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RESEARCH NOTE

# Evaluation of Column-Tree Moment Resisting Connection with End Plates and Haunched Beam

A. Bahaarmast, J. Razaaghi*

*Department of Civil Engineering, Faculty of Engineering, University of Guilan, Rasht, Iran*

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*ABSTRACT*

The column-tree constructional scheme allows for a reliable and convenient erection of moment resisting steel frames. Strengthening of column-tree connection with a haunch part can significantly improve seismic behavior of beam-column joint. In the current study, behavior of column-tree connections with haunched beams and end plate splices have been investigated. Special attention was paid for evaluation of the effect of haunch section length and presence of the beams bottom flange. Nonlinear finite element analysis has been used for numerical analyses. The results indicate an improvement of the load bearing capacity of specimens when a longer haunch was used and also when beam bottom flange existed in the haunch part. A direct relation between haunch length and the amount of stress in splice plates was also observed. However, all connections showed ductile behavior during cyclic analysis and specimens with longer haunch revealed more desirable behavior. It was concluded that despite some deficiencies observed during the numerical tests, in presence of a longer haunch and also the beam's bottom flange, a noticeable improvement of the general behavior of connection can be achieved.

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## 1. INTRODUCTION

Moment resisting frames are frequently used to dissipate earthquake energy in structures built in regions of high seismic risk. After damage and lack of ductile behavior observed during 1994 Northridge earthquake [1, 2], new details have been proposed and several studies have been conducted to improve the seismic performance of steel structures and specially their beam-to-column connections [3, 4]. All post Northridge connections were examined in order to reach maximum plastic capacity and form plastic hinge in beam section, away from beam-column interface and also to confirm essential condition of stiffness, resistance and ductility [5]. Haunch connections were one of the proposed post Northridge connections which widely studied [6-8] and declared as a suitable solution for modification of existing connections and also application in new structures.

A column-tree system is one of the constructional schemes used to build moment-resisting steel frames. The column-tree is fabricated by welding stub beams to the column in the shop. A mid-portion of the beam is then spliced to the stub beam to form the moment-resisting frame. Unlike the pre-Northridge connection case, the critical welding of the beam-to-column joint is performed in the shop, which can provide an excellent quality [9, 10], nonetheless a few number of deficiencies were reported during 1995 Kobe earthquake [11]. Several studies have been recently conducted to evaluate and improve the ductile behavior of column-tree connections. Chen et al. performed numerical and experimental investigations on widened flange connections without weld access hold and tapered flanges as a modified configuration of column-tree connections [9, 12]. Lee et al. experimentally tested and investigated the effect of stub beam length on cyclic behavior of column tree connections [13]. Since haunched column-tree connections are commonly utilized in steel moment resisting frames, this study aims to evaluate the effect of haunch length and beam's bottom flange on the behavior of column-tree moment resisting connections. In order to reach this goal, six specimens were simulated and numerically analyzed by

*Corresponding Author's Email:javadr@guilan.ac.ir (J. Razaaghi)

