ingegneria sismica
International Journal of Earthquake Engineering
Anno XXXIII – Speciale CTA 2015 – Num. 3

# IMPORTANCE OF LINK MODELS IN THE ASSESSMENT OF THE SEISMIC RESPONSE OF MULTI-STOREY EBFS DESIGNED BY EC8

*Melina Bosco, Aurelio Ghersi, Edoardo M. Marino, Pier Paolo Rossi**

Department of Civil Engineering and Architecture, University of Catania, Italy

SUMMARY: *The nonlinear seismic response of eccentrically braced frames is strongly correlated to the cyclic response of links, i.e. the dissipative members of this structural type. In this paper, a simple but refined link model recently proposed by the authors is used to assess the effectiveness of some common and simpler models in which the effect of the isotropic hardening is not taken into account explicitly. The comparison is carried out by comparing the dynamic response of 12 multi-storey eccentrically braced frames designed as per Eurocode 8. The structures are characterised by different number of storeys and link lengths. The investigation considers both global and local response parameters. The results show which model gives the best estimate of the ultimate response of the frames. It is also shown that the major differences between the responses predicted by the models are recorded for systems with long links.*

KEYWORDS: *modelling, isotropic hardening, kinematic hardening, eccentrically braced frames, non-linear dynamic analyses*

## 1. INTRODUCTION

Several models have been used in the past to replicate the cyclic behaviour of links. When the nonlinear response of eccentrically braced structures is determined, the adopted link models are generally simple and usually based on the Euler-Bernoulli beam with concentrated flexural plastic hinges [Roeder and Popov, 1977; Ricles and Popov, 1987; Ramadan and Ghobarah, 1995; Richards and Uang, 2003]. Ending springs with an elastic plastic with kinematic hardening or an elastic perfectly plastic behaviour are often considered to simulate the inelastic shear and flexural response of the link [Rossi and Lombardo, 2005; Bosco and Rossi, 2009; 2013b; Badalassi *et al*, 2013; Montuori et al., 2014a-b; 2015a-b]. On the other hand, finite element models are also developed to predict the response of links in more detail and generalize the results of the laboratory tests to ranges of geometric and mechanical properties not fully investigated yet. [Ghobarah and Ramadan, 1991; Itani *et al*., 2003; Richards and Uang, 2005; Chao et al., 2005; Berman and Bruneau, 2008; Prinz and Richards, 2009; Daneshmand and Hashemi, 2012; Della Corte *et al*., 2013]. However, because of the computation burden and complexity of these models, they are not suitable for inelastic dynamic analyses of structures with eccentric bracings. Recently, the authors have proposed a simple but refined link model, which consists of an elastic beam with concentrated flexural and shear plastic hinges [Bosco *et al*., 2015a]. The uniaxial material model proposed by Zona and Dall'Asta [2012] dictates the response of the hinges.

*Corresponding author: Pier Paolo Rossi, Department of Civil Engineering and Architecture, University of Catania, Italy.
Email: prossi@dica-unict.it

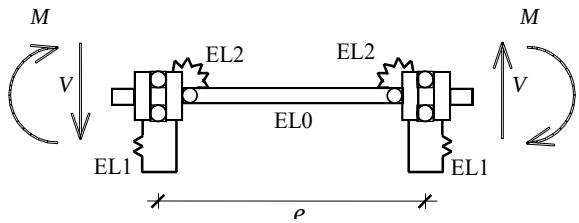


Figure. 1. *Elements of the link model*

This uniaxial material was proposed as an improvement over the elastic-plastic models with only kinematic hardening used in studies on buckling restrained braces [Kim and Choi, 2004; Bosco and Marino, 2013]. The proposed link model takes into account both kinematic and isotropic hardening [Formisano *et al*., 2006; De Matteis *et al*., 2012; Brando and De Matteis, 2014; Rossi, 2015] but neglects the effect of the axial force that may develop in the link [D'Aniello *et al.*, 2006; Della Corte *et al.*, 2007; Mazzolani *et al.*, 2009]. Thus, this model is proper to replicate the cyclic response of links arranged in the split K configuration. In a previous investigation [Bosco *et al*., 2015a], the proposed model and other simple existing models have been adopted to replicate the cyclic response of short, intermediate and long links for which laboratory test data are available [Hjelmstad and Popov, 1983; Malley and Popov, 1983; Engelhardt and Popov, 1989; Arce, 2002; Galvez, 2004; Okazaki *et al*., 2005; 2006; 2007; 2009; Drolias 2007]. The obtained results have proved the effectiveness of the proposed model. Based on these results, the new model is considered here to assess the effectiveness of some common and simpler models (in which the effect of the isotropic hardening is not taken into account explicitly) in predicting the dynamic response of multi-storey structures subjected to artificial accelerograms. The structures are characterised by different number of storeys and link lengths and are designed as per Eurocode 8 [2003]. The investigation considers both global and local response parameters, namely ultimate peak ground acceleration, plastic rotations of links and residual drifts.

## 2. DESCRIPTION OF THE MODELS

Three different models are compared in this paper. All of them consist of elements connected in series (Fig. 1). The central element (EL0) has the same length and moment of inertia of the link and simulates the flexural elastic behaviour of the link (the shear stiffness of this element is infinite). The two elements at each end of the link (EL1 and EL2) are zero length and connect the beam segments outside the link to the central element EL0. The first of these two elements (EL1) simulates the elastic and inelastic shear behaviour of half a link, while the second (EL2) simulates the inelastic flexural behaviour of the ending part of the link (a very high value of the elastic stiffness is assigned to this element). The nodes of EL1 may have only relative vertical displacements while those of EL2 only relative rotations. The models differ because of the inelastic response of the elements.

