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NEW METHOD FOR PROVIDING FAVORABLE SEISMIC PERFORMANCE IN PANEL ZONE REGIONS OF MOMENT RESISTING CONNECTIONS OF BEAMS TO FLANGED CRUCIFORM COLUMN SECTION

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Abstract. *It is common in structural design to use separate load resisting system in each principal direction of the buildings. In these systems, columns which are located in the intersection of two perpendicular axes play a significant role in resisting gravitational loads as well as seismic demands in both directions. In seismic design, it is necessary to use practically similar sections for columns in both axes.*

Flanged cruciform sections, having relatively similar behavior about both principal axes, seem to be a proper choice for this purpose. They have apparently much simpler fabrication process in beam-column joint regions than the closed box sections. Since having two perpendicular webs, usage of common doublers plates leads to discontinuation of load paths and is not applicable. On the other hand, the local increase in the web thickness is not possible in the cruciform sections built using hot rolled sections. Therefore, it is necessary to consider an alternative path to provide a suitable performance in the beam column joints.

In this paper, by means of nonlinear numerical analysis, influences of connection details on the behavior of flanged cruciform column are studied. Since dynamic behavior of flanged cruciform column is depended on the connection behavior, in order to detect column behavior, it is subjected to monotonic and cyclic lateral displacement loading. Analysis shows connection details play a significant role in moment distribution of column parts. In addition, a new load path is suggested to provide a proper way to transfer panel zone's shear force and its effectiveness in improving behavior of flanged cruciform column is presented.

1 INTRODUCTION

1.1 Column sections

It is common in structural design to use separate load resisting system in each principal direction of the buildings. In these systems, columns which are located in the intersection of two perpendicular axes play a significant role in resisting gravitational loads as well as seismic demands in both directions. In seismic design, it is necessary to use practically similar sections for columns in both axes.

W sections are not suitable since having one strong principle axis and a weak one. Their strength and stiffness about their weak principal axis is low. In addition, design and application of seismic connection to this weak axis is a complicated issue.

Box sections, although having two similar high bending and torsional strength as well as high stiffness, are not supposed as an ideal choice for above mentioned performance, due to difficulties in continuity plate fabrication and out of plane action of the column flanges in beam-column joint regions.

Flanged cruciform sections, having relatively similar behavior about both principal axes, seem to be a proper choice for this purpose. They have apparently much simpler fabrication process in beam-column joint regions than the closed box sections.

In the following the characteristics and weaknesses of flanged cruciform sections are reviewed.

1.2 Fabrication of flanged cruciform columns

This section consist of two W sections where one of these sections is cut into two pieces symmetrically at the centre line of the web along its longitudinal axis, and connected to the mid-depth of other section with welding. Fabrication procedure for this section is complicated because enough space is needed for welding of two W shape sections. Most of W12 and W14 sections are not suitable for making flanged cruciform sections due to their geometric dimensions.

1.3 Axial capacity and lateral stiffness of flanged cruciform columns

The axial capacity of column is usually calculated based on the area and compression resistance. The compression resistance depends on the effective length of the column and the radius of gyration of column section. Since this section has not minor axis and behaves similarly in both principal axes, there is no reduction in axial capacity of this columns due to buckling about minor axis.

Tahir et al, conducted experimental program to study axial capacity of these sections. They demonstrated that the behavior of the flanged cruciform column made up uniform beam sections is similar to behavior of uniform beam section with axial load bending on the major axis. Due to high lateral stiffness of this section, they suggested it for using in unbraced frames to increase stiffness of the frame and decrease sway of frame [1].

1.4 Bending capacity of flanged cruciform columns

This section consists of two W sections that are perpendicular to each other. In bending about principal axis of flanged cruciform sections, one of them bends about its major axis and other bends about its minor axis. In the analysis and design of this section ,it is a assumed that plane section of the column remains plane, so moment distribution in each part of column section follows proportionally the weak axis stiffness to strong axis stiffness ratio of W sections.

1.5 Connection of flanged cruciform columns

In order to use of flexure capacity of both perpendicular W sections at the same time, the connection of beam to columns must have enough strength and stiffness to constrain two W sections together. As the strong axis connections are investigated extensively, prequalified connections for seismic moment frames are represented in AISC prequalified connections [2, 3].

This standard mentions that the behavior of the prequalified connections with flanged cruciform columns is expected to be similar to that of a rolled wide-flange column and the behavior of the assemblies employing this section would be acceptable, as long as such column sections meet the limitations for I-shaped sections and connection assemblies are designed to ensure that most inelastic behavior occurred within the beam as opposed to the column [2].

It is necessary to study the connection characteristics needed to push two parts of section to behave in the same manner in order to whole section plane remains plane.

1.6 Panel zone regions of flanged cruciform columns

Since having two perpendicular webs, usage of the common doublers plates in both directions leads to discontinuity of load paths and is not applicable. On the other hand, the local increase in the web thickness is not possible in the flanged cruciform sections with hot rolled sections.

Therefore, in order to reduce the shear force demands on the column's web, it is necessary to consider an alternative path that can transfer the panel zone shear force and provide a suitable performance in the beam column joints.

1.7 Objective

In this paper several moment resisting connections of beams to flanged cruciform column are developed to study their seismic performance. The design of the connection details is based on AISC prequalified procedure. A model of each connection was created and analyzed, using the finite element analysis program ABAQUS and their behavior to monotonic and cyclic lateral loading was studied. A new load path is suggested to provide a proper way to transfer panel zone shear force and its effectiveness is evaluated. In addition, moment resisting participation of each part of column section as well as shear resisting are studied.

