PROPOSED LIMITS OF STIFFENER SPACING REQUIREMENTS FOR EBF LINKS WITHIN THE FRAMEWORK OF EUROCODE 8

Nikolaos G. Skretas

Ph.D., M.Sc., Dipl.-Eng.

Athens. Greece

e-mail: n.g.skretas@gmail.com

Theodore L. Karavasilis

Professor

Dept. of Civil Engineering, University of Patras

Patras, Greece

e-mail: karavasilis@upatras.gr

Dimitrios G. Lignos

Professor, Department Chair

École Polytechnique Fédérale de Lausanne (EPFL)

Lausanne, Switzerland

e-mail: dimitrios.lignos@epfl.ch

1. SUMMARY

Design specifications for Eccentrically Braced Frames (EBFs) provided by both the current AISC and Eurocode (EC) 8 seismic provisions are primarily based on physical tests conducted in the early 1980s. The tests investigated the behavior of, mostly short, EBF links that featured an ASTM A36 steel grade (i.e., nominal $f_y = 250$ MPa). This paper first presents a thorough review of the EC8 stiffener spacing requirements. Finally, we propose new stiffener spacing requirements for short and intermediate length links by formulating and solving analytically the classic plastic plate buckling problem of idealized EBF web plates. The proposed limits are validated via continuum finite element studies. A summary of these studies is presented.

2. INTRODUCTION

Eccentrically braced frames (EBFs) concentrate inelastic deformations in a specific portion of the beam, referred to as the "*link*" hereinafter. Due to their large potential for prefabrication and accelerated construction, interest in implementing EBFs in practice was noticed after the 2010-2011 earthquake series in New Zealand [1]. Since then, further research on EBFs has been carried out, particularly on replaceable links [2-6].

Depending on the employed design and detailing criteria, the maximum inelastic rotation angle, $\gamma_{\mathrm{p,max}}$, of the EBF link should meet pre-established plastic deformation capacity limits. Referring to Fig. 1a, $\gamma_{\mathrm{p,max}}$ is defined as the maximum inelastic rotation angle at which the EBF link sustains at least one full cycle of loading before its shear resistance drops to 80% of the maximum recorded shear demand, V_{max}. Moreover, the design of the adjacent non-dissipative structural elements, is based on capacity design principles that mobilize the associated overstrength, Ω, which is defined as the ratio of the maximum developed shear force over the expected shear strength, $V_n = max[V_{\mathrm{p}}; 2M_{\mathrm{p}}/e]$ where e is the link's length, M_{p} is its plastic flexural resistance, and V_{p} is its plastic shear resistance. The definitions of $\gamma_{\mathrm{p,max}}$ and Ω are provided schematically in Fig. 1a. The geometric parameters used hereinafter are defined in Fig. 1b.

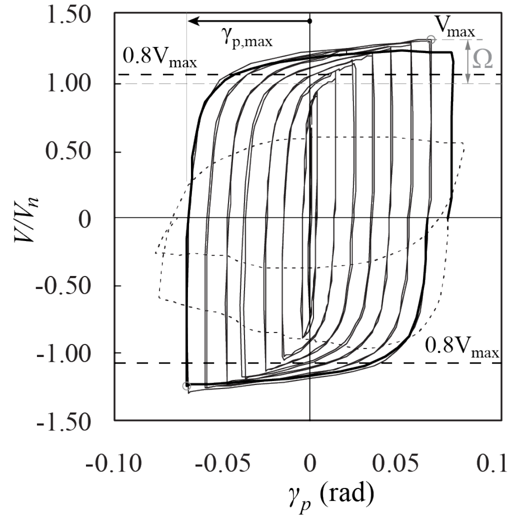

Figure 1a: Definitions of overstrength and inelastic shear distortions at 20% loss of the peak strength of links.

