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# SEISMIC TIMBER FRAMES WITH STEEL LINK: MECHANICAL CHARACTERIZATION THROUGH NUMERICAL INVESTIGATION

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**ABSTRACT:** Since timber has elastic-fragile behavior, for the design of seismic resistant timber frame structures the dissipative capacity is usually concentrated into steel connections. However, these are components with an important role in the structural system, so they should be preserved by damage. To this purpose, with regards to timber framed structures, innovative timber joints with steel link, with a dissipation function based on cycles of plastic deformation, can be introduced. In this context, the paper focuses on the mechanical characterization of a dissipative Moment Resisting Frame (MRF) structure, equipped with steel links at the joints and designed by applying hierarchy resistance criteria. Monotonic non-linear static analysis is performed by using the ABAQUS structural program, aimed at investigating both the global and joints behavior. The results confirmed the achievement of the plastic deformation in the link before joints and timber members failures, validating both the efficiency of the system and the proposed design method.

**KEYWORDS:** Seismic resistant dissipative timber moment resisting frames, steel link, capacity design for timber structures, FE modeling of timber MRFs, monotonic non-linear static analysis of timber MRFs.

## 1 – INTRODUCTION

At present, seismic resistant timber frame structures are assumed to dissipate input energy through plastic deformations of steel connectors, since timber has an elastic-fragile behavior. However, joints, having an important structural role, should be preserved by damage. Some authors have studied different alternatives to dissipative connections, showing a good potential to improve the seismic behavior of timber structures combining the excellent ductile characteristics of steel with the lightness of timber. Among others, Gilbert and Gohlich [1] and Miller et al. [2] studied hybrid steel-timber structures, such as timber frames equipped with steel Buckling Restrained Brace (BRB), Blomgren et al. [3] as well, but with a Timber BRB. Chen et al. [4] studied a beam to column joint characterized by a steel panel box in the column, connected to the timber members through glued steel bars. Tomasi et al. [5] and Andreolli et al. [6], proposed to apply steel links at the beam ends in moment resistant frames, as the same Montuori and Savarese [7] a steel link but with reduced beam sections, derived from steel MR frames practice. In this context, at the University of Naples Federico II since some years the solution with steel links is deepened through theoretical, numerical and experimental studies, either implementing the capacity design [8, 9] at the level of structural element and at the level of the joint components, or developing the joint details [10, 11]. Studies have been carried out also in cooperation with the University of Trento in Italy [12] and with the University of Minho, at Guimaraes, in Portugal [13, 14]. In this framework, the paper focuses on the mechanical characterization of a MR portal frame, equipped with dissipative steel links at the joints, designed according to the proposed capacity design criteria, in order that timber elements and steel connections remain in the elastic field while the link undergoes plastic deformations. Therefore, a refined FE model is set up and a monotonic non-linear static numerical analysis is performed through the software ABAQUS (v.2022), for grasping the peculiarities of both global structural behavior of the frame and local behaviour of joints under horizontal actions. The aim of the investigation is to demonstrate the effectiveness of the system and of the proposed design criteria.

## 2 – BACKGROUND

The work falls in the context of the research project DPC–ReLUIS (2024-2026), with regards to innovative timber frame structural systems. The peculiarity implemented is the integration of modern steel connection technology into timber structures, for avoiding brittle failure modes and improving the overall seismic performance. The project aims at both conceiving the constructional dissipative joint details and

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investigating the global and local structural performances of seismic resistant timber frames equipped with the dissipative devices, through campaigns of numerical analyses and experimental tests. The final goals are to provide specific design rules and constructional details, further to qualify the mechanical behaviour of joints.