2 DESIGN OF MOMENT CONNECTION DETAILS

Three separate specimens are adopted to examine performance of flanged cruciform column. First specimen (specimen 1) is without continuity plates, the second one (specimen 2) includes continuity plates and the third specimen (specimen 3) including continuity plates and inclined vertical plates. The details of the moment connection for specimen 1, specimen 2 and specimen 3 are shown in Figures 1(a),(b) and (c) respectively.

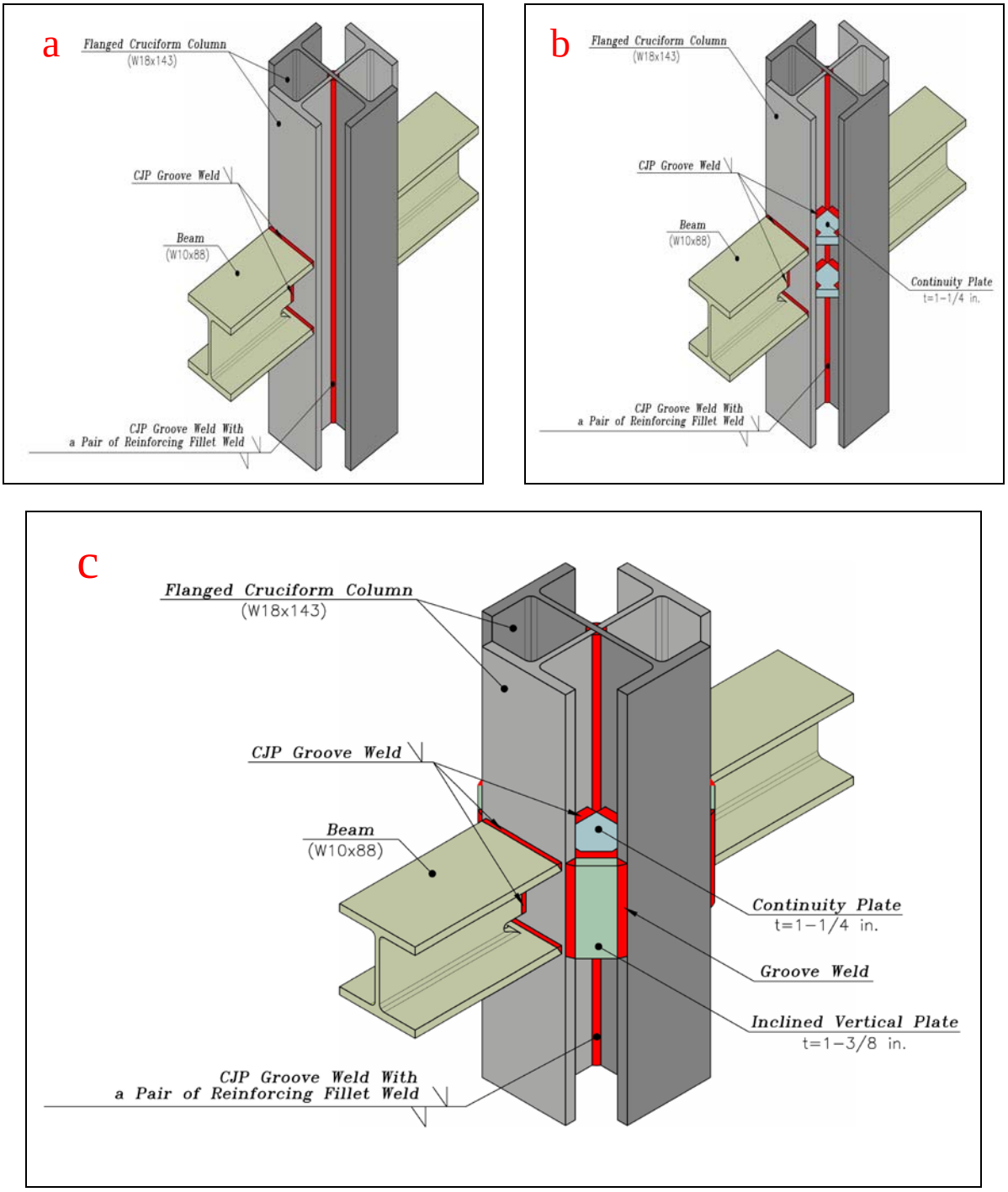

Figure 1. The details of the moment connection: (a) for specimen 1, (b) for specimen 2 and (c) for specimen 3.

As shown in Figure 1(c), inclined vertical plates with similar thickness of column flange plates, connected to flange edges of two perpendicular W sections in the panel zone region, provide a proper additional path to transfer panel zone's shear forces. Also these inclined vertical plates are welded to continuity plates. In order to show details of this specimen clearer, the details of the moment connection of specimen 3 in panel zone region are shown in Figure 2(a) and (b).