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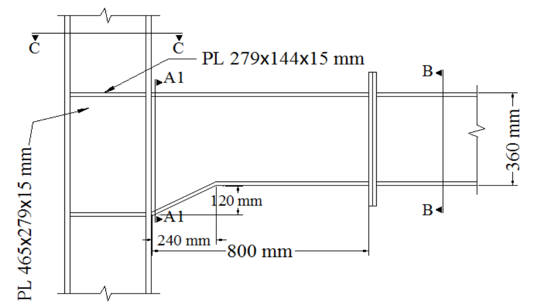
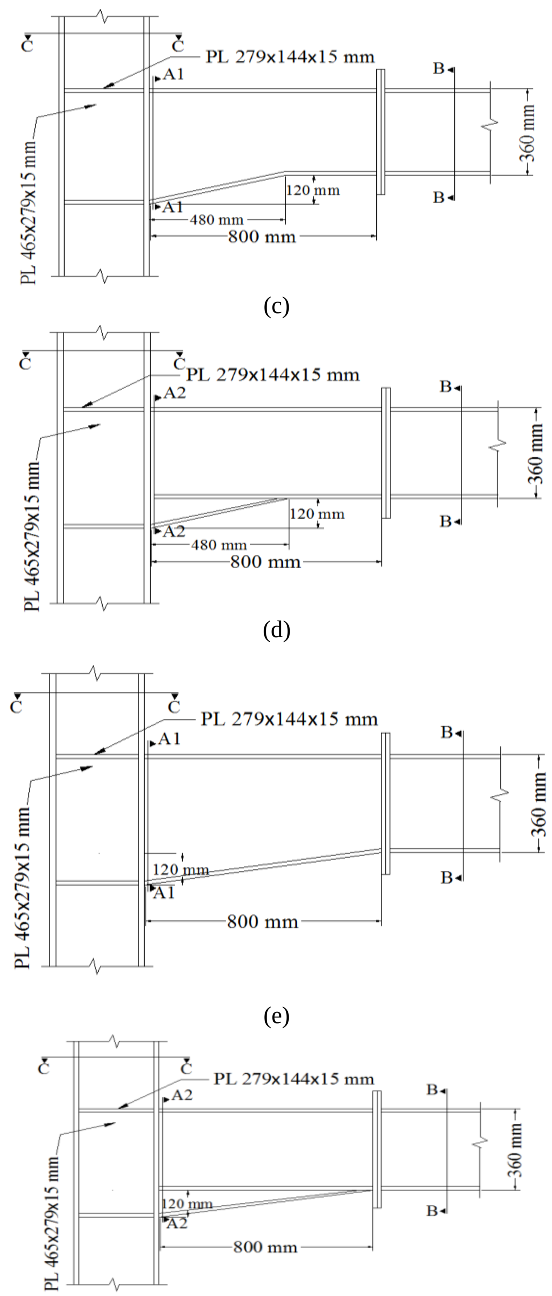
finite element technique and their behavior were carefully studied and compared.

## 2. CONNECTION DETAILS AND MATERIAL ROPERTIES

The beam and column properties were chosen from the first story of a purposely designed six story building, 20x20 meters in plan and 18 meters in height. Six different configurations of H-shaped beams and columns have been considered for modeling and analysis. Beams dimension are 360x250x10x15 mm and column dimensions are 320x300x11.5x20.5 mm. The numbers represent sections depth, flange width, web thickness and flange thickness, respectively. The beam length used for finite element modeling (FEM) was 2500 mm and the column height was 3000 mm. Other parts include haunches, beam splice plates, continuity plates and doubler plates which have been properly used for each specimen. Stub and link beams are connected by end plates and 20 mm bolts. The specimens were divided to three groups with two different configurations at each one. The first group includes specimens of 0.3L´ (240 mm) haunch length while the second and third groups haunches length are 0.6 L´ (480 mm) and L´ (800 mm), respectively where L´ is the length of stub beam. Beam bottom flange was supposed to be discontinued in the haunched area of one specimen in each group.




(a) (b) (c) (d) (e) (f)

**Figure 1.** Specimens geometry and configuration
(a) HA-F (b) HA-F-NC (c) HA-F-LWF (d) HA-F-LF (e) HA-F-CWF (f) HA-F-CF

The specimens were coded as HA-F, HA-F-NC, HA-F-LWF, HA-F-LF, HA-F-CWF and HA-F-CF and are presented in Figures 1 and 2. Bilinear elastic-perfectly plastic stress-strain curve of ST37 has been considered for modeling steel components. The same behavior has been hired to model the A490 bolts. Material properties are defined in Table1.

