints of an existing non-conforming steel building. Therefore, the monotonic and cyclic behaviors of internal beam-to-column joints are investigated by means of finite element analyses (FEAs). The main peculiarities of these joints are the continuity of beams and hollow square columns interrupted at each level and connected both by bolts and fillet welds to the tapered flanges of the beams. On-site surveys allowed fully characterizing the geometry of the joints. The joint assemblies were sub-structured from the moment resisting frames and analyzed against both the vertical loads and seismic actions. The results showed that these types of joints exhibit very poor seismic behavior, concentering most of the damage within the welds. Therefore, different local strengthening interventions were designed and numerically checked. The comparison between the response of the unreinforced and the various strengthening joints is described, and the best solutions in terms of both cyclic behavior and technological feasibility are subsequently identified.

Keywords: Existing structures, Beam-to-column joints, Seismic retrofit, ductile behavior, finite element model.

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1 INTRODUCTION

Nowadays many studies investigated the behavior of the steel structures in seismic areas [1-12], with the aim to obtain ductile structures that could resist to the earthquake action without showing any brittle behavior; however, in many cases, these studies and innovative solutions could be applied not only to the new structures but could represent a viable solution also for the existing structures.

The existing steel buildings are characterized by a great variability in terms of structural conception, typology and distribution of resisting systems, and details adopted for the connections. Moreover, old steel buildings were often designed without considering any seismic action or, even when some lateral loads were considered, no hierarchy criteria between structural members were adopted. Thus, in the assessment of the existing steel building particular attention should be given to the joint's details; indeed, as deeply investigated in literature for the new structures [13-22], also in the analyses of the existing buildings the joint could influence the global structural behavior.

In this framework, an existing steel multi-storey building located in Naples has been selected as a case study due to its structural peculiarities. The design was carried out in accordance to the Italian provisions that were in force during the 1960 as reported in [23]; therefore, only gravity and wind loads were considered at that time. The P-Delta effects were also completely neglected despite of the height of the building (22.5 m). The structure was conceived and manufactured not in conformity with the majority of similar contemporary buildings.

The main aim of this research is to investigate the local behavior of the moment resisting joints and to investigate the efficiency of some local intervention to strengthen and improve the joint behavior. The paper is divided in four main parts; initially the structural feature of the building is described; then the main aspects of the finite element model (FEM) are presented. The third part is mainly focused on the results of beam-to-column joints under both vertical and horizontal action. Finally, the efficiency of four possible retrofitting solutions is presented.

2 DESCRIPTION OF THE STRUCTURE

2.1 History and general features

The multi-storey building serves for offices and depository of documents. The structure was built between 1960 and 1961 to replace a former two-storeys masonry construction used as public office. The V-shape of the original plan of the building was preserved (see Figure 1), with a footprint of about 1400 m^2. The design was carried out in accordance to the Italian provisions that were in force during the 1960; therefore, only gravity and wind loads were considered. MRFs were adopted on the longitudinal axes of the construction, while various types of concentric braces (X and Y shaped) were located in the transverse direction.

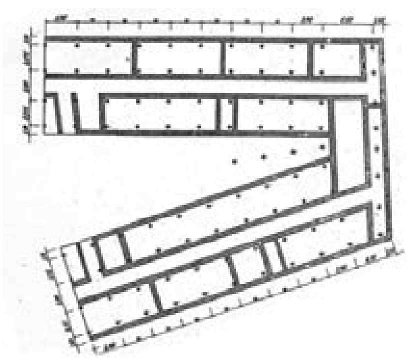


Figure 1: Planar view of the building.

2.2 History and general features

The columns of the MRF systems have a hollow squared tubes cross section, with a constant external size along the height of the building (140x140 mm), but the thickness of the tubes varies from 18 to 6 mm from the bottom to the top. The main beams are made with IPN 320 profiles at all storeys. Floors were made using corrugated steel sheets filled with concrete for a total height of 55 mm. Geometrical features of MRF joints are summarised in Fig. 2.

