# Evaluation of Panel Zone Shear Strength in Cruciform Columns, Box-Columns and Double-Web Columns

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***Abstract*—Moment Resisting Frames (MRFs) 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. On the other hand, in these systems, column performance has special importance as the main part of tolerating resistant demands made by earthquake in both directions. The most dominant factor in a MRF is how to transfer moment between parts of the frame. Panel zone is a column region which is surrounded by the continuity plates and column flanges. In this study, firstly evaluates the adequacy of the panel zone relationship in AISC for cruciform columns, box-columns and double-web columns using non-linear finite element analysis method. Then the panel zone shear capacities of these columns that have the same plastic capacity are compared with each other. Part of the results of this study indicate AISC relations that only have considered the effect of web column in shear strength ,so that for cruciform column underestimate shear capacity about 20% less than accurate values and overestimate about 30% in boxed shape column and double-web column.**

***Index Terms*—panel zone, shear strength, cruciform columns, box-columns, double-web columns, beam-column connection**

## I. Introduction

To provide enough strength and stiffness in two orthogonal directions, using sections with similar behavior about two main axes seem essential for the column. Box-columns are frequently used in orthogonal moment resisting frames due to inherent characteristics such as a large flexural capacity and stiffness about their main axes. In the box-shaped columns, implementation of continuity plates, particularly the welding of the fourth side is associated with remarkable difficulties that sometimes these problems causes this important member will not be implemented in these type of column. Fig. 1 shows a box-shaped column.

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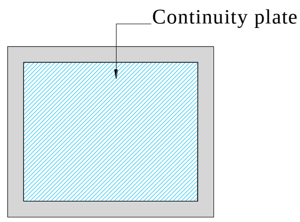


Figure 1. Box column with continuity plates

Double-web I-shaped column is of more torsional stiffness compared to the one with H-shaped cross section, it doesn't have operational issues of continuity plates in box-shaped column and in some cases, removing their continuity plate is possible [1]. Fig. 2 shows a double web column.

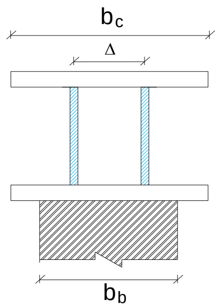


Figure 2. Double -web I-shaped column [1].

Cruciform sections can be named as sections with similar behavior in both directions.

In the following, more detailed investigation will be carried out on cruciform and double-web I-shaped columns.

### A. Introducing of the Cruciform Column

These sections consist of two I-shaped sections that are perpendicularly attached to each other at the mid-point of their webs after splitting one of them into two symmetric T-shaped sections (Fig. 3). The main benefits of these columns can mention to following items:



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1) Proper option for columns in orthogonal moment-resisting frames.
2) Have similar flexural strength and stiffness about both principal axes.
3) Could allow for a simpler construction process due to open and accessible shape of the section, especially for preparing continuity plates and panel zone regions.
4) Have more capacity rather than H-shaped column.
5) Increase axial capacity of column.
6) Reducing the consumption of steel.

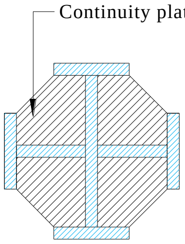



Figure 3. Cruciform column with continuity plates.

### B. *Introducing of the Double-Web I-Shaped Column*

Double-web I-shaped column was first suggested by Saffari *et al* [1] in order to omitting the continuity plates in the box columns. In this case the strength and rotational stiffness of the connection was provided by nearing the column webs to each other. Double web columns have more torsional stiffness in comparison with the H-shaped columns; furthermore because of reducing of local flange bending, it is easily possible to omit the continuity plates. According to Fig. 2 and ref. [1], the relationship between parameters on the double web column is as follows:

$$\alpha = \frac{b_b}{b_c} \quad (1)$$

$$\beta = \frac{\Delta}{b_b} \quad (2)$$

$$\beta = 3.979\alpha^2 - 5.42\alpha + 2.313 \quad (3)$$

Wide experimental and analytical studies have been carried out starting from the '70s, mainly by Krawinkler *et al*. [2], Bertero *et al*. [3], and Popov [4], in order to examine the behavior of PZ under monotonic and cyclic loadings. In this paper, firstly evaluates the adequacy of the panel zone relationship in AISC for cruciform columns, box-columns and double-web columns using non-linear finite element analysis method. Then the panel zone shear capacities of these columns that have the same plastic capacity are compared with each other.