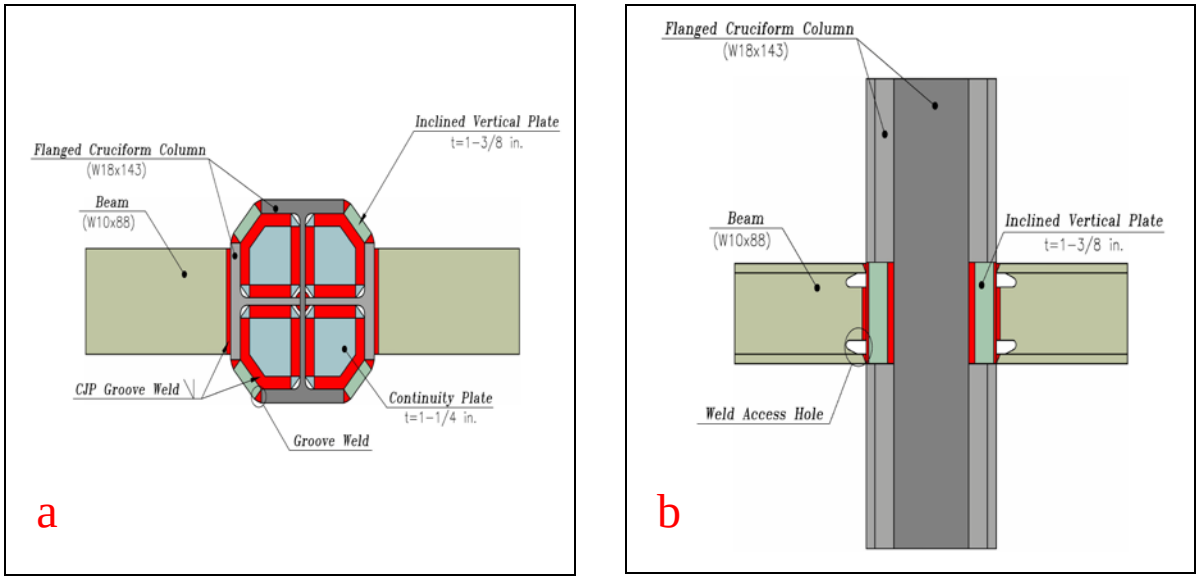

Figure 2. (a) and (b): The details of specimen 3 in panel zone region.

The specimens consist of cruciform interior connection subassemblies with beams attached to the column opposite faces. The subassemblies are extracted from interior joints of moment frames. The height of column is 160 in. and two beams 120 in. long framed into both face of column at mid height. In moment frames, deflected under lateral load, the inflection points are formed near the mid-span of beams and mid-height of columns. By this assumption, the inflection points of moment frames are considered to be the ends of subassembly beams and columns with hinged supports. The column base is supported by a hinged connection and the beam ends were supported by adjustable ones with lateral movement capability to provide the roller boundary condition. The column top end is subjected to horizontal displacement-controlled loading.

In order to avoid global and local instabilities, lateral bracing of beams and column are provided as requirements of AISC seismic provisions [4].

The flanged cruciform column consists of two W18*143 section which are suitable sections to be fabricated as flanged cruciform section. The beams in opposite side of the column are W10*88 sections. The geometric properties of the sections are shown in Table 1.

Section	d (in.)	b_f (in.)	t_w (in.)	t_f (in.)	I_x ($in.^4$)	I_y ($in.^4$)	Z_x ($in.^3$)	Z_y ($in.^3$)
W18*143	19.5	11.2	0.73	1.32	2750	311	322	85.4
W10*88	10.8	10.3	0.605	0.99	534	179	113	53.1
Flanged cruciform column(W18*143)	19.5	11.2	0.73	1.32	3061	3061	407.4	407.4

Table 1: Geometric properties of sections

To calculate geometric properties of flanged cruciform section, it is assumed that plane section of column remains plane so the second moment of area for flanged cruciform column as well as plastic section modulus are calculated by the summation of properties of area of the x–x axis and y–y axis of the corresponding w sections.

According to the AISC seismic provisions, width-thickness ratios for the flanges and web of the beam and column conform to the requirement of the seismically compact elements [4].

Material properties of all elements in the specimens are according to ASTM-A36 and showed in Table 2 [4, 5].

F_y(ksi)	F_u(ksi)	E(ksi)	ν	R_y
36	58	29000	0.3	1.5

Table 2: Material properties of all elements

In accordance with the requirements of AISC prequalified connection, the web of the tee-shaped sections shall be welded to the web of the continuous I-shaped section with CJP groove welds with a pair of reinforcing fillet welds within a zone extending from 12 in. above the upper beam flange to 12 in. below the lower beam flange [2, 3].

Welded Unreinforced flange–welded web (WUF–W) moment connections are selected to connect the beams to the column faces. In this type of connection, Beam flanges and beam web are connected to column flanges using complete-joint penetration (CJP) groove welds. These welds conform to the requirements for demand critical welds in the AISC seismic provisions. The plastic hinge location is taken to be at the face of the column [3].

The probable maximum moment at the plastic hinge, M_{pr}, is calculated in accordance with AISC prequalified connections with

$$M_{pr} = C_p R_y F_y Z_x = 8{,}543 \text{ kip} - \text{in.} \tag{1}$$

where
F_y = specified minimum yield stress of the yielding element, ksi
R_y = ratio of the expected yield stress to the specified minimum yield stress F_y ,as specified in AISC seismic provisions (presented in Table1)
Z_x = beam plastic section modulus x-axis, in.^{3}
C_{pr} = factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions. Since experimental data shows a high degree of strain hardening, AISC prequalified connections demonstrate that the value of C_{pr} shall be taken as equal to 1.4 [3].