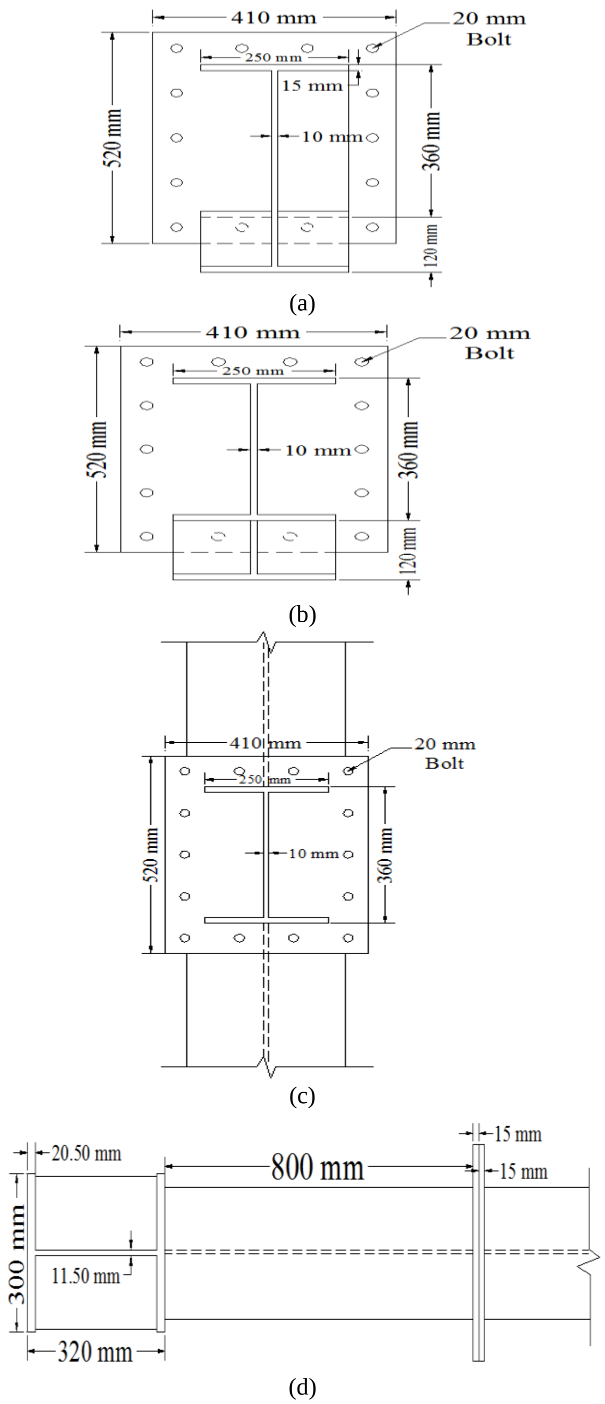



**Figure 2.** Specimens cross sections (a) A1-A1 (b) A2-A2 (c) B-B (d) C-C

**TABLE 1.**Material Properties

| Member | Steel Type | Stress (MPa) | Strain |
|---|---|---|---|
| Beam, Column, Haunch, Plates | A37 | 240 | 0.02 |
| | | 370 | 0.2 |
| Bolts | A490 | 794 | 0.00386 |
| | | 1035 | 0.0135 |

## 3. ANALYSIS PROCEDURE AND FE MODEL

Nonlinear analyses were performed by making use of ABAQUS package. It uses a displacement convergence criterion with the norm of $5\times10^{-5}$ as well as a combination of incremental and iterative procedures and Newton method to solve the nonlinear algebraic equations [14]. Software default values were chosen for minimum and maximum size of displacement increment, i.e. $10^{-6}$ and 0.1.

All specimens were subjected to the loading sequence specified in AISC seismic provision [15] for cyclic testing of moment-resisting connections. The protocol contains 6 cycles at the story drift angles of 0.375, 0.5 and 0.75% rad. It will continue with 4 cycles of 1% rad and 2 cycles of 1.5, 2, 3% and goes on until failure occurs. This loading sequence was hired for all specimens until 6% rad drift angle and is shown in Figure 3. To conduct the loading procedure, a predetermined cyclic displacement history was applied at the beam tip and Pinned boundary condition was also considered at the ends of the column. Figure 4 illustrates the location of loading sequences and specimen's boundary condition.

All beams with/without haunch were connected to column flange by CJP groove welds, which have no considerable effect on connection's stiffness hence the weld itself was not modeled in this study.

The areas in vicinity of beam-column interface were meshed with fine mesh sized between 10 to 20 mm while the areas away from the beam-column interface were modeled with a coarser mesh, sized between 20 to 40 mm. Three dimensional continuum solid brick shape elements with 8 nodes named as C3D8R were used in finite element models.

## 4. ANALYSIS RESULTS

The envelop curves shown in Figure 5 represent beam tip load versus story drift angle describing global behavior of specimens. The story drift angle is computed as the ratio of the beam tip displacement to the distance between the beam tip and the column centerline. The results indicated that application of longer haunches and also presence of beam's bottom flange led to a more reliable behavior. Comparison of beam tip load in corresponding positive and negative cycles revealed a less than 5% difference for each specimen which demonstrated the similarity of their global behavior.

Maximum beam tip load versus story drift angle curves are presented in Figure 6. While all specimens obtained almost the same elastic behavior before a story drift angle of 1%, specimens with longer haunches demonstrated higher load bearing capacity in inelastic cycles. Although presence of beam's bottom flange did not significantly change the load bearing capacity of specimens with short haunches, it improved the behavior of HA-F-CF and made a 11% increase on its load bearing capacity in comparison with HA-F-CWF.