### *2.1 Model 1: isotropic and kinematic hardening*

The first model (M1) has been proposed by Bosco *et al.* [2015a] and considers isotropic hardening explicitly. The response of the two elements EL1 and EL2 is defined by means of the uniaxial material model proposed by Zona and Dall'Asta [2012] for buckling restrained braces. This uniaxial material model considers a simple rheological scheme where a spring "0"

is connected in series with a friction slider in parallel with a spring "1". The stiffness $k_0$ of the spring "0" is equal to the initial elastic stiffness of the element while the stiffness $k_1$ of the spring "1" influences the kinematic hardening. The hysteretic response is described by the nonlinear relationship between force and deformation in the friction element. The full description of this model requires that values be given to the stiffness $k_0$ and $k_1$, to the initial yield force $F_{y0}$ and to the maximum yield force for the fully saturated isotropic hardening condition $F_{ymax}$. In addition, the values of two positive non-dimensional constants have to be specified. The first constant β controls the rate of the isotropic hardening; the second, α, controls the trend of the transition from the elastic to the plastic response.
The elastic stiffness $k_{L1,0}$ of element EL1 is calculated by means of the equation:

$$k_{L1,0} = \frac{GA}{\chi\, e/2} \tag{1}$$

where $A$ is the area of the link cross-section, $G$ is the tangent modulus of elasticity, $e$ is the link length and χ is the shear coefficient. This latter parameter is calculated [Bosco *et al.*, 2015a] as:

$$\chi = \frac{b\left(d-t_{\mathrm{f}}\right)^5}{240A\cdot i^4}\left[15\frac{\eta^2}{\xi}+10\eta+5\eta\frac{b^2}{\left(d-t_{\mathrm{f}}\right)^2}+2\xi\right] \tag{2}$$

where $i$ is the radius of gyration of the cross-section, $\xi = t_{\mathrm{w}}/b$ and $\eta = 2\,t_{\mathrm{f}}/\left(d-t_{\mathrm{f}}\right)$, $b$ is the width of the flange, $d$ is the depth of the section, $t_{\mathrm{f}}$ is the thickness of the flange, $t_{\mathrm{w}}$ is the thickness of the web. The initial yield force $V_{\mathrm{y}}$ of element EL1 is calculated taking into account only the contribution of the web, i.e.

$$V_{\mathrm{y}} = V_{\mathrm{p}} = 0.6\ f_{\mathrm{yw}}\ t_{\mathrm{w}}\left(d-2t_{\mathrm{f}}\right) \tag{3}$$

where $f_{yw}$ is the tensile yield strength. Based on several numerical tests on links, the following values of the parameters above have been suggested [Bosco *et al.*, 2015a] to simulate the cyclic response of links in which stiffeners are disposed as required by modern seismic codes

$$k_{\mathrm{L1,1}} = 0.442\,\%\ k_{\mathrm{L1,0}} \qquad V_{y,\max} = 1.308\ V_y \tag{4a}$$

$$\alpha_{\mathrm{L1}} = 0.485 \qquad \beta_{\mathrm{L1}} = 0.113 \tag{4b}$$

The ultimate shear force of short links (i.e. the shear force corresponding to a plastic rotation angle equal to 0.08 rad) corresponding to the fully saturated isotropic hardening is:

$$V_{\mathrm{u}} = 1.308\cdot V_{\mathrm{p}} + \frac{0.08\cdot e/2\cdot k_{\mathrm{L1,0}}}{k_{\mathrm{L1,0}}/k_{\mathrm{L1,1}}-1} \tag{5}$$

Element EL2 is characterised by a very high elastic stiffness $k_{L2,0}$ because the elastic flexural deformability of the link is simulated by element EL0. The bending moment at yield $M_{\mathrm{y}}$ is assumed equal to the plastic moment $M_{\mathrm{p}}$ of the entire cross-section:

$$M_{\mathrm{p}} = f_{\mathrm{yf}}\, b\, t_{\mathrm{f}}\left(d-t_{\mathrm{f}}\right) + f_{\mathrm{yw}}\frac{t_{\mathrm{w}}}{4}\left(d-2t_{\mathrm{f}}\right)^2 \tag{6}$$

where $f_{yf}$ is the tensile yield strength of the flanges. The suggested values of the post-elastic stiffness $k_{L2,1}$, that of the fully saturated bending moment $M_{y,max}$, and those of the parameters $\alpha_{L2}$ and $\beta_{L2}$ are:

$$k_{\mathrm{L2,1}} = 0.795\,\%\,\frac{6EI}{e} \qquad M_{y,\max} = 1.212\ M_y \tag{7a}$$

