# PANEL ZONE MODELLING OF BOX COLUMNS: AN ANALYTICAL AND NUMERICAL APPROCH

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**ABSTRACT:** Panel zone is a region of the column which is bounded by the column flanges and continuity plates and plays a vital role in determining stiffness and capacity of the frame. Despite the extensive researches conducted on the panel zone behavior, there are different viewpoints on how panel zones function. Since box columns are used widely in the moment-resisting frames, presentation of a mathematical model for the panel zone behavior for these columns seems essential. This study attempts to present some relations in order to determine the shear capacity, the linear stiffness and non-linear stiffness of the panel zones in the box columns. For this purpose, a comprehensive parametric study is conducted, using the ABAQUS software; accordingly, a mathematical model is introduced. Then, the results of the mathematical model are compared with the results of other sources, and the accuracy of the obtained results of the model is proved.

**Keywords:** Mathematical model, panel zone, box column, moment-resisting frame, steel structures

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## 1. INTRODUCTION

Moment resisting frames (MRFs) are one of the frequently used lateral load resisting systems that resist lateral forces through the flexural and shear strength of the beams and columns. One of the most important components of the MRFs is their connections; furthermore, the most prominent component of a flexural rigid connection is how to transfer the moment between the beam and the column. A rectangular zone of the column web, which is surrounded between the continuity plates and the column flanges has a significant role in the connection behavior; this zone is called the panel zone (see Figure. 1).

In lateral loadings like seismic conditions, the connection of the beam to the column in MRFs is extensively subjected to asymmetric moments, leading to shearing deformations in the column panel zones. The panel zone plays a vital role in general stiffness and capacity of the frame. In addition, the panel zone could have a significant effect on the distribution of plasticity and energy dissipation of the structure.

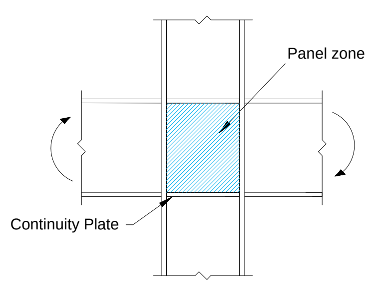



Figure 1. Geometry of a Panel Zone

## 2. A REVIEW ON MATHEMATICAL PANEL ZONE MODELS FOR H-SHAPED COLUMN

Previous studies show that the panel zone has ductile and stable behavior. In other words, when a MRF is subjected to lateral loads, huge shear forces are created in the panel zone and the resulted deflection can play an important role in the response of frame to both the elastic and inelastic cases.

Considering the aforementioned issues related to the panel zone, many researchers have been addressed to the study of the seismic behavior of the panel zone in the early 1970's; also for many years, the codes and instructions were presented with different equations related to the panel zone behavior. Most of the conducted researches have focused on the panel zone behavior of the I-shaped columns. Krawinkler *et al.* [1], Bertero *et al.* [2], and then Popov and Takhirov [3] showed that the panel zone has a high resistance even after yielding. High ductility, the stable hysteresis loops and the high cyclic work-hardening are some of the observations and findings of those researchers. The studies conducted after the 1994 Northridge earthquake showed that the panel zone's over-distortion could cause brittle fractures in welded of flange connections to column flange [4-9]. These studies led to determining new design criteria for the panel zone, which could consider a more real panel zone behavior and performance in response to the MRFs. The main problem was the suitable performance of the structures under the service and seismic loads. For this purpose, the presentation of a design, in which more contribution of the structure components to absorb energy is considered, seems essential because in case the structure remains in elastic range under seismic loads, it leads to high expenditure and the structure becomes uneconomical. On the other hand, to consider the inelastic behavior of the structure necessitates a full understanding of the performance and extensive studies in the subject. For this reason, the studies have focused on the better understanding of the inelastic behavior of the connections of the MRFs. To understand the effects of various loadings, different types of loadings were simulated in the experiments, whereby the gravity loads and the seismic cyclic loads were applied on the samples, and the effects of the axial loads on the performance of connections were considered in few cases. Based on the obtained results, the researchers suggested that the panel zone should be considered as an energy contributor while considering the yield mechanism that would be simultaneous to the beam mechanism [10].

As described before, the panel zone performance is important in the MRFs. Due to the high moment of the inertia of box sections in both directions, as well as the high torsional resistance, the box columns are vastly used in the MRFs. Therefore presenting a suitable design to describe the panel zone behavior in those columns seems essential. However, since most of the studies were conducted on the I-columns which they didn't had thick flanges. It is predicted that the existing relationships are not accurate enough to describe the panel zone behavior in the box columns. In this study, using the finite elements parametrically, the panel zone behavior in the box columns is studied. Based on the results, the mathematical models are proposed to be applied on the linear and non-linear behavior of the panel zone.

