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A mathematical steel panel zone model for flanged cruciform columns

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Abstract. Cruciform sections are an appropriate option for columns of orthogonal moment resisting frames for equal bending strength and stiffness about two main axes and the implementation is easier for continuity plates. These columns consist of two I-shaped sections, so that one of them is cut out in middle and two generated T-shaped sections be welded into I-shaped profile. Furthermore, in steel moment frames, unbalance moment at the beam-column connection leads to shear deformation in panel zone. Most of the obtained relations for panel zone strength derived from experimental and analytical results are on I-shaped columns with almost thin flanges. In this paper, a parametric study has been carried out using Finite Element Method (FEM) with effective parameters at the panel zone behavior. These parameters consist of column flange thickness, column web thickness, and thickness of continuity plates. Additionally, a mathematical model has been suggested to determine strength of cruciform column panel zone and has been shown its accuracy and efficiency.

Keywords: panel zone; shear strength; cruciform column; FEM; beam-column connection

1. Introduction

Moment resisting frames (MRF) are one of the widely used lateral load resisting systems that resist lateral forces through the flexural and shear strength of the beams and columns. MRF plays a crucial role in architectural and functional versatility as well. On the other hand, in these systems, column performance has special importance as the main part of tolerating resistant demands of seismic loadings in both directions. To provide sufficient strength and stiffness in two orthogonal directions, using sections with similar behavior about two main axes seem essential for the column. Cruciform sections are known as sections whose behavior is the same in both directions (see Fig. 1). Moreover, beam to column connection is one of the most important parts in MRFs and the most dominant factor in a MRF is the transferring condition of the moment between parts of the frame. Panel zone is a column region which is surrounded by the continuity plates and column flanges as shown in Fig. 2. A panel zone can undergo inelastic deformations during an earthquake, which can participate in the energy dissipation capacity of a MRF.

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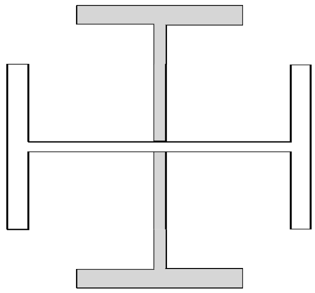


Fig. 1 Column with cruciform section

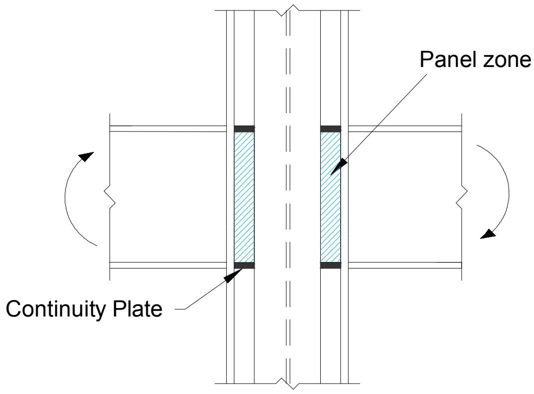


Fig. 2 Panel zone in steel frames

Seismic behavior of the panel zone has drawn attention of many researchers for a long time. Researches in this regard, have begun since late 60s and early 70s, and regulations and guidelines during these years have represented different relations concerned with behavior of panel zone. The performed studies have been focused on seismic behavior of panel zone in I-shaped columns. Krawinkler *et al*. (1971), Bertero *et al*. (1973) and Popov (1987) demonstrated that panel zone has high strength after yielding. High deformable, stable hysteresis loops and high cyclic re-hardening were observations and findings of these researchers.

In addition, experiments carried out on panel zone revealed that shear distortions have a remarkable effect on its behavior (Jin and El-Tawil 2005). In the case of preventing from local crippling and yielding of column web and distortion of column flange, the panel zone can have good features of energy dissipation in shear till reaching great inelastic distortions (Saffari *et al*. 2013a). Yielding mainly initiates in the middle and then extends radially throughout the panel zone (Jin and El-Tawil 2005). Subsequently, shear distortion is maximum at the center of panel zone and minimum at the corners.

Generally, when a connection subjected to unbalanced bending moments, a complex mode of stress occurs in a panel zone. This includes normal stresses which are basically due to axial forces applied to the column, and shear stress resulting from moment transferred from the beams. Experimental studies have shown that panel zone behavior in elastic range is mainly a function of shear distortions (Krawinkler *et al*. 1971 and Becker 1975). After yielding, shear strength decreases substantially and bounding frame of panel zone which is formed by column flanges and continuity plates will provide extra strength. Based on the experimental observations, it will be logical to assume that full yielding occurs in a level of distortions which is four times of the first one (Krawinkler *et al*. 1971).

Most of experiments on panel zone have been conducted on I-shaped columns. Accordingly, mathematical equations to determine the yield shear strength and ultimate shear capacity of the panel zone have been proposed (Krawinkler and Mohasseb 1987, Fielding and Huang 1971, Mansouri and Saffari 2014). Comparison of experimental and analytical results suggest that even for I-shaped columns with more than 2.5 cm flange thickness, these models are not sufficiently accurate (Saffari *et al*. 2013b).

Nasrabadi *et al*. (2013) have conducted a comprehensive survey on improving the performance of cruciform column panel zone. They proposed a new panel zone configuration for flanged cruciform columns by a practical solution. Concerning the proposed detail, new behavior models and design equations were suggested by them.

In this study a comprehensive mathematical model is suggested to determine the strength of cruciform column panel zone. A comparison between the results of proposed method herein with FEM analyses shows the accuracy and efficiency of the proposed model.