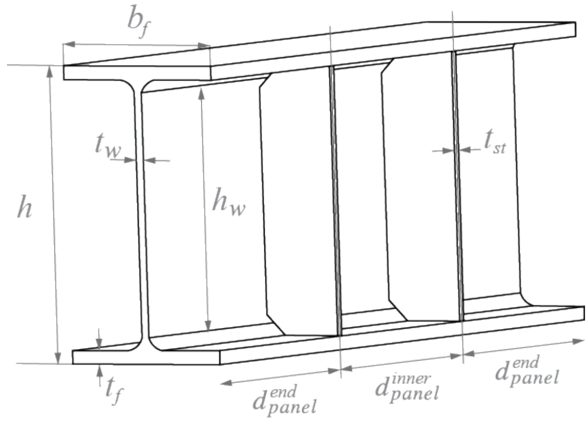

Figure 1b: Definitions of geometric parameters for EBF links.

Links are classified as short, intermediate, and long based on the developed plastic mechanism. The three categories are defined in Fig. 2 according to AISC-341-16 [7] and EN1998-1 [8]. There is a difference between EN1998-1 and AISC-341-16 regarding the upper bound of intermediate length links. According to AISC-341-16, a link is considered to be intermediate when $1.6 < e/(M_{\mathrm{p}}/V_{\mathrm{p}}) < 2.6$. Engelhardt and Popov [14] tested A36 links close to $2.6\, M_{\mathrm{p}}/V_{\mathrm{p}}$ and noticed a strong influence of the inelastic shear deformation on the link behavior. As such, they suggested that the transition length range from shear-dominated to flexural-dominated behavior be taken as $1.6 < e/(M_{\mathrm{p}}/V_{\mathrm{p}}) < 3.0$. EN1998-1 followed this suggestion. Hereinafter the EN1998-1 bounds are considered. Fig. 2 shows that the vast majority of the tested EBF links are in the short length range, whereas, to date, limited experimental data are found for intermediate ($1.6 < e/(M_{\mathrm{p}}/V_{\mathrm{p}}) < 3.0$) and long length links ($e/(M_p/V_p) \geq 3.0$). The maximum permitted design inelastic rotation angles per EN1998-1for the design of EBF links is given as follows:

$$\gamma_{\mathrm{pd,max}} = \begin{cases} 0.08\ [\mathrm{rad}]\,, & eV_p/M_p \leq 1.6 \\ 0.02 + \frac{0.06}{1.4}\left(3.0 - eV_p/M_p\right)\ [rad], & 1.6 < eV_p/M_p < 3.0 \\ 0.02[\mathrm{rad}]\,, & eV_p/M_p \geq 3.0 \end{cases} \tag{1}$$

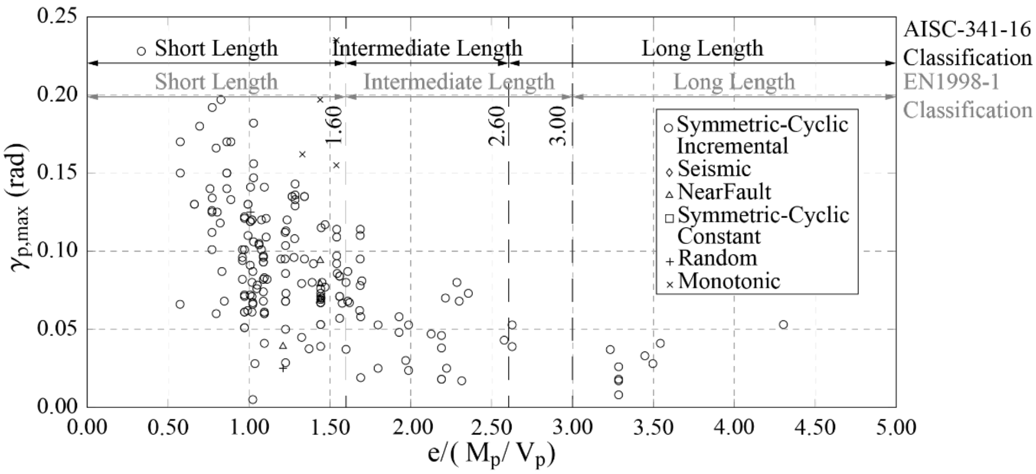

Fig. 2 Attained inelastic rotation angles of 224 tested links along with the classification of links based on their dimensionless length.