## 3. PANEL ZONE SHEAR STRENGHTH CAPACITY IN THE AISC

In recent seismic criteria of the AISC Code of Standard Practice that is based on LRFD design, the design resistance of the panel zone is categorized based on the axial forces applied on the column as follows [11,12]:

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

i) for $P_r \leq 0.4\ P_c$

$$R_n = 0.6 F_y\ d_c\ t_{cw} \tag{1}$$

ii) for $P_r > 0.4\ P_c$

$$R_n = 0.6\ F_y\ d_c\ t_{cw} \left(1.4- \frac{P_r}{P_c}\right) \tag{2}$$

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

i) for $P_r \leq 0.75\ P_c$

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

ii) for $P_r > 0.75\ P_c$

$$R_n = 0.6\ F_y\ d_c\ t_{cw} \left(1 + \frac{3b_{cf}\ t_{cf}^{\ 2}}{d_b\ d_c\ t_{cw}}\right) \left(1.9- \frac{1.2P_r}{P_c}\right) \tag{4}$$

In the above formulae, $F_y$ is the yield stress of the column, $d_c$ is the depth of the column section, and $t_w$ is the web thickness of the column. In addition, $b_{cf}$ is the flange width of the column, $t_{cf}$ is the flange thickness of the column, and $d_b$ is the beam depth. Moreover, $P_r$ is the factored axial force of the column and $P_c$ is the axial yield resistance of the column.

## 4. FINITE ELEMENT MODELS OF BOX COLUMN PANEL ZONE

To reach a suitable mathematical model, first an extensive parametric study regarding the effective factors on the behavior of panel zone is conducted out by ABAQUS [24] software. These parameters are: column flange thickness ($t_{cf}$), column web thickness ($t_{cw}$), beam flange thickness ($t_{bf}$) and axial force. All parametric studies were done for BSAC group, which represent a wide range of connections of different beam overall depths (from 450 to 912 mm). Details of these specimens are presented in Table 1. B-SAC specimens comprised of a built-up box-shape column and a wide flange I-shape beam. The connection details of these specimens are exactly the same as SAC specimens presented in [13], (i.e., all specimens have post-Northridge connection detail) except that the box shape columns have been used instead of I-shape columns. Box columns have the same flanges and webs used in SAC specimens presented in [13].

Table 1. BSAC Samples Specifications

| Spesimen | Type | Section/size(mm) | Yield stress (MPa) |
|---|---|---|---|
| BSAC3 | Beam | W24×68 | 306 |
| | column | Box (d=368, b=373, $t_f$=24, $t_w$=7.4) | 318 |
| BSAC5 | Beam | W30×99 | 306 |
| | column | Box (d=386, b=399, $t_f$=33, $t_w$=11) | 318 |
| BSAC7 | Beam | W 36×150 | 306 |
| | column | Box (d=417, b=406, $t_f$=48, $t_w$=15) | 318 |

To consider the column web thickness effect in the reached model acquired by the software, the webs thickness sections have been multiplied by the coefficients of 1 and 0.75 in the Table 1. To consider the column flange effects, the thickness of the column flange have been multiplied by the coefficients of 0.5, 0.75, 1, and 1.25; and to consider the beam flange effects, the beam flange thicknesses have been considered to be 0.85, 1, 1.5, and 2 times the introduced sections in the Table 1. In addition, to consider the effect of the axial force, the axial force have been considered 0.0, 0.2, 0.5, 0.6, 0.75, 0.8, and 0.9 times of the yield axial force of the column. Therefore, the number of the built samples equaled 672 samples, that is (3 samples) × (2 column web thicknesses) × (4 column flange thicknesses) × (4 beam flange thicknesses) × (7 axial force ratios). In Table 2, the connection details of the SAC samples are shown. Moreover, the details of the SAC7 sample are fully shown in Figure 2.

The ABAQUS software was used for modelling and 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 by several researchers [7]. To reduce the computational efforts, dense meshes have been used in the panel zone region while the other regions have coarse meshes. Column flanges plate are modelled in 5 layers of elements. The free end of beam moves vertically under displacement control analysis. (see Figure 3)

Table 2. Connection Details of the SAC Samples [7]

| specimen | Shear tab | No. of A325 SC Bolts (mm) | Continuity Plate (mm) | Weld type and size (mm) | |
|---|---|---|---|---|---|
| | | | | Beam flange | Shear tab |
| SAC3 | 457X127X9.50 | 6Φ22 | 355X335X16 | CJP, root opening = 9 mm, Angle = 30° and E70TG- | Fillet 8mm E70T-7 |
| SAC5 | 610X127X12.70 | 8Φ25 | 375X345X19 | | Fillet 8mm E70T-8 |
| SAC7 | 762X127X15.88 | 10Φ25 | 350X330X25 | | Fillet 8mm E70T-7 |

The considered analysis is non-linear static and it is introduced in the form of the General Static in the ABAQUS. Loading is considered as a monotonic displacement at the end of the beam and it is regarded 178 millimeters. The maximum imperfection value is considered with a factor of 1% of the beam flange thickness from first buckling mode [14-17] and distribution of geometric imperfections matched the first eigenvector of the loaded connection configureuration. The supports of column ends are considered hinged (as in experiments). In all samples, the length of the beam is considered to be 3400 mm and the length of the column is considered to be 3650 mm. Two lines of nodes at each end of the column were restrained against translation only (i.e., a pinned connection) to approximately replicate the support conditions used for the laboratory tests [13]. The behavior of materials is considered using the Von Mises yield criterion and the materials non-linearity was considered according to the reference [13], along with the young modulus of 200000 MPa and the Poisson ratio of 0.3. Stress-strain diagram of steel is considered bilinear [13] as seen in Figure 4.