2. Mathematical models for panel zones of H-shaped columns

Mathematical models for panel zone are usually based on equivalent shear forces applied on the panel zone (see Fig. 3). The available boundary forces of panel zone can be transformed into an approximate equivalent shear force as follows

$$V_{eq}=\frac{M_{bl}+M_{br}}{d_b\text{-}t_{bf}}-\frac{V_{ct}+V_{cb}}{2} \tag{1}$$

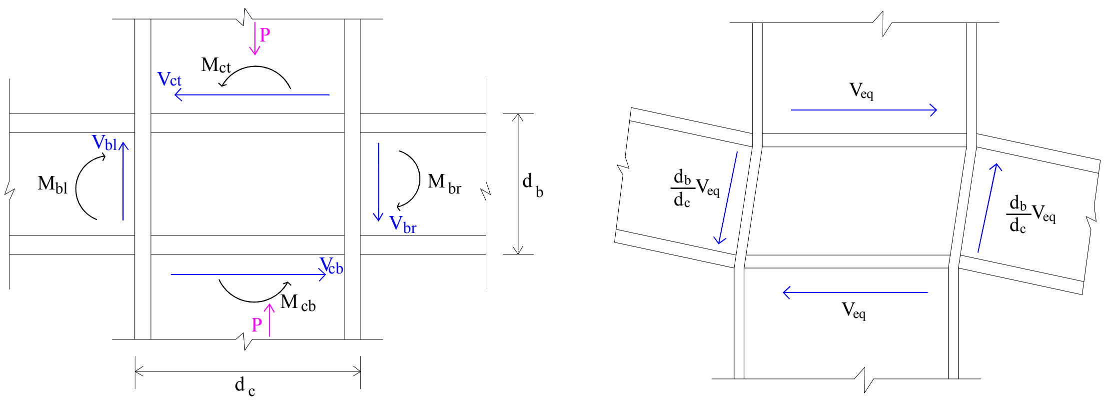
Where t_{bf} is beam flange thickness, d_b is the beam depth, V_{ct} is shear in upper column, V_{cb} is shear in lower column, M_{bl} is the moment in left beam, M_{br} is the moment in right beam and d_b is beam depth. In order to obtain shear forces in column sections outside panel zones, it is usually assumed that inflection points occur in the middle of the top and the bottom columns. Therefore, Eq. (2) can be arranged as follows

$$V_{eq}=\frac{M_{bl}+M_{br}}{d_b\text{-}t_{bf}}(1-\rho), \qquad M^{\rho\alpha}\frac{(1\text{ - }\rho)}{d_b-t_{bf}} \tag{2}$$

Where $\rho=\dfrac{d_b-t_{bf}}{H_c}$, $M^{\rho\alpha}=M_{bl}+M_{br}$ is moment of panel zone and H_c is the average height of the top and the bottom columns.

2.1 Linear range

The mathematical relation between the shear force of panel zone, V, and the shear deformation



(a) Panel boundary forces (b) Equivalent panel shear forces

Fig. 3 Equivalent panel shear forces and panel boundary forces (Kim and Engelhardt 2002) of panel zone, γ, could be changed into the relation of the panel zone moment, $M^{\rho\alpha}$, and panel zone shear deformation, γ, by means of Eq. (2).

$$M^{\rho\alpha} = V_{eq} \frac{d_b - t_{bf}}{(1-\rho)} \tag{3}$$

By replacing the relation between shear force and shear deformation of the panel zone, $V_{eq} = \tau A_{eff} = G\gamma A_{eff}$, in Eq. (3), the following equation would be conceived, which is accepted among most researchers for calculating the panel zone hardness in the linear range (Krawinkler *et al*. 1971, Fielding and Huang 1971, Wang 1988)

$$K_e = \frac{M_y^{\rho\alpha}}{\gamma} = \frac{G\, A_{eff}\left(d_b - t_{bf}\right)}{(1-\rho)} \tag{4}$$

Where G is the Shear modulus of elasticity and A_{eff} is the effective shear area whose value in *H*-shaped columns is $(d_c - t_{cf})t_{cw}$ by the suggestion of Fielding and Huang 1971 and Krawinkler *et al.* (1971), and $(d_c - 2t_{cf})t_{cw}$ by the suggestion of Wang (1988). So that, d_c is the column depth, t_{cf} is the column flange thickness and t_{cw} is the column web thickness. The panel zone yield moment $M_y^{\rho\alpha}$ can be calculated as follows

$$M_y^{\rho\alpha} = \frac{\bar{\tau}_y\, A_{eff}\left(d_b - t_{bf}\right)}{(1-\rho)} \tag{5}$$

In which $\bar{\tau}_y$ is the yield shear stress of the column web which could be calculated based on Von Mises criterion as following

$$\bar{\tau}_y = \frac{\sigma_y}{\sqrt{3}} \sqrt{1 - \left(\frac{P}{P_y}\right)^2} \tag{6}$$

Where P and P_y are axial force and axial yield strength of the column, respectively, and σ_y is yield stress of the column web.

2.2 Non-linear range

Fielding and Huang (1971) have proposed a bi-linear relationship for the nonlinear considerations, which includes elastic stiffness, K_e, and post-elastic stiffness, K_1, as shown in Fig. 4.

Post-elastic stiffness, K_1, in the abovementioned bi-linear relation is suggested as follows

$$K_1 = \frac{5.2\, G\, b_{cf}\, t_{cf}^{\;3}}{d_b\,(1-\rho)} \tag{7}$$

Where t_{cf} and b_{cf} are thickness and width of the column flange, respectively, and d_b is the beam depth.

Krawinkler *et al*. (1971) have proposed $M^{\rho\alpha} - \gamma$ tri-linear relation, including elastic stiffness, K_e, which is followed by two linear post-elastic stiffness factors, K_1 and K_2 (Fig. 5).