In this paper, we propose limits of stiffener spacing requirements for short and intermediate length EBF steel links within the framework of EN1998-1. The proposed limits have been developed on the basis of a numerical solution of the inelastic plate buckling problem and have been verified and refined by means of numerical simulations through continuum finite element (CFE) analyses of characteristic link geometries. Links designed according to the proposed stiffener spacing requirements can attain the $\gamma_{\mathrm{pd,max}}$ of the respective length.

3. BACKGROUND ON INTERMEDIATE STIFFENER REQUIREMENTS USED IN EN1998-1

For short links (i.e., $e \leq 1.60 M_p/V_p$), inelastic shear buckling of the web and ultra-low-cycle fatigue are the most governing failure modes. Transverse web stiffeners are used to control inelastic web buckling in short links, thereby enhancing their ability to dissipate hysteretic energy during seismic events. Semi-analytical and experimental studies on isolated links were carried out in 1980s [9-14]. These resulted in the first design procedures for EBFs. Malley and Popov [10] first proposed stiffener spacing requirements for links. Kasai and Popov [12] employed the classical problem of inelastic plate buckling theory with a semi-empirical method [15] and revised the above requirements. The simple rule proposed by Kasai and Popov [12] was incorporated in EN1998-1 for short links in the following form:

$$d_{panel} \leq min\left[52 - 366.7(\gamma_{\mathrm{pd}} - 0.02)\,;\ 52\right]t_w - h/5 \tag{2}$$

where γ_{pd} is the targeted design inelastic rotation angle, which shall not be greater than $\gamma_{\mathrm{pd,max}}$. During 2002 to 2005, a series of EBF tests were conducted at the University of Texas at Austin [16], i.e., the UTA tests. This study suggested that the established stiffener spacing requirements at that time may be conservative. These findings were further substantiated by more recent studies [17].

In long links, failure mechanisms typically involve coupled web and flange local buckling, and/or lateral torsional buckling. Engelhardt and Popov [14] studied intermediate and long links that featured A36 (i.e., nominal $f_y = 250$ MPa) steel cross sections. They found that locating stiffeners near the ends of long links delays lateral torsional buckling. Also, they noticed that the onset of flange local buckling is not substantially delayed, although the initiation of flange buckling in a stiffened long link is not concerning. Furthermore, they proposed that placing stiffeners too close to the link ends may be disadvantageous by constraining flange buckles within a small region. As such, they proposed a design rule and located transverse stiffeners at a distance $1.5b_f$ away from the link ends. Conversely, for intermediate length links, since shear buckling of the web can occur along with flange buckling and/or lateral torsional buckling, they suggested the use of stiffener spacing requirements for long links along with the stiffening of the web within the remaining central portion. Moreover, they noted that the use of stiffener spacing criteria for the remaining central portion of short links appears to be conservative. The above recommendations, which were not meant to be final, were only based on limited test data [18]. However, placing stiffeners at $d_{panel}^{end} = 1.5b_f$ while the remaining part of the link is 'stiffener-free' was adopted by EN 1998-1 for long links. The same seismic provisions require that the web stiffener spacing of intermediate length links should meet the requirements of short and long links.

Richards and Uang [19] found that many intermediate links did not achieve the targeted $\gamma_{\mathrm{pd,max}}$. They attributed this behavior to the associated web stiffener spacing. They also stated that (a) it seems somewhat nonconservative to extend the web stiffener spacing developed for short links to intermediate length links; and (b) the flexure–shear interaction may be appreciable at the web end panels, thereby triggering web buckling. Daneshmand and Hashemi [20] reported that some intermediate links did not meet the required inelastic rotation capacities as per AISC-341-16 [7]. They also reported that: (a) the use of double-sided web stiffeners significantly increased the rotation capacity of intermediate length links relative to their one-sided web stiffener counterparts; (b) narrowly spaced stiffeners would increase the rotation capacities of intermediate links; and (c) the $\gamma_{\mathrm{p,max}}$ becomes sensitive to the web slenderness ratio. More recently, Skretas et. al [21] showed through CFE analysis that many intermediate links designed for the maximum permitted inelastic rotation angle per AISC-341-16 did not achieve the required design rotation.