By dividing this value (M_{pr}) by the distance from the plastic hinge to the beam end, the corresponding shear force at the column faces (V_h) found to be 71.2 kips. The beam design shear strength (V_b) is larger than required shear strength at the column faces (V_h).

For special moment frames (SMFs), the panel zone shear force demand, per the AISC seismic provisions [4], are determined from the summation of the probable maximum moments at the face of the column and then using

$$V_{pz} = \frac{2M_{pr}}{d_b - t_{bf}} = 1{,}743 \text{ kips} \tag{2}$$

where
d_b = overall beam depth
t_{bf} = beam flange thickness
and M_{pr} is as defined previously.

The design panel zone shear strength are calculated per AISC prequalified connections with

$$V = \phi_v R_n = \phi_v 0.6 F_y t_{cw} d_c \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_c d_c t_{cw}}\right) = 424.6 \text{ kips} \tag{3}$$

where

d_c= overall column depth
b_{cf} = width of column flange
t_{cf} = column flange thickness
t_{cw}= column web thickness Main headings
ϕ_v=Resistance factor for shear strength of panel zone of beam-to-column connections, AISC prequalified connections demonstrate that the value of ϕ_v shall be taken as equal to 1.0 and d_b , F_y are as defined previously [2,3,6].

Since the panel zone shear force demand are larger than the design panel zone shear strength, providing doubler plates is necessary. To detect influences of inclined vertical plates in resisting panel zone shear force demand of connection, doubler plates are not provided intentionally either specimen 1 or specimen 2.

According to AISC seismic provisions, providing continuity plates is needed in these connections [4]. In specimen 1, the continuity plates are not provided to investigate its influences on mobilizing of each part of column section. All Continuity plates are 1-1/4 in. thick which are 1/4 in. thicker than beam flange thickness and welded to column flanges and column web and inclined vertical plates with CJP groove welds.

The connection satisfy the strong column-weak beam requirement given by AISC seismic provisions [4].

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} = 1.11 \geq 1 \tag{4}$$

Where
ΣM^*_{pb} = the sum of the moments in the beams at the intersection of the beam and column centerlines
ΣM^*_{pc}= the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines with a reduction for the axial force in the column

By extrapolation of the probable maximum moment at the plastic hinge (M_{pr}) to the centerline of the column, ΣM^*_{pb} is calculated with

$$\sum M^*_{pb} = 2(M_{pr} + \frac{V_u d_c}{2}) = 18{,}474 \text{ kips} - \text{in.} \tag{5}$$

ΣM^*_{pc} is calculated , assuming axial load is P_{uc}= $0.3F_yA_g$, with

$$\sum M^*_{pc} = 2Z_c(F_{yc} + \frac{P_{uc}}{A_g}) = 20{,}533 \text{ kips} - \text{in.} \tag{6}$$

where
Z_c=plastic section modulus of the column, in.^{3}
F_{yc}=specified minimum yield stress of column, ksi
P_{uc}=required compressive strength using LRFD load combinations, kips
A_g=gross area of column, in.^{2}

3 ANALYTICAL STUDY

3.1 Finite element modeling

The model of each connection was created and analyzed, using the finite-element analysis program ABAQUS. In this regard the shells elements were used for modeling in order to achieve a computationally efficient model. Here, a quadrilateral four-node shell elements (S4 element) is used.

Analysis have two steps, first only column is subjected to the axial load, the top column end is loaded by imposing lateral displacement. Nonlinear geometric behavior is considered in analysis. Each model is analyzed twice, once in order to evaluate the elastic performance of connections, considering only the elastic properties of the material. An additional analysis is carried out for each model to investigate the influences of plasticization of material in connection assemblies.

While it is assumed that the connected parts were completely joined together by the welds, neither complete joint penetration welds nor fillet welds were explicitly modeled. Additionally, beam weld access holes are not included in the models.

In order to avoid global and local instabilities, the out-of-plane movement of the beam flange is restrained near the plastic hinges and beam ends. In addition lateral bracing of column is provided at the both ends of the column and near connection region.

The overall view of the finite element model of specimen 3 is shown in Figure 3.