According to the design report [23], all members are made with Aq 42 Steel, with an allowable stress of 1600 kg/cm^2, with the only exception of the tubes, which are manufactured with Aq 55 Steel, with an allowable stress of 2000 kg/cm^2. High-strength bolts (8.8) has been considered.

A total of six beam-to-column joints (one for each floor) have been investigated under both vertical and horizontal actions. Therefore, a specified nomenclature was introduced to identify each investigated joint in function of the floor ("Fx") and the load scenario (vertical or seismic).

3 MODELING ASSUMPTIONS

The finite element models (FEMs) were developed using ABAQUS 6.14 [24]. The modelling assumptions are the same as described by the Authors in previous publications [25-26]. Therefore, only the main features of the models are summarized in the following for the sake of brevity.

The beam-to-column joints were modelled considering a sub-assemblage of the whole structure, which is obtained by extracting both the columns and the beams at the inflection points of the bending moment diagram induced by gravity and horizontal loads on the MRF. All elements were discretized using C3D8R solid element type. The characteristic dimension of mesh was set equal to 5 mm for bolts, welds and plates, and 20 mm for the beams and the columns.

According to the design report, a yield stress of 240 MPa was assigned to all the existing members, with the only exception of the columns, for which the yield stress was set to 300 MPa.

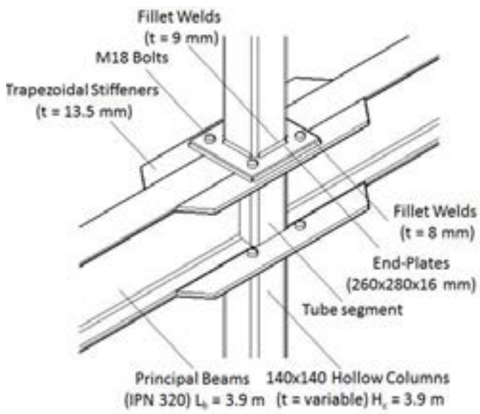


Figure 2: Detail of investigated MRF beam-to-column joints.

Steel yielding was modelled using the Von Mises criterion. The kinematic and isotropic hardening were implemented. The constitutive law for high-strength bolts (class 8.8) was modelled according to D'Aniello et al. [27]. The weld steel was modelled with the Ramberg-Osgood constitutive law; in absence of more detailed information from the design report, the yielding strength of existing welds was assumed to be equal to the highest between the existing members (300 MPa). Finally, an European S355 steel grade was used for all strengthening elements.

Displacement histories were applied at the tip of the beams in the sub-assemblages, reaching a maximum chord rotation of 0.06 rad. AISC 341 [28] protocol was adopted for cyclic analyses.

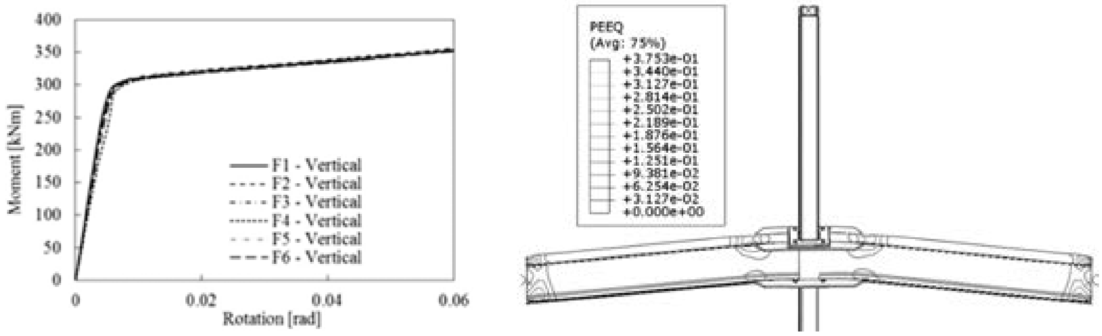
4 RESULTS FROM FINITE ELEMENT ANALYSES

4.1 Behavior of MRF beam-to column joints under vertical loads

The behaviour of MRF beam-to-column joints under vertical loads was investigated imposing maximum vertical displacements at the beam tips equal to 120 mm.