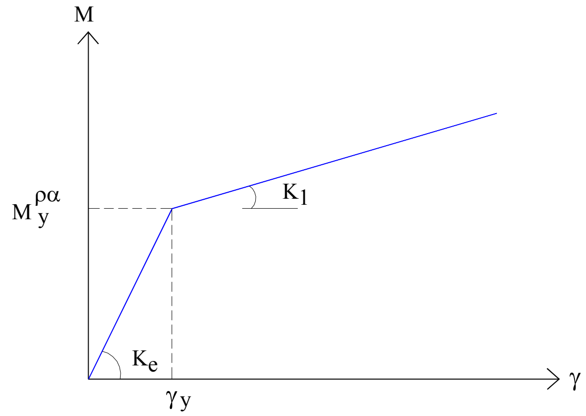


Fig. 4 Bi-linear relationship for panel zone (Fielding and Huang 1971)

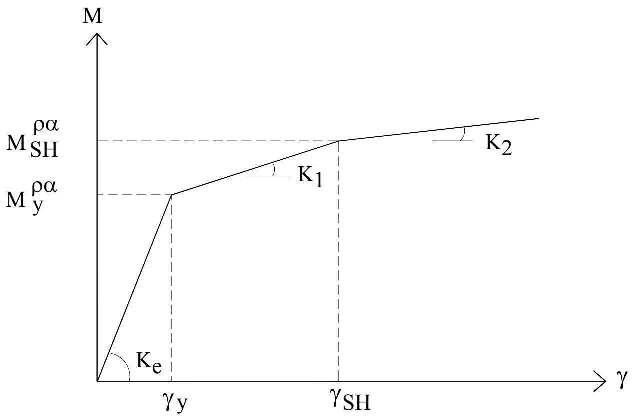


Fig. 5 Tri- linear relationship for panel zone (Krawinkler *et al*. 1971)

Relations for post-elastic stiffness K_1 and moment $M_{SH}^{\rho\alpha}$ was provided by them as well

$$K_1 = \frac{1.04\ G\, b_{cf}\ t_{cf}^{\ 2}}{1-\rho} \tag{8}$$

$$K_1 = \frac{1.04\ G\, b_{cf}\ t_{cf}^{\ 2}}{1-\rho} \tag{9}$$

Additionally, they presumed that strain hardening begins at $\gamma_{SH} = 4\gamma_y$.
Stiffness of the strain hardening part, K_2, has been proposed in their study as following

$$K_2 = \frac{G_{st}\ A_{eff}\ d_b}{1-\rho} \tag{10}$$

Where G_{st} is the shear module of strain hardening.

3. Panel zone shear strength capacity in the AISC 2010

In seismic regulations of AISC (2010), which is based on LRFD method, design strength of panel zone in H-shaped column regarding whether panel zone deformation is considered or not, and based on the axial force applied to the column, is defined as follows:

(a) When the effect of panel zone deformation on frame stability is not considered in the analysis:

(i) for $P_u \leq 0.4\, P_c$

$$R_n = 0.6\, F_y\, d_c\, t_{cw} \tag{11}$$

(ii) for $P_u > 0.4\, P_c$

$$R_n = 0.6\, F_y\, d_c\, t_{cw}\left(1.4 - \frac{P_u}{P_c}\right) \tag{12}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

(i) for $P_u \leq 0.75\, P_c$

$$R_n = 0.6\, F_y\, d_c\, t_{cw}\left(1 + \frac{3 b_{cf}\, t_{cf}^{\,2}}{d_b\, d_c\, t_{cw}}\right) \tag{13}$$

(ii) for $P_u > 0.75\, P_c$

$$R_n = 0.6\, F_y\, d_c\, t_{cw}\left(1 + \frac{3 b_{cf}\, t_{cf}^{\,2}}{d_b\, d_c\, t_{cw}}\right)\left(1.9 - \frac{1.2 P_u}{P_c}\right) \tag{14}$$

Where P_u is the column axial force, P_c is column axial strength and F_y is yield stress of the column web. Other parameters are the same as mentioned before.

4. Finite element model of cruciform column panel zone

4.1 Modeling process

To achieve an appropriate model, a parametric study regarding the effective parameters on the behavior of panel zone is carried out by ABAQUS (2013) software, in the first place. These parameters consist of column flange thickness (t_{cf}), column web thickness (t_w), and thickness of continuity plates (t_{cp}). Since experimental results on seismic performances of cruciform columns do not exist in pre-qualified connections data-base, the results of a well-known experimental program on “SP7 of SAC01” (Lee *et al*. 2000), are considered in order to validate the modeling accuracy.

All parametric studies were performed for CSP3, CSP5 and CSP7 specimens whose columns are constructed by two W- shape sections as shown in Table 1. It should be noted that column

Table 1 Columns sections (all dimension in mm)

Specimen	Section	Flange width	Flange thickness	Web thickness	Outside height
CSP3	W21×101	312.42	20.32	12.7	543.56
CSP5	W27×146	355.6	24.765	15.367	695.96
CSP7	W33×201	398.78	29.21	18.161	855.98

sections of CSP3, CSP5 and CSP7 are selected from equalization of their plastic capacity with SP3, SP5 and SP7 column sections of SAC01 (Lee *et al*. 2000), respectively. This is for using the experimental results of SAC01 (Lee *et al*. 2000), for verifying the finite element modeling methodology and general assumptions on the material behavior and nonlinear analysis in the mentioned study. Furthermore, to avoid yielding in beams before yielding in panel zone, beam sections used in CSP3, CSP5 and CSP7 are selected in such a way that yielding in panel zone precedes beams yielding. Column sections of CSP3, CSP5 and CSP7 specimens are presented in Table 1.