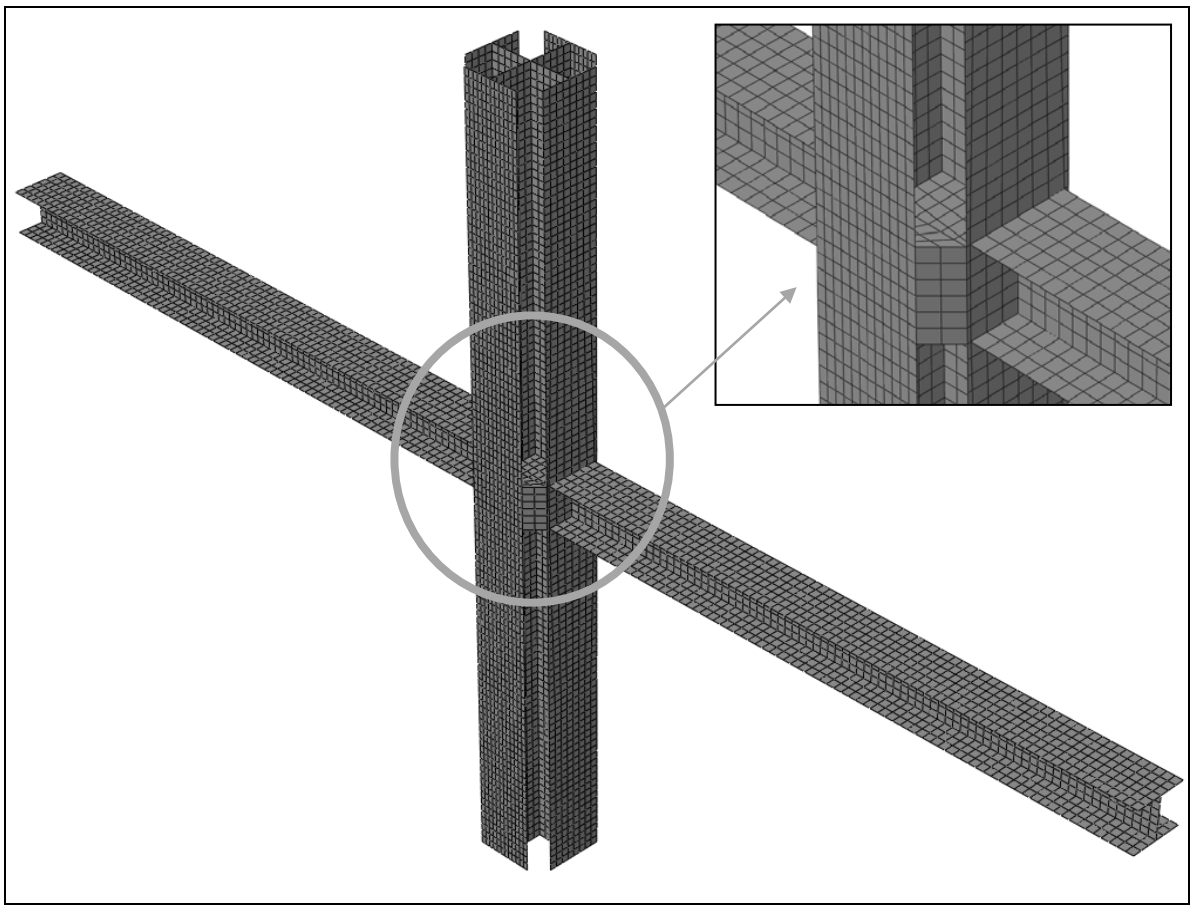

Figure 3: The overall view of the finite element model of specimen 3

3.2 Finite element analysis

A tri-linear stress-strain curve was used to represent the material properties. The Von Mises yield criterion and kinematic hardening rule were adopted to consider the plasticity behavior. The yield strength (F_y=36,000 ksi) and the ultimate strength (F_u=58,000 ksi) of the materials were considered. The Young's modulus of elasticity, tangent modulus and Poisson's ratio were assumed as 29,000 ksi, 0.01 Young's modulus and 0.3, respectively, for all materials of the analysis.

As requirements of AISC seismic provisions [4], the connection used in SMFs, must accommodate a story drift angle at least 0.04 rad. and the measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 rad.. In all models, the top column end is loaded monotonically by imposing a displacement until 0.04 rad. of story drift angle.

Since neither specimen 1 nor specimen 2 provide sufficient panel zone shear strength, only in specimen 3, the top column end is loaded axially by $P_{uc}=0.3F_yA_g$ accompanied by a prescribed quasi-static cyclic lateral displacement. The total story drift was calculated by dividing the column tip displacement by the height of column.

3.3 Finite element analysis results

Specimen 1

As there is no continuity plate placed in this specimen, parts of section are joined by webs, the compatibility of two parts of section is not taken place sufficiently due to low out-of-plane strength of webs. Shear force and flexural moment of each part of column are presented separately to investigate moment and shear resisting participation of each part of column. Maximum shear force in the column developed in the mid height of panel zone region (Section 1) and maximum flexural moment developed in the level of continuity plates (Section 2). In both elastic and plastic analysis of specimen 1, the W section part of flanged cruciform column bent about its strong principal axis, named herein after as "strong part", resists major portion of shear force and flexural moment in the column section and the W section part of flanged cruciform column bent about its weak principal axis, named herein after as "weak part", resists only minor portion of flexural moments as well as shear forces.

The developed shear force and shear force participation ratio of each part of column at section 1 versus column tip displacement are shown in Figures 4(a) and 4(b) respectively.

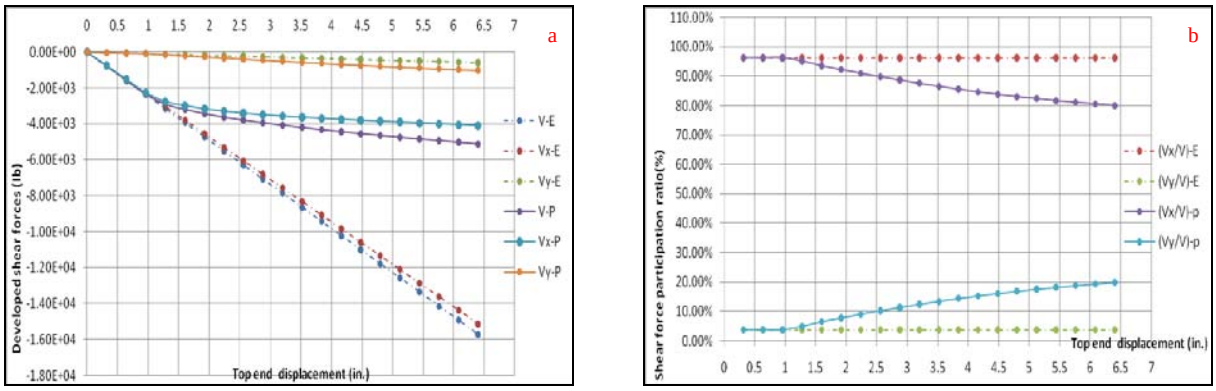

Figure 4. (a): The developed shear force of each part of column at section 1 versus column tip displacement; (b): Shear force participation ratio of each part of column at section 1 versus column tip displacement.