## 3 – PROJECT DESCRIPTION

The MRF case study is a single storey structure 3m high, with a rectangular plan layout having one bay, 4.5m long, in the y transverse direction and three bays, 6m long, in the longitudinal x-direction, for a total length of 12m (Fig. 1). The floor structure is oriented along the y-direction. The vertical seismic resistant system consists of 4 MRFs, 2 for each direction. The roof is assumed as composed by timber planks with a thin concrete slab. The structural and non-structural permanent loads are equal to $G_{1k}$=0.22kN/m$^2$ and $G_{2k}$=1.30kN/m$^2$, respectively. The variable service load is equal to $Q_k$=2.00kN/m$^2$ (NTC2018) [15]. The seismic action is defined with reference to the seismic zone 1, according to OPCM 3274 (03/20/2003), corresponding to the peak ground acceleration $a_g$= 0.35g and a “category B” soil. The structure is designed as both non-dissipative (ND) and dissipative (D), by using, respectively, behaviour factors $q_d$=1 and $q_d$=4, in Medium Dissipative Capacity (DCM), as for the same structural types made of steel, according to Eurocode 8 (EN 1998-1-1, 2005, [16]) [17], considering that the ductility is in charge of the steel links. The response spectra are defined at Life Safety (LS) and Damage (D) Limit States. Loads are combined according to NTC2018 (§ 2.5.3, [15]). With regards to materials, for timber members, beam-to-column and column-to-foundation steel link, GL24h, S355 and S235 grades, respectively, are used.

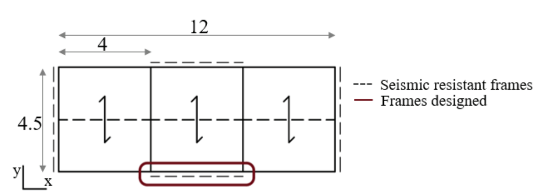


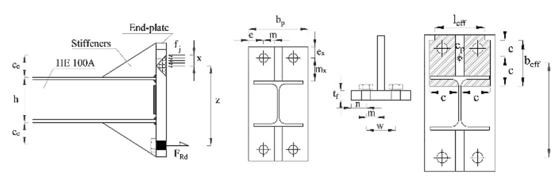

*Figure 1. Plan layout [m].*

## 4 – DESIGN PROCESS

### 4.1 DESIGN CRITERIA OF TIMBER FRAMES WITH DISSIPATIVE STEEL LINKS

The MRF structure is designed in accordance with the proposed design criteria based on the application of the capacity design [8, 10], which is applied at the levels of macro-components (beam, columns, beam-to-column and column-to-foundation links) and sub-components (link connection elements, such as, end-plate, stiffeners and bars). Dissipative zones are assumed to be located in the steel links, which should have high ductile capacity, with cross sectional classes 1, 2. To ensure yielding of the dissipative zones for cycles of bending moments, all non-dissipative members, such as timber beams, columns and connections, should be designed according to hierarchy resistance criteria, on the bases of the design strength of the ductile parts, through the application of an overstrength factor Ω, which should be adequately defined according to the EC8 [8, 16]. In particular, the lowest $\Omega_i$ value, among those calculated at beam-to-column and column-to-foundation links should be considered.

For sub-components, the joint is modelled as an assembly of individual parts, each evaluated separately for contributing to overall strength, stiffness and rotational behaviour, according to the component method (EC3 part 1-8, [18]). The beam-to-column and column-to-foundation link connection is assumed as rigid. Under these conditions, the joints design bending resistance is then determined as specified in Iovane et al. [11], considering the design resistance of the joint, assumed as the resistance of the weakest joint component referred to all the following possible collapse modes of the joint: *T-stub in tension* (Fig. 2b), *T-stub in compression* (Fig. 2c), yielding failure of *steel stiffeners*, *glued-in steel bars in tension (pull-out)*.




a) Stiffened endplate joint b) T-Stub in tension c) T-Stub in compression

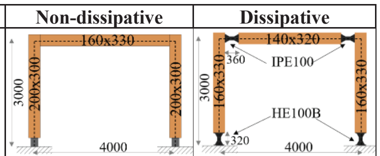
*Figure 2. Stiffened endplate joint and Equivalent T-stub models (EN 1993-1-8: 2005, [18]).*

### 4.2 STRUCTURAL DESIGN AND SEISMIC PERFORMANCE EVALUATION

For the design of MRF structure, linear dynamic analysis is carried out, through the structural calculation program SAP2000 (v18). The design procedure consists at first in the design of structural members (beams, columns and dissipative links), applying the capacity design criteria for macro-components (§4.1), secondly in the design of joints according to the hierarchy collapse criteria for sub-components (§4.1) in order to achieve the required ductile capabilities, based on the Overstrength Component $OC_i$, evaluated as the ratio $M_{i,Rd}/M_{l,pl,Rd}$, between the bending resistance of the joint $M_{i,Rd}$ corresponding to the design resistance of the i-th joint components and the bending design resistance of the link

$M_{l,pl,Rd}$, assuming that the dissipative link is the first component to reach the elastic limit (yielding). Design results are presented in terms of structural sizes, design force at LS ($F_{d,LS}$), structural mass (M) and structural mass variation of the dissipative (D) structures as respect to the non-dissipative (ND) ones through the ΔM factor, equal to $\Delta M=(M_{ND}-M_D)/M_{ND}$, where $M_{ND}$ and $M_D$ are the structural mass of ND- and D-structures. Corresponding data are given in Table 1.

