PROPOSED PANEL ZONE MODEL FOR BEAM-TO-COLUMN JOINTS IN STEEL MOMENT RESISTING FRAMES

Andronikos Skiadopoulos
Postdoctoral Researcher
École Polytechnique Fédérale de Lausanne (EPFL)
Lausanne, Switzerland
e-mail: andronikos.skiadopoulos@epfl.ch

Ahmed Elkady
Lecturer
University of Southampton
Southampton, United Kingdom
e-mail: a.elkady@soton.ac.uk

Dimitrios Lignos
Full Professor
École Polytechnique Fédérale de Lausanne (EPFL)
Lausanne, Switzerland
e-mail: dimitrios.lignos@epfl.ch

1. ABSTRACT

In capacity designed steel moment resisting frames (MRFs), beam-to-column joints (i.e., panel zones) are designed to remain elastic. To potentially exploit the beneficial aspects of the stable panel zone hysteretic response in shear, a robust panel zone model is required. To assess the available panel zone models in the literature, a database of more than 100 experiments was systematically assembled. Experimental evidence suggests that available models may overestimate the panel zone shear strength by 40%. Regarding the model utilised in Europe, the column flange contribution to panel zone shear strength is disregarded, if continuity plates are not present, thereby underestimating the panel zone shear strength by 20%, on average. Moreover, only one doubler plate (if two are needed) is considered in the panel zone strength calculation. This paper examines realistic shear stresses in various panel zone geometries based on continuum finite element (CFE) simulations validated with available experimental data. The parametric analysis results are then leveraged to develop a new panel zone shear strength model for the seismic design of steel MRFs. Comparisons of the proposed panel zone shear stiffness and strength reveal a noteworthy accuracy of less than 10% error and decreased variation compared to existing models. Complementary CFE analyses on panel zones with doubler plates revealed that these are effective withstanding the shear strains developed in the column web. Thus, the panel zone thickness to be used in the proposed panel zone strength model should include the column web and the doubler plate(s).

2. INTRODUCTION

In capacity-designed steel moment resisting frames (MRFs), column web panel zones are usually designed to remain elastic, while the dissipation of the earthquake-induced energy is mostly realised in the steel beam ends [1], [2]. Back in 1970s, research on small-scale beam-to-column connections with highly inelastic panel zones [3] highlighted the beneficial aspects of panel zone shear yielding in the seismic performance of beam-to-column connections. Similar findings have been demonstrated experimentally in recent studies on full-scale beam-to-column connections [4]–[10]. The current design standards limit the pane zone inelastic distortions to $4\gamma_y$ (where γ_y is the panel zone shear distortion at yield). Moreover, the current design equations are questionable for higher inelastic shear distortions. Therefore, to potentially exploit the beneficial aspects of panel zone shear yielding, a robust panel zone model is timely.

Referring to Fig. 1a, the Krawinkler model [11] comprises an elastic branch with stiffness K_e, given in Eq. 1a, which assumes shear deformation mode dominance within the panel zone (see Fig. 1b). The bending deformation mode (see Fig. 1c) is disregarded from the panel zone stiffness equation. The panel zone is assumed to yield uniformly within the column web, as per Eq. 1b. The contribution of the flanges in the elastic branch is disregarded, while it is assumed that they solely contribute to the plastic deformations of the panel zone, until $4\gamma_y$. The panel zone shear strength at $4\gamma_y$ is given in Eq. 1c and was calibrated to limited experimental data with t_{cf} ranging from 10mm to 24mm. After $4\gamma_y$, a constant stiffness of $0.03K_e$ is assumed. Available models in the literature are similar in nature to the described model. In Europe [2], panel zones are designed based on their yield stress, similarly to Eq. 1b. In case continuity plates within the panel zone are utilized in design, an additional term to V_y is considered, as per Eq. 1d.

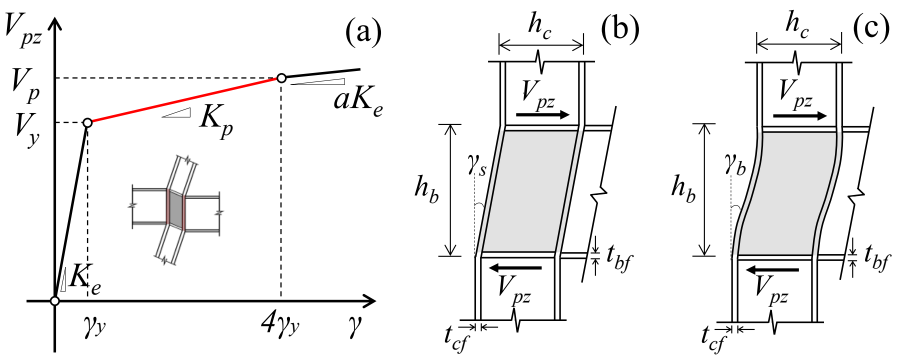

Fig. 1 (a) Typical panel zone mathematical model, (b) panel zone bending deformation mode, and (c) panel zone shear deformation mode