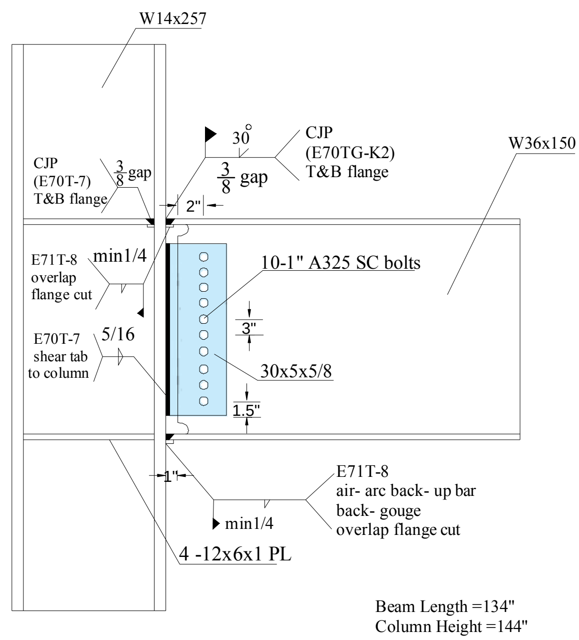



Figure 2. Details of the SAC7 Sample [13]

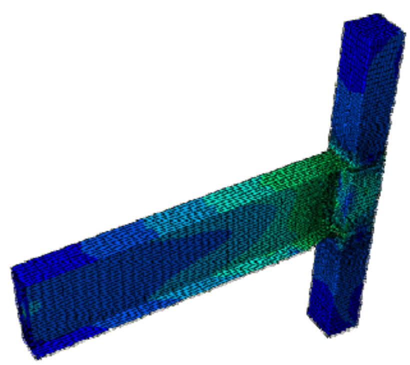


Figure 3. Finite Element Modeling

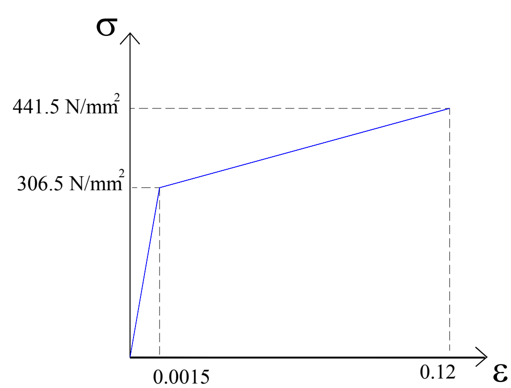



Figure 4. Stress-strain Diagram of Steel [13]

## 4.1 Shear Computing Method

To calculate the panel zone shear ($V_{pz}$) in the conducted modeling, the proposed relationship derived from the reference [18] have been used. In addition, to calculate the panel zone strain ($\gamma$), the proposed relationship derived from the reference [19] have been used.

$$V_{pz}=\frac{PL}{h_t}(1-\frac{h_t}{H}) \tag{5}$$

$$\gamma=\frac{\Delta^+-\Delta^-}{2}\frac{\sqrt{d_{pz}^2+b_{pz}^2}}{d_{pz}b_{pz}} \tag{6}$$

In the above relationships, P is the applied force to the end of the beam, L is the distance between the applied force and the column face, $h_t$ is the distance between the centers of the flanges of the beam, H is the height of the column, $\Delta^-$ and $\Delta^+$ are the diagonal displacements of the panel zone, and $d_{pz}$ and $b_{pz}$ are the vertical and horizontal dimension of the panel zone, respectively (see Figure 6).

## 5. VERIFICATION STUDY

As validation is essential in numerical studies, specimen SP7 [13], is modeled by the ABAQUS software and compared with experimental results, before the main study in this research is being carried out. As seen from Figure 10, results of the SP7 specimen modeling in ABAQUS software are in a good agreement with the experimental results.

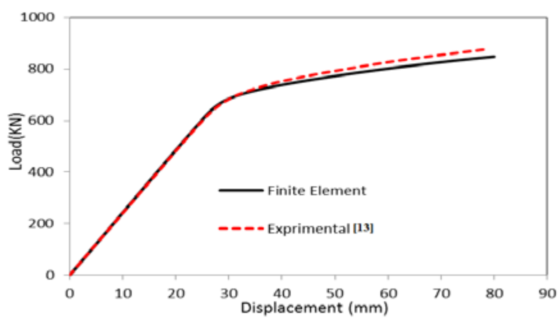



Figure 5. Comparing Experimental Results and Finite Element Results for SP7 Sample.