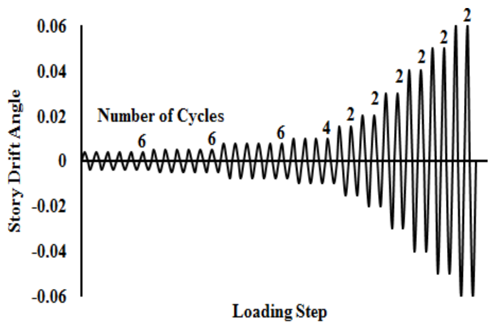



**Figure 3.** Loading sequence

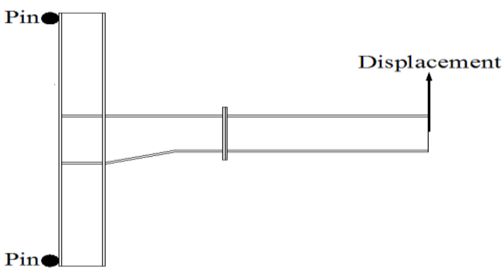



**Figure 4.** Loading sequences location and boundary condition

All specimens were able to develop their full load bearing capacity and even in highly inelastic story drift angles, no sign of deterioration was observed. HA-F and HA-F-CF showed the least and the most load bearing capacities equal to 183.01 KN and 237.52 KN in story drift angle of 6%, leading to a 23% difference.

The envelop curves of moment at the column face versus story drift angle of specimens are presented in Figure 7. To elucidate the effect of haunch section in flexural strength of specimens, the moments were compared with the plastic moment of a similar haunch-less model. The flexural strength ratio of each specimen against the plastic moment of the unreinforced model is presented in Table 2. $M/M_p$ ratio can be considered as a criterion to compare the flexural strength of specimens.

The moment-rotation relations of specimens are displayed in Figure 8. Specimens with longer haunches represented higher moment capacity. While HA-F experienced considerably higher amounts of rotations, all other models rotated almost the same. The maximum rotation value of HA-F was more than twice as the other specimens. On the other hand, flexural resistance of HA-F-CF was notably higher than others specimens at the same rotations. It was also revealed that presence of beam bottom flange can increase the flexural strength of connection.

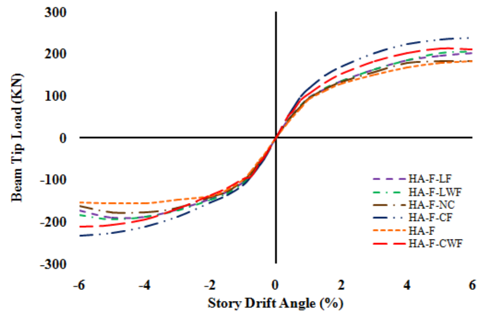



**Figure 5.** Beam tip load versus story drift angle envelop curves

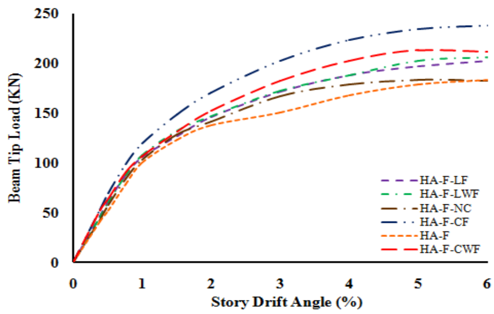



**Figure 6.** Maximum beam tip load versus story drift angle

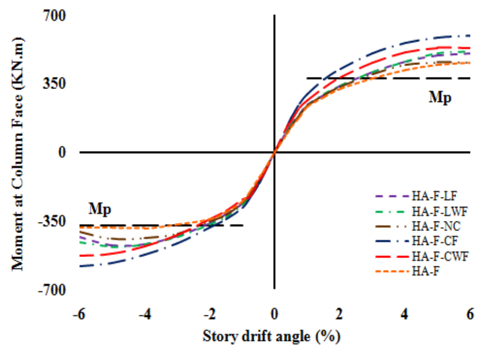



**Figure 7.** Moment at column face versus story drift angle

**TABLE 2.** Flexural strength of specimens

| FEM | MomentAt column Face (KN) | Plastic Moment (KN) | $M/M_p$ |
|---|---|---|---|
| HA-F | 457.5 | 375.84 | 1.22 |
| HA-F-NC | 459.2 | 375.84 | 1.22 |
| HA-F-LWF | 513.6 | 375.84 | 1.37 |
| HA-F-LF | 505.9 | 375.84 | 135 |
| HA-F-CWF | 527.9 | 375.84 | 1.40 |
| HA-F-CF | 593.8 | 375.84 | 1.58 |

