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A SEMI-ANALYTICAL MODEL FOR DISSIPATIVE SHEAR LINKS: EXPERIMENTAL TESTS AND NUMERICAL ANALYSES

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Abstract. *Eccentrically braced frames (EBF) represent an optimal structural solution for seismic prone areas, being characterized by a high dissipative capacity, able to withstand strong seismic events, and by a relevant elastic stiffness, able to avoid (or limit) the damage of non-structural elements during low-to-moderate seismic events. The knowledge of the cyclic behaviour of the dissipative link elements is required to perform reliable and accurate nonlinear analyses and, in particular, to finally optimize the design of EBFs and their structural performance. Notwithstanding such aspects, in the current scientific literature few models are proposed for the modelling of EBFs equipped with short/shear links, mainly adopting simple bilinear springs with kinematic hardening constitutive laws that are not always able to correctly represent the real experimental behaviour of EBFs, especially for what concerns the influence of the hardening phenomena. In the present work, a semi-analytical model for short/shear link is proposed and calibrated basing on the results of experimental tests on real-scale one store/one bay EBFs equipped with horizontal and vertical dissipative elements. The main aim of the proposed model consists in the ability to represent the cyclic/seismic behaviour of EBFs with more accuracy with respect to what actually used in the current scientific works and, at the same, with a low computational burden with respect to solid models, becoming then an optimal solution for the implementation in design models and for the representation of multi-storey buildings' behaviour.*

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1 INTRODUCTION

The seismic events of the 1980s-1990s caused severe damages to steel buildings highlighting critical features of structural details and components [1-3]; most of the observed damages were related to the initiation and propagation of cracks in correspondence of connections and to local buckling phenomena of dissipative elements. This situation evidenced the inadequacy of the current technical guidelines and, at the same time, the need to provide indications able to guarantee the human safety together with the reduction of the economic losses.

Technical codes were then improved through the introduction of the *Performance Based Design* [4, 5], foreseeing the reaching of specific seismic performances in relation to different hazard levels, described as a function of the seismic action return period (T_R). In this context, among steel structural typologies, Eccentrically Braced Frames (EBF) systems, joining together the ductility of Moment Resisting Frames (MRF) and the stiffness of Concentrically Braced Frames (CBF) [6-9] and whose dissipative performance are fully devoted to the hysteretic behaviour of 'link' elements, became then the most promising structural solution for earthquake prone areas.

The possibility to optimize the design of EBF structures fully exploiting link performance, is related to the correct representation of their actual behaviour towards horizontal/seismic events; this, otherwise, evidences the need to perform reliable nonlinear analyses through the exact modelling of the dissipative elements (links) including, in particular, their flexural and shear behaviour. Nonlinear modelling approaches, spreading from simple concentrated plasticity models [10] to distributed fibre elements and, finally, to complex continuum models [11], shall be properly defined depending on the purpose of the analysis, being their accuracy strongly dependent on the computational burden.

While the dissipative behaviour of long/bending link, whose plastic deformation is governed by flexural stresses, can be represented adopting the common reliable constitutive models used for steel beams [12], in the case of short/shear links the same cannot be stated and further investigations are needed.

In the present paper, a new semi-analytical model able to take into consideration the two different contributions of hardening phenomena, both kinematic and isotropic – strongly influencing the cyclic/seismic behaviour, is presented. The proposed formulation allows to reduce the computational effort related, for example, to solid models and can be simply applied to multi-storey buildings, being more accurate respect to simplified models commonly adopted in the current scientific literature [13-15]. The calibration of the proposed model, based on the results of experimental tests on real-scale prototypes, is also presented.

The work has been developed in the framework of the European research project *'MATCH: material choice for seismic resistant structures'*, funded by the Research Fund for Coal and Steel and aiming at defining the influence of material variability on the structural performance of steel structures.

2 METHODOLOGY

The methodology adopted for the achievement of the above mentioned objective can be organized in the following steps.

Step 1. *Elaboration of a semi-analytical model*. A semi-analytical model has been elaborated to represent the monotonic and cyclic behaviour of link elements and to include the two contribution of kinematic and isotropic hardening.