The Young's modulus of elasticity, E, and Poisson's ratio, v, were assumed to be 200 GPa and 0.3, respectively. Stress-strain diagram of steel is considered bi-linear (Lee *et al*. 2000) as seen in Fig. 6. For all specimens, beam length and column length are 342.9 and 365.8 cm, respectively. Other geometric parameters of these specimens are available in Table 2. Both the shear tab and continuity plates were ASTM A36 (yield stress = 250 MPa), and E70TG-K2 electrodes were used for welds.

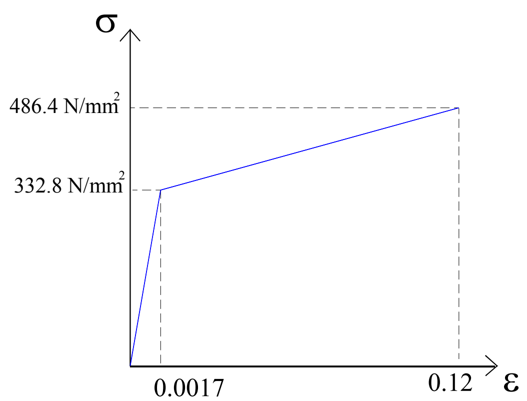


Fig. 6 Stress-strain diagram of steel (Lee *et al*. 2000)

Table 2 Geometric parameters of specimens

Specimen	Shear tab (mm)	No. of A325 SC bolts (mm)	Continuity plate (mm)	Weld type and size (mm)	
				Beam flange	Shear tab
CSP3	400×127×10	6φ22	270×270×16	CJP, root opening = 9 mm, Angle = 30° and E70TG-K2	Fillet, 8 mm, E70T-7
CSP5	610×127×13	8φ25	375×375×19		Fillet, 8 mm, E70T-8
CSP7	765×127×16	10φ25	455×455×25		Fillet, 8 mm, E70T-7

Quadrilateral four-node shell elements (the S4R element) are used for constructing three-dimensional models of subassemblies. Shell element has been taken into account successfully in several studies (El-Tawil *et al*. 1998, Saffari *et al*. 2015 and Hedayat *et al*. 2013). Concerning the assessment of panel zone behavior to be the main objective of this study, the specification of the beam and connection are defined solid to guarantee yielding in panel zone region. In the finite element model, the bolts were not modeled exactly; however, shear tab, bolt holes and interaction between the shear tab and the beam web were modeled to achieve a realistic model. In order to model the interactions between welded component parts, tie constraints are used, so that no relative motion between the surfaces in contact would be possible. It should be noted that other parts of connection like beam, column and continuity plates are merged in ABAQUS software (2013).

To reduce the computational efforts, dense meshes have been used in the panel zone region while the other regions have coarse meshes. Column flanges are modelled in 5 layers of elements. Imperfection value is considered with a factor of 1% of the beam flange thickness from first buckling mode and the supports of column ends are considered hinged (as in experiments). The free end of beam moves vertically under displacement control analysis. (see Figs. 7 and 8)

For producing the models, thickness value for specimens column flange, t_{cf}, is 0.75, 1, 1.25, and 1.5 times of reference specimens, t_{cf0}. Likewise, thickness of column web specimens, t_{cw}, is equal to 0.75, 1, 1.25, and 1.5 times of reference specimens, t_{cw0}. Moreover, to analyze the effect of continuity plate's thickness on panel zone behavior, the reference continuity plates thickness, t_{cp0}, has been multiplied by the values of 0.75, 1, 1.25, and 1.5. Therefore, total numbers of produced specimens in ABAQUS (2013) are:

192 specimens = (4 continuity plate thicknesses) × (4 column web thicknesses) × (4 column flange thicknesses) × (3 specimens (CSP3, CSP5 and CSP7)).

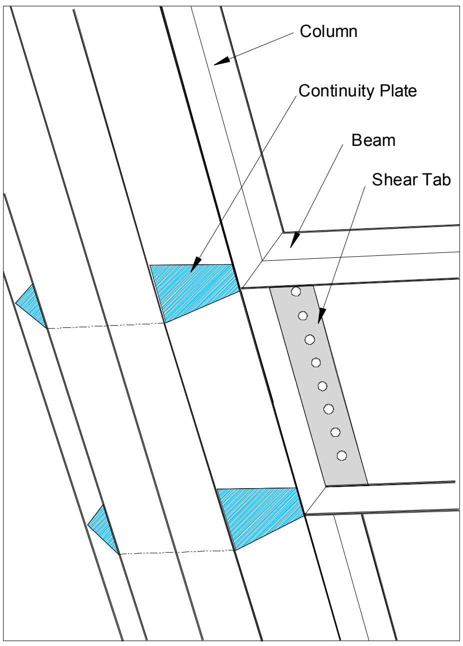


Fig. 7 Sample connection detail

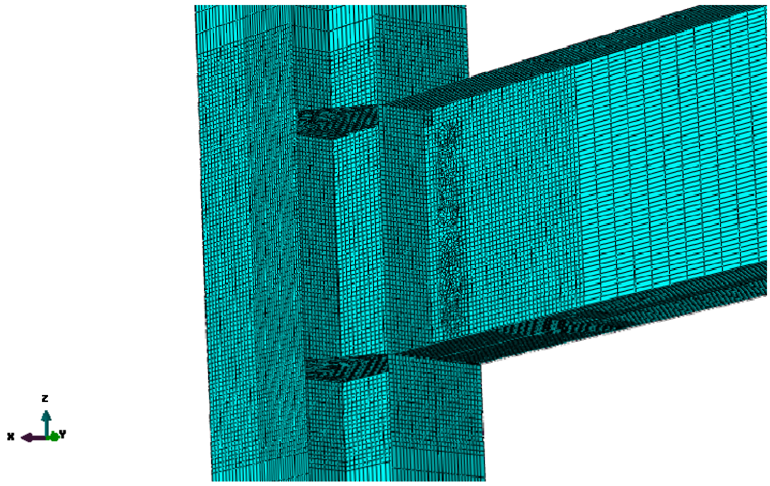


Fig. 8 Finite element modeling

4.2 Shear computing method

To obtain panel zone shear force the following relation considered (Brandonisio *et al*. 2012)

$$V_{pz} = \frac{M_b}{h_t}(1-\rho) \tag{15}$$

Where $h_t = d_b - t_{bf}$, $\rho - \dfrac{h_t}{H - d_b}$.