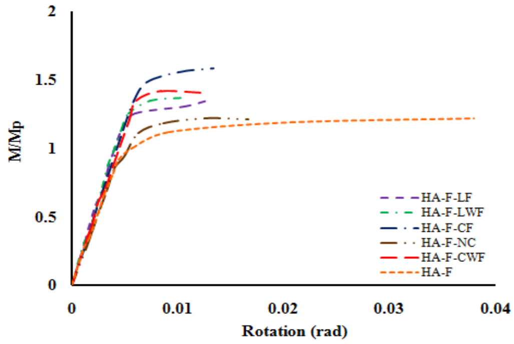



**Figure 8**. Moment-rotation relation

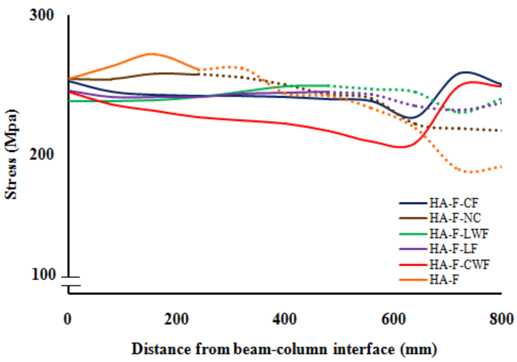
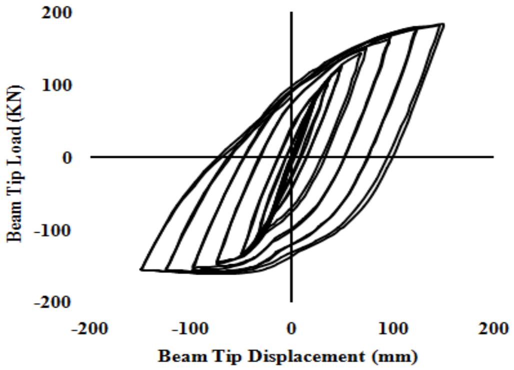



**Figure 9.** Von Mises stress distribution along the top flange of stub beam

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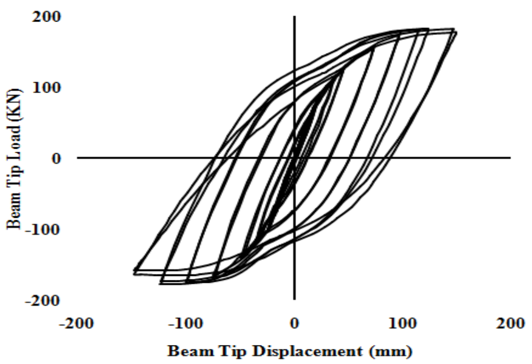


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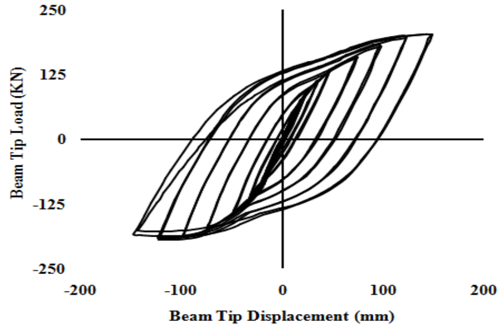


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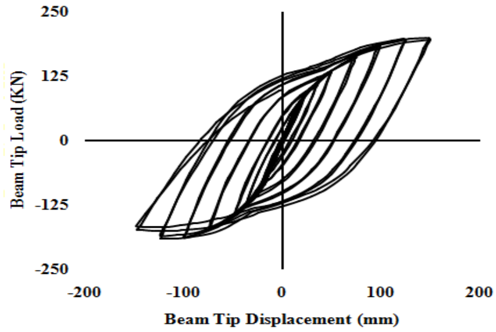


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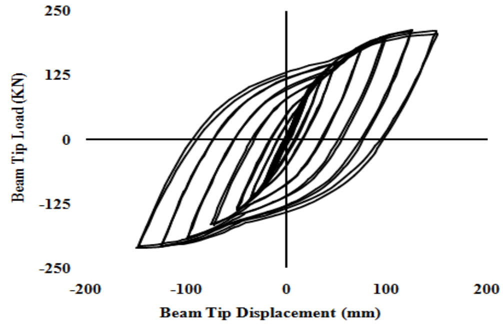