$$K_e = \frac{V_y}{\gamma_y} = 0.95 h_c \cdot t_{cw} \cdot G \tag{1a}$$

$$V_y = \frac{f_y}{\sqrt{3}} \cdot 0.95 h_c \cdot t_{cw} \tag{1b}$$

$$V_p = \mathrm{V_y} \cdot \left(1 + 3K_p/K_e\right) \tag{1c}$$

$$V_{wp,add,Rd} = 4M_{pl,fc,Rd}/d_s, \text{ with } M_{pl,fc,Rd} \le M_{pl,st,Rd} \tag{1d}$$

Where, G is the shear modulus; f_y is the yield stress of the steel material; t_{cw} is the column web thickness; h_c is the column depth; K_p is the panel zone post-yield stiffness; $M_{pl,fc,Rd}$ is the plastic moment resistance of the column flange; $M_{pl,st,Rd}$ is the plastic moment resistance of the continuity plate; d_s is the distance between the continuity plates.

Fig. 2a compares the experimental elastic panel zone stiffnesses, $K_{e,m}$, with the predicted ones, K_e, as per the CEN [2] model, based on a dataset assembled by Skiadopoulos and Lignos [12]. The data are divided into connections with and without doubler plates. The straight lines denote the respective trends. It is observed that the elastic stiffness K_e is overestimated by 20% for the cases without doubler plates because the contribution of the bending deformation on the total panel zone deformation is neglected [6], [13]. In the test specimens featuring doubler plates, the elastic stiffness diverges by more than 20% from the experimentally derived values. The reason is that when two doubler plates are needed, the CEN [2] model accounts for only one in the respective calculations. Moreover, if the doubler plate thickness is less than that of the column web, no contribution to the strength and stiffness is assumed. These assumptions are questionable, as also concluded by Fig. 2a.

Fig. 2b depicts the ratio of the predicted over the measured panel zone strength ratio, $V_p/V_{p,m}$ with respect to t_{cf}. For sections without doubler plates and thick flanges, the CEN [2] model underestimates the panel zone strength by 20%, contrary to AISC [14], because the column flange contribution is neglected when continuity plates are not present. This is a common design condition when shallow (i.e., $h_c \leq 600$ mm) and stocky cross sections (i.e., $t_{cf} \geq 40$ mm) are used as columns. However, the expected column flange contribution is substantial in this case. For sections with flange thicknesses less than 30 mm, the panel zone strength is overestimated by up to 60%, since Eq. 1d does not accurately depict the mechanics of shear stresses in the column flanges. Consequently, the panel zone strength is underestimated, especially when two doubler plates are present.

Within such a context, this paper proposes a new mechanics-based panel zone model for the seismic design of steel MRFs. The model is informed by continuum finite element (CFE) analyses to comprehend the mechanics of panel zones experiencing high inelastic shear strains. The proposed model is validated to available test data in the literature.