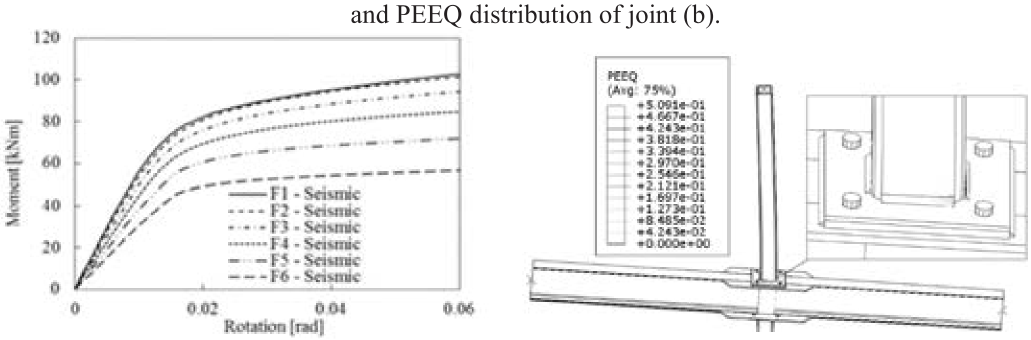
As expected, the internal beam-to column joints showed a good overall response, with the activation of plastic hinges at the extremity of the beams with basically no involvement of the column that remains in elastic range (see Fig. 3). Plastic hinges start to develop nearby the continuity restraint, while increasing the vertical displacements also the activation other two plastic hinges at the beam extremity can be observed. Moreover, since the beam profiles do not change among the different floors, the response in terms of bending resistance is almost the same for all the investigated joints, as shown in Fig. 3a.

However, a very small difference in terms of elastic stiffness can be observed owing to the variation of the column bending stiffness at each floor.



(a) (b)

Figure 3: Results of MRF joints at 3rd floor under vertical loads in terms of moment-rotation curve (a) and PEEQ distribution of joint (b).



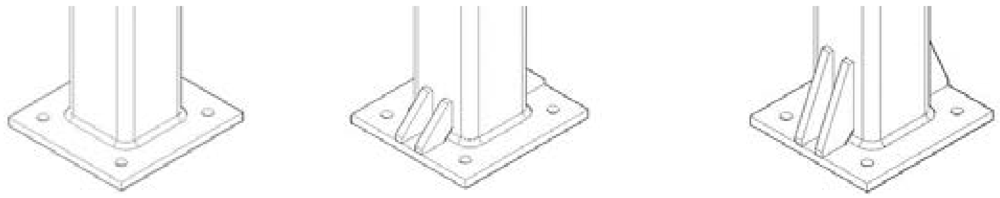
(a) (b)

Figure 4: Results of MRF joints at 3rd floor under seismic loads in terms of moment-rotation curve (a) and PEEQ distribution of joint (b).

4.2 Seismic behavior of MRF beam-to column joints

The seismic behaviour of MRF beam-to-column joints is depicted in Fig. 4 in terms of moment-rotation curves and distribution of equivalent plastic strains (PEEQ); in Fig. 4b it can be notice that all plastic deformations are concentrated in the columns and in the fillet welds. The beams always remain in an elastic range; the same type of failure mode can be observed at each floor of the building. The maximum bending resistance is showed by the joint at the first floor, as shown in Fig. 4a; indeed, due to the variation of the column thickness a reduction of the resistance along the structural height can be observed.