*Table 1: Member sizes for non-dissipative and dissipative MRF.*

| | Non-dissipative | Dissipative |
|---|---|---|
| **Member size [mm]** |  | |
| Timber beam | *GL24h* 160x330 | *GL24h* 140x320 |
| Timber column | *GL24h* 200x300 | *GL24h* 160x330 |
| Link beam-column | / | *S355* IPE100 |
| Link column-foundation | / | *S235* HE100B |
| **Design force [kN]** | 57.8 | 26.0 |
| **Structural mass** | | |
| M [kN] | 2.2 | 1.91 |
| ΔM [%] | 13 | |

The length of the links is selected to ensure the formation and development of the plastic hinge. It is noted that the D structure has 13% reduction of the structural mass compared to the ND one.

For dissipative MRF, at the design stage the links are the first components to reach the yielding. Then the collapse hierarchy of the structural elements is imposed to achieve a ductile failure, according to the following order: 1) beam-to-column link; 2) column-to-foundation link; 3) Timber beam; 4) Timber column. In Table 2, the collapse hierarchy, the bending moment resistance $M_{j,Rd}$, the coefficient of structural overstrength $\Omega_i$ and the component overstrength $OC_i$ as respect to the link are given.

*Table 2: Collapse hierarchy of the D-MRF structural elements.*

| Collapse hierarchy | $M_{i,Rd}$ [kNm] | $\Omega_i$ | $OC_i$ |
|---|---|---|---|
| Link beam-column yielding | 13.32 | 1.12 | 1.00 |
| Link column-foundation yielding | 23.32 | | 1.79 |
| Timber beam in bending | 43.50 | | 3.27 |
| Timber column in bending | 52.80 | | 3.96 |

Both beam-to-column (IPE100 joint) and column-to-foundation (HE100B joint) joints consist of a steel link equipped with two welded S275 steel end-plates, 120x230 $mm^2$ (IPE100) and 120x300 $mm^2$ (HE100B), 15mm thick; four S355 stiffeners, 110x65 $mm^2$, 15mm thick. The end-plates are connected to the timber beam by means of 4 glued threaded bars, M16, 10.9, 540mm (IPE100) and 740mm (HE100B) long, respectively (Fig. 3). Such connection is assumed as rigid.

At the design stage, the collapse hierarchy of the IPE100 and HE100B joints components is imposed to achieve a ductile failure, according to the following order: 1) link, 2) stiffeners (IPE100) / endplate (HE100B), 3) endplate (IPE100) / stiffeners (HE100B), 4) bars, 5) timber beam.

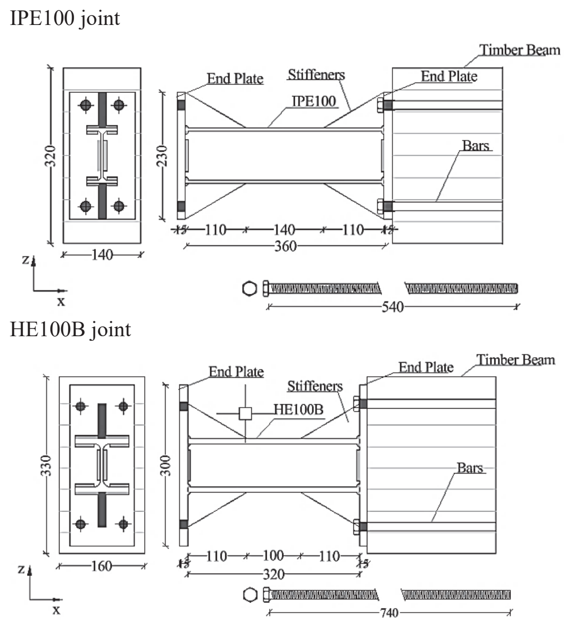



*Figure 3. Geometrical features of the joints [mm].*

The results of the capacity design for sub-components are presented in Table 3, in terms of the joint component design resistance $F_{i,Rd}$, corresponding bending resistance $M_{i,Rd}$, and overstrength ($OC_i$). In particular, in both joints the dissipative link is the first component to reach the elastic limit (yielding), while all the other components have an overstrength as respect to the steel link.