Step 2. *Execution of the experimental tests*. Experimental tests on real scale EBF systems have been performed to characterize the cyclic/seismic behaviour of links till the collapse achievement.

Step 3. *Calibration of the semi analytical model*. The semi-analytical model is calibrated basing on the experimental tests' results on real scale specimens.

3 ELABORATION OF SEMI-ANALYTICAL MODEL FOR LINK ELEMENTS

The proposed semi-analytical model, simply represented in Figure 1, consists of two nonlinear springs working in parallel, respectively representing the shear/displacement and the bending/rotation behaviour.

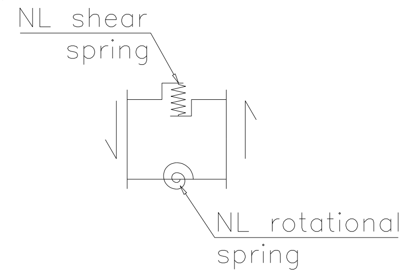


Figure 1: Schematic overview of the two in parallel springs for shear and flexural behaviour.

Since the axial stress is usually negligible in the dissipative links of EBF systems, only the bending behaviour was introduced in the model through a nonlinear spring characterized by kinematic behaviour. The shear behaviour was, otherwise, reproduced through the adoption of an additional spring whose V-γ (shear/distortion) relationship was calibrated basing on the results of experimental tests. Flexural and shear nonlinear springs were then linked to work in parallel.

The model was firstly elaborated using OpenSees software and the available relationships. Two different constitutive laws were adopted for the preliminary model of the V-γ relation of the link: the linear (LKH) and nonlinear (NLKH) kinematic hardening relationships; the last one, accounting for the nonlinear kinematic hardening, based on the Menegotto-Pinto model [16], and for the isotropic hardening, was directly represented using the *Steel4* material model.

The definition of the LKH law requires only 3 parameters: the initial stiffness, the yielding force and the tangent modulus of the plastic range. The first two parameters are directly determined by means of analytical considerations, while the third parameter is tuned considering two different conditions, corresponding to what foreseen according to Eurocode 8 (i.e. ultimate strength $V_u=1.5V_y$ reached in correspondence of an angular distortion equal to 80 mrad - LKH1) or, in alternative, considering the real ultimate strength and maximum shear deformation, estimated, for example, on the base of experimental tests (LKH2).

Besides the three aforementioned parameters, the definition of the NLKH law requires three additional parameters for the kinematic hardening, five parameters for the isotropic hardening, and information about the ultimate strength limit at which the isotropic behaviour is saturated (that is when only the kinematic hardening with yield plateau continues). Such parameters need to be calibrated on the basis of the results of the experimental tests.

3.1 Design of EBF prototypes

Experimental tests have been executed on real-scale one storey/one bay EBF frames with horizontal and vertical links. The prototypes are characterized by a span length equal to 5.0 and a storey height up to 3.0 m and by the adoption of short shear links. Steel grade S355 was used for all the elements. The design of the frames was executed in compliance with actual seismic design regulations [17, 18], optimizing as much as possible dissipative elements' sections and pursuing easiness in joints and connections.

The EBF systems have been designed considering a 'limit' angular rotation of links equal to 110 mrad: this value, in agreement with FEMA 356, is higher than what actually foreseen by Eurocode 8 (i.e. 80 mrad) but reliable in relation to what observed during experimental tests revealing a higher rotational capacity and, besides, higher over-strength factors ([19]-[20]-[21]). A safety/confidence factor equal to 1.30 has been also applied during the design. Basing on above mentioned assumptions and considerations, a HEA100 profile with length equal to 300 mm for the horizontal link and a HEB120 profile of length equal to 150 mm for the vertical link have been selected. Table 1 summarizes the maximum expected horizontal forces ($F_{h,max}$) and displacements ($d_{max,110mrad}$) for links in horizontal and vertical configurations.