$$\alpha_{L2} = 0.025 \qquad \beta_{L2} = 0.054 \tag{7b}$$

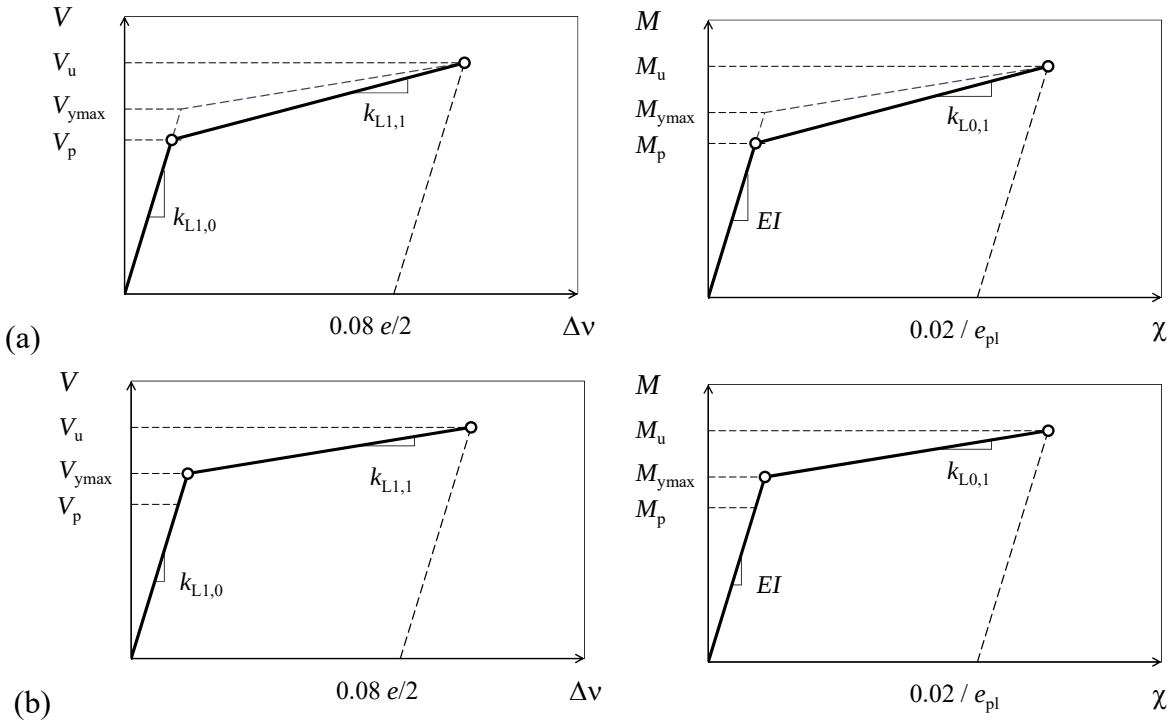


Fig. 2. *Response of element EL1 and plastic hinge of element EL0 according to model: (a) M2, (b) M3*

Owing to this, the ultimate bending moment of long links (i.e. the bending moment corresponding to a plastic rotation angle equal to 0.02 rad) is:

$$M_u = 1.212 \cdot M_p + 0.02 \cdot k_{L2,1} \tag{8}$$

### *2.2. Models 2 and 3: kinematic hardening*

In the second model (M2) and in the third model (M3) the behaviour of elements EL1 is elastic-plastic with kinematic hardening (Fig. 2a, 2b). The effect of the isotropic hardening is not considered explicitly in these models. In particular, in model M2 an equivalent kinematic strain hardening is used to include the effects of both isotropic and kinematic hardening while in model M3 the effect of the isotropic hardening is considered assuming that the yield shear force and the yield bending moments are higher than the plastic values provided by Eqs. (3) and (6). Specifically, in model M2, the elastic stiffness and the plastic shear force of element EL1 are equal to those assigned to model M1. The post elastic stiffness is such that the shear force corresponding to a plastic rotation angle equal to 0.08 rad is equal to that provided by model M1 at the same plastic rotation angle, i.e.

$$k_{L1,1} = \frac{V_u - V_p}{0.08 \cdot e/2 \cdot k_{L1,0} + \left(V_u - V_p\right)} \cdot k_{L1,0} \tag{9}$$

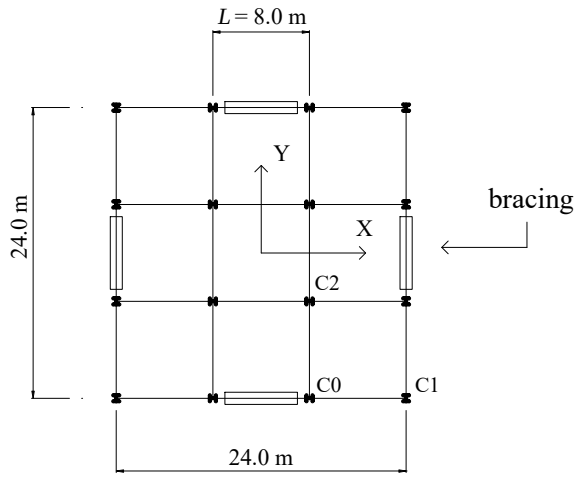


Figure 3. *Plan of the building*

Elements EL2 are not included in the model because element EL0 is a beam with hinge element. The plastic hinges of element EL0 are characterised by a length $e_{pl}$ equal to 1/100 $e$ and by an elastic-plastic with kinematic hardening moment-curvature relationship. This moment-curvature relationship is characterised by a plastic bending moment equal to $M_p$, an elastic flexural stiffness equal to $EI$ and a post elastic stiffness equal to:

$$k_{L0,1} = \frac{M_u - M_p}{0.02 \cdot EI/e_{pl} + \left(M_u - M_p\right)} \cdot EI \tag{10}$$

The flexural stiffness in the equation above is such that the bending moment corresponding to a plastic rotation angle equal to 0.02 rad is equal to that provided by model M1 at the same plastic rotation angle. Instead, in model M3, the yield shear force and the yield bending moments are equal to the values corresponding to the fully saturated isotropic hardening condition, i.e. $V_y = V_{y,max}$ and $M_y = M_{y,max}$. In addition, the post yield stiffness $k_{L1,1}$ of element EL1 is equal to the corresponding stiffness of model M1 while the post yield stiffness of the plastic hinge of element EL0 is:

$$k_{L0,1} = \frac{M_u - M_{y,max}}{0.02 \cdot EI/e_{pl} + \left(M_u - M_p\right)} \cdot EI \tag{11}$$

## 3. ANALYSED MULTI-STOREY STRUCTURES

The eccentrically braced frames investigated in this paper were designed in previous research [Bosco *et al.*, 2014, 2015b] and constitute simplified models of the structure of an apartment building having squared plan (24m x 24m) and geometric and mass properties equal at all floors (Fig. 3). The structure of this building is defined by the intersection of two groups of four three-span plane frames oriented along two orthogonal directions and located symmetrically to the geometric centre. Frames on the perimeter of the building are designed to resist the entire horizontal force and are endowed with eccentric braces disposed in the central span according to the split K-braced configuration. The interstorey height is equal to 3.3 m. To nullify the interaction between deck and links, two beam members are introduced at each level of the eccentrically braced frame instead of the traditional single section. Systems with

different dynamic and mechanical properties are obtained by varying the number of storeys $n_s$ from 4 to 12 (in step of 4) and the link length $e$. This length is equal to either 0.1 to 0.3 times the length $L$ of the braced span. Vertical dead and live loads are constant on every floor level and are defined by characteristic values ($G_k$ and $Q_k$) equal to 4.4 and 2.0 kN/m², respectively. The structures stand on soft soil (soil C according to Eurocode 8) and are designed assuming a peak ground acceleration equal to 0.35 g and a behaviour factor equal to 5. The design internal forces on members are determined by either the modal response spectrum analysis (MRSA) or the lateral force method of analysis (LFMA). The non-dissipative members are designed according to the capacity design principles as per Eurocode 8. All the members that do not belong to the braced frames are designed to sustain gravity load only. In the following, the frames are identified by a label obtained by adding the number of storeys (04, 08 or 12), the link length (10 or 30 for systems with $e/L$ equal to 0.10 or 0.30, respectively) and the design method of analysis (M for MRSA or S for LFMA).