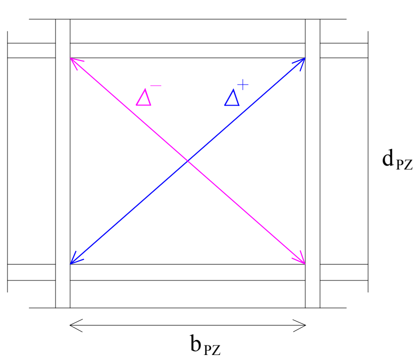



Figure 6. Geometry of Panel Zone to Determine Panel Zone Distortion [20]

## 6. PROPOSED MATHEMATHICAL MODEL

### 6.1 Initial Stiffness of the Panel Zone

As shown in Figure 7, the panel zone behavior is modeled as a column which its supports have been shown in Figure 8.

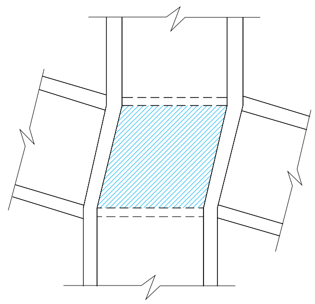


Figure 7. PZ Deformation caused by Applied Forces

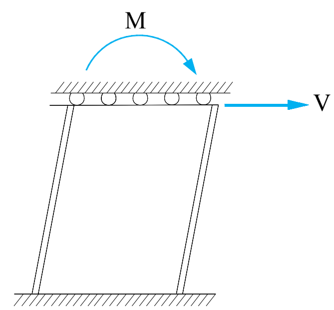



Figure 8. The Proposed Model of a PZ

To determine the initial stiffness of the panel zone, at first, a unit load is applied at the end of the panel zone in the horizontal direction and the correspondent displacement is obtained. Then, since the inverse of the obtained displacement is actually the stiffness, the panel zone initial stiffness is found by inversing the displacement. Therefore, first, the horizontal unit load is applied at the end of the panel zone, then the corresponding displacement (Δ) is obtained as follows;

$$\Delta=\frac{d_b^3}{12EI}+\frac{d_b}{\eta GA} \tag{7}$$

In the above relationship, $d_b$ is the vertical distance between continuity plates of the panel zone, E is the elasticity modulus, I is the moment of inertia of column, η is the shape factor, G is the shear modulus and A is the sectional area of the column. By inversing the above relationship, the panel zone stiffness ($k_e$) have been obtained as follows;

$$K_{e0}=\frac{1}{\frac{d_b^3}{12EI}+\frac{d_b}{\eta GA}} \tag{8}$$

Since the shear-distortion relationship is often used in researches, the Eq. 8 multiplied by $d_b$; consequently, the stiffness of the linear district of the panel zone, is resulted as follows;

$$K_e=\frac{1}{\frac{d_b^2}{12EI}+\frac{1}{\eta GA}} \tag{9}$$

Considering E=2.6G, estimating the moment of inertia by the relationship $I=A_{cf}d_c^2/2$ (that is ignored from the effect of the web in the moment of inertia), and by placing them into the Eq. 9 the Eq. 10 is obtained as follows:

$$K_e = \frac{\eta G A}{1 + 0.064\eta \frac{A}{A_{cf}} \left(\frac{d_b}{d_c}\right)^2} \tag{10}$$

In the Eq. 10, if it is supposed that $A=A_w$, where $A_w$ is the sum of the areas of the two webs of the column, and assuming that $\eta=1$, the Eq. 11 is obtained:

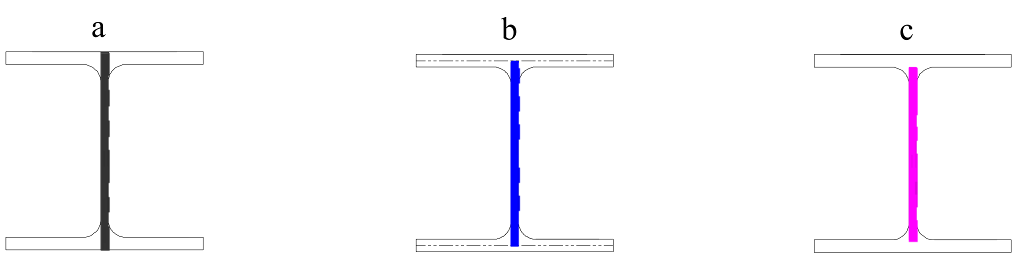
$$K_e = \frac{G A_w}{1 + 0.064 \frac{A_w}{A_{cf}} \left(\frac{d_b}{d_c}\right)^2} \tag{11}$$

Thus, the stiffness of the elastic range is obtained. The relationship between the yield shear strain of $\gamma_y$ and the yield shear force of $V_y$ is stated as follows:

$$K_e \gamma_y = V_y \tag{12}$$

## 6.2. Shear Force of the Panel Zone

After obtaining the initial stiffness of the panel zone, the shear force of the panel zone will be studied. There are several theories regarding the panel zones of the I-columns. These theories compare the shear force of the panel zone with the yield shear force obtained from the finite elements models of the box columns. In Figure 9, the considered shear area in some models is shown.




