Parametric study of reduced web section of beam to column welded connection with castellated beam having hexagonal openings and subjected to cyclic loading

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Abstract. In this paper, a finite element (FE) analysis is carried out on a beam to column welded connections with multiple hexagonal web openings. The openings are distributed along the length of the beam which is subjected to cyclic loading protocol proposed by the SAC and FEMA 350 (2000). This study focuses on the interaction of the connections from columns to the perforated beam and mobilization of stresses. Two parameters were addressed: the distance from the face of the column (S), and the web opening spacing (S₀) including closely spaced opening. The study revealed that the design of mostly controlled reduced web section (RWS) connections should be fundamentally based on the distance of the first opening from the face of the column. Three values of the distance (S) are presented (200 mm, 350 mm, and 520 mm). A model with an ideal distance (S) of 350 mm has resulted in an ultimate rotation of 0.039649 rad, which is less than other models, in which rotations were 0.049386 rad and 0.050000 rad, respectively.

1. Introduction

Castellated beam is a name commonly used for a type of expanded beam. It is made by expanding a standard rolled shape in a manner which creates a regular pattern of holes in the web. Castellated beams have been used in a wide variety of applications, such as roof beams and rafters in both simple span and cantilever construction, floor beams and girders for heavy as well as light floor loads, tier buildings, rafter portions of rigid frames, pipe bridges, girts and other special applications. These uses take advantage of the increased strength and the economy of castellated beams. [1]. Castellated steel beams were proved their efficacy in terms of rotational capacity under static and seismic loading. They became a preferred choice for engineers and contractors because of comparable strength and material savings. The behavior of beam-to-column connection using castellated beams is not fully understood; especially in regards to web opening size and spacings. A semi-rigid steel beam-to-column connection with regular hexagon web opening subjected to cyclic loading was examined in the current parametric study. Web opening of regular hexagons has six equal sides and angles and is composed of six equilateral triangles. The web-post buckling behavior was investigated for multiple spaced web opening along the length of the beam with the location of nodes B1, B2, C1, and C2. It has been used for the moment-rotation curves [2].

In the literature; Jamadar A. M. et al.[3] conducted a parametric study of a castellated beam with circular and diamond shaped openings. It was discovered that a castellated beam with diamond-shaped openings proved to be better than circular shaped openings in respect to shear transfer area and local failure.
Hosain and Speirs (1973) analyzed experimental results of twelve castellated steel beam samples to study the effect of holes number on behavior of beams that have the same span length and expansion ratio. The test results had also revealed that the shearing strain distribution along the web weld is not parabolic. The shear strains at the free ends of the weld line are not zero. Test results revealed that elastic optimization (more castellation in a given span) has little effect on limit load and elastic stiffness but affects considerably the ductility of the beam. The beam with the maximum number of castellation recorded the maximum rotation capacity [4]. Hayder W. Al-Thabawee [5] studied the experimental results of six specimens of castellated steel beams and compares them with the control beam (Parent section). It investigated the effect of hexagonal hole dimensions on the ultimate strength and stiffened response of the castellated steel beam. The ultimate strength of the castellated steel beam was increased about (50%) stronger than the original beam.

2. Description of the specimens

The geometrical parameter of the beam to column connection and detailed configuration are illustrated in figure 1 and table 1 as follows:

- The side length of regular hexagon (dₒ).
- The distance from the face of the column to the centerline of the first web opening (S).
- The distance between adjacent web openings taken from their respective centerlines (Sₒ).
- For all parametric analyses, (dₒ) was taken as equal to 0.8hb, where hb is the height of the beam, hence: dₒ = 0.8 * hb = 0.8 * 298.9 = 239.12 mm
- (af) is a weld throat thickness of flange and (aw) is a weld throat thickness of web
- Sₒ = 1.2 * dₒ = 1.2 * 0.8 * hb = 1.2 * 0.8 * 298.9 = 286.94 mm (closely spaced)

Three values of the distance (S), were examined (200 mm, 350 mm and 520 mm). Sₒ is approximately equal to the 96% of beam depth, for closely spaced openings; therefore, it was taken for the current study equal to 1.2 dₒ.
Figure 1. Semi-rigid beam to column connection

Table 1. Summarizing of finite element of parametric studies

| Model | Number of holes | Column face distance (S) | First and Second Web openings |
|-------|-----------------|--------------------------|------------------------------|
| Solid | N/A             | N/A                      | N/A                          |
| 1     | Fully perforated| 200                      | 1.2                          |
| 2     | Fully perforated| 350                      | 1.2                          |
| 3     | Fully perforated| 520                      | 1.2                          |