Link profile	e [mm]	F_y [kN]	$F_{h,max}$ [kN]	d_y[mm]	$d_{max(110mrad)}$ [mm]
HEA100 (hor.)	300	90.1	162.3	0.8	24
HEB120 (vert.	150	145.2	217.8	0.4	19.5

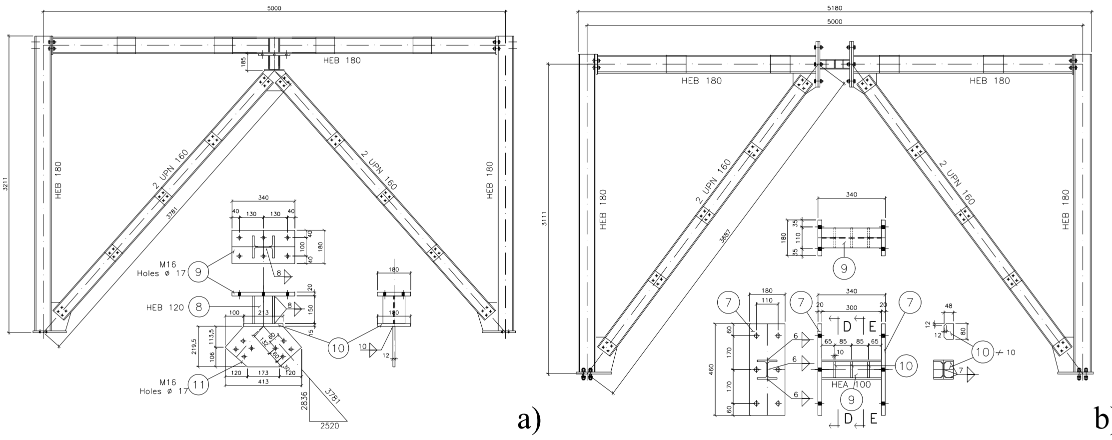
Table 1: Summarizing characteristics and maximum expected forces and displacements for links.

The non-dissipative elements (i.e. beams, columns and braces) were then designed to remain elastic until the links reached the ultimate shear $V_{u,link}$. To this aim, according to EN 1998-1:2005, being $F_{h,y}$ the horizontal force needed to yield the link, Equation (1) can be adopted:

$$F_{h,y} \cdot 1.1 \cdot \gamma_{ov} \cdot \Omega \tag{1}$$

being $\Omega = 1.5 \cdot V_{y,link} / V_{Ed}$ the design over-strength factor of the dissipative element and γ_{ov} the material overstrength factor, equal to 1.25.

Since during the experimental test is expected the complete plastic deformation of the link, the ratio $V_{y,link} / V_{Ed}$ was assumed unitary in the design; to account for eventual higher over-strength ratio of links, the Ω factor was assumed equal to 2.0. Figure 2 shows the final configuration of the EBFs systems with vertical and horizontal links. For both the configurations, columns and beams were realized using HEB180 sections, while for braces 2UPN160, coupled with 3 bolted connections equally spaced, were employed.



a) b)

Figure 2: EBFs with a) vertical links and b) horizontal links.

3.2 Experimental test set – up and organization

Real-scale prototypes were realized according to what described in the previous paragraph. In order to avoid possible out-of-plane and buckling phenomena (i.e. global instability of the tested frames, buckling of the top beam), additional components were necessary to safely connect the EBFs systems to the bidirectional concrete slab of the Laboratory of Pisa University. To this aim, four additional structures/substructures were opportunely designed and adopted, being:

(i). A rigid basement connecting the testing prototypes to the slab of the Laboratory (Figure 3).

(ii). A stabilizing frame avoiding the warping of the columns (Figure 3).

(iii). A loading transfer frame (Figure 3). This last, in particular, connects the EBF system to the hydraulic jackets and, at the same time, restrains both the top beam and the whole frame from buckling phenomena.

(iv). A contrast frame, to which the actuator are fixed to (Figure 3).

The load was applied at the height of the top beam (~3000 mm) with two hydraulic jackets with maximum stroke equal to ± 150 mm and maximum load capacity equal to ± 200 kN each one, able to apply a cyclic loading history. For both the two configurations - with vertical and horizontal link - the applied load was recorded using two load cells placed between the hydraulic jackets and the loading transfer frame. The sensors were placed in order to monitor the shear force-displacement behaviour of the links (or, that is the same, the shear - angular deformation one). LVDTs were organized to account for the possible slip due to hole-to-bolt clearances. With reference to Figure 4 and Figure 5 respectively for the vertical and horizontal links, the effective displacement of the link due to shear deformation, depurated from all the slip to which the total displacement of the hydraulic actuators can be affected by, was evaluated monitoring directly the transversal deformation and the displacement of the diagonals.