(a) AISC (b) Krawinkler (c) Wang

Figure 9. The Considered Shear Area

Figure 9(a) shows the shear area considered by the AISC Code of Standard Practice [6] that is equal to ($d_c t_{cw}$). Figure9(b) demonstrates the considered area in the reference [1] that is equal to ($d_c$-$t_{cf}$) $t_{cw}$; and Figure 9(c) illustrates the considered shear area by the reference [21] whose value equals ($d_c$-2$t_{cf}$) $t_{cw}$. In the above relationships, $t_{cw}$ is the web thickness of the column; and the rest of the parameters previously defined.

In addition to the differences existing among the three presented models while selecting the shear area, there are also differences in selecting the shear yield stress of $\tau_y$ in the relationship of $V_y = A_v \tau_y$ (where Av is the considered shear area in each model in Figure 9). Both references [1] and [21] have presented the yield shear stress according to the von Mises criterion in accordance with the Eq. 13.

$$\tau_y = \frac{F_y}{\sqrt{3}} \left( \sqrt{1 - \left(\frac{P}{P_y}\right)^2} \right) \tag{13}$$

While the amount of the yield shear stress in the AISC Code of Standard Practice is in accordance with Eq. 14.

$$\tau_y=\begin{cases}0.6F_y & \text{for} \quad \dfrac{P}{P_y}\le 0.75\\[2ex] 0.6F_y\left(1.9\text{-}1.2\dfrac{P}{P_y}\right) & \text{for} \quad \dfrac{P}{P_y}> 0.75\end{cases} \tag{14}$$

In these relationships, $F_y$ is the yield stress of the column materials, P is the axial force of the column, and $P_y$ is the axial yield resistance of the column that is equal to $F_yA_g$ where the $A_g$ is the gross area of the column.

From comparing the results of 672 box column panel zone models which are explained in the next sections and the previously presented models, it is derived that the amount of the yield shear force corresponds with the model presented by Krawinkler with an acceptable error percentage up to the loadings of $P_y \le 0.5$. Nevertheless, in higher loadings, this model underestimates the yield shear force; therefore, this model has been modified in the higher axial loading amounts. In Figure 10, the considered shear area for the box column in present study is shown.

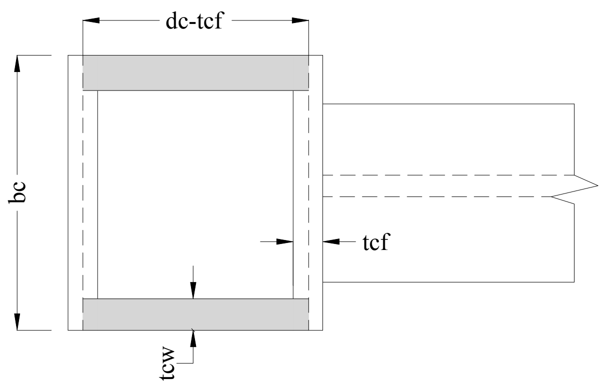



Figure 10. The Considered Shear Area for the Box Column

After analyzing the results obtained from the finite elements parametrical study, it is concluded that in case the yield shear stress is considered to be $\tau_y=\alpha F_y/\sqrt{3}$, and the decreasing coefficient of $\alpha$ is regarded by the Eq. 15, the finite elements of parametrical results will suitably correspond with the mathematical model.

$$\alpha=\begin{cases}\sqrt{1\text{-}\left(\dfrac{P}{P_y}\right)^2} & \text{for} \quad \dfrac{P}{P_y}\le 0.5\\[2ex] 01.066\text{-}0.4\dfrac{P}{P_y} & \text{for} \quad \dfrac{P}{P_y}> 0.5\end{cases} \tag{15}$$

Therefore, using the shear area adopted by Krawinkler and the Eq. 15, the yield shear force have been calculated as follows:

$$V_y=\frac{\alpha F_y}{\sqrt{3}}\left(d_c\text{-}t_{cf}\right)2t_{cw} \tag{16}$$

As demonstrated in the Eqs. 3 and 4, the amount of the decreasing coefficient in the AISC Code of Standard Practice is as follows:

$$\alpha_A = \begin{cases} 1 & \text{for} \quad \dfrac{P}{P_y} \le 0.75 \\ 1.9\text{-}1.2\dfrac{P}{P_y} & \text{for} \quad \dfrac{P}{P_y} > 0.75 \end{cases} \tag{17}$$

In the above relationship, $\alpha_A$ is considered to be the decreasing coefficient of the AISC Code of Standard Practice. Moreover, the amount of the decreasing coefficient in the von Mises theory is as follows:

$$\alpha_v = \sqrt{1\text{-}\left(\frac{P}{P_y}\right)^2} \tag{18}$$

In the above relationship, $\alpha_v$ is the considered decreasing coefficient according to the von Mises relationship. In Figure 11, the comparison between the adopted decreasing coefficient in the proposed relationship, the AISC Code and the Von Mises relationship is presented.