N/A: Not Applicable

3. Finite element model

The numerical results were obtained from the finite element analysis of models by Abaqus CAE v.6.14 software [6]. It has been compared with the experimental results by Janss et al. [7], the finite element results from Diaz et al. [2], K. D. Tsavdaridis et al. [8] and the results obtained using Eurocode 3 [9], as shown in figure 2. The dimensions of the FE model connection are summarized in table 2. The material properties for all components S355 steel grade and mechanical properties used for the connection are summarized in table 3. Young's modulus, E, equal to 210,000 MPa, and Poisson's ratio, υ, equal to 0.3 [8]. The 10-node solid elements of quadratic tetrahedral were used to mesh beam, column, welds, and load stiffener.
Figure 2. Comparison of moment-rotation (Mj- φj) curves

Table 2. Dimensions of the connection (all in mm and mm²) [7]

| Beam (IPE 300) | Column (HE 160B) | Load and stiffener | Weld throat thicknesses |
|----------------|-------------------|--------------------|-------------------------|
| bfb 150        | Width of column bfc 160 | Lload 3000 | afix 6.0 |
| Width of beam | hb 300 Height of column hc 160 | | awfix 4.0 |
| Height of beam | Thickness of flange tfb 10.7 | | |
| Thickness of web twb 7.1 | Thickness of web twc 8 | | |
| Length of column (H) 3625.0 | | | |

Weld throat thickness of flange and Weld throat thickness of web

Table 3. Mechanical properties of the connection (all in MPa) [2]

| Beam | Column | Weld |
|------|--------|------|
| Fu,beam 445.0 | Fu,column 422.2 | Fu,weld 463.4 |
| Fy,beam 308.5 | Fy,column 289.4 | Fy,weld 291.5 |
4. Loading and analysis

Point load (P) (42.26 kN) is applied on solid beam, model 1, model 2, and model 3 as shown in figure 1. It has incrementally of ultimate force at a distance Load (m) from the face of the column onto the top of the beam flange of ultimate moment approximate (126.78 kN.m) with the experimental results by Janss et al. [7]. The monotonic sequence of analysis in figure 2 is described as a comparison of the moment-rotation curves of FE models. Comparison of experimental results of ultimate moment and rotation of solid beam, model1, model 2 and model 3 by Janss et al. [7], where the ultimate moment of solid beam increased about 4% and reduced about (4.6%, 4% and 3.3%) for model1, model2 and model3 respectively. Regarding to ultimate rotation all models increased about (39.6%, 8.1%, 10.2% and 11.9%) for solid beam, model1, model2 and model3 respectively (table 6).

The specimen is loaded cyclically following the SAC loading protocol recommended by [10], in arrange to distinguish its primary seismic reaction characteristics. A total of 32 cycles, equivalent to 64 applied displacements, are computed for all FE models as written in the results in table 5, model 2 had depicted of SAC cycling loading sequence at last two accumulative of load-steps (60 and 64) approximately with maximum displacement 0.122168 m as plotted in figure 3. Table 4 summarizes the displacements values, ∆Lc, applied as load-steps [10]. Maximum end displacement for (64) load-steps (accumulative) of model 1 and model 3 are 0.1540625 m as shown in figure 3 and figure 5. Model 2 achieves (61) load-steps (accumulative) with end displacement 0.13095 m as depicted in figure 4.

- Distance LLC from the centerline of the column to the centerline of the load stiffener is equal to: $LLC = L_{load} + hc/2 = 3000 + 162.5/2 = 3.08125$ m

| Number of cycles | Peak deformation φ (radians) | Load-steps (accumulative) | End displacements ∆Lcd (m) |
|------------------|-------------------------------|---------------------------|---------------------------|
| 6                | 0.00375                       | 12                        | 0.011554688               |
| 6                | 0.005                         | 24                        | 0.01540625                |
| 6                | 0.0075                        | 36                        | 0.023109375               |
| 4                | 0.01                          | 44                        | 0.0308125                 |
| 2                | 0.015                         | 48                        | 0.04621875                |
| 2                | 0.02                          | 52                        | 0.061625                  |
| 2                | 0.03                          | 56                        | 0.0924375                 |
| 2                | 0.04                          | 60                        | 0.12325                   |
| 2                | 0.05                          | 64                        | 0.1540625                 |

$^{d}$ End displacements ∆Lc equal to: LLC × φ
Figure 3. SAC cycling loading sequence of model 1