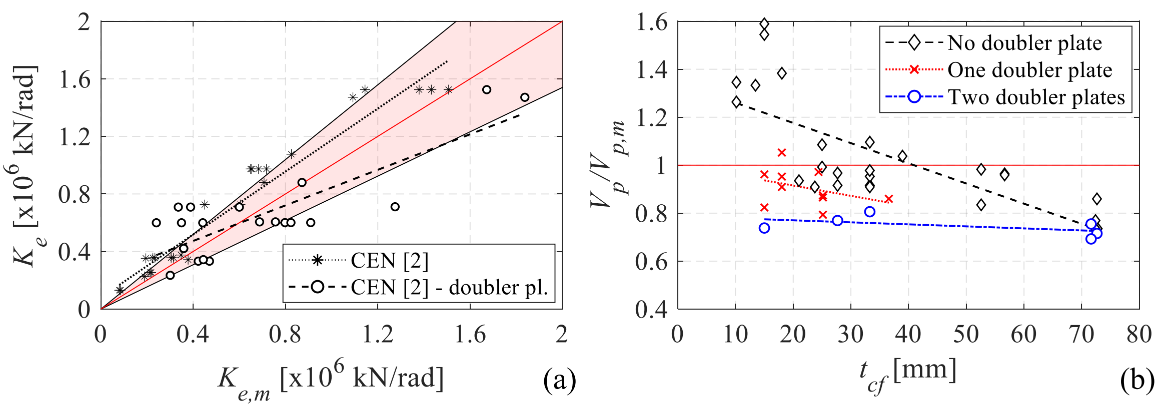

Fig. 2 Comparison of experimental panel zone response variables with panel zone model predictions as per CEN [2]: (a) panel zone stiffness, and (b) panel zone strength V_p

3. PARAMETRIC FINITE ELEMENT ANALYSES

The CFE models are developed through the commercial finite element analysis software ABAQUS (version 6.14-1) [15]. The CFE modelling assumptions of the developed models are first validated to full-scale experiments. As illustrated in Fig. 3a, the interior beam-to-column connection test by Shin [8] is employed for this purpose. In brief, the material model employs a combined multiaxial isotropic/kinematic hardening law [16] with input model parameters identified as per the optimization approach proposed by de Castro e Sousa et al. [17]. Local imperfections and residual stresses in the steel beams are considered as discussed in Elkady and Lignos [18] and Young [19]. Twenty-node quadratic brick elements with reduced integration (C3D20R) are employed. A finer mesh is employed at the critical panel zone and beam end regions based on a mesh sensitivity study. The model boundary conditions represent those of the experiment (see Fig. 3a). To expedite the computations, a reduced-order panel zone model is considered with the boundary conditions shown in Fig. 3a. Fig. 3b shows the panel zone test response compared with the detailed and the reduced-order CFE analysis. The comparison demonstrates the validity of the employed assumptions for both the detailed and the reduced-order CFE models in terms of strength and stiffness.

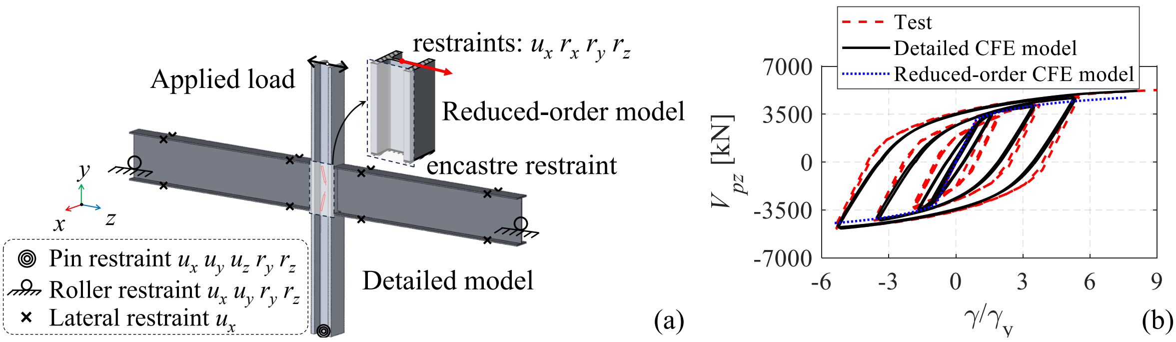

Fig. 3 (a) Detailed and reduced-order CFE models, and (b) panel zone response comparison between test and CFE models

The development of the panel zone model relies on eight panel zone geometries. The variables of the parametric analysis are the panel zone aspect ratio, h_b/h_c, the column flange width, b_{cf}, and the column flange thickness, t_{cf}. The reduced-order models are subjected to monotonic loading up to $6\gamma_y$ and the deduced parameters are; (a) $K_{e,m}$ (in terms of shear resistance-to-distortion, $V_{pz} - \gamma$, relation); (b) V_y, based on the onset of yielding at the panel zone; and (c) the panel zone shear strength at $4\gamma_y$ and $6\gamma_y$.

Fig. 4 shows normalised stress distributions at the mid-height of the panel zones with slender and stocky geometries, characterised with $h_b/h_c = 1.5$ and $t_{cf} = 25$ mm, and $h_b/h_c = 1.0$ and $t_{cf} = 50$ mm, respectively. The distributions correspond to shear distortions of $4\gamma_y$, while planes of average shear stresses in the web and the flanges are superimposed. It is demonstrated that, although the web shear stress distributions are similar after the onset of panel zone yielding and equal $1.1\tau_y$, shear stresses at the flanges vary significantly. For instance, in the former case, shear stresses at the flanges are infinitesimal (nearly $1\% \cdot t_y$), while in the latter case, shear stresses at the flanges exceed $10\% \cdot t_y$. Considering that the area of flanges with respect to the total area of stocky sections is considerable, the flange contribution in this case should be estimated accurately.