## 4. Numerical Analyses and Response Parameters

In this section, the response of the multi-storey structures is determined by incremental nonlinear dynamic analysis. The numerical analyses are performed by means of the program OpenSEES [Mazzoni *et al.*, 2003]. The numerical model includes both the braced frames and the gravity columns, which are pinned at the base. To promote a more uniform distribution of drift demand along the height of the building [Tremblay, 2003; Brandonisio *et al.*, 2006] the columns are continuous over the entire height of the building. Links are modelled by each of the considered models; beam segments outside links, braces and columns are non-dissipative members and for this reason they are modelled by means of elastic elements. As a consequence, the differences between the responses of the corresponding frames are only due to the considered model of the link. Beam-to-column connections are assumed perfectly pinned; the numerical model neglects the effect of the panel zone deformation because yielding and buckling of the panel zone are assumed to be prevented by a proper choice of its geometrical slenderness [Brandonisio *et al.*, 2011, 2012]. The connections at the end of braces are welded. The Rayleigh formulation is used to introduce damping. Mass and stiffness proportional damping coefficients are defined so that the first and the third modes of vibration of the structure are characterised by an equivalent viscous damping factor equal to 0.05. In accordance with Ricles and Popov [1987], no stiffness proportional damping is considered for the zero length elements of the link. The seismic input consists of 10 artificially generated accelerograms [Amara *et al.*, 2014]. $P$-$\Delta$ effects are included in the numerical analyses. The peak ground acceleration is scaled in steps of 0.04g. For each considered value of peak ground acceleration, seismic demands of dissipative and non-dissipative elements are verified not exceed the corresponding capacities in terms of deformations and strengths, respectively. Thus, the peak ground acceleration is scaled up to the value $a_{gu}$ that leads to the first achievement of the ultimate rotation capacity $\varphi_u$ of links or to yielding or buckling of non-dissipative members. Specifically, the plastic rotation capacity $\varphi_u$ of links is calculated according to Eurocode 8 as:

$$\varphi_u = \begin{cases} 0.08\,rad & if \quad \dfrac{eV_p}{M_p} \leq 1.6 \\ 0.02\,rad & if \quad \dfrac{eV_p}{M_p} \geq 3.0 \\ linear\ intepolation & if \quad 1.6 < \dfrac{eV_p}{M_p} < 3.0 \end{cases} \tag{12}$$

The safety level of non-dissipative members is calculated with reference to both yielding ($SL_y$) and instability ($SL_b$). In particular, at each time $t$ of the time history, the bending moments $M(t)$ and axial forces $N(t)$ are calculated. Then, $SL_y$ is calculated as the maximum ratio of the bending moment to the flexural strength $M_{N,Rd}$ reduced because of the axial force while $SL_b$ is determined as the maximum ratio of the axial force to the buckling resistance $N_{b,Rd(M)}$ reduced because of the bending moment. Yielding and buckling resistances for combined values of axial force and bending moment of non-dissipative members are calculated according to Eurocode 3 [2005] assuming that the partial safety factors $\gamma_{M0}$ and $\gamma_{M1}$ are equal to 1.
For each accelerogram scaled to $a_{gu}$, the heightwise distribution of the normalised plastic rotation of links and that of the residual drift angle is investigated. Specifically, the normalised plastic rotation of links is calculated as the ratio of the maximum plastic rotation $\varphi$ required at the link of the $i$-th storey to the corresponding rotation capacity $\varphi_u$. The residual drift angles $\Delta_{res}$, i.e. the ratio of the residual drifts to the interstorey height, are calculated according to the procedure described in Bosco *et al.*, [2015c].

## 5. COMPARISON OF THE MODELS

In this section, the values of $a_{gu}$, $\varphi/\varphi_u$ and $\Delta_{res}$ predicted by the considered models are averaged over the number of the considered accelerograms and compared. The average values of $a_{gu}$ obtained for the considered models are compared in Fig. 4. In the figure, different colours are used to represent results obtained by the considered models. The figure shows that the models provide values of $a_{gu}$ very close to each other when applied to predict the response of low-storey systems with short links (0410M, 0410S) or high storey systems designed by LFMA (0810S, 1210S, 0830S, 1230S). The greatest differences between the values of $a_{gu}$ are obtained when considering systems designed by MRSA, especially in the case of long links (0830M, 1230M). Model M2 (dark grey bars) gives the values of $a_{gu}$ closest to those provided by model M1 (black bars), although it significantly underestimates the ultimate peak ground acceleration in the case of systems with long links designed by MRSA. Model M3 (light grey bars) provides the lowest values of $a_{gu}$. In order to investigate deeper on the above-mentioned differences, it is worth noting that in systems with short links the value $a_{gu}$ is always related to the first achievement of the ultimate rotation capacity of links; instead, in systems with long links, the bending moment of the beam segments outside links may reach values higher than the flexural strength ($SL_y$ larger than unity). In particular, for the considered structures with long links, models M2 and M3 always predict the achievement of the yielding resistance of the beam segment outside the link while, according to model M1, the achievement of the ultimate plastic rotation of the link is recorded only for some accelerograms. Thus, the differences on the predicted values of $a_{gu}$ are partially due to the different failure mechanism and are related to the different values of the bending moment that, according to the considered models, are transmitted by the link for assigned values of plastic deformation. Fig. 5 shows the comparison between the normalised plastic rotations of links predicted by the models for the considered 12-storey structures. The solid line is used to represent the results predicted by model M1, whereas different types of hatches are used for the other models. For the structure with short links ($e/L = 0.10$) designed by MRSA (Fig. 5a), the normalised plastic rotation of links is significant at all storeys. However, the values predicted by the models are scattered especially at the lower storeys. As an example, at the third storey, the percentage difference between the normalised plastic rotation provided by models M2 and M3 and that given by model M1 is 30% and -58%, respectively. In the corresponding system designed by LFMA (Fig. 5b), the