Figure 4. SAC cycling loading sequence of model 2

Figure 5. SAC cycling loading sequence of model 3
5. Results of FE Analysis

5.1. Hysteretic behavior

The moment–rotation (M–θ) curves were determined from each investigation to show the hysteretic execution of the beam-to-column connections and particular characteristics including the starting rotational stiffness, rotational capacity, strength (ultimate moment capacity), and the amount of energy dissipated. Such hysteretic curves were established from data inferred by [6]. Vertical y-axis nodal displacements were recorded at the column centerline and at the end of the beam where the stiffener is found, as well as nodal responses comparing to the applied displacement for each load-step. The rotations were determined from:

$$\varphi = \frac{\text{Stiffener End Displacement} - \text{Column Centerline Displacement}}{\text{Distance from Column Centerline to Stiffener Location}}.$$  

The moment capacity was taken at the column centerline and corresponds to:

$$M = |\text{Reaction}| \times \text{Distance from Column Centerline to Stiffener Location}.$$  

The initial rotational stiffness is calculated from the first cycle of the analysis and corresponds to:

$$K_i = \frac{M}{\varphi}.$$  

The web opening area (WOA) corresponds to the area of the opening:

$$\text{WOA} = \frac{1}{2} \times 3 \times (3)^{1/2} \times (d_0)^2 = \frac{1}{2} \times 3 \times (3)^{1/2} \times (239.12)^2 = 148553.77 \text{ mm}^2.$$  

The energy dissipated, $E$, is equivalent to the area under the hysteretic curve which is computed using the trapezoidal rule where:

$$\text{Area Increment} = (\varphi_2 - \varphi_1) \times \left[ \frac{1}{2} \times (M_1 + M_2) \right].$$  

Table 5 summarizes the results derived from the hysteretic curves. Initial rotational stiffness was a great value for the solid beam model. The rotational stiffness decreased by 32.1%, 14.1%, and 6.2% for model 1, model 2, and model 3, respectively. Also, rotational ductility for solid beam was higher than other models. The rotational ductility decreased by 42.9%, 46.4%, and 19.2% for model 1, model 2, and model 3, respectively. The losing in dissipated energy for the solid beam model was a bigger magnitude than others. The (E) decreased by 34.2%, 55.9%, and 36.7% for model 1, model 2, and model 3, respectively. High decrease of percent of dissipated energy loss in model 2 that it is given a great indication on his analysis.

### Table 5. Summarizing of results of hysteretic curves for models

| Specimen       | Number of hexagonal opening | Yield moment $M_y$ (kN.m) | Ultimate moment $M_u$ (kN.m) | Yield rotation $\varphi_y$ (rad) | Ultimate rotation $\varphi_u$ (rad) |
|----------------|-----------------------------|---------------------------|-----------------------------|----------------------------------|------------------------------------|
| Solid Beam     | N/I                         | 74.15                     | 129.65                      | 0.006254                         | 0.059334                           |
| 1              | 10                          | 73.56                     | 124.13                      | 0.009111                         | 0.049386                           |
| 2              | 10                          | 70.55                     | 118.84                      | 0.007789                         | 0.039649                           |
| 3              | 9                           | 60.26                     | 124.79                      | 0.006523                         | 0.050000                           |

N/I: Not Included

| Specimen       | Rotation ductility $D\varphi$ | Initial rotational stiffness $K_i$ (kN.m/rad) | Web opening area (mm$^2$) | Dissipated energy $E$ (kN.m)(rad) |
|----------------|-------------------------------|-----------------------------------------------|---------------------------|----------------------------------|
| Solid Beam     | 9.49                          | 8602.08                                       | N/I                       | 4.851                            |
| 1              | 5.42                          | 5839.37                                       | 148553.77                 | 3.194                            |
Table 6. Summarizing of results of monotonic curves for models

| Specimen | Number of hexagonal opening | Yield moment My (kN.m) | Ultimate moment Mu (kN.m) | Yield rotation φy (rad) | Ultimate rotation φu (rad) |
|----------|-----------------------------|------------------------|---------------------------|-------------------------|---------------------------|
| Solid Beam | N/I                         | 79.84                  | 131.93                    | 0.011907                | 0.069799                  |
| 1        | 10                          | 77.75                  | 120.98                    | 0.012495                | 0.054046                  |
| 2        | 10                          | 74.89                  | 121.70                    | 0.011755                | 0.055100                  |
| 3        | 9                           | 73.59                  | 122.58                    | 0.011366                | 0.055948                  |

N/I: Not Included

5.2. Stress distribution
The Von-Mises stresses for all four models were plotted (Figures 7, 9, 11, and 13) to examine the behavior of the connections. The stress distribution in the vicinity of the beam to column connection was highlighted independently. To prepare the stresses away from the column and the connection gathering, a Vierendeel mechanism ought to be formed. Local bending moments, known as Vierendeel moments lead to the creation of numerous plastic hinges above and below the connection at specific points [10,11]. Stress values in Shear Panel Zone (SPZ) beam and column in have reached their ultimate values, which indicate that a failure has occurred in these locations and the authors were aware of that overloading up to the maximum value to determine failure load. Failure of the specimen is identified by attaining the specimen the ultimate stresses because the stress-strain curve drops down beyond this point and the fracture will initiate. However, fracture has not been considered as it is out of the scope of the current study.