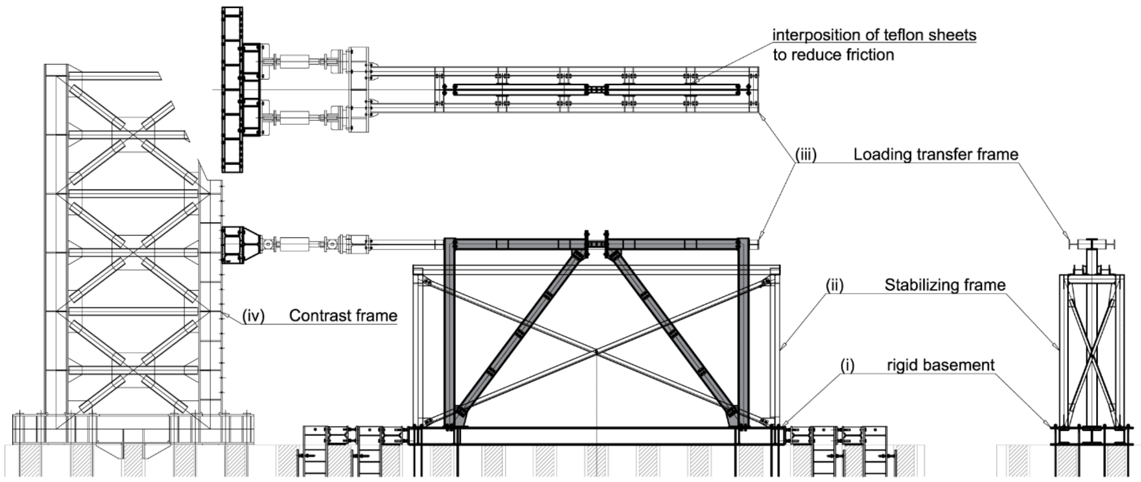


Figure 3: General set up of the test.

Figure 4: LVDTs on the vertical link EBF

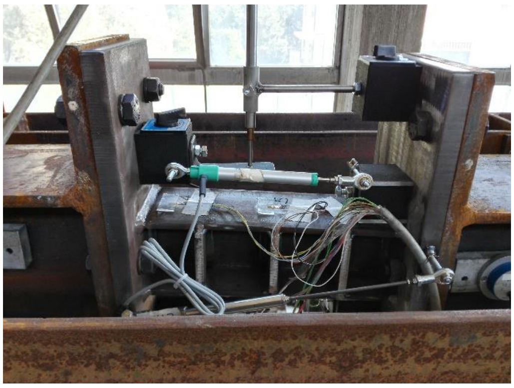


Figure 5: LVDTs on the horizontal link EBF

The cyclic tests have been performed following the ECCS45 protocol [19]; the value of e_y (i.e. displacement corresponding to the yielding) was selected to achieve an angular distortion equal to 130 mrad within 16 steps of 3 cycles each, without considering the first elastic part and allowing to reduce the duration of the whole test. After the detection of the crack initiation in the specimen, the test was prosecuted without increasing the displacement amplitude until the total loss of shear capacity.

3.3 Experimental tests' results

Experimental tests evidenced a stable hysteretic behaviour of the EBF frames during all the steps of the cyclic loading application (Figure 6). The cyclic behaviour was characterized by a strong isotropic and nonlinear kinematic hardening, this last one presenting, in the plastic branch, a variable tangent modulus decreasing with the increase of the plastic strain. The isotropic hardening showed a variable yielding surface that can increase from one cycle to the next one. All the specimens experienced a crack within the web, that propagated parallel to the flanges, as represented in Figure 7 and Figure 8, respectively in the case of vertical and horizontal links. A rather steep loss of shear capacity was observed in the case of specimens tested using the ECCS protocol.