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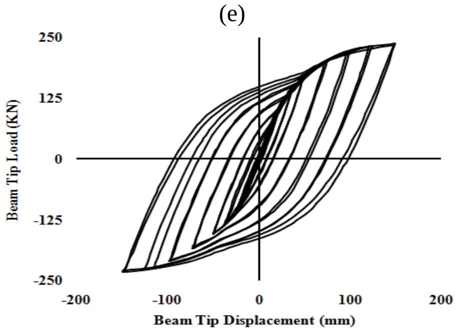



(f)

**Figure 10.** Beam tip load versus beam tip displacement hysteresis loops
(a) HA-F (b) HA-F-NC (c) HA-F-LWF (d) HA-F-LF (e) HA-F-CWF (f) HA-F-CF

Von Mises stress distribution at the beam's top flange along the web centerline of stub beam is presented in Figure 9. The evaluation has been conducted in story drift angle of 4%. The solid parts of the curves represent the haunch-reinforced length of the beams and the dotted parts show the rest of the stub beams length. The amount of stresses decreased continuously along the length of the beam's top flange for HA-F and HA-F-NC. In HA-F-LWF and HA-F-LF which had longer haunches than the first group, the stress distribution was observed almost steady. The results indicated that in HA-F-CWF and HA-F-CF, which had the longest haunches, the amounts of stress gradually decreased as the distance from column face increased. However, it significantly increased at the end plates location and reached to the same and even greater amounts of the column face.

The load-displacement hysteresis loops indicated that all specimens have shown ductile behavior and exhibited proper energy dissipation under cyclic loading, Figure 10. However, all models showed acceptable behavior, the larger area under the load-displacement curves of specimens with beam's bottom flange indicates more efficient energy dissipation ability and also more reliable ductile behavior. Comparing the length of the specimens, HA-F-CWF and HA-F-CF which had the longest haunches, represented the most proper cyclic behavior. As a conclusion, it can be stated that the specimens with the benefit of longer haunch and beam's bottom flange are able to accomplish a more reliable cyclic behavior.

It was demonstrated that maximum values of PEEQ took place around the centerline of beam top flange groove weld. This is true for all specimens except for HA-F where maximum PEEQ was observed at the extreme edge of flange. Although the results were almost the same in other specimens, HA-F-LWF and HA-F-LF had better behavior and PEEQ values were smaller in these two specimens

Plastic equivalent strains (PEEQ) of specimens have been studied to evaluate the inelastic behavior of connections in vicinity of column face. Plastic equivalent strain has been used by other researchers as a criterion of plastic strain demand in pervious numerical studies [9, 16-18]. A higher PEEQ index indicates a higher demand for plastic strain [9], hence higher amount of PEEQ index shows the damage possibility in beam- column interface. Since special moment resisting frames are supposed to sustain at least story drift angle equivalent to 4%, PEEQ indices are presented in story drift angle of 4%. Distribution of PEEQ indices of different configurations along the beam top flange groove weld and haunch flange groove weld are presented in Figure 11.

PEEQ distribution in haunch flange had the same condition; all specimens represented the same results except for HA-F. However, the maximum value in this specimen took place around its centerline.

Splice plates behavior was evaluated through Von Mises Stress distribution in different story drift angles. Figure 12 illustrates the maximum stresses in each specimen. Although all specimens behaved similarly; the amounts of stresses were different. Comparing the stresses values in 4% story drift angle, it was concluded that the amount of stress in all specimens is higher than 200 MPa. Generally, HA-F had the lower limits of stresses, HA-F-NC, HA-F-LWF and HA-F-LF showed almost the same amount of stresses until 5% story drift angle while in last steps of loading, HA-F-LWF and HA-F-LF experienced greater stresses. The maximum values observed in HA-F-CWF and HA-F-CF were higher than 240 MPa (equal to their yield stress) in 4% story drift angle.