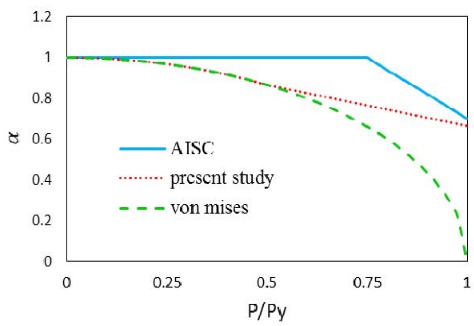



Figure 11. The Comparison between the Adopted Decreasing Coefficient in the Proposed Formula with the AISC and the Von Mises Formula

### 6.3 Yield Shear Strain of the Panel Zone

After the calculation of the yield shear force of the panel zone and the initial stiffness, the only remaining unknown, i.e. $\gamma_y$, is calculable, using the Eq. 12.

Replacing the Eq. 12 by the Eqs. 11 and 16, the yield shear strain of the panel can be derived zone as follows:

$$\gamma_y = \frac{F_y \alpha (d_c - t_{cf})}{G d_c} \left( 0.577 + 0.037 \left( \frac{A_w}{A_{cf}} \right) \left( \frac{d_b}{d_c} \right)^2 \right) \tag{19}$$

Taking into consideration that most box column designs consider the web and the flange thicknesses to be equal, and the box column is in square-shape with equal dimensions, the amount of the $A_{cf}$ equals 2 in Eq. 19, and the equation is simplified as follows:

$$\gamma_y = \frac{F_y \alpha (d_c - t_{cf})}{G d_c} \left( 0.577 + 0.074 \left( \frac{d_b}{d_c} \right)^2 \right) \tag{20}$$

## 6.4 Inelastic Region of the Panel Zone

The surrounding elements of the panel zone continuity plates and column flanges help to increase the shear resistance of the panel. It should be mentioned that this only occurs when the continuity plates exist. In case the plastic moment of the column flange is $M_{y,cf}$, the moment equals to [22]:

$$M_{y,cf}=F_{y,cf}\frac{b_{cf}\,t_{cf}^{2}}{4}\alpha_V \tag{21}$$

Consequently, the shear force increment of the panel zone in the non-linear district, with the elastic district of $\Delta V_{PZ}$, will be obtained as in [22].

$$\Delta V_{PZ}=\frac{4M_{y,cf}}{h_t}=\frac{F_{y,cf}b_{cf}\,t_{cf}^{2}}{h_t}\alpha_V \tag{22}$$

In the above relationships, $b_{cf}$ is the flange width of the column, and $F_{y,cf}$ is the yield stress of the web materials of the column; the rest of the parameters are defined as before. In Figure 12, the details of the shear force increment caused by the boundary elements contribution are shown. It should be noted that the final capacity of the panel zone, according to the suggestions of Krawinkler [20] and Lin *et al.*[23], is considered to be four times the yield shear strain ($\gamma = 4\gamma_y$) because of the contribution of the shear force boundary elements related to the shear strain.

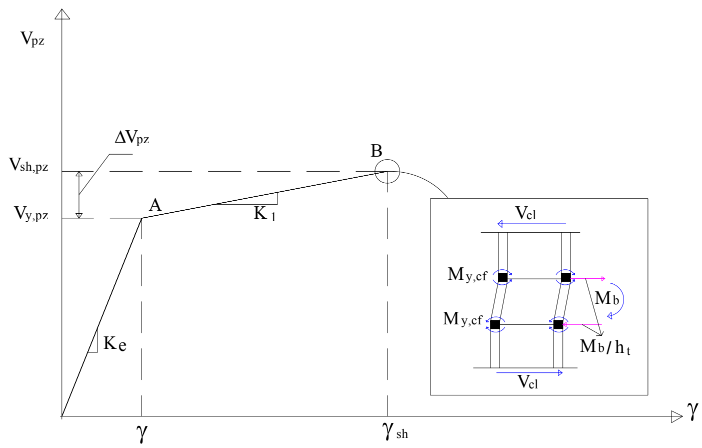



Figure 12. Shear Resistance Increment of the PZ Resulted from the Boundary Elements Contribution