*Table 3: Collapse hierarchy of joints components.*

| Collapse hierarchy | $F_{i,Rd}$ [kN] | $M_{i,Rd}$ [kNm] | $OC_i$ |
|---|---|---|---|
| **IPE100 joint** | | | |
| Link yielding | - | 13 | 1.00 |
| Stiffeners yielding | 104 | 18 | 1.38 |
| T-stub in tension (Mode 1) | 124 | 22 | 1.69 |
| Bar pull-out (Mode c) | 136 | 24 | 1.85 |
| T-stub in compression | 165 | 29 | 2.23 |
| Bar pull-out (Mode d) | 206 | 36 | 2.77 |
| Bar failure in tension | 226 | 40 | 3.08 |
| Timber beam in bending | - | 44 | 3.38 |
| **HE100B joint** | | | |
| Link yielding | - | 23 | 1.00 |
| T-stub in tension (Mode 1) | 138 | 31 | 1.35 |
| Stiffeners yielding | 158 | 35 | 1.52 |
| Bar pull-out (Mode c) | 160 | 36 | 1.57 |
| T-stub in compression | 164 | 37 | 1.61 |
| Bar failure in tension | 226 | 51 | 2.22 |
| Bar pull-out (Mode d) | 235 | 52 | 2.26 |
| Timber column in bending | - | 53 | 2.30 |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### 4.3 MECHANICAL CHARACTERIZATION THROUGH MONOTONIC NON-LINEAR STATIC NUMERICAL ANALYSIS

The mechanical behaviour of the dissipative MR portal frame is examined through a monotonic non-linear static numerical analysis using the structural calculation program ABAQUS (v.2022). The implicit dynamic analysis is performed by applying a load history in displacement control. The displacement is imposed at the beam-column node, with an increment of 20mm up to collapse, corresponding to a speed of 0,2 mm/s. The outputs are here presented in terms of Force-Displacement (F-u) and Moment-Rotation curves obtained at the load application point, stress and AC yield (ACtively yielding) distributions that identifies the attainment of yield stresses. The performance is analyzed at specific points, corresponding to yielding ($P_Y$), complete plasticization ($P_P$) and collapse ($P_C$) of the links (Fig. 4).

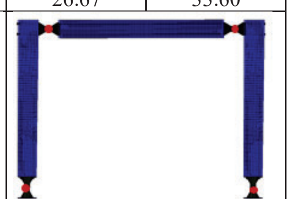
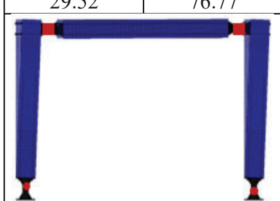
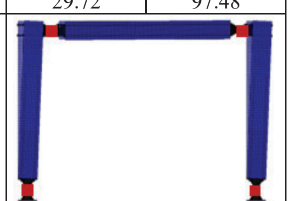
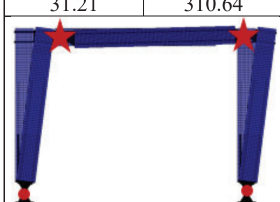
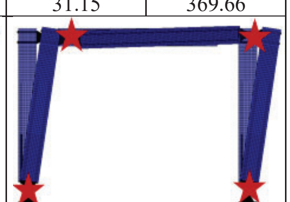
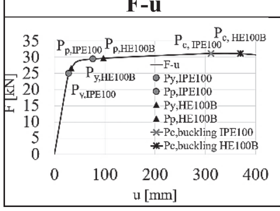
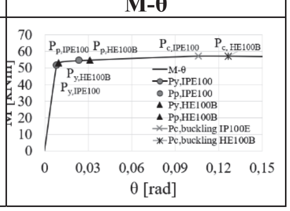
| F [kN] | u [mm] | F [kN] | u [mm] |
|---|---|---|---|
| $P_{Y,IPE100}$ | | $P_{Y,HE100B}$ | |
| 25.06 | 28.75 | 26.67 | 33.60 |
|  | |  | |
| $P_{P,IPE100}$ | | $P_{P,HE100B}$ | |
| 29.52 | 76.77 | 29.72 | 97.48 |
|  | |  | |
| $P_{C,IPE100}$ | | $P_{C,HE100B}$ | |
| 31.21 | 310.64 | 31.15 | 369.66 |
|  | |  | |
| F-u | | M-θ | |
|  | |  | |