normalised plastic rotation is very high only at the upper storey while the plastic behaviour of the links of the other storeys is limited. Indeed, this system is characterised by scattered values of the overstrength and low damage distribution capacity (Bosco and Rossi, 2013a). For this reason, all the models provide a similar distribution of the considered response parameter. In the systems with long links designed by either MRSA or LFMA ($e/L = 0.30$), the plastic rotations predicted by models M2 and M3 are very low because they are obtained for ultimate peak ground accelerations that are significantly lower than those corresponding to the model M1. Even if not shown in any figure, these results are confirmed when considering 4- and 8-storey frames. Fig. 6 shows the comparison between storey drift angles predicted by the three considered models for the same structures analysed in Fig. 5 in terms of normalised plastic rotations of links. When short links are considered, model M2 is that providing the results closest to those of model M1. However, the percentage differences between the predicted residual drift angles are significantly greater than those obtained with reference to the normalised plastic rotation of links. Model M3 provides larger residual drifts, especially in the upper storeys. When long links are considered, models M2 and M3 give residual drifts close to zero. This result is not surprising owing to the low values of the plastic rotations of links corresponding to the ultimate peak ground accelerations predicted by these models.

## 6. CONCLUSIONS

This paper investigates the effectiveness of some simplified models of steel link beams commonly used in the seismic assessment of split K eccentrically braced frames. The benchmark is represented by the seismic response obtained by means of a simple but refined link model (M1 model) that has recently been proposed by the writers and that takes into account both kinematic and isotropic hardening. The comparison is carried out on the seismic response at collapse of 12 eccentrically braced frames designed as per Eurocode 8. The seismic response at collapse of the analysed frames is determined by incremental nonlinear dynamic analysis. The frames differ because of the number of storeys, the geometric link length and the design method of analysis. The seismic response is expressed in terms of the ultimate peak ground acceleration, link normalised plastic rotations and residual storey drift angles.