## II. Panel Zone Shear Capacity

### A. *Modeling Process*

To achieve an appropriate model, first, a parametric study regarding the effective parameters on the behavior of panel zone is carried out by ABAQUS [5] software. These parameters consist of column flange thickness (tcf), column web thickness (tw), and thickness of continuity plates (tcp). Since, experimental results on seismic performances of cruciform columns and double web I-shaped columns do not exist in pre-qualified connections data-base, accordingly, experimental results of a well-known experimental program on “SP7 of SAC01” [6] are considered to validate modeling accuracy.

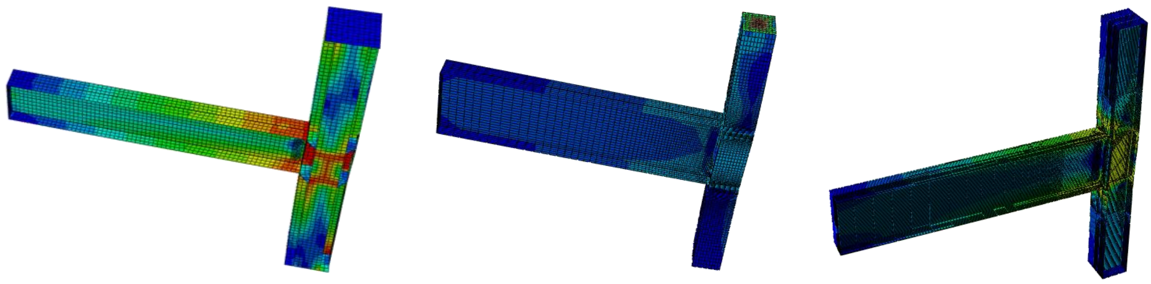
All parametric studies were performed for CSP3, CSP5 and CSP7 specimens which their columns shown in Table I. It should be noted that column sections of CSP3, CSP5 and CSP7 are selected from equalization of their plastic capacity with of SP3, SP5 and SP7 column sections of SAC01[6] respectively, since experimental results of SAC01[6], for verifying the finite element modeling methodology and general assumptions on the material behavior and nonlinear analysis, are available in ref [6]. 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 I.

TABLE I. Columns Section (All Dimensions in mm).

| Specimen | Column | Flange Width | Flange thickness | Web thickness | Outside height |
|---|---|---|---|---|---|
| | Box | 312.42 | 20.32 | 12.7 | 543. 56 |
| CSP3 | cruciform | 312.42 | 20.32 | 12.7 | 543. 56 |
| | Double web | 312.42 | 20.32 | 12.7 | 543. 56 |
| | Box | 355.6 | 24.765 | 15.367 | 695.96 |
| CSP5 | cruciform | 355.6 | 24.765 | 15.367 | 695.96 |
| | Double web | 355.6 | 24.765 | 15.367 | 695.96 |
| | Box | 398.78 | 29.21 | 18.161 | 855.98 |
| CSP7 | cruciform | 398.78 | 29.21 | 18.161 | 855.98 |
| | Double web | 398.78 | 29.21 | 18.161 | 855.98 |

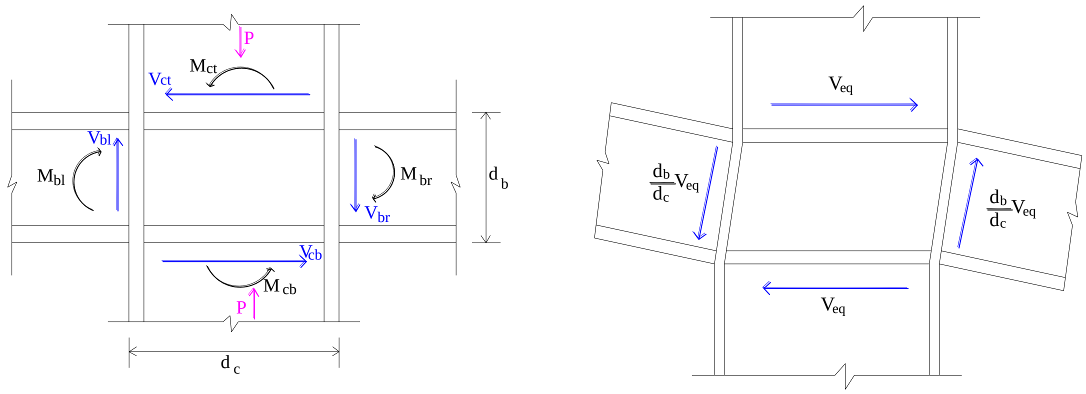
TABLE II. GEOMETRIC PARAMETERS OF SPECIMENS.