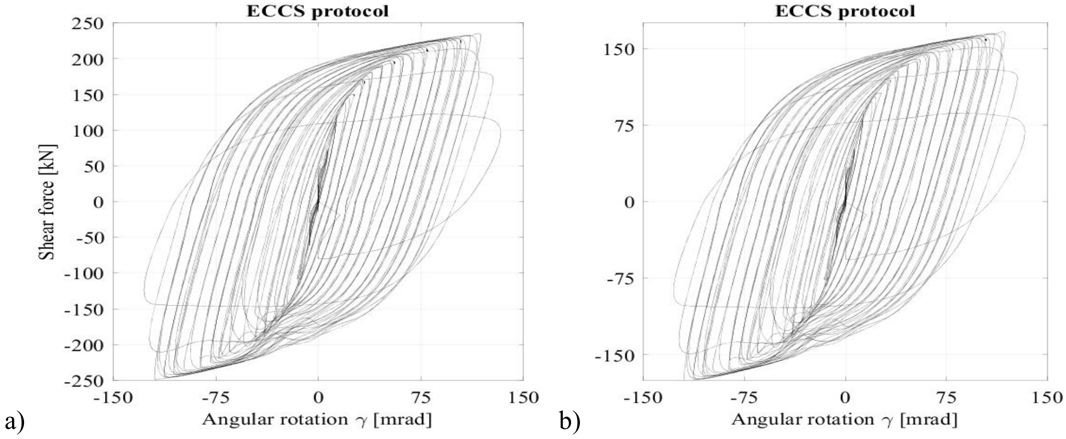


Figure 6: Force/angular distortion diagrams (F-γ) for a) the vertical link and b) the horizontal link.

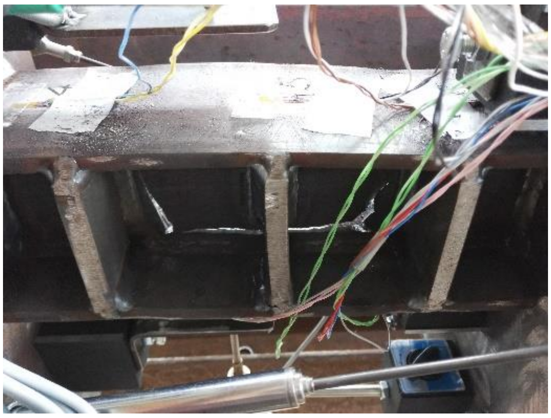


Figure 7: Collapse mechanism of the horizontal link.

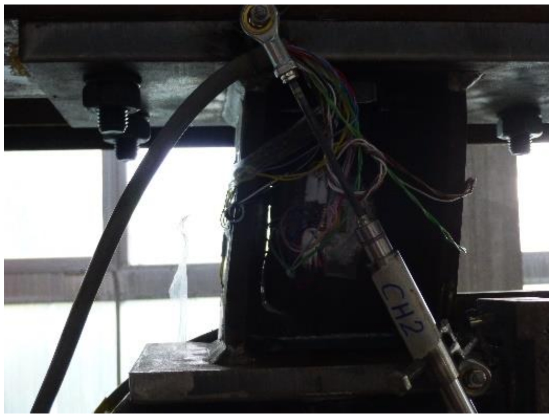


Figure 8: Collapse mechanism of the vertical link.

In the case of EBF prototypes with horizontal links, the presence of equally spaced stiffeners within the web hindered the propagation of the crack, slowing the loss of shear capacity. All the specimens reached an average angular rotation γ close to 125 mrad (**Errore. L'origine riferimento non è stata trovata.**), higher than the imposed collapse limitation defined by EN1998-1:2005 and FEMA 356, respectively equal to 80 and 110 mrad.

What above presented highlighted the general oversizing of dissipative structural elements of EBFs designed according to the capacity design approach imposed by EN1998-1:2005 [18], that is, otherwise, more relevant in the case of frames equipped with horizontal links.