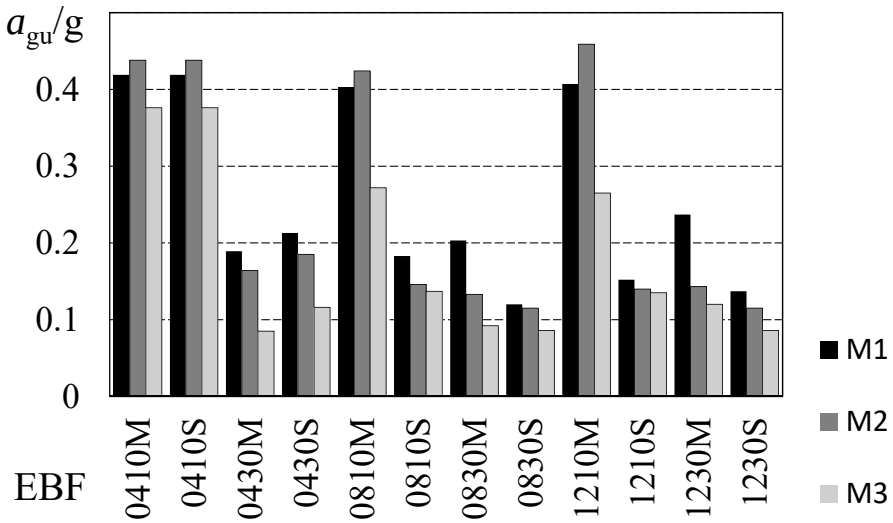


Figure 4. *Comparison between the predicted ultimate peak ground accelerations*

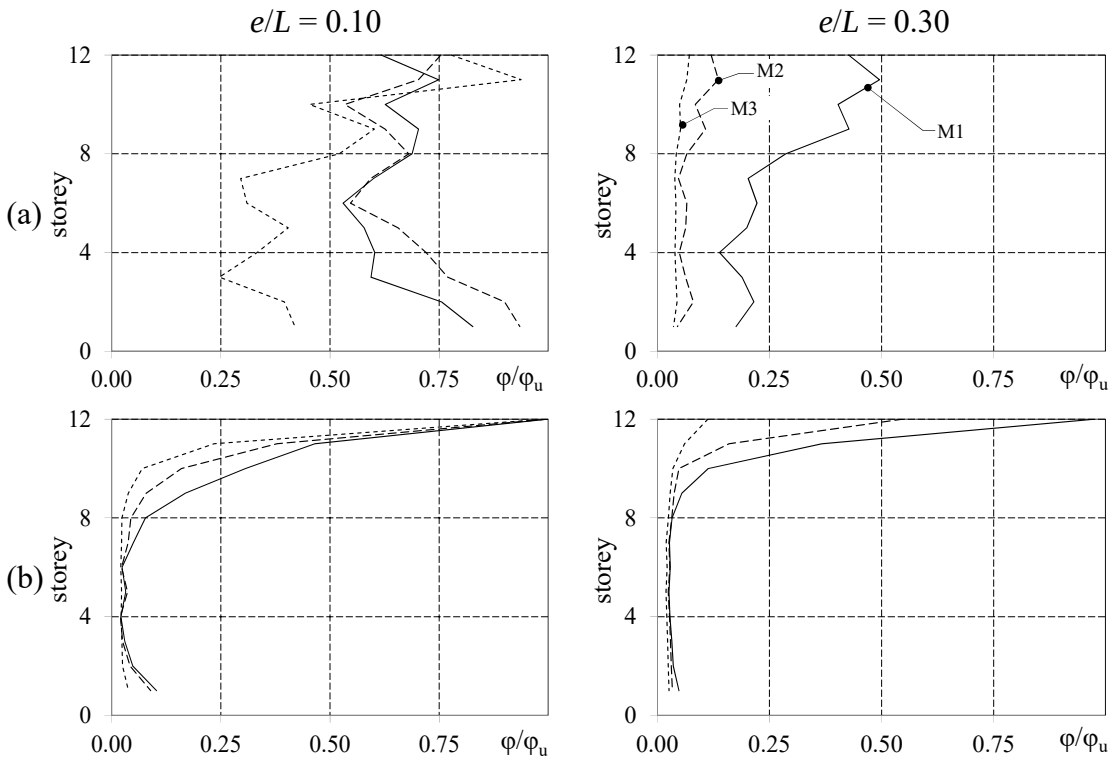


Figure 5. *Comparison between the predicted normalised plastic rotations of links: systems designed by (a) MRSA, (b) LFMA*

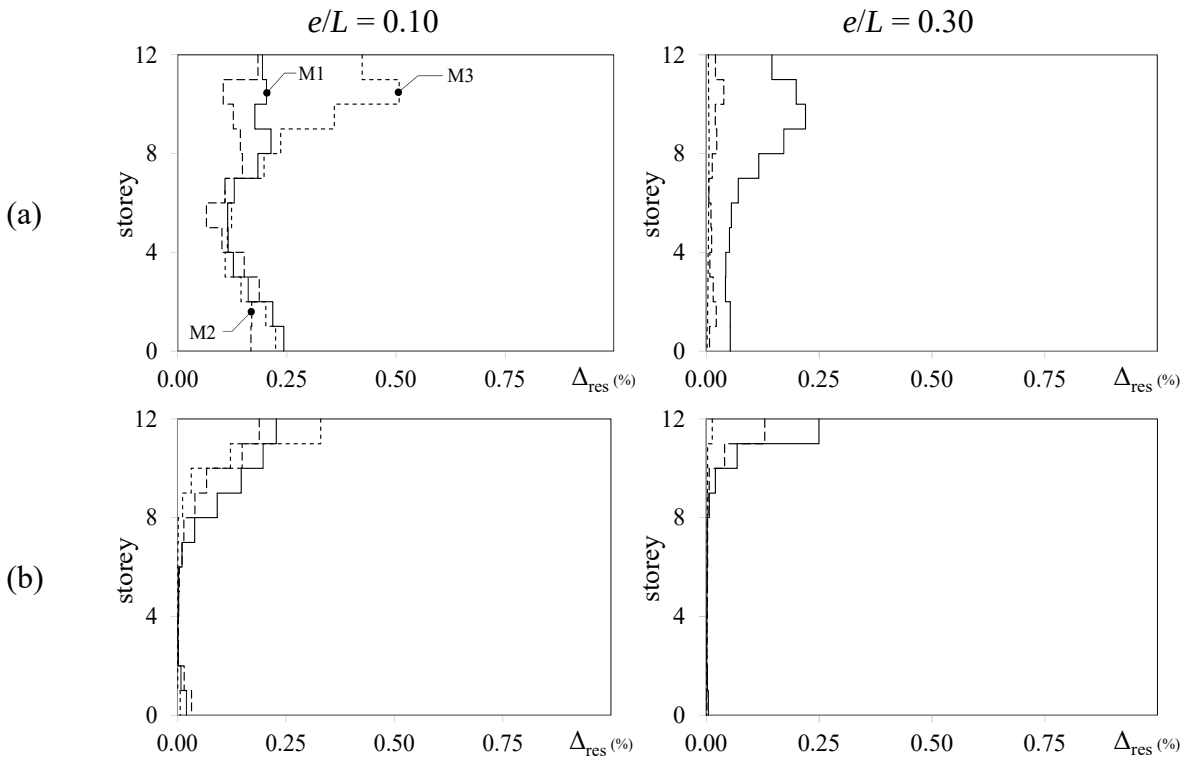


Figure 6. *Comparison between the predicted residual storey drift angles: systems designed by (a) MRSA, (b) LFMA*

The results of incremental nonlinear dynamic analyses show that the simplified model in which the effect of isotropic hardening is represented by an equivalent (increased) kinematic hardening (model M2) gives the values of ultimate peak ground acceleration closest to those provided by the reference model for systems with short links. Instead, the simplified model in which the effect of isotropic hardening is represented by increasing the value of the yielding shear force and bending moments (model M3) underestimates significantly the same response parameter.

The major differences between the responses predicted by the analysed models are recorded for systems with long links. Indeed, in this case, models M2 and M3 predict the yielding of the beam segment outside link while this behaviour is recorded only for some accelerograms when the reference model is adopted. The greatest differences between the models are obtained for residual drift prediction.

In order to generalize the results obtained in this study, the comparison should be extended to eccentrically braced frames designed according to different design procedures. Finally, the variability of the response related to the model should be compared to that related to the seismic input.

## 7. REFERENCES

Amara, F., Bosco, M., Marino, E. M., Rossi, P. P. [2014] "An accurate strength amplification factor for the design of SDOF Systems with P-Δ Effects", *Earthquake Engineering and Structural Dynamics* 43, 589–611.

Arce, G. [2002] *Impact of higher strength steels on local buckling and overstrength oflinks in eccentrically braced frames*, Master's thesis, Dept. of Civil and Environmental Engineering, University of Texas at Austin, Austin, TX.

Badalassi, M., Braconi, A., Caprili, S., Salvatore, W. [2013] "Influence of steel mechanical properties on EBF seismic behaviour", *Bulletin of Earthquake Engineering* 11, 2249–2285

Berman, J. W., and Bruneau, M. [2008] "Tubular links for eccentrically braced frames. I: finite element parametric study", *Journal of Structural Engineering* 134(5), 692-701.