According to the presented relationships in the AISC Code of Standard Practice, the amount of increment in the shear force in the district after the elastic district is considered to be as follows:

$$\Delta V_{PZ}=\frac{1.8F_y\,b_{cf}\,t_{cf}^{2}}{d_b}\alpha_A \tag{23}$$

Studying the results of the finite elements parametrical studies shows that the coefficient for applying the shear resistance increment caused by the boundary elements is 1.5; consequently, the shear force increment of the district after the elastic district is obtained as follows:

$$\Delta V_{PZ}=\frac{1.5F_y\,b_{cf}\,t_{cf}^{2}}{h_t}\alpha \tag{24}$$

Thus, the final shear capacity of the panel zone ,$V_{sh,PZ}$ , results as follows:

$$V_{sh,PZ}=V_{y,PZ}+\Delta V_{PZ} \tag{25}$$

Placing the Eqs. 16 and 24 into the Eq. 25, we obtain the result in Eq. 26.

$$V_{sh,PZ}=\frac{F_y\,\alpha}{\sqrt{3}}\left(d_c\text{-}t_{cf}\right)2t_{cw}+\frac{1.5f_y b_{cf}\,t_{cf}^{\;2}\,\alpha}{h_t} \tag{26}$$

To calculate the stiffness of the panel zone in the non-elastic district next to the elastic district, supposing that the final shear capacity of the panel zone occurs in $\gamma = 4\gamma_y$ , the stiffness of the non-elastic district of the $K_{sh}$ panel zone is obtained as follows:

$$K_{sh}=\frac{\Delta V_{PZ}}{3\gamma_y} \tag{27}$$

Placing the Eqs. 19 and 24 into the Eq. 27, we obtain:

$$K_{sh}=\frac{b_{cf}t_{cf}^{\;2}Gd_c}{h_t\left(d_c\text{-}t_{cf}\right)\left(1.154+0.074\left(\frac{A_w}{A_{cf}}\right)\left(\frac{d_b}{d_c}\right)^2\right)} \tag{28}$$

Again, considering that the box columns with square shape and equal thicknesses of flange and web, the equation can be simplify Eq. 29 as follows:

$$K_{sh}=\frac{b_{cf}t_{cf}^{\;2}Gd_c}{h_t\left(d_c\text{-}t_{cf}\right)\left(1.154+0.148\left(\frac{d_b}{d_c}\right)^2\right)} \tag{29}$$

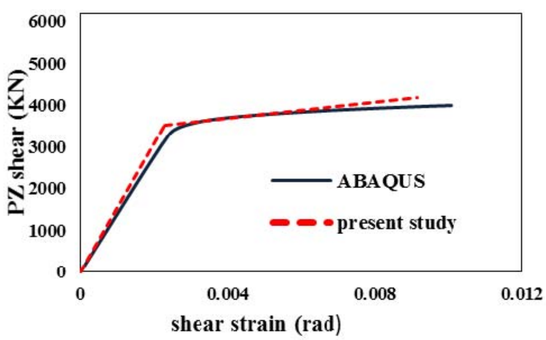
## 7. COMPARING THE ACCURACY OF DIFFERENT METHODS

To illustrate the accuracy of the presented relationships in this article, they have been compared with the finite elements results (see Figures 13(a)-(f)). In Table 3, the specifications of these samples are shown: $t_f$ is the thickness of the flange section, $t_w$ is the thickness of the web, d is the depth of the section and $b_f$ is the flange width of the section.

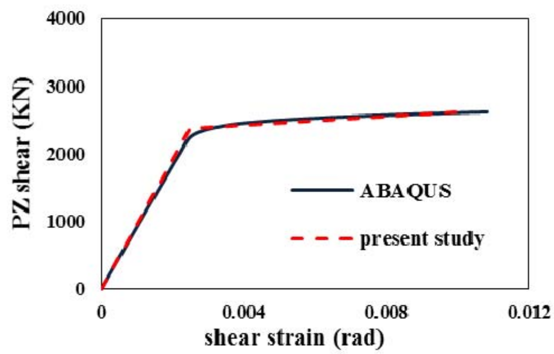
Table3. Specifications of the Samples 1, 2 and 3

| Sample | Section | $t_f$ (mm) | $t_w$ (mm) | d (mm) | $b_f$ (mm) |
|---|---|---|---|---|---|
| 1 | Column | 60 | 30 | 428 | 406 |
| | Beam | 47.7 | 16 | 935.7 | 305 |
| 2 | Column | 33 | 21 | 386 | 399 |
| | Beam | 34 | 13 | 762 | 266.7 |
| 3 | Column | 11 | 11 | 356 | 373 |
| | beam | 30 | 10.5 | 617 | 228 |

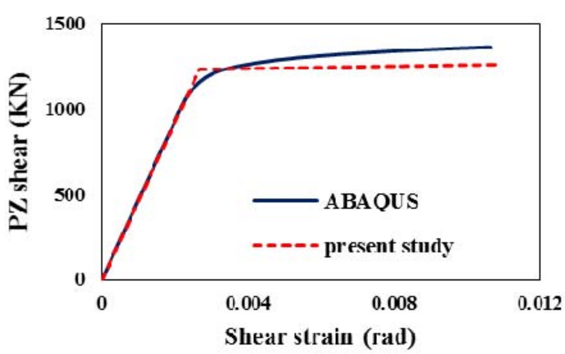
Figures 13(a) and 13(b) are related to the columns with thick and medium flange thicknesses respectively, along with the axial force ratio of 0.5. As it can be seen in the two figures, there is a suitable correspondence between the finite elements results and the present study.