4. PROPOSED NEW STIFFENER SPACING LIMITS FOR EBF LINKS

The proposed limits for the intermediate stiffener spacing are described by the following design equation:

$$\frac{d_{panel}}{h_w} \leq \frac{d_{panel,max}}{h_w} = \begin{cases} \sqrt{\frac{1541/(h_w/t_w)^2}{\mu^* - 1155/(h_w/t_w)^2}}, & h_w/t_w > (h_w/t_w)^* \\ 1 + \kappa^* \left(\frac{1}{h_w/t_w} - \frac{1}{(h_w/t_w)^*}\right), & otherwise \end{cases} \tag{3}$$

The above design equation has been developed via a rigorous numerical solution of the inelastic plate buckling of a rectangular plate. The deformation theory of plasticity [22] based on (a) small strain approximation; and (b) first-order shear deformation theory [23] for the plastic buckling analysis of a plate subjected to pure shear were adopted. The Ramberg–Osgood constitutive relationship was used for the steel plate material [24]. The

μ^*, κ^* and $(h_w/t_w)^*$ coefficients are defined in [25] using the parameters $\tau_{cr}/0.6f_y$ and $\bar{\alpha}_{b,cr}$ where $\bar{\alpha}_{b,cr} = \sigma_{bend}/\sigma_{cr}$ and f_y the nominal yield stress.

In short links, the parameters are constant with $\bar{\alpha}_{b,cr} = 0$ and $\tau_{cr}/0.6f_y = 1.30$. An additional constraint for short links is that the end panels shall have a width less or equal to the 80% of the inner panels ($d_{panel}^{end} \leq 0.80\, d_{panel}^{inner}$) [26].

In intermediate links the $\tau_{cr}/0.6f_y$ is defined as follows:

$$\tau_{cr}/0.6f_y = \begin{cases} 1.15 - 0.25\left[e/\left(M_{\mathrm{p}}/V_{\mathrm{p}}\right) - 2.00\right], & 1.60 < e/\left(M_{\mathrm{p}}/V_{\mathrm{p}}\right) \leq 2.00 \\ 0.39 + 1.52\left[e/\left(M_{\mathrm{p}}/V_{\mathrm{p}}\right)\right]^{-1}, & e/\left(M_{\mathrm{p}}/V_{\mathrm{p}}\right) > 2.00 \end{cases} \tag{4}$$

while σ_{bend} and σ_{cr} are estimated by the following equations:

$$\begin{aligned} \sigma_{bend}/f_y &= 0.33e/\left(M_{\mathrm{p}}/V_{\mathrm{p}}\right) \\ \sigma_{cr}/f_y &= -3.26 + 4.25\bar{\lambda}_{\sigma}^{-0.07} - 0.09\bar{\lambda}_{\sigma} \end{aligned} \tag{5}$$

$$\bar{\lambda}_{\sigma} = \frac{h_w}{t_w}\sqrt{\frac{12(1-\nu^2)}{23.88\pi^2}}\sqrt{\frac{f_y}{E}}$$

A basic difference between short and intermediate links is that in the former the inner stiffeners shall be equally spaced while in the latter Eq. (3) is applied to every panel. For the inner panels of intermediate links σ_{bend} is reduced by the following equation:

$$\sigma_{bend,i} = \frac{e - \sum_{j=0}^{i-1} 2d_{panel,j}}{e}\sigma_{bend} \tag{6}$$

where $i > 0$. Subscript 0 on $\sigma_{bend,i}$ corresponds to end stiffeners, 1 to the next ones to the end stiffeners and so on. $d_{panel,j}$ are the applied panel widths for each panel, i.e., $d_{panel,0}$ are the applied d_{panel} to the end stiffeners and so on. Applied stiffener spacings have to be symmetric around the center of a link. An additional constrain for intermediate links is that $\bar{\alpha}_{b,cr}$ parameter is upper bounded by the value of 0.75.