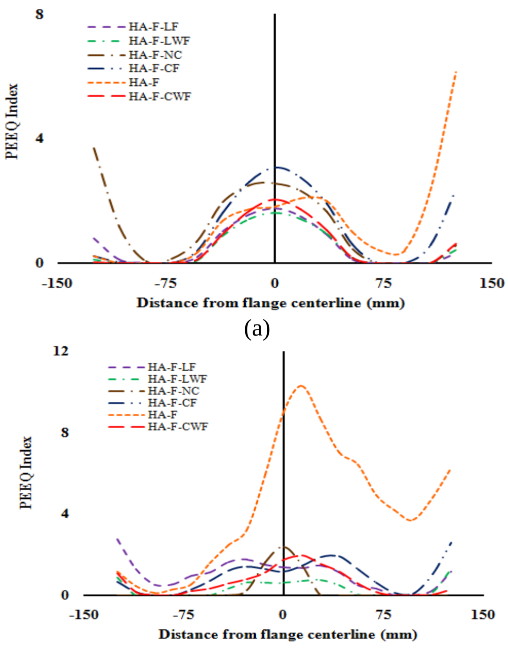



**Figure 11.** Distribution of plastic equivalent strain (PEEQ) at 4% story drift angle
a) Along beam top flange groove weld
b) Along haunch flange groove weld

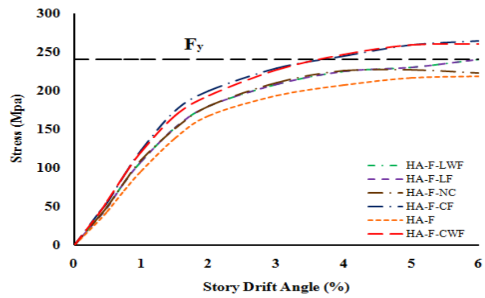
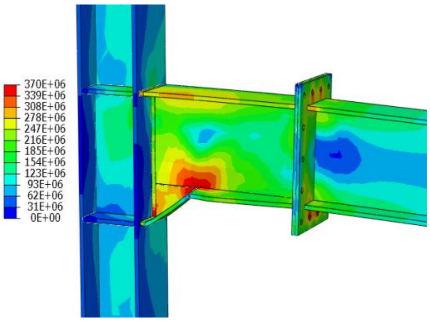



**Figure 12.** Von Mises stress versus story drift angle in splice plates

Therefore, early yielding of splice plate which is one of the failure modes of column-tree connections can take place in this configuration. Therefore, it requires further researches to check the adequacy of design parameters for them.

As special moment resisting frames are supposed to sustain the story drift angle of at least 4% rad, there should be no signs of lateral-torsional or local buckling in proposed connections. Ergo fulfilling this criterion is one of the most important conditions in order to use a connection in special moment resisting frames. To investigate the buckling phenomenon in beams web and flanges and also evaluate the stress distribution in connections, Von Mises stress contours and deformed shapes of specimens in story drift angle of 4% are presented in Figure 13.

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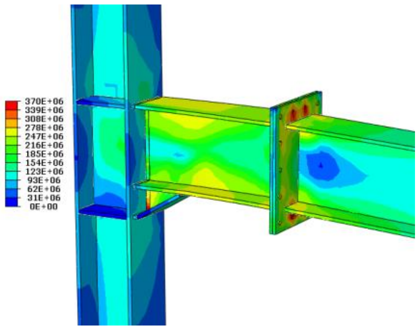


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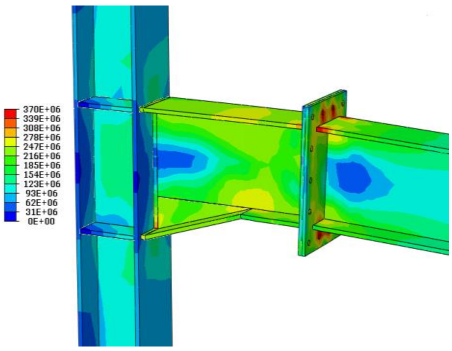


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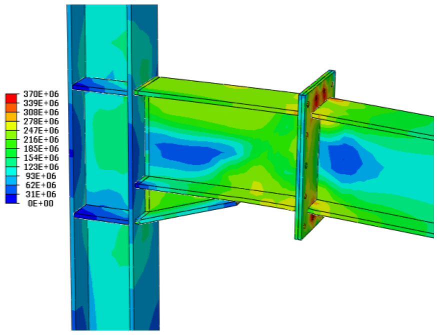


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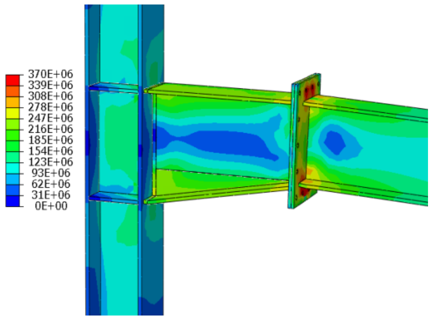