Bosco, M., and Rossi, P. P. [2009] "Seismic behaviour of eccentrically braced frames", *Engineering Structures* 31, 664-674.

Bosco, M., and Rossi, P. P. [2013a] "A design procedure for dual eccentrically braced systems: Analytical formulation", *Journal of Constructional Steel Research* 80, 440-452.

Bosco, M., and Rossi, P. P. [2013b] "A design procedure for dual eccentrically braced systems: Numerical investigation", *Journal of Constructional Steel Research* 80, 453-464.

Bosco, M., and Marino, E. M. [2013] "Design method and behavior factor for steel frames with buckling restrained braces", *Earthquake Engineering and Structural Dynamics* 42(8), 1243-1263.

Bosco, M., Marino, E. M, and Rossi, P. P. [2014] "Proposal of modifications to the design provisions of Eurocode 8 for buildings with split k eccentric braces", *Engineering Structures* 61, 209-223.

Bosco, M., Marino, E. M., Rossi, P. P. [2015a] "Modelling of steel link beams of short, intermediate or long length", *Engineering Structures* 84, 406-418.

Bosco, M., Marino, E. M, and Rossi, P. P. [2015b] "Critical review of the EC8 design provisions for buildings with eccentric braces", *Earthquakes and Structures* 8(6), 1407-1433.

Bosco, M., Marino, E. M., and Rossi, P.P. [2015c] "Design of steel frames equipped with BRBs in the framework of Eurocode 8", *Journal of Constructional Steel Research* 113, 43–57.

Brando, G., and De Matteis, G. [2014] "Design of low strength-high hardening metal multi-stiffened shear plates" *Engineering Structures* 60, 2-10.

Brandonisio, G., De Luca, A., Grande, E., and Mele E. [2006] "Non-linear response of concentric braced franmes", *Proc. of the International Conference STESSA 2006*, Japan, 399-405.

Brandonisio, G., De Luca, A., and Mele E. [2011] "Shear instability of panel zone in beam-to-column connections", *Journal of Constructional Steel Research* 67, 891–903.

Brandonisio, G., De Luca, A., and Mele, E. [2012] "Shear strength of panel zone in beam-to-column connections", *Journal of Constructional Steel Research* 71, 129–142.

Chao, S. H., Khandelwal, and El-Tawil, S. [2006] "Ductile web fracture initiation in steel shear links", *Journal of Structural Engineering* 132(8), 1192-1200.

D'Aniello, M., Della Corte, G., and Mazzolani, F. M. [2006] "Seismic Upgrading of RC Buildings by Steel Eccentric Braces: Experimental Results vs. Numerical Modeling", *Proc. of the International Conference STESSA 2006*, Japan, 809-814.

Daneshmand, A., and Hashemi, B. H. [2012] "Performance of intermediate and long links in eccentrically braced frames", *Journal of Constructional Steel Research* 70, 167-176.

De Matteis, G., Brando, G., and Mazzolani, F. M. [2012] "Pure aluminium: An innovative material for structural applications in seismic engineering", *Construction and Building Materials* 26(1), 677-686.

Della Corte, G., D'Aniello, M., and Mazzolani, F. M. [2007] "Inelastic response of shear links with axial restraints: numerical vs analytical results". *Proc. of the 5th International conference on Advances in Steel Structures*, ICASS 2007, 651-656

Della Corte, G., D'Aniello, M., and Landolfo, R. [2013] "Analytical and numerical study of plastic overstrength of shear links", *Journal of Constructional Steel Research* 82, 19-32.

Drolias, A. [2007] *Experiments on link-to-column connections in steel eccentrically braced frames*, Master's thesis, Department of Civil Engineering, University of Texas, Austin.

Engelhardt, M. D., and Popov, E. P. [1989] *Behavior of long links in eccentrically braced frames*, Report No. UCB/EERC-89/01, Earthquake Engineering Research Center, University of California, Berkeley, 1989.

Eurocode 3. [2005] *Design of steel structures – Part 1–1: General rules and rules for buildings*, ENV 1993–1–1. European Committee for Standardization, Bruxelles.

Eurocode 8. [2003] Design provisions for earthquake resistance of structures. European Committee for Standardisation, ENV 1998-1-1/2/3.

Formisano, A., Mazzolani, F. M., Brando, G., and De Matteis, G. [2006] "Numerical evaluation of the hysteretic performance of pure aluminium shear panels", *Proc. of the International Conference STESSA 2006*, Japan, 211-217.

Galvez, P. [2004] *Investigation of factors affecting web fractures in shear links*, Master's thesis, Department of Civil Engineering, University of Texas, Austin.

Ghobarah, A., and Ramadan, T. [1991] "Seismic analysis of links of various lengths in eccentrically braces frames", *Canadian Journal of Civil Engineering* 18, 140-148.

Hjelmstad, K. D., and Popov, E. P. [1983] *Seismic Behavior of Active Beam Links in Eccentrically Braced Frames*, Report No. UBC/EERC-83/15, Earthquake Engineering Research Center, University of California, Berkeley.

Itani, A. M., Lanaud, C., and Dusicka, P.[2003] "Analytical evaluation of built-up shear links under large deformations", *Computers and Structures* 81, 681-696.

Kim, J., and Choi, H. [2004] "Behavior and design of structures with buckling-restrained braces", *Engineering Structures* 26(6), 693–706.

Malley, J. O., and Popov, E. P. [1983] *Design Considerations for Shear Links in Eccentrically Braced Frames*, Report No. UBC/EERC-83/24, Earthquake Engineering Research Center, University of California, Berkeley.

Mazzolani, F. M., Della Corte, G., D'Aniello, M. [2009] "Experimental analysis of steel dissipative bracing systems for seismic upgrading", *Journal of Civil Engineering and Management* 15(1), 7-19.

Mazzoni, S., McKenna, F., Scott, M. H., Fenves, G. L., and Jeremic, B. [2003] *OpenSEES Command Language Manual*. Pacific Earthquake Engineering Research Center. University of California, Berkeley.