In Table 2 results from each scenario are reported in terms of maximum bending moments.



a) RS1 b) RS2 c) RS3

Figure 5: Different retrofitting solutions for the beam-to-column joint: weld collar (a), 65 mm high ribs (b), 150 mm high ribs (c)

Floor	Vertical [kNm]	Seismic [kNm]
Floor 1	356.1	102.8
Floor 2	355.4	101.4
Floor 3	355.0	94.8
Floor 4	354.3	85.2
Floor 5	353.9	72.2
Floor 6	351.9	56.5

Table 2: Results from numerical analyses in terms of maximum bending moments.

4.3 Strengthening solutions

The results of the numerical analyses showed that MRF joints have a good behaviour under gravity loads while exhibit a poor ductile response, with a large plastic concentration in the column's tip and in the adjacent fillet welds (see Fig. 4b). Therefore, three different strengthening interventions have been designed and numerically tested to improve the seismic performance of the joints by preventing welds brittle failure. For sake of brevity only results of the joint at the third floor (with the medium value of column thickness) are reported hereinafter.

The main geometrical features of the interventions (see Fig. 5) are summarised as follows:

1. Complete replacement of the fillet welds with a continuous weld collar made with stronger material (e.g. yielding stress equal to 450 MPa), see Fig. 5a (RS1).
2. Replacement of the welds and introduction of four 65x10 mm rib stiffeners made with steel S355, two on each side of the column, shaped with an angle of 40°, see Fig. 5b (RS2).
3. Replacement of the welds and introduction of four 150x10 mm rib stiffeners made with steel S355, two on each side of the column, see Fig.5c (RS3).

The results of monotonic analysis for each RS scenario are depicted in Fig. 6. Maximum bending moments and the relative increment with respect to unreinforced joint response are also reported in Table 3. As it can be observed, the retrofitting solutions provide a large variation of both resistance and elastic stiffness; indeed, with the only exception of RS1, that only implies the change of the welds, the introduction of the rib stiffeners increases the assembly stiffness and the resistance. The maximum increase of the bending capacity is not very high (RS3, 17% with respect to the unreinforced joint) but the real benefit of the presence of the ribs is to obtain a more ductile mechanism moving the failure from the weld to the column allowing a larger rotational capacity. This aspect can be better observed in case of cyclic analyses (see Fig. 7).

Thus, comparing the unreinforced solution with the RS3 (the one with two ribs) it can be observed that the first case a large concentration of plastic deformation appears in the welds and at the column tip. Contrariwise, the reinforced solution provides a slightly larger resistance and a more stable energy dissipation without pinching. In this case plastic strains are concentrated in the columns, leaving the welds in elastic range.

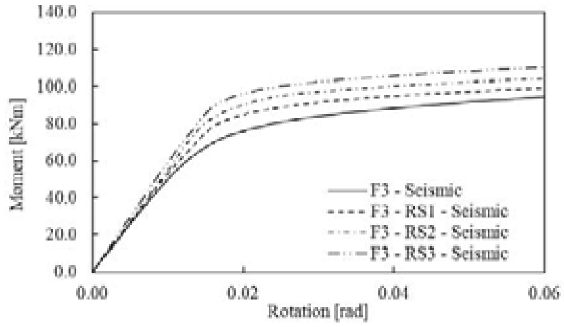
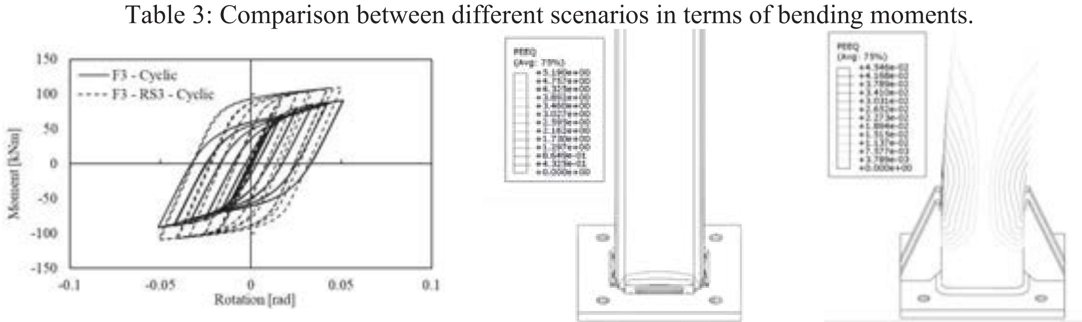


Figure 6: Moment – rotation curves for all different strengthening solutions.