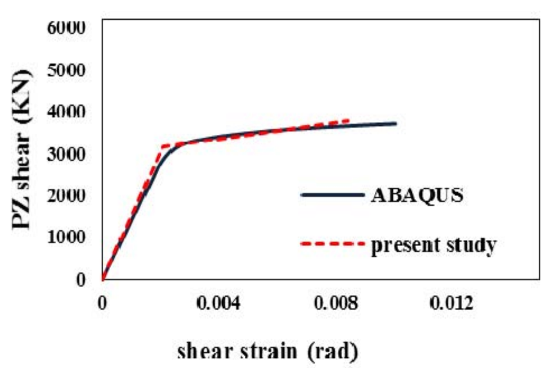

(a) Sample No.1



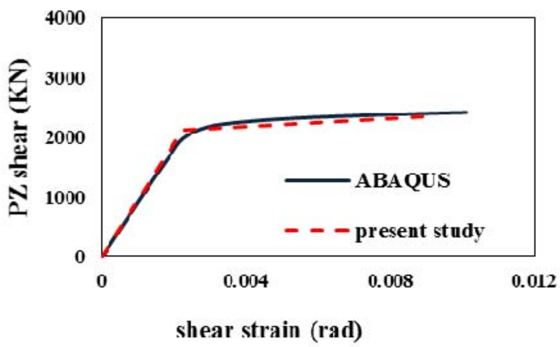

(b) Sample No.2

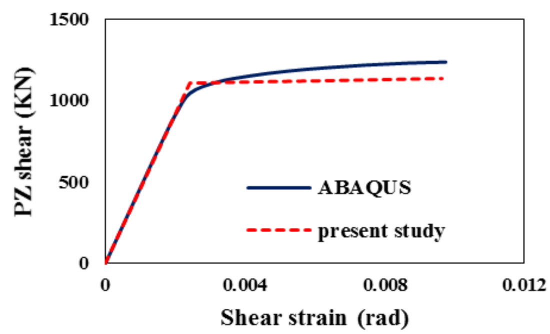



(c) Sample No.3




(d) Sample No.1




(e) Sample No.2 (f) Sample No.6

Figure 13. Comparison of the Presented Relationships and ABAQUS

Figure 13(c) is related to a column with a thin flange thickness and the axial force ratio of 0.5. As demonstrated in the figure, the presented results in this article are slightly lower (in the non-linear district) than the finite elements results, but the amount of the difference is negligible. Figure. 15(d) is related to a column with thick flange thickness and the axial ratio of 0.75. As illustrated in this figure, there is a suitable correspondence among the presented relationships and the finite elements results.

Figures 13(e) and 13(f), in a row, are related to columns with medium and thin flange thicknesses respectively, and the axial force ratio of 0.75. It is obvious that there is a suitable correspondence between the presented results and the finite elements results in Figure 13(e). However, in Figure. 13(f), the results of the presented relationships of this study are slightly lower than the finite elements results, but this difference is negligible.

The comparisons of the proposed with the results of finite element show that the average error for yield shear resistance of the panel zone is 2.19% and maximum error is 8%, For the final shear resistance of the panel zone the average error is 3.4% and maximum error is 9%. It show that the proposed relations has a good agreement with the result of finite element models.

## 8. SUMMARY AND CONCLUSIONS

The present relationships of the panel zone have been obtained from the analytical and experimental results of the I-shape columns in the panel zone. Yet, in most of these experiments, the columns with thick flange thicknesses and the vast existence of the axial force present on the columns are not considered. With regard to the importance of the box columns in the MRFs in seismic areas, we have concluded that the presentation of a mathematical model for the panel zones of the box columns that would estimate the behavior of these panel zones with higher accuracy seems essential. In this study, to reach a suitable mathematical model for extensive parametrical studies in relation to the effective parameters on the performance of the panel zone, we have implemented the ABAQUS software. The parameters include the flange thickness of the column, the web thickness of the column, and the flange thickness of the beam. Then, the mathematical relationships for the behavior of the panel zones in box columns are presented. The comparison of the presented relationships and the obtained results from the modeling shows a suitable correspondence between the relationships and the finite elements results. However, it should be mentioned that this correspondence is at the non-linear district of the shear force diagram; and the shear strain in columns with medium and thick flange thicknesses is more than the columns with thin flange thicknesses. Of course, considering that the error in the columns with thin flange thickness is negligible, it is concluded the presented model gives acceptable results for different types of flange thicknesses of the columns.

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