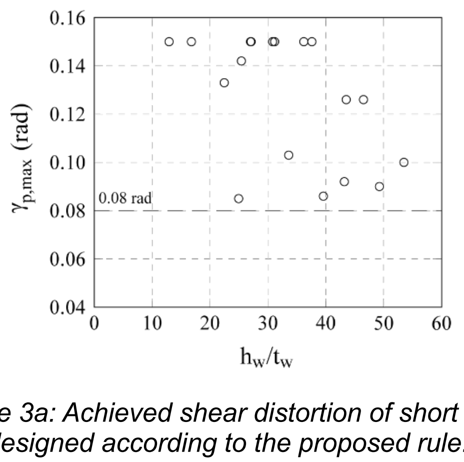

Figure 3a: Achieved shear distortion of short links designed according to the proposed rule.

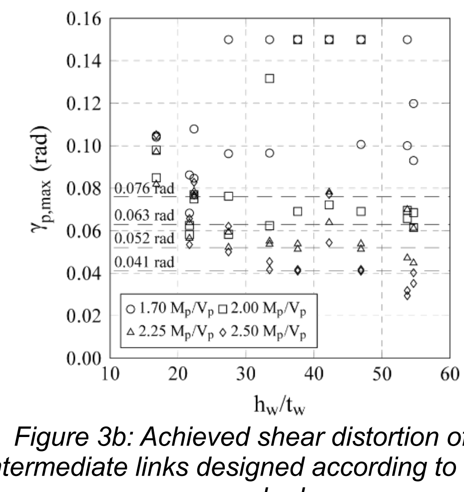

Figure 3b: Achieved shear distortion of intermediate links designed according to the proposed rule.

The proposed rules for short and intermediate links were verified through CFE analyses [25, 26]. The considered simulation cases featured short links with $12.0 < h_w/t_w < 53.5$ and $5.2 < b_f/t_f < 9.2$ [26], whereas intermediate length links featured $16.7 < h_w/t_w < 54.7$ and $5.9 < b_f/t_f < 8.5$. The results are depicted in Fig. 3. All 18 short links attain the maximum permitted inelastic rotation of 0.08 rad (Fig. 3a). On the other hand, nine out of 78 intermediate length links did not attain the maximum permitted inelastic rotation angle as per EN1998-1. Links with $h_w/t_w > 50$ and/or $b_f/2\,t_f > 8.4$ exhibited unacceptable cyclic performance.

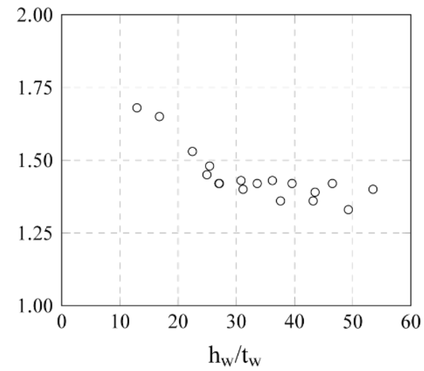

Figure 4a: Overstrength of short links designed according to the proposed rule.

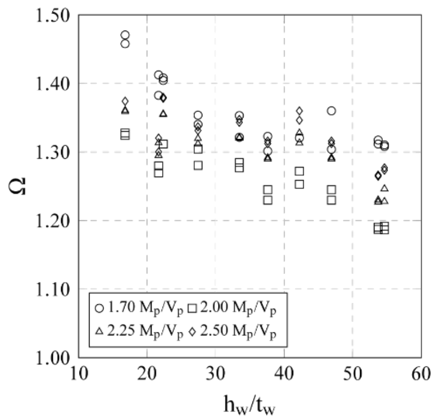

Figure 4b: Overstrength of intermediate links designed according to the proposed rule.

5. CONCLUSIONS

In the present study we proposed new stiffener spacing requirements for both short and intermediate length links in the design framework of EN1998-1 [8]. The stiffener distances that were examined correspond to the maximum permitted inelastic rotation angle as per EN1998-1. The proposed rules have been developed based on a rigorous numerical solution of the inelastic plate buckling problem and were verified through finite element analyses [25, 26]. All short links achieved the inelastic rotation angle of 0.08 rad while some intermediate links failed to achieve the required rotation angle. The proposed rules should be further examined via complementary experimental studies both at the material and member level.