Intervention	$M_{j,Max}$ [kNm]	Resistance increment [%]
RS0 (Unreinforced)	94.8	-
RS1 (Replaced Welds)	99.2	+5%
RS2 (Ribs 65 mm)	104.5	+10%
RS3 (Ribs 150 mm)	110.2	+16%

Table 3: Comparison between different scenarios in terms of bending moments.



(a) (b) (c)

Figure 7: Moment-rotation curves for the unreinforced column and the RS3 proposal (a) and distribution of equivalent plastic strains under cyclical horizontal actions for the same cases (b-c).

5 CONCLUSIONS

An existing non-code conforming six-storey steel building has been investigated. Particular attention was paid on both vertical and lateral behaviour of the MRF beam-to-column joints. From the results of FEAs the following observations can be pointed out:

- Moment resisting joints are able to support the vertical loads showing only the activation of the plastic hinges at very large value of chord rotation.
- Lateral loads induce stress concentration in the welds around the columns, which represent a brittle component. Therefore, local interventions have been designed to retrofit joints.
- The first retrofitting intervention (RS1) involves the replacement of the original welds; results show that the increment of weld resistance is not enough to change the failure mode.
- The second (RS2) and third (RS3) interventions consist in the use of rib stiffeners to increase the resistance of the connection. The numerical results show that both under monotonic and cyclic action the introduction of rib stiffeners allows to reduce the plastic demand within the welds. Therefore, the local ductility of the joints is preserved despite additional consideration should be done form a global point of view.

REFERENCES

[1] F. Di Lauro, R. Montuori, E. Nastri, V. Piluso, Partial safety factors and overstrength coefficient evaluation for the design of connections equipped with friction dampers. *Engineering Structures*, **178**, 645-655, 2019.

[2] R. Montuori, E. Nastri, V. Piluso, Problems of modeling for the analysis of the seismic vulnerability of existing buildings. *Ingegneria Sismica*, **36**(2), 53-85, 2019.

[3] A. Poursadrollah, M. D'Aniello, A. De Martino, R. Landolfo, Preliminary study on the seismic performance of hybrid steel structures with truss lightweight girders and plug-and-play connections. *Ingegneria sismica* **37**,102-114, 2020.

[4] L. Fiorino, T. Pali, R. Landolfo, Out-of-plane seismic design by testing of non-structural lightweight steel drywall partition walls. *Thin-Walled Structures*, **130**, 213-230, 2018.

[5] V. Macillo, S. Shakeel, L. Fiorino, R. Landolfo, Development and Calibration of Hysteretic Model for CFS Strap braced stud walls. *International Journal of Advanced Steel Construction*, **14**(3), 337-360, 2018.

[6] A. Campiche, Numerical modelling of CFS three-story strap-braced building under shaking-table excitations. *Materials*, **14**(1), 1–13, 2021

[7] A. Campiche, L. Fiorino, R. Landolfo, Numerical modelling of CFS two-storey sheathing-braced building under shaking-table excitations, *J. Constr. Steel Res.* **170**, 2020.

[8] S. Shakeel, L. Fiorino, R. Landolfo, Behavior factor evaluation of CFS wood sheathed shear walls according to FEMA P695 for Eurocodes. *Engineering Structures,* **221,** 111042, 2020.

[9] L. Fiorino, O. Iuorio, V. Macillo, R. Landolfo, Performance-based design of sheathed CFS buildings in seismic area. *Thin-Walled Structures*, **61,** 248-257, 2012

[10] R. Montuori, G. Gabbianelli, E. Nastri, M. Simoncelli, Rigid plastic analysis for the seismic performance evaluation of steel storage racks. *Steel and Composite Structures*, **32** (1), 1-19, 2019.