Where in latter equations, t_{bf} is beam flange thickness, d_b is the depth of the beam cross section, M_b is the moment in the beam and H is the average value of the story heights.

4.3 Computing panel zone distortion

The proposed relation by Ricles *et al*. (2004) is used to calculate the panel zone distortion

$$\gamma = \frac{\Delta^+ - \Delta^-}{2}\frac{\sqrt{d_{pz}^2 + b_{pz}^2}}{d_{pz}\, b_{pz}} \tag{16}$$

Where Δ^+ and Δ^- diagonal deformations of panel zone, and d_{pz} and b_{pz} are vertical and horizontal dimension of panel zone, respectively (see Fig. 9).

4.4 Verification of study

As validation is essential in numerical studies, specimen SP7 (Lee *et al*. 2000), is modeled by the ABAQUS software and compared with experimental results, before the main study in this research is being carried out. As seen in Fig. 10, results of the SP7 specimen modeling in ABAQUS software are in a good agreement with the experimental results.

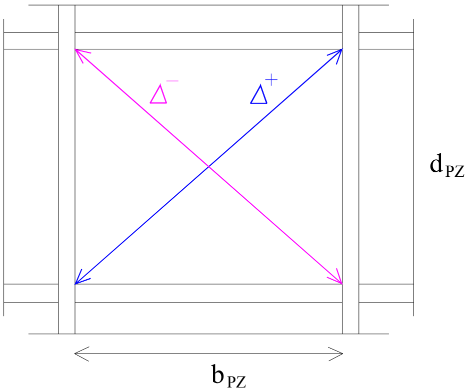


Fig. 9 Geometry of panel zone to determine panel zone distortion (Ricles *et al*. 2004)

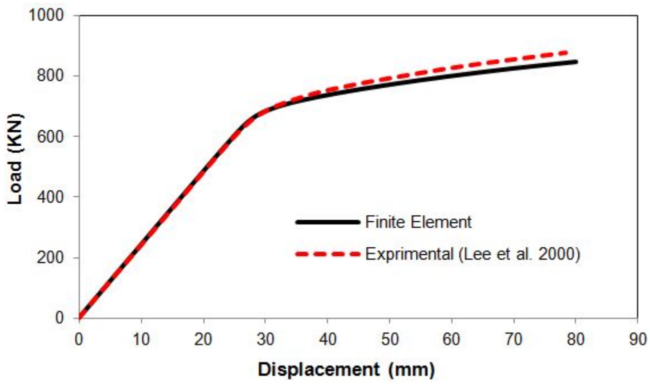


Fig. 10 Comparing experimental results and finite element modeling results for specimen SP7

5. Proposed analytical model

5.1 Proposing a new relation for yield capacity of panel zone

The effective shear area in cruciform section is shown by black, dotted and crossed areas in Fig. 11. The portion of the shear force resisted by the web for a cruciform section can be determined as follows.

To calculate shear portion of web, shear in flanges is calculated first, and next, shear portion of web is achieved by deducting this quantity from total shear

$$V = V_1 + V_2 + V_3 \rightarrow V_3 = V - (V_1 + V_2) \tag{17}$$

Where V_1 is shear portion of the upper and the lower flanges (crossed area), V_2 denotes shear portion of flanges parallel to web (dotted area), V_3 is shear portion of web (black area), and V represents total shear exerted to cross section. It should be noted that the shear portion at the web perpendicular to shear force is neglected.

Shear portion of each part can be computed as follows

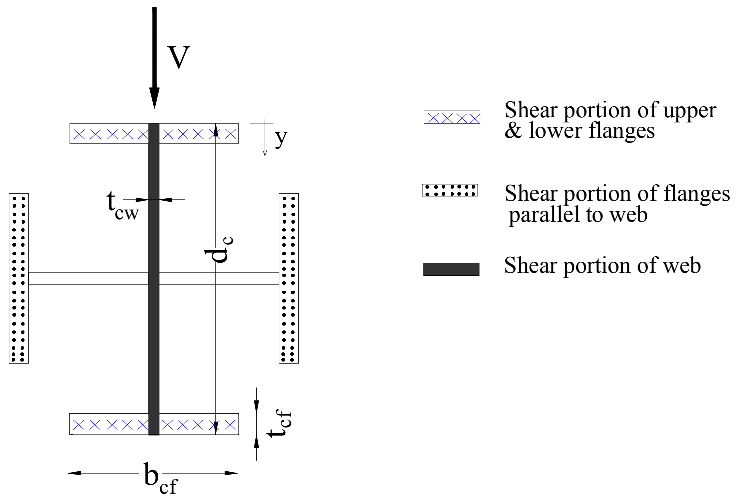


Fig. 11 Portion of shear force resisted by web and flanges

$$V_i = \frac{V}{It} \int Q_i \ dA_i \tag{18}$$

The shear portion of upper and lower flanges, could be determined as follows

$$V_1 = \frac{V}{I\, b_{cf}} \int_0^{t_{cf}} Q_1 \ dA_1 \tag{19}$$

In which

$$Q_1 = b_{cf}\ y \left(\frac{d_c\text{-}y}{2}\right), \qquad dA_1 = \left(\frac{b_{cf} - t_{cw}}{2}\right) dy$$