6. REFERENCES

[1] Bruneau, M., and MacRae, G. "Building structural systems in Christchurch's post earthquake reconstruction." *Earthquake Spectra*, Vol. 35, No. 4, 2019, pp. 1953–1978.

[2] Mansour, N., Christopoulos, C., and Tremblay, R. "Experimental validation of replaceable shear links for eccentrically braced steel frames." *Journal of Structural Engineering*, ASCE, Vol. 137, No. 10, 2011, pp. 1141–1152.

[3] Stratan, A., Dubina, D., and Dinu, F. "Control of global performance of seismic resistant EBF with removable link." *STESSA 2003 - Behaviour of Steel Structures in Seismic Areas: Proceedings of the 4th International Specialty Conference*, F. M. Mazzolani, ed., Naples, Italy, Routledge, 2003, pp. 455–461.

[4] Tan, K., and Christopoulos, C. "Development of replaceable cast steel links for eccentrically braced frames." *Journal of Structural Engineering*, ASCE, Vol. 142, No. 10, 2016, pp. 04016079.

[5] Ioan, A., Stratan, A., Dubin, D., Poljanek, M., Molina, F., Taucer, F., Pegon, P., and Sabu, G. "Experimental validation of re-centring capability of eccentrically braced frames with removable links." *Engineering Structures*, Vol. 113, 2016, pp. 335 – 346.
[6] Bozkurt, M., and Topkaya, C. "Replaceable links with direct brace attachments for eccentrically braced frames." *Earthquake Engineering & Structural Dynamics*, Vol. 46, No. 13, 2017, pp. 2121–2139.
[7] AISC Seismic provisions for structural steel buildings. ANSI/AISC 341-16. 2016 Chicago: AISC
[8] CEN EN1998-1-1. Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings. *Eurocode 8. 2005*. Brussels, Belgium
[9] Hjelmstad, K. D., and Popov, E. P. "Cyclic behavior and design of link beams." *Journal of Structural Engineering*, ASCE, Vol. 109, No. 10, 1983, pp. 2387–2403.
[10] Malley, J. O., and Popov. E. P. "Shear links in eccentrically braced frames." *Journal of Structural Engineering*, ASCE, Vol. 110, No. 9, 1984, pp. 2275–2295.
[11] Kasai, K., and Popov. E. P. "General behavior of WF steel shear link beams." *Journal of Structural Engineering*, ASCE, Vol. 112, No. 2, 1986, pp. 362–382.
[12] Kasai, K., and Popov. E. P. "Cyclic web buckling control for shear link beams." *Journal of Structural Engineering*, ASCE, Vol. 112, No. 3, 1986, pp. 505–523.
[13] Ricles, J. M., and Popov. E. P. "Composite action in eccentrically braced frames." *Journal of Structural Engineering*, ASCE, Vol. 115, No. 5, 1989, pp. 2046–2066.
[14] Engelhardt, M. D., and Popov. E. P. "Experimental performance of long links in eccentrically braced frames." *Journal of Structural Engineering*, ASCE, Vol. 118, No. 11, 1992, pp. 3067–3088.
[15] Gerard, G. "Critical shear stress of plates above the proportional limit." *Journal of Applied Mechanics*, ASME, Vol. 15, No. 1, 1948, pp. 7–12.
[16] Okazaki, T., and Engelhardt, M. "Cyclic loading behavior of EBF links constructed of ASTM A992 steel." *Journal of Constructional Steel Research*, Elsevier, Vol. 63, No. 6, 2007, pp. 751–765.
[17] Azad, K. S., and Topkaya. C. "A review of research on steel eccentrically braced frames." *Journal of Constructional Steel Research*, Elsevier, Vol. 128, Jan, 2017, pp. 53–73.
[18] Engelhardt, M. D., Popov E. P. "Behaviour of long links in eccentrically braced frames". *Pacific Earthquake Engineering Research Center* UCB/EERC-89/01 1989.
[19] Richards, P., and Uang C. "Effect of flange width-thickness ratio on eccentrically braced frames link cyclic rotation capacity." *Journal of Structural Engineering*, ASCE, Vol. 132, No. 8, 2005, pp. 1183–1191.
[20] Daneshmand, A., and Hashemi B. "Performance of intermediate and long links in eccentrically braced frames." *Journal of Constructional Steel Research*, Elsevier, Vol. 70, Mar, 2012, pp. 167–176.
[21] Skretas, N. G., Karavasilis, T. L., and Lignos, D.G. "Assessment of stiffener spacing design requirements for intermediate EBF links via nonlinear finite element analysis." Fifth National Conference on Earthquake Engineering, Athens, Greece, October 20-22, 2022.
[22] Hencky, H. "Zur theorie plastischer deformationen und der hierdurch im material hervorgerufenen nachspannungen." *Journal of Applied Mathematics and Mechanics*, Wiley, Vol. 4, No. 4, 1924, pp. 323–334.
[23] Mindlin, R. D. "Influence of rotatory inertia and shear on flexural motions of isotropic, elastic plates." *Journal of Applied Mechanics*, ASME, Vol. 18, No. 1, 1951, pp. 31–38.