[11] R. Montuori, E. Nastri, V. Piluso, Modelling of floor joists contribution to the lateral stiffness of RC buildings designed for gravity loads. *Engineering Structures*, **121**, 85-96, 2016.

[12] A. Campiche, S. Shakeel, V. Macillo, M.T. Terracciano, B. Bucciero, T. Pali, L. Fiorino, R. Landolfo, Seismic behaviour of sheathed CFS buildings: Shake table tests and numerical modelling. *Ingegneria sismica,* **35**, 106-123, 2018.

[13] E. Elettore, F. Freddi, M. Latour, G. Rizzano, Design and analysis of a seismic resilient steel moment resisting frame equipped with damage-free self-centering column bases. *Journal of Constructional Steel Research*, **179**, 2021.

[14] S. Di Benedetto, A.B. Francavilla, M. Latour, G. Ferrante Cavallaro, V. Piluso, G. Rizzano, Pseudo-dynamic testing of a full-scale two-storey steel building with RBS connections. *Engineering Structures*, **212**, 2020.

[15] G. Ferrante Cavallaro, A.B. Francavilla, M. Latour, V. Piluso, G. Rizzano, Cyclic response of low yielding connections using different friction materials. *Soil Dynamics and Earthquake Engineering*, **114**, 404-42, 2018.

[16] A. Lemos, L.S. da Silva, M. Latour, G. Rizzano, Numerical modelling of innovative DST steel joint under cyclic loading. *Archives of Civil and Mechanical Engineering*, **18** (3), 687-701, 2018

[17] M. D'Antimo, J.F. Demonceau, J.P. Jaspart, M. Latour, G. Rizzano, Experimental and theoretical analysis of shear bolted connections for tubular structures. *Journal of Constructional Steel Research*, **138**, 264-282, 2017.

[18] M. Latour, G. Rizzano, Mechanical modelling of exposed column base plate joints under cyclic loads. *Journal of Constructional Steel Research*, 2019.

[19] A.F. Santos, A. Santiago, M. Latour, G. Rizzano, Analytical assessment of the friction dampers behaviour under different loading rates. *Journal of Constructional Steel Research*, **158**, 443-459, 2019.

[20] S. Di Benedetto, M. Latour, G. Rizzano, Assessment of the stiffness of 3D cut welded connections with CHS columns and through I-BEAMS. *Structures*, **27**, 247-258, 2020

[21] S. Di Benedetto, M. Latour, G. Rizzano, Chord failure resistance of 3D cut welded connections with CHS columns and through I-BEAMS. *Thin-Walled Structures,* **154**, 2020.

[22] R. Tartaglia, M. D'Aniello, G.A. Rassati, J.A. Swanson, R. Landolfo, Full strength extended stiffened end-plate joints: AISC vs recent European design criteria. *Engineering Structures*, **159**,155-171, 2018.

[23] G. Travaglini, F. De Miranda, L'ampliamento della sede uffici del Genio Civile di Napoli, Costruzioni Metalliche, **2**, 198-209, 1962 [in Italian].

[24] Dassault (2014), Abaqus 6.14 User's Manual, Dassault Systèmes Simulia Corp.

[25] R. Tartaglia, M. D'Aniello, M. Zimbru, Experimental and numerical study on the T-Stub behaviour with preloaded bolts under large deformations. *Structures*, **27**, 2137-2155, 2020.

[26] R. Tartaglia, M. D'Aniello, Influence of Transvserse Beams On the Ultimate Behaviour of Seismic Resistant Partial Strength Beam-To-Colummn Joints. *Ingegneria Sismica*, **37**(3), 50-65, 2020.

[27] M. D'Aniello, D. Cassiano, R. Landolfo, Simplified criteria for finite element modelling of European preloadable bolts. *Steel and Composite Structure*, **24**, 643–658, 2017.

[28] AISC (American Institute of Steel Construction) (2016), Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341–16.