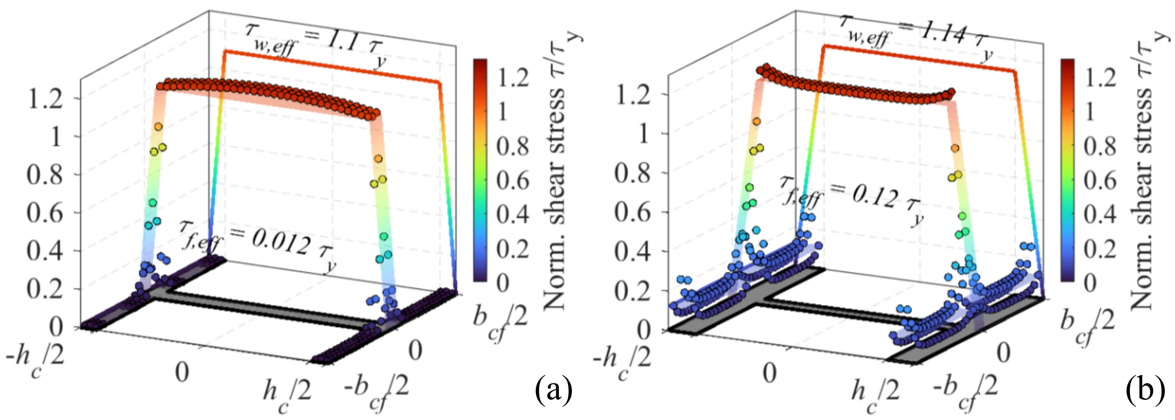

Fig. 4 Shear stresses in panel zones of variable geometries: (a) slender (h_b/h_c = 1.5 and t_{cf} = 25 mm), and (b) stocky (h_b/h_c = 1.0 and t_{cf} = 50 mm)

4. PROPOSED PANEL ZONE MODEL

4.1 Elastic stiffness

The elastic stiffness of the proposed panel zone model, K_e, considers both shear and bending deformation modes in series (see Figs. 1b and 1c), as per Eq. 2. The effective area withstanding the shear stresses in the web equals $h_c - t_{cf}$, according to Charney et al. [20]. The computation of the bending stiffness considers the panel zone in contraflexure along its strong axis. In case doubler plates are attached to the column web, t_{pz} equals the thickness of the column web and the total thickness of the doubler plates. Moreover, the second moment of area of the doubler plates should be accounted for in second moment of area, I, with respect to the strong axis of the column, where E is the Young's modulus.

$$K_e = \frac{12EI \cdot t_{pz} \cdot (h_c - t_{cf})}{t_{pz} \cdot (h_c - t_{cf}) \cdot h_b^2 + 24I \cdot (1 + v)} \quad (2)$$

4.2 Shear strength

The foregoing highlighted the contribution of the flanges in the panel zone shear resistance after the onset of yielding at the web, especially for stocky geometries with $K_f/K_e > 0.06$ (K_f defined in Eq. 3). The proposed panel zone shear strength should rely on a parameter that predicts shear forces in the panel zone flanges. Therefore, the stiffness of the flanges, K_f, is defined as per Eq. 3, which considers both shear and bending modes within the panel zone flanges. The shear forces sustained by the flanges equal $(K_f/K_e) \cdot V_{pz}$.

$$K_f = \frac{2E \cdot b_{cf} \cdot t_{cf}^3}{h_b^2 + 2 \cdot (1 + v) \cdot t_{cf}^2} \quad (3)$$

Fig. 5 shows the predicted-to-measured panel zone elastic stiffness and strength at $4\gamma_y$ with respect to K_f/K_e. For slender panel zones with $K_f/K_e < 0.03$, geometries that disregard the bending deformation mode overpredict the panel zone elastic stiffness by up to 60%. On the other hand, the proposed panel zone elastic stiffness shows remarkable accuracy.

Moreover, the Krawinkler model overpredicts the panel zone shear strength at $4\gamma_y$ by more than 20% for stocky panel zone geometries, contrary to the later proposed model that is accurate regardless of K_f/K_e.