Vertical link	Test 1 ECCS	Test 2 ECCS
Angular rotation at collapse	125	119
Maximum applied force	244.03	248.73
Horizontal link	Test 1 ECCS	Test 2 ECCS
Angular rotation at collapse	128	123
Maximum applied force	172.62	175.95

Table 2: Results of the experimental tests.

4 CALIBRATION OF THE PROPOSED MODEL

In order to calibrate the proposed model, the experimental tests were modelled in OpenSees [22, 23]. To assess the limitations of each model, a two level comparison has been performed, including a first *'graphical'* assessment of the results in terms of cyclic behaviour (i.e. shape of the cycle and degradation of the hysteretic capacity) and, secondly, evaluating the differences in terms of cumulated energy (global and per cycle). The erroneous characterization of the hysteretic energy of the EBF system might strongly affect the assessment of its seismic capacity. The above mentioned comparison procedure was performed considering the two LKH and NLKH constitutive laws. A graphical interpretation of the results is provided in

Figure **9** and

Figure **10**.

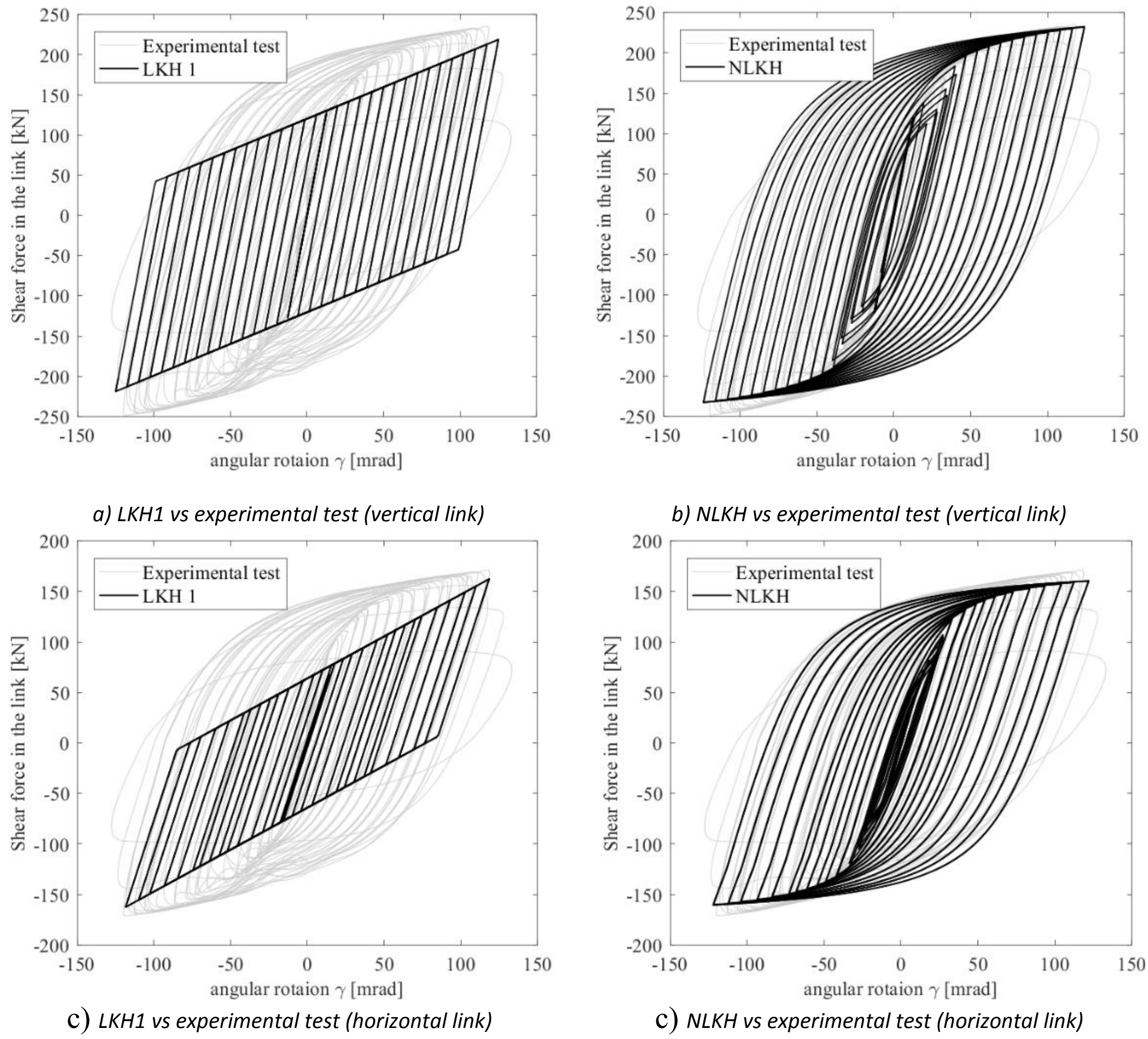
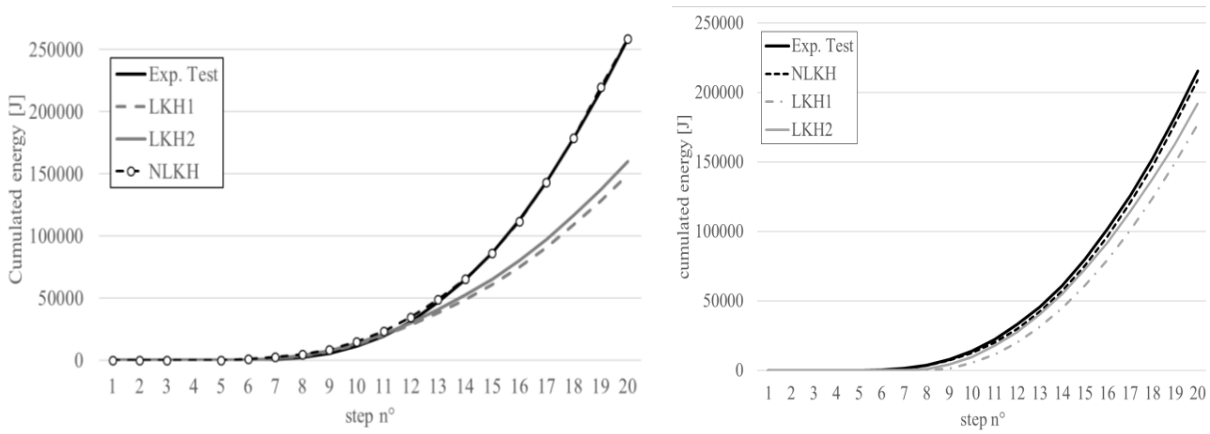