| Specimen | Shear tab (mm) | No. of A325 SC Bolts (mm) | Continuity plate (mm) | Weld type and size (mm) | |
|---|---|---|---|---|---|
| | | | | Beam flange | Shear tab |
| CSP3 | 400X127X10 | 6φ22 | 270X270X16 | CJP, root opening = 9 mm, Angle=30 °and E70TG-K2 | Fillet, 8mm, E70T-7 |
| CSP5 | 610X127X13 | 8φ25 | 375X375X19 | | Fillet, 8mm, E70T-8 |
| CSP7 | 765X127X16 | 10φ25 | 455X455X25 | | Fillet, 8mm, E70T-7 |



(a) Cruciform column (b) Box-shaped column (c) Double web I-shaped column

Figure 4. Finite element modeling.




(a) Panel Boundary Forces (b) Equivalent Panel Shear Forces

Figure 5. Equivalent panel shear forces and panel boundary forces [7].

The Young's modulus of elasticity, E, and Poisson's ratio, ν, were assumed to be 200 GPa and 0.3 respectively. Stress-strain diagram of steel is considered bilinear [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 II. Quadrilateral four-node shell elements (the S4R element) are used for constructing three-dimensional models of subassemblies. The free end of beam moves vertically under displacement control analysis. (Fig. 4)

## B. Shear Computing Method

To obtain panel zone shear force the below relation considered [7]: (Fig. 5)

$$V_{pz}=\frac{M_{b1}+M_{b1}}{d_b-t_{bf}}(1-\rho),\rho=\frac{d_b-t_{bf}}{H} \tag{8}$$

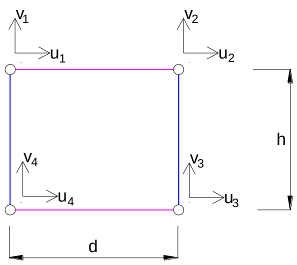



Figure 6. Geometry of panel zone to determine panel zone distortion [8].

### C. Computing Panel Zone Distortion

The proposed relation by Mulas [8] is used to calculate the panel zone distortion: (Fig. 6)

$$\gamma = \frac{1}{2}\left( \frac{u_1 - u_2 - u_3 - u_4}{h} + \frac{-v_1 + v_2 + v_3 - v_4}{d} \right) \tag{9}$$

### D. Verification Study

As validation is essential in numerical studies, specimen SP7 [6], is modeled by the ABAQUS software and compared with experimental results, before the main study in this research is being carried out.

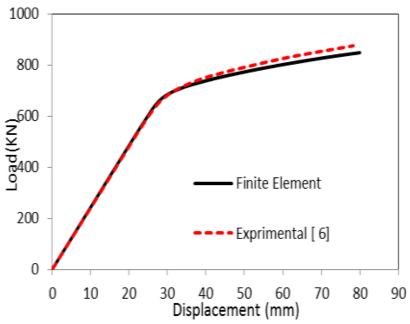



Figure 7. Comparing experimental results and finite element modeling results, for specimen SP7.

As seen from Fig. 7, results of the SP7 specimen modeling in ABAQUS software are in a good agreement with the experimental results. After making all the specimens, shear-rotation diagram of each panel zone specimen is drawn and compared with the amounts in AISC [9] (Fig. 8 - Fig.10). Tables III and IV depict comparison of the relationships of AISC regulations and results of FEM.