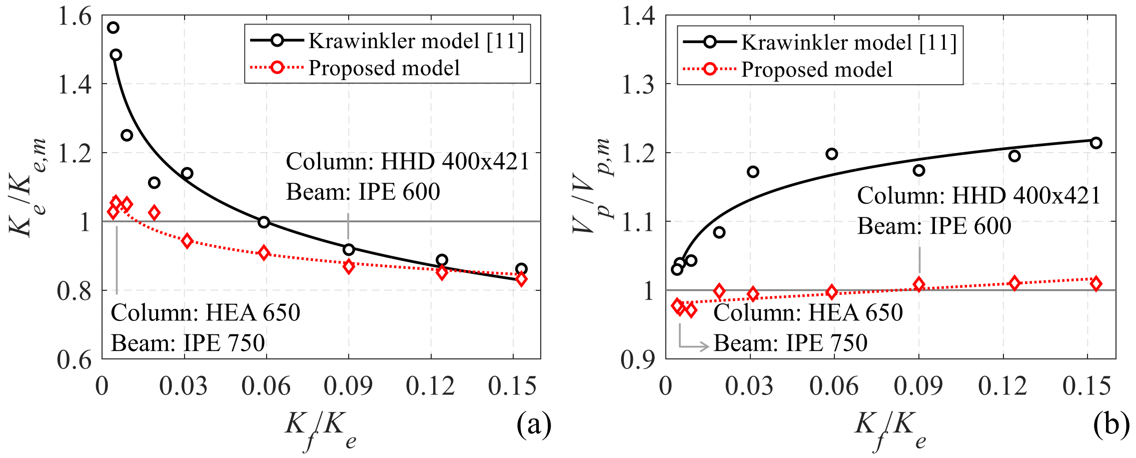

Fig. 5 Predicted-to-measured panel zone elastic stiffness and strength with respect to K_f/K_e

The proposed panel zone shear strength model is based on integration of the realistic shear stress distributions at the panel zone mid-height plane. The adopted double integration is given in Eq. 4, where A is the column cross-sectional area, A_f is the area of each column flange, and A_w is the column web area. To simplify the proposed model, average web and flange shear stresses are introduced in Eq. 5 by computing the normalized average shear stresses in the web, $a_{w,eff}$, and the flange, $a_{f,eff}$. Statistical analysis to examine the relationship between K_f/K_e and $a_{w,eff}$ or $a_{f,eff}$ at shear distortions of interest (i.e., $[1,4,6] \cdot \gamma_y$) showed a strong dependence, with the coefficient of determination being higher than 0.95. This demonstrated the efficiency and sufficiency of K_f/K_e in describing the shear stress evolution within a panel zone. Although these results are not shown due to brevity, more details are found in Skiadopoulos et al. [14].

$$V_{pz} = \iint_A \tau dA = \iint_{A_w} \tau dA_w + 2\iint_{A_f} \tau dA_f \tag{4}$$

$$V_{pz} = \frac{f_y}{\sqrt{3}} \cdot \left[a_{w,eff} \cdot \left(h_c - t_{cf}\right) \cdot t_{cw} + a_{f,eff} \cdot \left(b_{cf} - t_{cw}\right) \cdot 2t_{cf}\right] \tag{5}$$

Table 1 summarises the proposed simplified equations according to Skiadopoulos et al. [14] that could be directly used in prospective seismic design provisions of steel MRFs. It is observed that the column flanges do not contribute to the panel zone shear strength at yield. For slender panel zone geometries, with $K_f/K_e < 0.02$, the shear stress distribution in the web is parabolic, contrary to stocky panel zone geometries with $K_f/K_e > 0.06$ that are uniform. This is explained by the coefficients $a_{w,eff}$ which equal 0.9 and 1.0, respectively. For higher panel zone distortion angles, $a_{w,eff}$ is constant regardless of the K_f/K_e, as demonstrated in Fig. 4. However, $a_{f,eff}$ depends on the panel zone geometry itself. For stocky panel zone geometries, $a_{f,eff}$ is equal to 0.1, while for slender panel zone geometries, $a_{f,eff}$ equals 0.02-0.03 (see Fig. 4).