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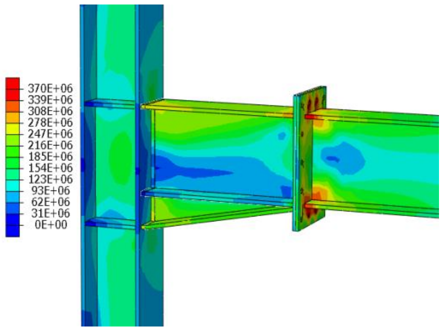


(f)

**Figure 13.** Von Mises stress distribution and deformed shape of subassemblages at 4% rad story drift angle
a) HA-F (b) HA-F-NC (c) HA-F-LWF (d) HA-F-LF (e) HA-F-CWF (f) HA-F-CF

All specimens sustained the loading cycles except for HA-F. Local buckling is observable at haunch flange and beam's bottom flange after haunched area and it means that HA-F cannot be used in special moment resisting frames. The other matter of importance is that enlarging the haunched area led to a larger stress free zone and absence of the beam's bottom flange caused the neutral axis to move upward.

## 5. CONCLUSIONS

This paper investigated the behavior of the column-tree moment resisting connection with end plates and haunched beam. Nonlinear finite element analysis employed to conduct a comprehensive numerical study. The main objective of the study was to evaluate the effect of haunch length and beam's bottom flange in haunched area. Analysis results indicated that configurations with longer haunches have greater load bearing capacity and flexural strength and will act more properly in presence of beam's bottom flange. The same result obtained through evaluation of hysteresis loops of specimens. To elucidate the inelastic behavior of specimens in beam-column interface and groove welds distribution of plastic equivalent strain was studied. Among the evaluated models, the specimens with haunch length of 0.6 stub beam length represented the best results leading to a smaller possibility of the beam weld fracture and other brittle damages in this area under cyclic loading. Moment-rotation relation demonstrated that except for one specimen, all models showed almost the same amounts of rotations. It was also resulted that with a longer haunch and also in presence of beam's bottom flange a greater flexural strength can be obtained. Stress distribution in splice plates in different story drift angles revealed that specimens with longer haunches confronted higher amounts of stresses. In the specimens with the longest haunches, splice plates were yielded before the threshold story drift angle of 4%. Therefore, in order to improve their seismic behavior, it is recommended to take measures like using thicker splice plates or reinforcement of the splice plates by stiffeners. Further numerical studies and experimental researches are needed in order to generalize the conclusions and evaluate the design parameters.

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A. Bahaarmast, J. Razaaghi

*Department of Civil Engineering, Faculty of Engineering, University of Guilan, Rasht, Iran*

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Ductile Behavior.

**چکیده**

مطالعه ی حاضر رفتار اتصال ستون- درختی تقویت شده با ما هیچه را تحقیق می کند. هدف اصلی مطالعه دریافت میزان تأثیر طول ماهیچه و نیز وجود یا عدم وجود بال پایین تیر در رفتار اتصال می باشد. اتصال ستون- درختی یکی از طرح های اجرایی سازه ای است که به دلیل محاسنش اخیراً بیشتر مورد استفاده قرار می گیرد. تقویت اتصال ستون- درختی با ماهیچه سبب بهبود رفتار اتصال شده و احتمال رفتار شکل پذیر مورد نظر را در آن افزایش می دهد. در این تحقیق جهت دست یابی به نتایج از تحلیل اجزای محدود غیر خطی استفاده شده است. نتایج تحقیق نشان می دهد که در صورت استفاده از ماهیچه های بلندتر و حضور بال پایین تیر ظرفیت باربری نمونه ها بهبود می یابد. رابطه ی مستقیم بین طول ماهیچه و میزان تنش های ورق های اتصال نشان داده شد. اگرچه توانایی رفتار شکل پذیر تمامی نمونه ها با ارزیابی رفتار چرخه ای آن ها مشاهده شد، نمونه های دارای ماهیچه ی بلندتر رفتلر بهبودیافته تری از خود نشان دادند. با وجود برخی نتایج نا مناسب، این نتیجه گرفته شد که استفاده از ماهیچه های بلند تر و وجود بال پایین تیر درحالت کلی باعث بهبود رفتار نمونه می گردند.

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