Therefore, by assuming $\alpha = \frac{t_{cf}}{d_c}$, $\beta = \frac{b_{cf}}{d_c}$, and after computing and simplifying the above equation V_1 can be presented as follows

$$V_1 = V \frac{\left(3\alpha - 2\alpha^2\right)}{3 + \beta^2} \tag{20}$$

Similarly, by means of above relations (Eq. (18)), V_2 can be determined as follows

$$V_2 = V \frac{\beta^2}{3 + \beta^2} \tag{21}$$

Finally, the shear portion of web can be written as

$$V_3 = V - (V_1 + V_2) = \lambda V \tag{22}$$

Where $\lambda = 1 - \frac{\left(3\alpha - 2\alpha^2\right) + \beta^2}{3 + \beta^2}$

In addition, according to Von Mises criterion for shear

$$3\tau^2 = F_y^2 \tag{23}$$

Moreover

$$\tau_y = F_y / \sqrt{3}, \quad \tau_y = G\gamma_y \quad \rightarrow \quad \gamma_y = F_y / \sqrt{3}G \tag{24}$$

Thus

$$\tau_y = V_{0y} / A_{eff} \quad \rightarrow \quad V_{0y} = 0.6F_y \, d_c \, t_{cw} \tag{25}$$

The proposed relation for modification of yield capacity of panel zone is as follows

$$\lambda \, V_y = V_{0y} \tag{26}$$

Where V_{0y} is the one presented in AISC (2010)

$$V_{0y} = 0.6 \, F_y \, d_c \, t_{cw} \tag{27}$$

Finally, the shear yield strength of cruciform section is

$$V_y = \frac{1}{\lambda} \, 0.6 \, F_y \, d_c \, t_{cw} \tag{28}$$

5.2 Proposing a new relation for ultimate capacity of panel zone

In order to determine the ultimate capacity of panel zone by assuming that this resistance happens in $\gamma_P = 4\gamma_y$ (Krawinkler *et al.* 1971), one can say that

$$V_p = V_y + 3\gamma_y \, K_p \tag{29}$$

In which, K_p is (Krawinkler and Mohasseb 1987)

$$K_p = \frac{1.095 b_{cf} \, t_{cf}^2 \, G}{d_b} \tag{30}$$

Eventually, after simplification and by means of obtained results of finite element simulation on 192 samples, the ultimate capacity of cruciform column panel zone can be presented as follows

$$V_p = 0.6F_y \, d \, t_{cw} \, \rho \tag{31}$$

Where

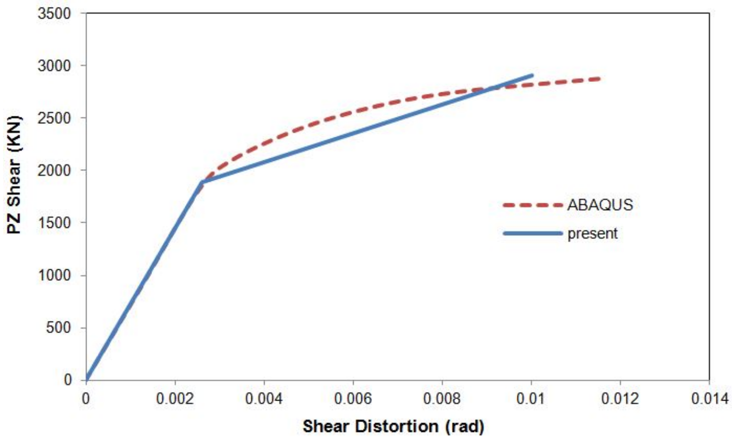


Fig. 12 Comparison of present model for CSP3

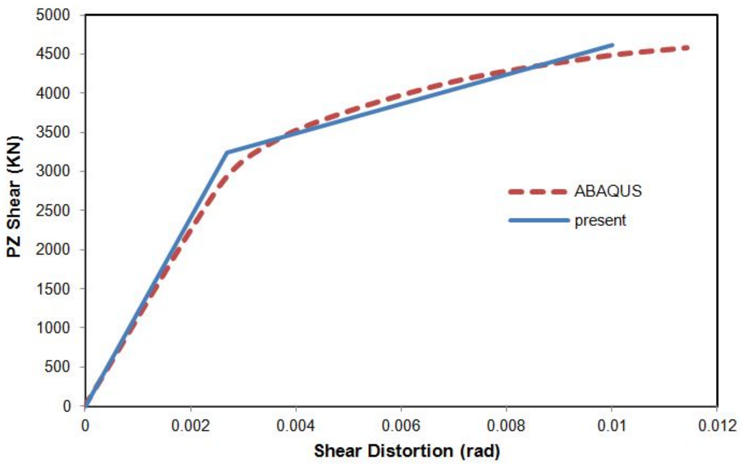


Fig. 13 Comparison of present model for CSP5

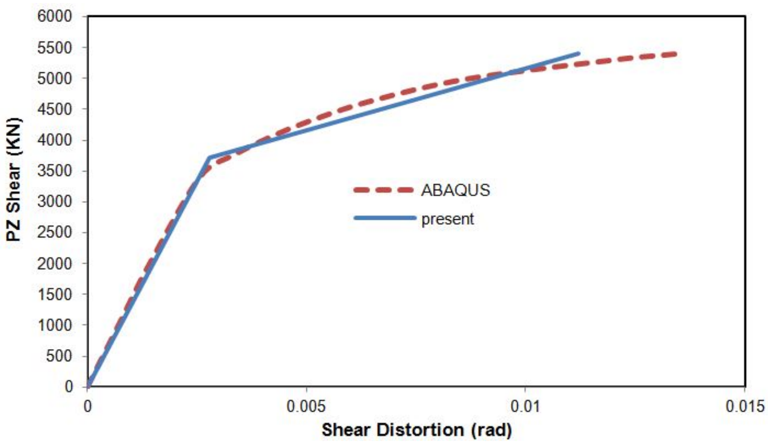


Fig. 14 Comparison of present model for CSP7

$$\rho = \frac{1}{\lambda} + \frac{3.17\, b_{cf}\, t_{cf}^2}{d_b\, d_c\, t_{cw}} \tag{32}$$