Geometry	Web ($a_{w,eff}$)			Flange ($a_{f,eff}$)		
	γ_y (V_y)	$4\gamma_y$ (V_p)	$6\gamma_y$ ($V_{6\gamma y}$)	γ_y (V_y)	$4\gamma_y$ (V_p)	$6\gamma_y$ ($V_{6\gamma y}$)
Slender	0.9	1.1	1.15	0.0	0.02	0.03
Stocky	1.0			0.0	0.1	

Table 1 Panel zone shear strength model coefficients

4.3 Model validation

The proposed model is validated to available experimental data in literature of variable panel zone geometries. Herein, representative comparisons are shown for a slender panel zone geometry with $K_f/K_e = 0.01$ [3] and a stocky geometry with $K_f/K_e = 0.05$ [8]. The former comprises W10x15 (~ IPE 240) beams and a W8x24 (~ HEB 200) column without doubler plates, while the latter features W36x150 (~ HEB 900) beams and a W14x398 (~ HHD 400x463) column with two doubler plates of 13 mm thickness each. In the former case, the CEN model [2] overestimates the panel zone shear strength due to the assumption of uniform shear strength distribution at V_y and the overestimation of the panel zone shear area. On the other hand, for stocky panel zone geometries, the CEN model [2] underestimates the panel zone shear strength by 60%. This is due to the fact that only the contribution of one doubler plate is considered in the CEN model [2]. This assumption is not justifiable as shown herein. The accuracy of the proposed panel zone model in both cases is noteworthy.

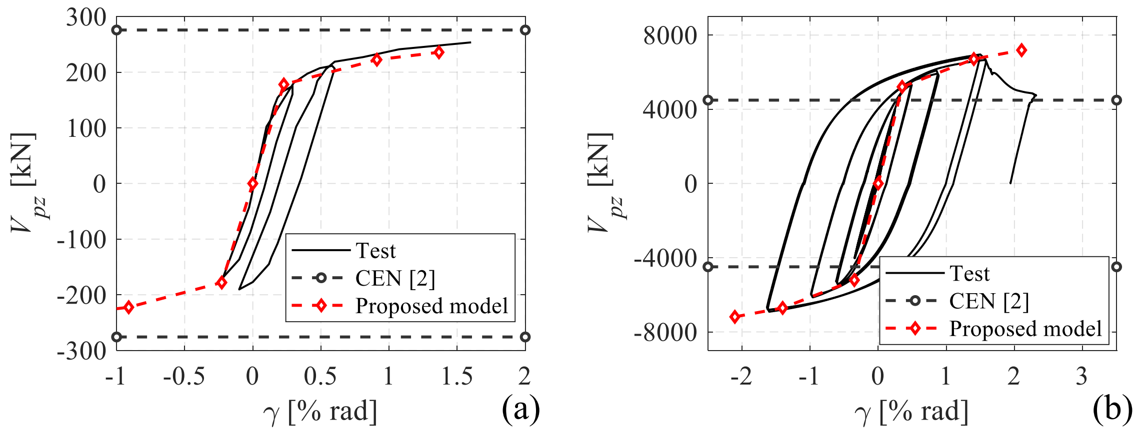

Fig. 5 Predicted-to-measured panel zone elastic stiffness and strength with respect to K_f/K_e

5. SUMMARY AND CONCLUSIONS

A pane zone model for the seismic design of steel MRFs is proposed in this study based on thoroughly validated continuum finite element simulations. The proposed model is based on realistic shear stress distributions in the web and the flanges. The model comprises an elastic panel zone stiffness that considers all deformations within the panel zone and three branches that are benchmarked at shear distortions of interest to the engineering profession. The primary conclusions are as follows:

The CEN model that disregards the bending deformation mode of the panel zone overestimates the panel zone elastic stiffness by more than 20%, especially for slender panel zone geometries with beam-to-column depth ratios, $h_b/h_c \geq 1.5$. For panel zones featuring doubler plates, the CEN model underpredicts the panel zone elastic stiffness, especially in cases where two doubler plates are present. The proposed elastic stiffness expression is accurate, since it considers both shear and bending panel zone deformation modes and both doubler plates, if present.

Analysis results demonstrated that the uniform web shear stress assumption assumed by CEN is realistic only for stocky panel zones. For slender panel zone geometries, this assumption leads to overestimation of the actual shear strength by more than 10%.

While the current CEN model overpredicts V_p by more than 30% in slender panel zone geometries ($t_{cf} < 30$mm), due to the non-justifiable shear area, the proposed equation for predicting V_p provides a remarkable accuracy as demonstrated by direct comparisons with available physical data.

6. ACKNOWLEDGMENTS

This study was supported by a Nippon Steel Corporation collaborative grant as well as an EPFL internal grant for the first and second authors. The financial support is gratefully acknowledged. The authors would like to sincerely thank Prof. Dr. Bozidar Stojadinovic from ETH-Zürich, for providing test data for the development of the inelastic panel zone database.