These figures include results of both elastic and plastic analysis to determine yielding effect of connection parts. The elastic analysis results are shown with suffix (-E) and plastic analysis results are shown with suffix (-P). In these figures (Mx) and (Vx) show flexural moment and shear forces, developed in strong part of column and (My) and (Vy) show flexural moment and shear forces, developed in weak part of column. Also (M) and (V) are resultant flexural moment and shear force, developed in the whole section of column.

In addition, the developed flexural moment and flexural moment participation ratio of each part of column at section 2 versus column tip displacement are shown in Figures 5(a) and 5(b) respectively.

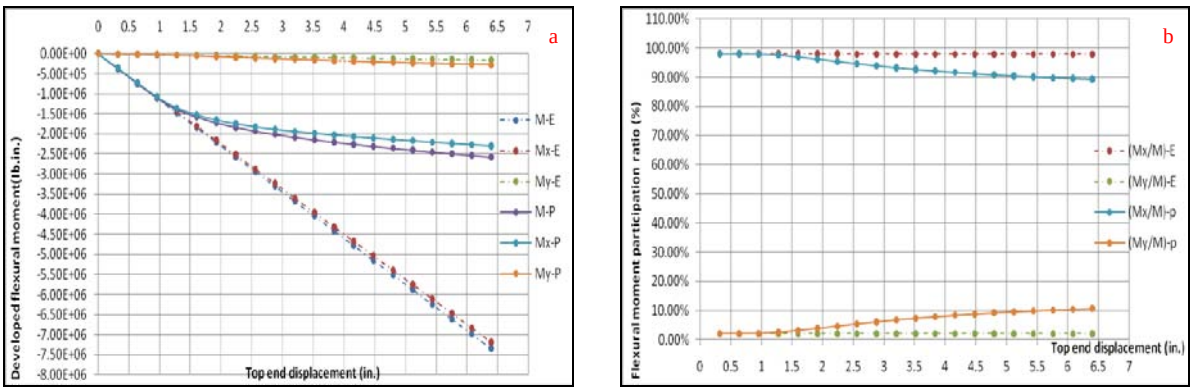

Figure 5 (a): The developed flexural moment of each part of column at section 2 versus column tip displacement; (b): Flexural moment participation ratio of each part of column at section 2 versus column tip displacement.

As shown, the compatibility of two parts of section are not remained the same which means the plane of the whole section does not remains plane and flexural moment distribution of column do not follow proportionally the weak axis stiffness to strong axis stiffness ratio of each W sections .

Due to yielding of panel zone and decrease in stiffness of strong part, the weak part participation in resisting flexural moment and shear force is higher in plastic analysis, as compared to results from elastic analysis.

Specimen 2

In this specimen which strong part and weak part are joined together through the continuity plates at beam flanges level as well as webs of two parts. Because of high in-plane capacity of continuity plates, it is predicted that the weak part has reasonably more share in the resisting flexural moment and shear force than specimen 1.

The developed shear force and shear force participation ratio of each part of column at section 1 versus column tip displacement are shown in Figures 6(a) and 6(b) respectively.

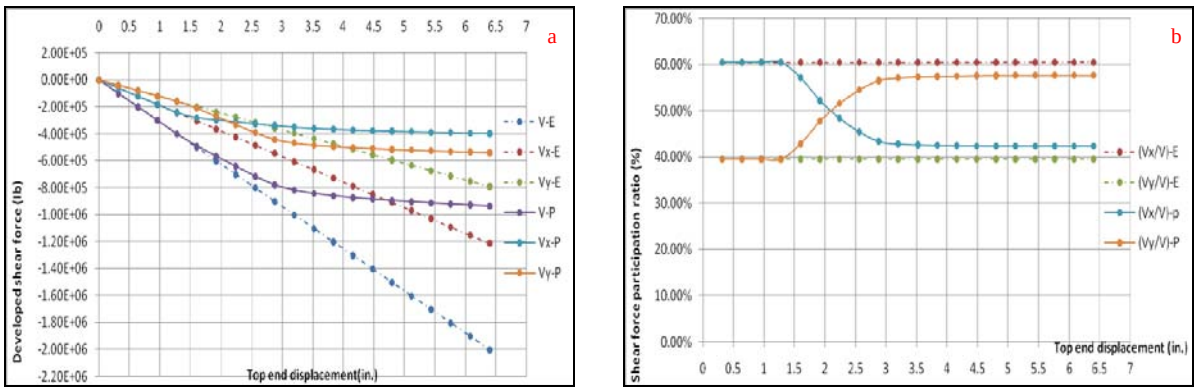

Figure 6. (a): The developed shear force of each part of column at section 1 versus column tip displacement; (b): Shear force participation ratio of each part of column at section 1 versus column tip displacement.