Figs. 12-14 display the comparison between the obtained results of the proposed relation in this paper, and the results of finite element simulation for SP3, SP5 and SP7 cruciform specimens.

As seen from Figs. 12 to 14, new model proposes reasonable results for a wide range of column flange thickness. Additionally, finite element analysis reveals that the thickness of continuity plates has slight efficacy on shear capacity of panel zone.

In order to present all of the results of finite elements simulation, changes of column flange thickness in non-dimensional yield capacity (V_y/V_{ABAQUS}) for CSP3, CSP5 and CSP7 are shown in Figs. 15-17.

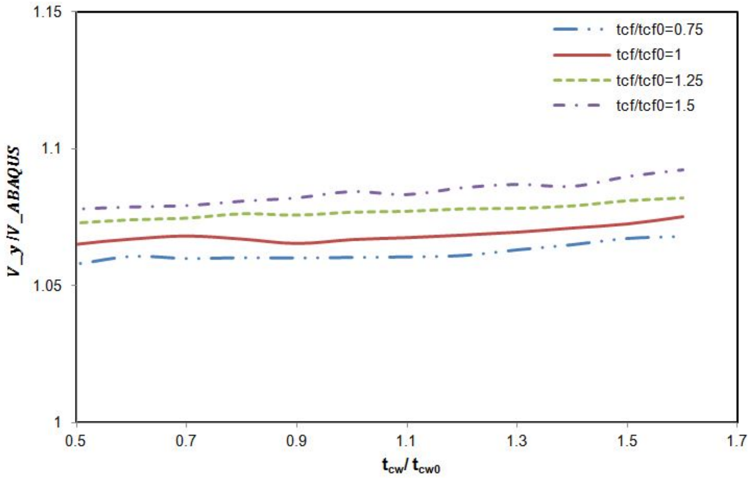


Fig. 15 Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from CSP3

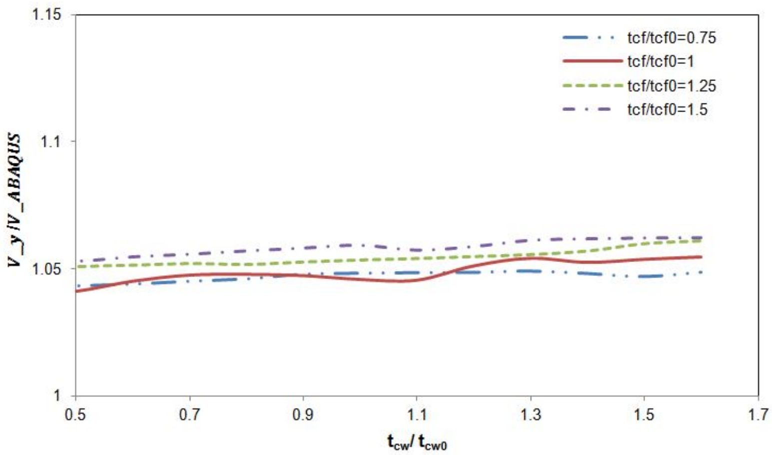


Fig. 16 Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from CSP5

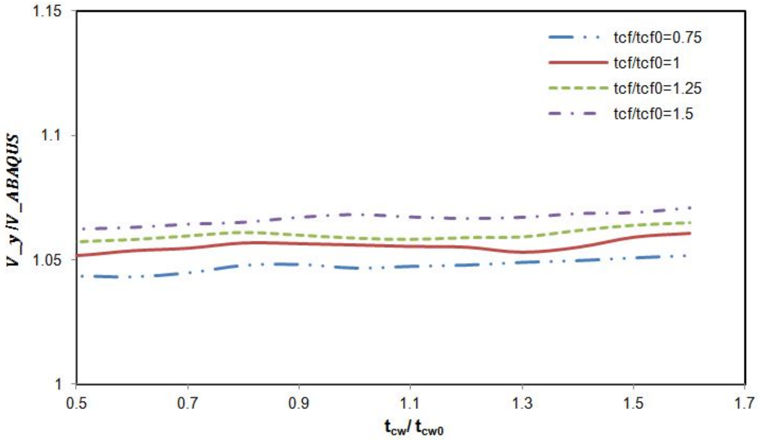


Fig. 17 Variations of column flange thickness in non-dimensional shear yield strength of panel zone in generated specimens from CSP7

Table 3 Error in 192 finite element samples for estimating yield capacity of panel zone

Error	Proposed model
Average error (%)	6.25
Max error (%)	9.24

Table 4 Error in 192 finite element samples for estimating ultimate capacity of panel zone

Error	Proposed model
Average error (%)	8.13
Max error (%)	11.08

As demonstrated in figures above, general trend of diagrams is upward which seems reasonable, as thickness of flange and web in manufactured specimens is growing. This leads to increasing the value of (V_y/V_{ABAQUS}) for different specimens and increases error of the proposed relation in estimating yield shear capacity. Nevertheless, the error rate is always in acceptable range whose value will be presented in following states.