7. REFERENCES

[1] AISC 'Seismic provisions for structural steel buildings', Chicago, IL, USA: American Institute of Steel Construction, Report Nr. ANSI/AISC 341-16.

[2] CEN 'EN 1993-1-8: Eurocode 3: Design of steel structures – Part 1-8: Design of joints', Brussels, Belgium: European Committee for Standardization.

[3] KRAWINKLER, H., BERTERO, V.V. and POPOV, E.P. 'Inelastic behavior of steel beam-to-column subassemblages', University of California, Berkeley, CA, USA, Report Nr. 71/07.

[4] LEE, C.H., JEON, S.W., KIM, J.H. and UANG, C.M. 'Effects of panel zone strength and beam web connection method on seismic performance of reduced beam section steel moment connections', *Journal of Structural Engineering*, Vol. 131, No. 12, 2005, pp. 1854–1865, https://doi.org/10.1061/(ASCE)0733-9445(2005)131:12(1854).

[5] LEE, D., COTTON, S.C., HAJJAR, J., DEXTER, R.J. and YE, Y. 'Cyclic behavior of steel moment-resisting connections reinforced by alternative column stiffener details I. connection performance and continuity plate detailing', *Engineering Journal*, Vol. 42, No. 4, 2005, pp. 189–214.

[6] LIN, K.C., TSAI, K.C., KONG, S.L. and HSIEH, S.H. 'Effects of panel zone deformations on cyclic performance of welded moment connections', in *Proc., 12th World Conference on Earthquake Engineering (WCEE)*, New Zealand National Society for Earthquake Engineering, Auckland, New Zealand, 2000.

[7] RAHIMINIA, F. and NAMBA, H. 'Joint panel in steel moment connections, part 1: experimental tests results', *Journal of Constructional Steel Research*, Vol. 89, 2013, pp. 272–283, https://doi.org/10.1016/j.jcsr.2013.07.002.

[8] SHIN, S. 'Experimental and analytical investigation of panel zone behavior in steel moment frames', PhD Thesis, Department of Civil, Architectural and Environmental Engineering, University of Texas at Austin, TX, USA.

[9] TSAI, K.C. and CHEN, W.Z. 'Seismic responses of steel reduced beam section to weak panel zone moment joints', in *In Proc., 3rd International Conference on Behavior of Steel Structures in Seismic Areas (STESSA)*, Montreal, Canada, 2000, pp. 279–286.

[10] SKIADOPOULOS, A., LIGNOS, D.G., ARITA, M. and HIROSHIMA, S. 'Full-scale experiments of cyclically loaded welded moment connections with highly dissipative panel zones and simplified weld details', *Journal of Structural Engineering*, 2023, https://doi.org/10.1061/JSENDH/STENG-12128.

[11] KRAWINKLER, H. 'Shear in beam-column joints in seismic design of steel frames', *Engineering Journal*, Vol. 15, No. 3, 1978, pp. 82–91.

[12] SKIADOPOULOS, A. and LIGNOS, D.G. 'Development of inelastic panel zone database', *Journal of Structural Engineering*, Vol. 147, No. 4, 2021, pp. 04721001, https://doi.org/10.1061/(ASCE)ST.1943-541X.0002957.

[13] KIM, K.D. and ENGELHARDT, M.D. 'Monotonic and cyclic loading models for panel zones in steel moment frames', *Journal of Constructional Steel Research*, Vol. 58, No. 5–8, 2002, pp. 605–635, https://doi.org/10.1016/S0143-974X(01)00079-7.

[14] SKIADOPOULOS, A., ELKADY, A. and LIGNOS, D.G. 'Proposed panel zone model for seismic design of steel moment-resisting frames', *Journal of Structural Engineering*, Vol. 147, No. 4, 2021, pp. 04021006, https://doi.org/10.1061/(ASCE)ST.1943-541X.0002935.

[15] ABAQUS 'ABAQUS analysis user's manual version 6.14-1', Dassault Systems Simulia Corp., Providence, RI: SIMULIA.

[16] LEMAITRE, J. and CHABOCHE, J.L. 'Mechanics of solid materials', Cambridge, UK: Cambridge University Press, 1990.

[17] DE CASTRO E SOUSA, A., SUZUKI, Y. and LIGNOS, D. 'Consistency in solving the inverse problem of the Voce-Chaboche constitutive model for plastic straining', *Journal of Engineering Mechanics*, Vol. 146, No. 9, 2020, pp. 04020097, https://doi.org/10.1061/(ASCE)EM.1943-7889.0001839.

