Investigation of Cold - Formed I - Section Castellated Beam with Cellular Openings

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***Abstract*—The cold formed steel structural members are made through cold forming a thin plate which is normally from 1.2 mm to 3 mm and has a section shape with the right purpose. One of the advantages of cold formed steels is that the strength to weight ratio is much higher than that of common hot rolled shapes, thus it can reduce the total weight of structures. Lots of study has been done in optimizing sizes of castellated beams with hexagonal openings, and hence there is need to optimize the beams with other shaped openings. While the local failure associated with the castellated beam can be minimized by providing other shaped openings like circular, diamond shapes etc. Investigation of cold formed I section castellated beam with cellular openings is carried out by varying the size of openings and the spacing between openings. The performance is analyzed using ABAQUS 6.13 by keeping the depth and width of the sections constant. Theoretical investigation is carried out by using North American specification for the design for cold formed steel AISI S-700:2007, Australian/New Zealand design code for cold formed steel AS/NZS 4600:2005. The results predicted using numerical analysis and theoretical analysis are compared and presented. Results showed that, the beam gives better strength results for cellular shaped opening with opening size of 0.4 times the overall depth of the beam. It is also observed that, castellated beams are mostly tends to fail in their local modes of failure.**

Keywords*— Castellated beam, Cellular web opening, Cellular beam, Finite Element Analysis, ABAQUS 6.13, Optimization.

I. 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. The name is derived from this pattern of web holes, because castellated means "built like a castle, having battlements, or regular holes in the walls, like a castle".

It is made by separating a standard rolled shape into two halves by cutting the web in a regular alternating pattern as shown. The halves are rejoined by welding, after offsetting one portion so that the high points of the web pattern come into contact. Some design conditions make it advantageous to increase the depth even more. This is done by adding web plates between high points of the tee sections. These added plates are called "increment plates".

II. REVIEW OF PREVIOUS STUDIES

In recent times, a lot of research work has been carried out for analysis and design of castellated beams, especially with hexagonal openings. There is no universally accepted design method for castellated beam because of complexity in geometry accompanied by complex mode of failure. At present, there are possibly six failure modes of castellated beam namely, formation of flexure mechanism, lateral torsional buckling, formation of Vierendeel mechanism, rupture of welded joint, shear buckling of web post and compression buckling of web post. Various research studies carried out for analysis and design of castellated beams are presented in the following section.

A. Erdal F. and Saka M. P [04]

The authors have studied the load carrying capacity of optimally designed castellated beam with various numbers of holes and spacing. Finite element analysis of same beams is also carried out under the application of centrally applied point load and failure patterns are studied and verified using ANSYS. Study shows that, even though the members are relatively of shorter spans, lateral supports are governing factor for the analysis of beams due to torsional buckling. It is concluded that, the beam fails in Vierendeel mode when the load is applied above the openings while it fails in web post buckling when load is applied in between space of the openings.

B. Jamadar A. M. and Kumbhar P.D [09]

They carried out experimentally as well as analytically using Abaqus (CAE 6.13) of castellated beams provided with circular and diamond shaped openings by following the guidelines given in EUROCODE 3. The software results were validated by comparing it with experimental results. The result indicates that the castellated beam with diamond shaped opening suffers least amount of local failure as more shear transfer area is available as compared to the castellated beams with circular opening. Also load carrying capacity is greater for diamond shape than circular opening.

C. Ehab Ellobody [10]

The author analyzed the castellated beam with circular openings by nonlinear analysis, where the combined modes of buckling of these beams were considered. The behavior was checked for high strength of beam by considering the parameters like imperfection of geometry, remaining stresses and also nonlinear material properties of material were considered. The nonlinear finite element method helped in predicting deflection, failure modes and also the loads causing failure. The result of parametric study shows the cellular beam fails because of combined action of web distortional as well as due to web post buckling mode which

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shows considerably decrease in failure load. Lateral torsional buckling was observed in cellular beams with normal strength while distortion of web and also the web post buckling was observed in cellular with high strength.