In addition, the developed flexural moment and flexural moment participation ratio of each part of column at section 2 versus column tip displacement are shown in Figures 7(a) and 7(b) respectively.

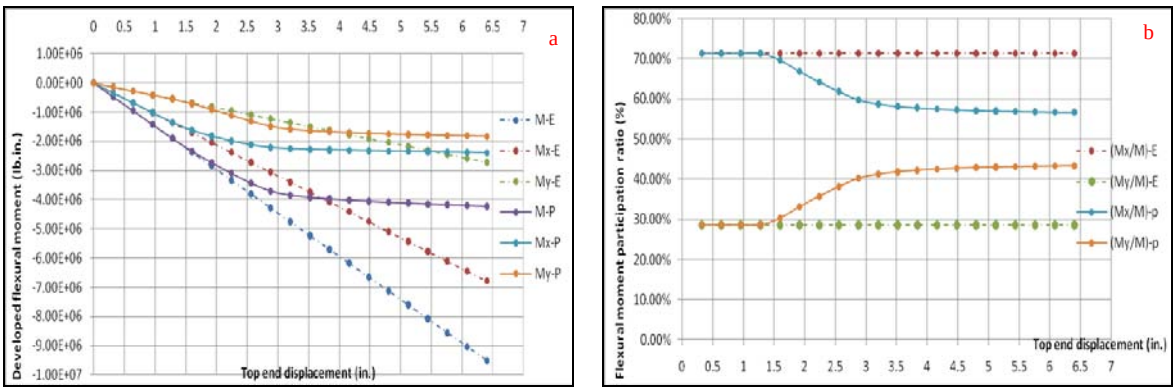

Figure 7. (a): The developed flexural moment of each part of column at section 2 versus column tip displacement; (b): Flexural moment participation ratio of each part of column at section 2 versus column tip displacement.

As shown, flexural moment distribution of column do not follow proportionally the weak axis stiffness to strong axis stiffness ratio of each W sections, so column section plane does not remains plane.

In the plastic analysis of specimen 2, as soon as panel zone start to yield, Stiffness of panel zone decreases so weak part resist more flexural moment as well as shear forces. Unlike specimen 1, panel zone yielding plays more important role in specimen 2.

Specimen 3

In this specimen, inclined vertical plates as well as continuity plates are provided. This inclined plates and flange plates of weak part create two extra load paths to transfer panel zone shear forces. In other word, there are three parallel load paths to resist shear forces of panel zone region, one directly through web of strong part and two accordion load path through inclined and flange plates.

The developed shear force and shear force participation ratio of each part of column at section 1 versus column tip displacement are shown in Figures 8(a) and 8(b) respectively.

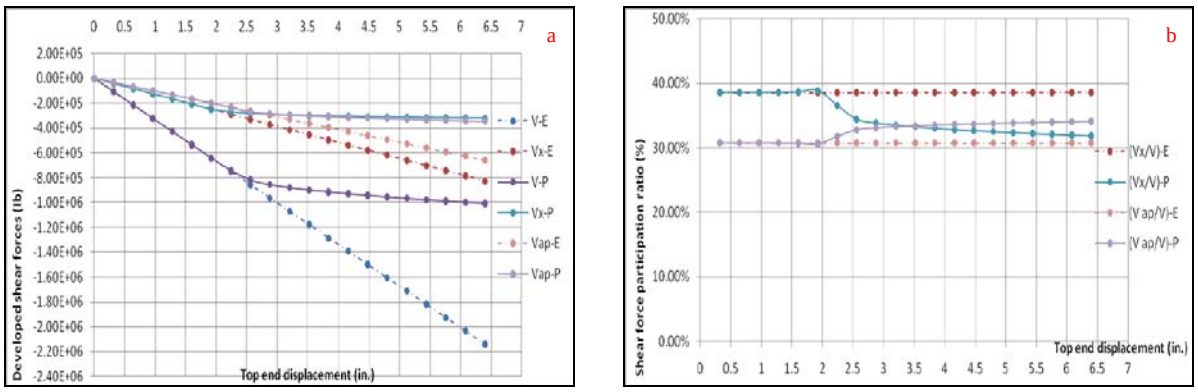

Figure 8 (a): The developed shear force of each part of column at section 1 versus column tip displacement; (b): Shear force participation ratio of each part of column at section 1 versus column tip displacement.

In these figures and (Vap) shows shear forces, developed in each accordion path of the panel zone regions. In addition, the developed flexural moment and flexural moment participation ratio of each part of column at section 2 versus column tip displacement are shown in Figures 9(a) and 9(b) respectively.