[24] Ramberg, W., and Osgood. W. “Description of stress-strain curves by three parameters”. *Washington, DC: National Advisory Committee for Aeronautics*, Rep. No. 902., 1943.
[25] Skretas, N. G. “Development of revised stiffener spacing requirements for seismic links in eccentrically braced frames.” Doctoral Dissertation, University of Patras, 2022.
[26] Skretas, N. G., Karavasilis, T. L., and Lignos, D.G. “Proposed stiffener spacing requirements for the seismic design of short links in eccentrically braced steel frames.” *Journal of Structural Engineering*, ASCE, Vol. 149, No. 6, 2023, pp. 04023058.

ΝΕΑ ΠΡΟΤΕΙΝΟΜΕΝΑ ΟΡΙΑ ΓΙΑ ΤΙΣ ΑΠΟΣΤΑΣΕΙΣ ΤΩΝ ΕΓΚΑΡΣΙΩΝ ΝΕΥΡΩΣΕΩΝ ΓΙΑ ΕΚΚΕΝΤΡΟΥΣ ΣΥΝΔΕΣΜΟΥΣ ΣΤΟ ΠΛΑΙΣΙΟ ΤΟΥ ΕΥΡΩΚΩΔΙΚΑ 8

Nikolaos G. Skretas

Ph.D., M.Sc., Dipl.-Eng.

Athens. Greece

e-mail: n.g.skretas@gmail.com

Theodore L. Karavasilis

Professor

Dept. of Civil Engineering, University of Patras

Patras, Greece

e-mail: karavasilis@upatras.gr

Dimitrios G. Lignos

Professor, Department Chair

Ecole Polytechnique Fédérale de Lausanne (EPFL)

Lausanne, Switzerland

e-mail: dimitrios.lignos@epfl.ch

ΠΕΡΙΛΗΨΗ

Ο προδιαγραφές για τους συνδέσμους με έκκεντρα πλαίσια (EBF) που παρέχονται τόσο από τις τρέχουσες σεισμικές διατάξεις του Αμερικάνικου κανονισμού AISC-341-16 όσο και από τον Ευρωκώδικα 8 βασίζονται σε πειραματικές δοκιμές που πραγματοποιήθηκαν στις αρχές της δεκαετίας του 1980. Οι δοκιμές διερεύνησαν τη συμπεριφορά, κυρίως βραχέων, συνδέσμων EBF που από ποιότητα χάλυβα ASTM A36 (δηλαδή ονομαστική $f_y = 250$ MPa) και Αμερικάνικες διατομές. Στην παρούσα δημοσίευση παρουσιάζεται μια λεπτομερής ανασκόπηση των απαιτήσεων για τις εγκάρσιες νευρώσεις του EC8. Τέλος, προτείνουμε νέες απαιτήσεις για τις αποστάσεις των νευρώσεων για βραχείς συνδέσμους και συνδέσμους ενδιάμεσου μήκους. Τα όρια αυτά επικυρώνονται μέσω μη γραμμικών αναλύσεων με συνεχή πεπερασμένα στοιχεία.