5.3 Comparing the accuracy of different methods

Tables 3 and 4 indicate the accuracy of the presented relations for estimating yield capacity and ultimate capacity of panel zone. As seen from these tables the introduced model in this study has a good performance.

6. Conclusions

The aim of this study is introducing a mathematical model to define panel zone behavior of cruciform column section. For this purpose to estimate yield strength and ultimate capacity of panel zone, new relations are presented. The proposed model considers only the shear deformation mode of panel zone and includes all elastic and inelastic ranges of behavior. The accuracy of the relationships is evaluated by modeling cruciform columns using ABAQUS software and performing nonlinear finite element analysis on a wide range of thicknesses and dimensions. This parametric study is conducted in which effective factors on cruciform column panel zone such as thickness of column web and flange, and thickness of continuity plates are considered. Results showed that proposed equations are compatible with the results of finite element simulations which reveal the accuracy, simplicity and efficiency of the proposed model.

Hence it might be concluded that among the three parameters, namely column flange thickness, column web thickness and continuity plate thickness, variations of continuity plate thickness has slight influence on shear capacity of panel zone, whereas flange and web thicknesses are of higher efficacy on shear capacity of panel zone. Furthermore, as thickness of flange and web in specimens is growing, error of the proposed relation in estimating yield shear capacity is increased. Nevertheless, the error rate is always in an acceptable range. It should be noted that to verify the behavior and shear capacity of cruciform columns panel zone more analytical, numerical and experimental studies should be conducted.

References

American Institute of Steel Construction (AISC) (2010), Specification for Structural Steel Buildings, Chicago, IL, USA.

Becker, R. (1975), “Panel zone effect on the strength and stiffness of steel rigid frames”, *Eng. J., AISC*, **12**(1), 19-29.

Bertero, V.V., Krawinkler, H. and Popov, E.P. (1973), “Further studies on seismic behavior of steel beam-to-column subassemblages”, EERC Rep. No. 73-27; University of California, Berkeley, CA, USA.

Brandonisio, G., De Luca, A. and Mele, E. (2012), “Shear strength of panel zone in beam-to-column connections”, *J. Construct. Steel Res.*, **71**, 129-142.

El-Tawil, S., Mikesell, T., Vidarsson, E. and Kunnath, S.K. (1998), “Strength and ductility of FR welded-bolted connections”, Report No. SAC/BD-98/01; SAC Joint Venture.

Fielding, D.J. and Huang, J.S. (1971), “Shear in steel beam–to–column connections”, *Welding Journal*, **50**(3), 313-326.

Hedayat, A.A., Saffari, H. and Mousavi, M. (2013), “Behavior of steel reduced beam web (RBW) connections with arch-shape cut”, *J. Adv. Struct. Eng.*, **16**(10) 1644-1662.

Jin, J. and El-Tawil, S. (2005), “Evaluation of FEMA-350 seismic provisions for steel panel zones”, *J. Struct. Eng.,ASCE*, **131**(2), 250-258.

Kim, K.D. and Engelhardt, M.D. (2002), “Monotonic and cyclic loading models for panel zones in steel moment frames”, *J. Construct. Steel Res.*, **58**(5-8), 605-635.

Krawinkler, H. and Mohasseb, S. (1987), “Effects of panel zone deformations on seismic response”, *J. Construct. Steel Res.*, **8,** 233-250.

Krawinkler, H., Bertero, V.V. and Popov, E.P. (1971), “Inelastic behavior of steel beam-to-column subassemblages”, EERC Rep. No. 71-7; University of California, Berkeley, CA, USA.

Lee, K.H., Stojadinovic, B., Goel, S.C., Margarin, A.G., Choi, J., Wongkaew, A., Reyher, B.P. and Lee, D.Y. (2000), “Parametric tests on unreinforced connections”, SAC Background Document, SAC/BD-00/01; SAC Joint Venture, Richmond, CA, USA.

Mansouri, I. and Saffari, H. (2014), “A new steel panel zone model including axial force for thin to thick column flanges”, *Steel Compos. Struct., Int J.*, **16**(4), 417-436.

Nasrabadi, M.M., Torabian, S. and Mirghaderi, S.R. (2013), “Panel zone modelling of flanged cruciform columns: An analytical and numerical approach”, *Eng. Struct.*, **49**, 491-507.

Popov, E.P. (1987), "Panel zone flexibility in seismic moment joints", *J. Construct. Steel Res.*, **8**, 91-118.

Popov, E.P. and Takhirov, S. (2001), "Experimental study of large seismic steel beam–to–column connections", PEER–2001/01; Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, USA.

Ricles, J.M., Zhang, X., Lu, L.W. and Fisher, J.W. (2004), "Development of seismic guidelines for deep-column steel moment connections", ATLSS Report No. 04-13.

Saffari, H., Hedayat, A.A and Nejad, M.P. (2013a), "Post-Northridge connections with slit dampers to enhance strength and ductility", *J. Construct. Steel Res.*, **80**, 138-152.

Saffari, H., Hedayat, A.A. and Goharizi, N.S. (2013b), "Suggesting double-web I-shape columns for omitting continuity plates in a box column", *Steel Compos. Struct., Int J.*, **15**(6), 585-603.

Saffari, H., Hedayat, A.A. and Goharrizi, N.S. (2015), "New alternatives for continuity plates in I-beam to box columns connections", *Asian J. Civil Eng.* (*BHRC*), **16**(2), 219-233.

SIMULIA. ABAQUS (2013), Analysis and theory manuals, SIMULIA, The Dassault Systèmes, Realistic Simulation, Providence, RI, USA.

Wang, S.J. (1988), "Seismic response of steel building frames with inelastic joint deformation", Lehigh University, Bethlehem, PA, USA.

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