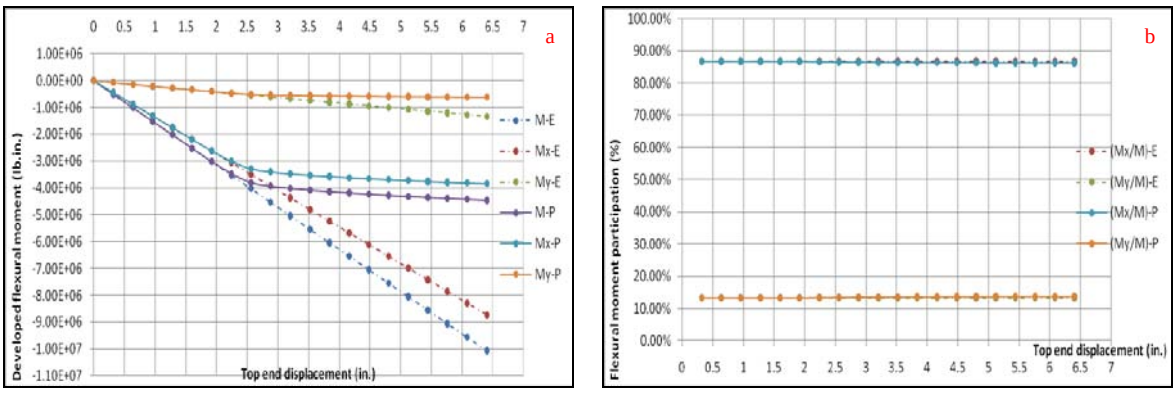

Figure 9. (a): The developed flexural moment of each part of column at section 2 versus column tip displacement; (b): Flexural moment participation ratio of each part of column at section 2 versus column tip displacement.

In the plastic analysis of specimen 3, after the yielding of web of the strong part, the flanges of weak part and inclined plates start to yield.

As shown, Although strong part web yielding cause decrease in panel zone shear stiffness, but does not considerably influence the flexural stiffness of the strong part of column, due to presence of two parallel accordion load path.

To examine connection performance in cyclic loading, this specimen is subjected to the cyclic lateral displacement loading. specimen shows quite stable inelastic behaviors and favorable energy dissipation capacities throughout cyclic loading. The connection performance exceeded the AISC seismic provision requirements.

The developed flexural moment of each part of column at section 2 versus total story drift angle of the specimens is shown in Figure 10.

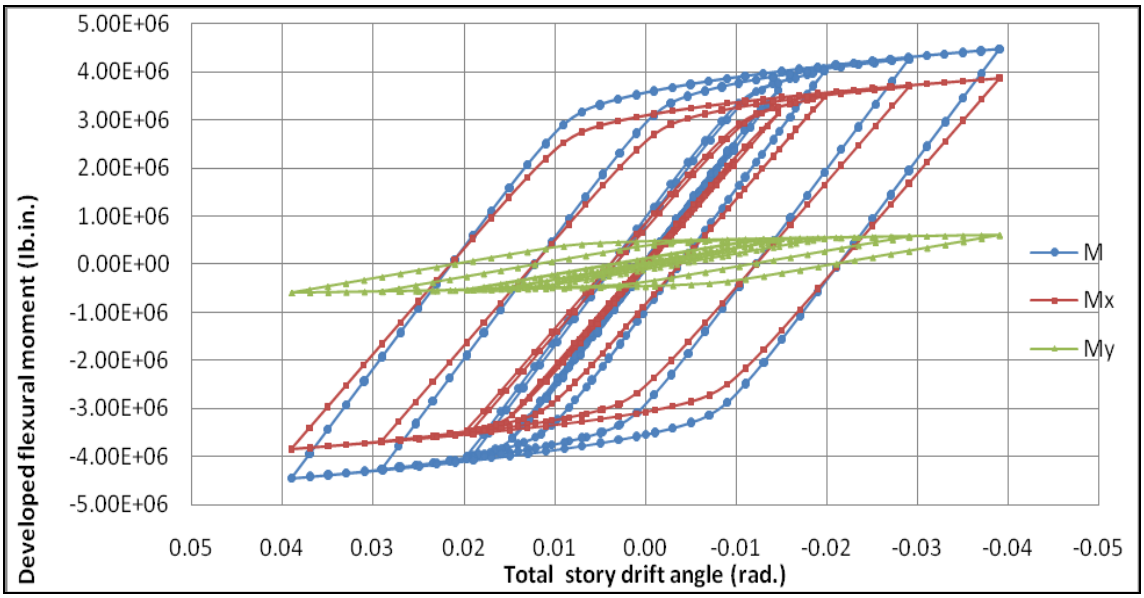

Figure 10: The developed flexural moment of each part of column at section 2 versus total story drift angle of the specimens in cyclic loading.

4 CONCLUSIONS

- While in the bending of flanged cruciform column, there is only out-of-plane bending of column web to constrain two section together and Due to low out-of-plane bending capacity of webs, extra measures is needed to push the two parts of section to behave in the same manner in order to whole section plane remains plane.
- The placement of the continuity plates in connection regions is necessary to mobilize flexural capacity of both W sections.
- Although providing continuity plates causes to develop flexural moment in both w sections, but column section plane does not still remains plane and flexural moment distribution of column does not follow proportionally the weak axis stiffness to strong axis stiffness ratio of each W sections.
- Due to the discontinuity of the load paths, it is not possible to use common doubler plates to reduce panel zone shear force in two directions and also local increase in column web thickness in the panel zone region is not possible for flanged cruciform section are fabricated from rolled sections, so it is needed to provide other paths to transfer the shear demands in panel zone regions.
- Inclined vertical plates with same thickness of column flanges, provide effective parallel accordion paths to resist panel zone shear forces. In addition, these plates cause that flexural moment participation of each part of flanged cruciform section becomes closer to the assumption of the weak axis stiffness to strong axis stiffness ratio of two perpendicular W sections.
- Due to existence of inclined vertical plates in panel zone regions, web shear yielding of strong part of column does not affect the flexural moment distribution in each part.
- These new load paths show quite stable inelastic behavior when subjected to cyclic lateral displacement loading.

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