[18] ELKADY, A. and LIGNOS, D.G. 'Improved seismic design and nonlinear modeling recommendations for wide-flange steel columns', *Journal of Structural Engineering*, Vol. 144, No. 9, 2018, pp. 04018162, https://doi.org/10.1061/(ASCE)ST.1943-541X.0002166.

[19] YOUNG, B.W. 'Residual stresses in hot rolled members', *IABSE reports of the working commissions*, Vol. 23, No. 1, 1975, pp. 25–38, https://doi.org/10.5169/SEALS-19798.

[20] CHARNEY, F.A., IYER, H. and SPEARS, P.W. 'Computation of major axis shear deformations in wide flange steel girders and columns', *Journal of Constructional Steel Research*, Vol. 61, No. 11, 2005, pp. 1525–1558, https://doi.org/10.1016/j.jcsr.2005.04.002.

PROPOSED PANEL ZONE MODEL FOR BEAM-TO-COLUMN JOINTS IN STEEL MOMENT RESISTING FRAMES

Ανδρόνικος Σκιαδόπουλος
Μεταδιδακτορικός Ερευνητής
École Polytechnique Fédérale de Lausanne (EPFL)
Λωζάνη, Ελβετία
Ηλεκτρονική αλληλογραφία: andronikos.skiadopoulos@epfl.ch

Ahmed Elkady
Επίκουρος Καθηγητής
University of Southampton
Σαουθάμπτον, Ηνωμένο Βασίλειο
Ηλεκτρονική αλληλογραφία: a.elkady@soton.ac.uk

Δημήτριος Λιγνός
Καθηγητής
École Polytechnique Fédérale de Lausanne (EPFL)
Λωζάνη, Ελβετία
Ηλεκτρονική αλληλογραφία: dimitrios.lignos@epfl.ch

1. ΠΕΡΙΛΗΨΗ

Ο ικανοτικός σχεδιασμός μεταλλικών καμπτικών πλαισίων επιβάλλει τον ελαστικό σχεδιασμό των κόμβων δοκού-υποστηλώματος. Πειραματική και αριθμητική έρευνα έχει δείξει ότι οι κόμβοι δοκού-υποστηλώματος προσφέρουν ευσταθή κύκλο υστέρησης όταν υποβάλλονται σε σεισμική διέγερση. Για να αξιοποιηθεί αυτή η ευσταθής συμπεριφορά, η ύπαρξη ενός προσομοιώματος συμπεριφοράς των κόμβων είναι απαραίτητη. Η σύγκριση διαθέσιμων προσομοιωμάτων συμπεριφοράς δοκού-υποστηλώματος στην βιβλιογραφία με πάνω από 100 πειραματικά δεδομένα που συλλέψαμε συστηματικά δείχνουν ότι το προσομοιώμα του Ευρωκώδικα υπερεκτιμά την διαθέσιμη διατμητική αντοχή κόμβων κατά 60%. Πέραν της αντοχής, η δυσκαμψία του κόμβου δοκού-υποστηλώματος υπερεκτιμάται κατά 20%, καθώς η καμπτική παραμόρφωση δεν λαμβάνεται υπόψη. Στο παρόν άρθρο, προτείνουμε ένα νέο προσομοίωμα συμπεριφοράς δοκού-υποστυλώματος για τον αντισεισμικό σχεδιασμό καμπτικών πλαισίων, το οποίο βασίζεται σε ρεαλιστικές διατμητικές τάσεις εντός του κόμβου, με βάση παραμετρικές αναλύσεις με συνεχή πεπερασμένα στοιχεία. Η προτεινόμενη σχέση για τον προσδιορισμό της δυσκαμψίας και της διατμητική αντοχής του κόμβου δοκού-υποστηλώματος έχουν σφάλμα λιγότερο από 10% και μειωμένη διασπορά σε σύγκριση με τις υπάρχουσες σχέσεις του Ευρωκώδικα 3. Συμπληρωματικές αναλύσεις με χρήση πεπερασμένων στοιχείων υποδηλώνουν ότι, όταν η χρήση συγκολλητών ελασμάτων στην περιοχή του κόμβου δοκού-υποστυλώματος είναι αναγκαία, τότε η διατμητική αντοχή του θα πρέπει να υπολογίζεται με βάση το συνολικό πάχος του κορμού του υποστυλώματος καθώς και αυτό των ελασμάτων.