*Figure 4. Progressive damage; F-u and M-θ curves: Y-yielding, P-plasticization, C-collapse.*

A refined FE model is set up. Regarding materials, timber is modeled as elastic fragile, while for steel an isotropic hardening model is adopted, according to EC3 part 1-1 [19], true stress-true strain equations are used. The MRF elements are modelled through Solid Elements, the mesh is automatically generated and then modified for adapting it and ensuring a regular mesh distribution. The interaction between shank and washer, shank and end-plate, washer and nut, end-plate and washer, end-plate and beam is a "*Surface to Surface Contact*", which simulates the simple direct contact between several components, by using a friction coefficient (0.4 steel-to-steel and 0.3 timber-to-steel), while the interaction between link and end-plate, shank and nut, shank and beam' holes is a "*Tie Constraint*", which simulates the welding between steel elements, as well as the gluing between the external surface of the steel bars shank and the internal surface of the holes in the timber beam. The interaction used are shown in Figure 5.

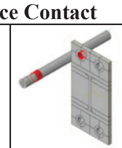
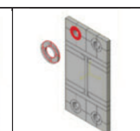
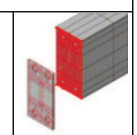
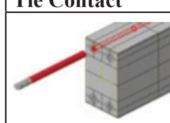
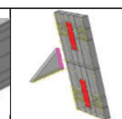
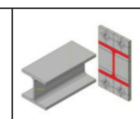
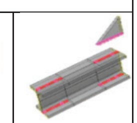
| **Surface to Surface Contact** | | | |
|---|---|---|---|
|  |  |  |  |
| Shank | Bar | End-plate | End-plate |
| Washer | End-plate | Nut | Beam |
| **Tie Contact** | | | |
|  |  |  |  |
| Bar | Stiffener | Link | Stiffener |
| Beam | End-plate | End-plate | Link |

*Figure 5. FE interaction used.*

## 5 – RESULTS

As for the global behavior, it is apparent from Figure 4 that the beam-to-column link reaches yield first, followed by yielding of the column-to-foundation link ($P_Y$), as well as failure is reached first in the beam-to-column link and then in the column-to-foundation link due to the flange buckling in both the joints ($P_C$). As for the joints behavior, the stress and the AC yield distributions in the joints are shown in Figure 6 at the collapse condition ($P_C$), where the maximum stress value ($\sigma_{max}$) of each joint component is also given. In addition, for each i-th component (L-link, EP-end plate, S-stiffeners, B-threaded bars and TB-timber beam) of the joints, the demand capacity ratio ($DCR_{i,j,el}=\sigma_{i,j}/\sigma_{i,el}$) between the maximum stress value evaluated at $P_C$ point ($\sigma_{i,PC}$) and the stress value at the elastic limit ($\sigma_{i,el}$) is provided.

The results of the capacity design for sub-components are presented in Table 4 in terms of overstrength ($OC_i$) of each component and $\Delta OC_i$ factor, equal to $\Delta OC_i$=($OC_{i,n}$-

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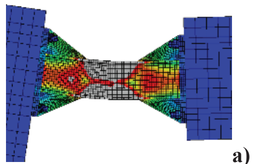
$OC_{i,a})/OC_{i,n}$, where $OC_{i,n}$ and $OC_{i,a}$ are the overstrength factor of the numerical and analytical studies.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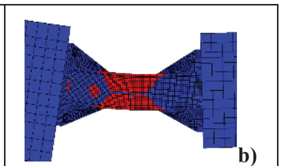
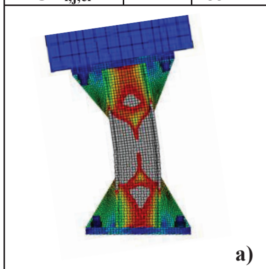

b)

| IPE100 | $L_B$ | $EP_B$ | $B_B$ | TB | $S_B$ |
|---|---|---|---|---|---|
| $\sigma_{max}$ | 403 | 236 | 375 | 14 | 284 |
| $DCR_{i,j,el}$ | 114 | 85 | 42 | 58 | 103 |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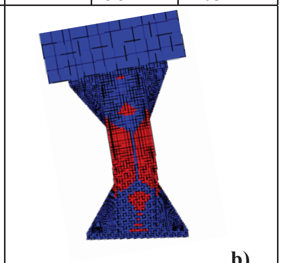
b)

| HE100B | $L_C$ | $EP_C$ | $B_C$ | TB | $S_C$ |
|---|---|---|---|---|---|
| $\sigma_{max}$ | 303 | 278 | 618 | 16 | 280 |
| $DCR_{i,j,el}$ | 129 | 101 | 69 | 67 | 102 |