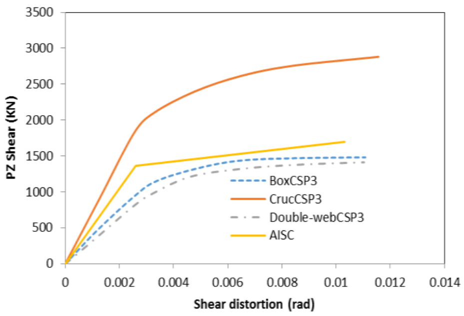



Figure 8. Comparison between the results obtained by the relations of AISC regulation and the results of finite element simulation for CSP3.

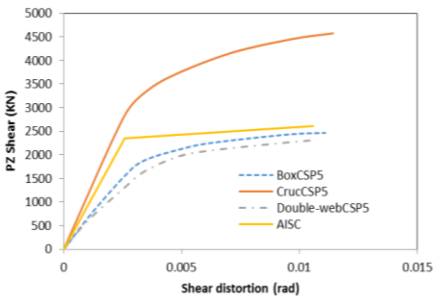



Figure 9. Comparison between the results obtained by the relations of AISC regulation and the results of finite element simulation for CSP5.

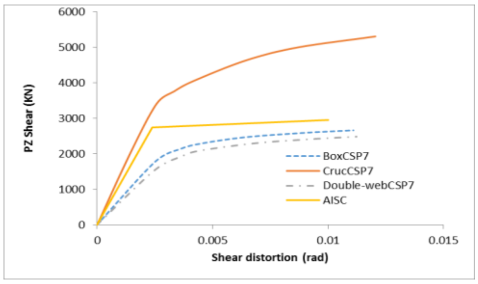



Figure 10. Comparison between the results obtained by the relations of AISC regulation and the results of finite element simulation for CSP7.

TABLE III. . RESULTS FOR ESTIMATING YIELD CAPACITY OF PZ

| Specimen | Column | $V_y$ abaqus (KN) | $V_y$ AISC (KN) | Error (%) |
|---|---|---|---|---|
| | Box | 935.1 | 1366.86 | 31.5 |
| CSP3 | cruciform | 1826.4 | 1366.86 | 25.1 |
| | Double web | 803.45 | 1366.86 | 41.2 |
| | Box | 1540.6 | 2350.85 | 34.5 |
| CSP5 | cruciform | 2795.6 | 2350.85 | 16 |
| | Double web | 1280.9 | 2350.85 | 45 |
| | Box | 1779.9 | 2744.51 | 35.1 |
| CSP7 | cruciform | 3310.7 | 2744.51 | 17 |
| | Double web | 1535.7 | 2744.51 | 44 |

TABLE IV. RESULTS FOR ESTIMATING ULTIMATE CAPACITY OF PZ

| Specimen | Column | $V_p$ abaqus (KN) | $V_p$ AISC (KN) | Error (%) |
|---|---|---|---|---|
| | Box | 1477 | 1700 | 13 |
| CSP3 | cruciform | 2881.4 | 1700 | 41 |
| | Double web | 1414.2 | 1700 | 17 |
| | Box | 2464.3 | 2610.8 | 5.6 |
| CSP5 | cruciform | 4576.4 | 2610.8 | 43 |
| | Double web | 2304.6 | 2610.8 | 11.7 |
| | Box | 2660.6 | 2956.1 | 10 |
| CSP7 | cruciform | 5308.1 | 2956.1 | 44 |
| | Double web | 2485.7 | 2956.1 | 16 |

## III. SUMMARY AND CONCLUSION

In this research attempts to propose a mathematical model using parametric study of finite element method in which effective factors on cruciform column panel zone such as thickness of column web and flange, and thickness of continuity plates are considered. The results obtained in this study are summarized as follows:

- In cruciform columns, double web columns and box-shaped column the effective area under shear is different than I-shaped columns. Thus AISC relations that are only considered the effect of web column do not have sufficient for these columns. So, it is necessary new relations provide to check of Panel zone capacity.
- Cruciform sections that have equal plastic capacity with double web sections and box-shaped sections show more shear strength in the panel zone. Because the flanges parallel with the web participate in Transmission of shear, so Cruciform sections have better performance in Transmission of shear capacity in panel zone in comparison with double web sections and box-shaped sections.

- There is not much difference between the results of the panel zone shear of Box-shaped columns and double web columns. But, since double-web I-shaped column is not engaged with the operational issues of continuity plate, and also, it is of much more torsional stiffness, it would be more suitable for buildings with moment resisting frames.

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