6. Results: Beams Models
6.1. Solid Beam Model
The solid beam model failed to converge at load-step 56 corresponding to cycle 28 (figure 6). It reaches a higher maximum moment and ultimate rotational capacity of 129.65 kN.m and 0.059334 rad respectively. In the solid beam model (figure 7) is noticed that the stress is concentrated in the SPZ of the column to beam connection. The SPZ in beam and column have reached to the ultimate strength of 460.8 MPa and 807.67 MPa respectively.

6.2. Model 1
Model 1 achieved all 64 load-steps corresponding to the 32 cycles (figure 8) with 10 hexagonal opening and having (S) equal to 200 mm. Ultimate moment and rotational capacity of 124.13 kN.m and 0.049386 rad, respectively are taken in FE analysis. High distribution stresses of Von-Mises are found surround to hexagonal opening along the beam with very high stresses in the vicinity of the welds plotted in figure 9. The SPZ in beam and column have reached to the ultimate strength of 559.8 MPa and 694.8 MPa respectively.

6.3. Model 2
Model 2 failed to converge at load-step 61 corresponding to cycle 31. The ultimate rotational capacity of 0.039649 rad and maximum moment capacity of 118.84 kN.m was achieved with 10 hexagonal
opening and having (S) equal to 350 mm. The stress in the column web panel zone about 735.62 MPa and the critical stresses of 544.79 MPa are found along the beam, with only some stress in the vicinity of the welds (figure 11).

Failure occurs obtained at a maximum load smaller than 20 % [9]. The initiation of web local buckling is related to strength degradation (figure 10). During the last two cycles (28 and 29), where the strength is reduced from 119.16 kN.m to 118.84 kN.m, the out-of-plane displacement increased by about 2%. Lateral-torsional buckling occurred because there were no bracing along the beam web neither by stiffeners nor by transverse beams/diaphragms. As such, lateral torsional buckling has commenced before local flange buckling because beam dimensions were relatively large enough to generate a global buckling mechanism.

6.4. Model 3

Model 3 achieved all 64 load-steps corresponding to the 32 cycles (figure 12) with 9 hexagonal opening and having (S) equal to 520 mm. Because of this, it achieves an ultimate rotational capacity of only 0.05 rad, approximately similar to model 1 with (S) equal to 200 mm. Also, model 3 attained a higher maximum moment of 124.79 kN.m than model 1. It is concluded that the changes in (S) of model 3 resulted in a very similar behavior to model 1. The SPZ in the column reaches to ultimate strength of 769.95 MPa. The critical zone (approximately 459.34 MPa) is found along the beam, with very low stresses in the vicinity of the welds (figure 13).

![Hysteretic curve of moment-rotation of solid beam](image)

**Figure 6.** The hysteretic curve of moment-rotation of solid beam
Figure 7. Von-Mises stress contour plots of Solid Beam

Figure 8. The hysteretic curve of moment-rotation of model 1
Figure 9. Von-Mises stress contour plots of Model 1

Figure 10. The hysteretic curve of moment-rotation of model 2
Figure 11. Von-Mises stress contour plots of Model 2

Figure 12. The hysteretic curve of moment-rotation of model 3
Figure 13. Von-Mises stress contour plots of Model 3

7. Conclusions: Based on the current study, the following conclusions can be drawn:

1. Both critical distance (S) 520 mm and 200 mm for first web opening, the narrow distance of 200 mm has resulted in the reduction in the ultimate rotational capacity from 0.05 rad to 0.049386 (model 1 and model 3).

2. Model 2 achieves the increase of web opening to 10 given a little value of ultimate rotation. The ideal distance (S) 350 mm has resulted in a perfect ultimate rotational 0.039649 rad. Also, there was a small loss in the dissipated energy and moderate rotational ductility and stiffness from other models.

3. The castellated steel beams are well accepted for using in long span roofing because of its economy and satisfactory serviceability requirement.

4. The behavior of the castellated steel beam is satisfactory for serviceability criteria because of increased depth of the castellated steel beam compared to the original beam. It increases the depth of the castellated steel beam up to 80% more than the height of the beam.

8. References

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