*Figure 6. Stress (a) and AC yield (b) distributions, maximum values in the joint components [MPa] and DCR [%] at $P_C$.*

*Table 4: Comparison of analytical and numerical collapse hierarchy of joints components.*

| Collapse hierarchy | $OC_{i,a}$ | $OC_{i,n}$ | $\Delta OC_{i,}$ [%] |
|---|---|---|---|
| **IPE100 joint** | | | |
| Link yielding | 1.00 | 1.00 | - |
| Stiffeners yielding | 1.38 | 1.28 | -7.8 |
| T-stub in tension (Mode 1) | 1.69 | | |
| Bar pull-out (Mode c) | 1.85 | - | - |
| T-stub in compression | 2.23 | | |
| Bar pull-out (Mode d) | 2.77 | | |
| Bar failure in tension | 3.08 | 3.23 | 4.6 |
| Timber beam in bending | 3.38 | 3.33 | -1.5 |
| **HE100B joint** | | | |
| Link yielding | 1 | 1.00 | - |
| T-stub in tension (Mode 1) | 1.35 | 1.37 | 1.5 |
| Stiffeners yielding | 1.52 | 1.47 | -3.4 |
| Bar pull-out (Mode c) | 1.57 | - | - |
| T-stub in compression | 1.61 | | |
| Bar failure in tension | 2.22 | 2.05 | -8.3 |
| Bar pull-out (Mode d) | 2.26 | - | - |
| Timber column in bending | 2.30 | 2.29 | -0.4 |

From Figure 6b, it is possible to note that plastic hinge forms in the beam to column and column base links (red colour), while the other structural components remain elastic, with adequate overstrength (blue colour), ranging between 1.35 (HEB100 joint stiffener) and 3.38 (timber beam; Table 4), until the collapse condition ($P_C$). This corresponds to the links failure, due to the buckling of the flange in compression. The following collapse hierarchy of the joint components is evidenced (Tab. 4): Link flange buckles in both the joints, stiffeners undergo plastic deformation, while end-plate, bars and timber beam are still in elastic field, with a $\Delta OC_i$ from -0.4% for timber column in bending to -7.8% for IPE100 bar failure in tension. Results confirmed the collapse hierarchy of the joint components, validating both the efficiency of the system and the proposed design method.

## 6 – CONCLUSION

The mechanical characterization carried out of a dissipative and a non-dissipative MR portal frames, equipped with steel link and designed through proposed capacity based criteria, has demonstrated the efficacy of the approach applied, which ensures the required dissipation under seismic actions, contemporary involving the reduction of the structural mass for the dissipative structure as respect to the non-dissipative one ($\Delta M$=13%). In particular, the numerical monotonic non-linear static analysis has confirmed both the formation of the plastic hinge in the links, which dissipate the energy through plastic deformation, and the collapse hierarchy of the joints components, where the links yield reaching the ultimate strength before the collapse of the connection components (end-plate, steel bars, stiffeners) and the timber beam and columns. Therefore, both the system and the proposed design method are validated, with an approximation (analytical vs numerical results) generally lower than 8.3%. From the global perspective, the study is in progress for the calibration of the design overstrength coefficients, based on parametric numerical analyses on multi-storey multi-span structures, considering the ultimate conditions to be achieved according to the reference limit state. From the local perspective, the study is in progress to carefully appraise the mechanical behavior of the joints and the accuracy of the proposed design criteria also through cyclic non-linear dynamic numerical analyses and through experimental campaigns to confirm the efficiency of the system. Definitely the study provides a significant contribution to the development of guidelines for the design of dissipative timber MRF with steel links. The topic is relevant nowadays also in the context of the ongoing activity for the improvement of chapter 8 of Eurocode 8, dedicated to timber structures in seismic area.

## Acknowledgments

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## 7 – REFERENCES

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