Figure 9: Experimental vs numerical cyclic behaviour (ECCS protocol).



a) horizontal link *b) vertical link*

Figure 10: Comparison in terms of cumulated energy (ECCS protocol)

The LKH models underestimates the plastic branch. Although the LKH2 is calibrated on the basis of the experimental results, the underestimation of the plastic branch strongly reflects on the reduction of the hysteretic energy. For the vertical link, considering the ECCS protocol, the difference in terms of hysteretic energy, evaluated at the collapse, is -22% for the LKH1 and -10% for the LKH2 (note that the negative value means an underestimation of the cumulated energy). Regarding the horizontal link, the difference increases to -42% for the LKH1 and -38% for the LKH2. It should be noted that such difference is reduced to half, if the hysteretic energy is computed at 80mrad of angular deformation. The NLKH model, on the contrary, is able to correctly represent the cyclic behaviour and also to correctly estimate the absorbed hysteretic energy.

5 CONCLUSIONS

A modelling approach for short/shear links, based on the adoption of two springs working in parallel, respectively representing the shear/displacement and the bending/rotation behaviour, has been proposed. The model has the capability to reproduce the cyclic/seismic behaviour of the dissipative element of EBF structures with a reduced computational burden respect to what actually proposed in the current scientific literature and is able to simulate the two contribution of hardening, i.e. isotropic and kinematic.

To calibrate the model and to validate the assumptions used for the semi-analytical formulation, a set of experimental tests have been carried out on real-scale one storey/one bay EBF system, equipped with vertical and horizontal links. Several studies are actually ongoing aiming to effectively prove the impact of the adoption of the proposed model on multi-storey EBF structures.

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