Montuori, R., Nastri, E., and Piluso, V. [2014a] "Theory of plastic mechanism control for eccentrically braced frames with inverted y-scheme", *Journal of Constructional steel research* 92, 122-135.

Montuori, R., Nastri, E., and Piluso, V. [2014b] "Rigid-plastic analysis and moment–shear interaction for hierarchy criteria of inverted Y EB-Frames", *Journal of Constructional steel research* 95, 71-80.

Montuori, R., Nastri, E., Piluso, V., [2015a] "Seismic Design of MRF-EBF Dual Systems with Vertical Links: Ec8 Vs Plastic Design", *Journal of Earthquake Engineering* 19(3), 480-504

Montuori, R., Nastri, E., Piluso, V., [2015b] "Seismic Response of EB-Frames with Inverted Yscheme: TPMC Versus Eurocode Provisions", *Earthquakes and Structures* 8(5), 1191-1214

Okazaki, T., Arce, G., Ryu, H. C., and Engelhardt, M. D. [2005] "Experimental study of local buckling, overstrength and fracture of links in eccentrically braced frames", *Journal of Structural Engineering* 131(10), 1526-1535.

Okazaki, T., Engelhardt, M. D., Nakashima, M., and Suita, K. [2006] "Experimental performance of link-to-column connections in eccentrically braced frames", *Journal of Structural Engineering* 132 (8), 1201-1212.

Okazaki, T., and Engelhardt, M. D. [2007] "Cyclic loading behavior of EBF links constructed of ASTM A992 steel", *Journal of Constructional Steel Research* 63(6), 751-765.

Okazaki, T., Engelhardt, M.D., Drolias, A., Schell, E., Hong, J. K., and Uang, C. M. [2009] "Experimental investigation of link-to-column connections in eccentrically braced frames", *Journal of Constructional Steel Research* 65(7), 1401-1412.

Prinz. G. S. and Richards, P. W. [2009] "Eccentrically braced frame links with reduced web sections", *Journal of Constructional Steel Research* 65(10–11), 1971-1978.

Ramadan, T., and Ghobarah, A. [1995] "Analytical model for shear-link behaviour", *Journal of Structural Engineering* 121(11), 1574-1580.

Richards, P., and Uang, C. M. [2003] *Development of testing protocol for short links in eccentrically braced frames*, Report No. SSRP-2003/08. Department of Structural Engineering, University of California, San Diego.

Richards, P., and Uang, C. M. [2005] "Effect of flange width-thickness ratio on eccentrically braced frames link cyclic rotation capacity", *Journal of Structural Engineering* 131(10), 1546-1552.

Ricles, J. M., and Popov, E. P. [1987] *Dynamic analysis of seismically resistant eccentrically braced frames*, Earthquake Engineering Research Center, Report No. UCB/EERC-87/07. University of California, Berkeley.

Roeder, C. W., and Popov, E. P. [1977] *Inelastic Behavior of Eccentrically Braced Steel Framed Systems Under Cyclic Loading*, EERC Report No. 77-18. Earthquake Engineering Research Center, University of California, Berkeley.

Rossi, P. P., and Lombardo, A. [2007] "Influence of the link overstrength factor on the seismic behaviour of eccentrically braced frames", *Journal of Constructional Steel Research* 63, 1529-1545.

Rossi, P. P. [2015] "Importance of isotropic hardening in the modeling of buckling restrained braces", *Journal of Structural Engineering* 141(4). 04014124, 1-11.

Tremblay, R. [2003] "Achieving a stable inelastic seismic response for multi-story concentrically braced steel frames", *Engineering Journal* AISC 40,111–129

Zona, A., and Dall'Asta, A. [2012] "Elastoplastic model for steel buckling-restrained braces", *Journal of Constructional Research* 68(1), 118-125.

ingegneria sismica
International Journal of Earthquake Engineering
Anno XXXIII – Speciale CTA 2015 – Num. 3

# INFLUENZA DELLA MODELLAZIONE DEL LINK NELLA VALUTAZIONE DELLA RISPOSTA SISMICA DI EBF

*Melina Bosco, Aurelio Ghersi, Edoardo M. Marino, Pier Paolo Rossi**

Department of Civil Engineering and Architecture, University of Catania, Italy

SOMMARIO: *La risposta ciclica dei link nei telai con controventi eccentrici è stata spesso modellata attraverso una trave elastica con molle di estremità che simulavano il comportamento inelastico flessionale ed il comportamento elastico ed inelastico a taglio. Il comportamento di tali molle era elasto-plastico con incrudimento cinematico o elastico-perfettamente plastico. L'utilizzo di tali modelli era giustificato dalla carenza di modelli in grado di simulare con accuratezza l'incrudimento isotropo. Recentemente, gli autori di questo articolo hanno proposto un modello in cui la risposta delle cerniere flessionali e a taglio è definita dal legame costitutivo proposto da Zona e Dall'Asta per i controventi ad instabilità impedita. Questo modello simula sia l'incrudimento cinematico sia quello isotropo e, come mostrato da precedenti analisi, è in grado di riprodurre la risposta ciclica dei link ottenuta nell'ambito di indagini sperimentali. Sono stati, inoltre, suggeriti i valori da utilizzare per i parametri del modello per simulare il comportamento di link provvisti degli irrigidimenti richiesti dalle attuali norme antisismiche. Nel presente lavoro, il modello proposto è assunto quale riferimento per verificare l'affidabilità dei modelli più semplici utilizzati in passato. Il confronto tra i modelli è condotto analizzando la risposta dinamica non lineare di un insieme di telai con controventi eccentrici caratterizzati da diverso numero di piani e lunghezza meccanica del link. La riposta è espressa in termini di enti di risposta locali e globali.*

*Corresponding author: Pier Paolo Rossi, Department of Civil Engineering and Architecture, University of Catania, Italy.
Email: prossi@dica-unict.it