D. Wakchaure M.R., Sagade A.V. and Auti V. A [02]

The authors have experimentally studied the behavior of simply supported castellated beams under two point loading (four point bending) by varying the depth of hexagonal openings (and hence the overall depth). Modes of failure of the castellated were examined for different depths of openings. From the experimentation, researchers conclude that the castellated beam behaves satisfactorily up to a maximum depth of 0.6 times the depth of opening (0.6D). Investigators recommend for providing reinforcement (stiffeners) in order to avoid Vierendeel effects caused due to openings.

E. B. Anupriya and Dr. K. Jagadeesan [13]

They studied the analytically shear strength and deflection properties of castellated beams with hexagonal openings using ANSYS14. Study shows that, as the depth of castellated beam increases, the stress concentration at corners as well as at the loading point increases. In order to avoid this, study was also carried out by provision of diagonal stiffeners and also with diagonal and vertical stiffeners (i.e. combined form) in the openings. The results indicate that minimum deflections occur in the castellated beam provided with diagonal and vertical stiffeners (combined form).

III. PARAMETERS OF THE SPECIMEN

Nine different castellated beams with cellular openings were selected by varying depth of openings and the spacing's of openings as shown in Table I.

TABLE I. PARAMETERS CONSIDERED FOR CELLULER SHAPED OPENINGS

Sp. No.	D mm	Do mm	S mm	D/Do Ratio	S/D Ratio	L mm
1	300	120	240	0.4	0.8	1800
2	300	120	300	0.4	1.0	1980
3	300	120	450	0.4	1.5	1860
4	300	150	240	0.5	0.8	1920
5	300	150	300	0.5	1.0	2100
6	300	150	450	0.5	1.5	1950
7	300	180	240	0.6	0.8	1620
8	300	180	300	0.6	1.0	2220
9	300	180	450	0.6	1.5	2040

D – Depth of the specimen
D_o – Depth of openings
S – Spacing of openings
L – Span of the Beam

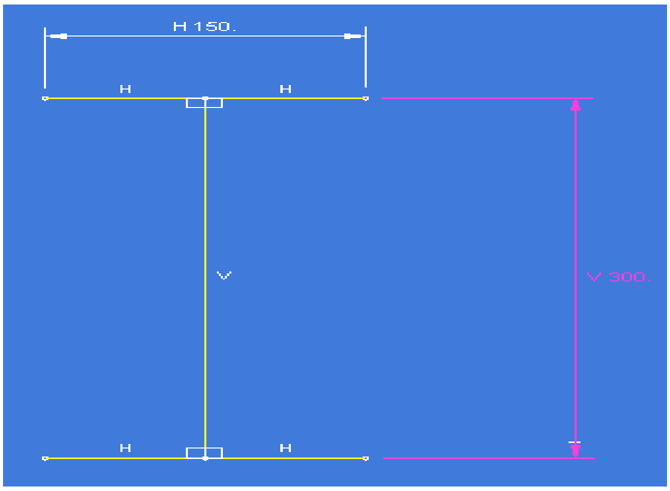

Fig. 1. Geometry of Castellated beam with cellular openings

IV. THEORETICAL INVESTIGATION

A. Design as per North American Specification of Cold formed Steel (AISI S – 100:2007)

1) Nominal flexural section strength

The nominal flexural strength (resistance) M_n, shall be minimum of lateral torsional buckling strength M_{ne}, local buckling strength M_{nl}, distortional buckling M_{nd}.

Effective initial yield moment, $M_y = S_e \times F_y$

Where, S_e = Effective section modulus.

F_y = yield stress.

2) Lateral Torsional buckling strength

The nominal flexural strength (resistance) M_{ne}, for lateral-torsional buckling shall be calculated in accordance with the following:

a) For $M_{cre} > 2.78 M_y$

$$M_{ne} = M_y$$

(No lateral buckling at bending moments less than or equal to M_y)

b) For $2.78 M_y \geq M_{cre} \geq 0.56 M_y$

$$M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10 M_y}{36 M_{cre}}\right)$$

c) For $M_{cre} < 0.56 M_y$

$$M_{ne} = M_{cre}$$

Where,

$$F_e = \frac{C_b \pi^2 E\, d\, I_{yc}}{S_f (L_y K_y)^2}$$

C_b - conservatively taken as unity for all cases
d - Depth of section.
I_{yc} - Moment of inertia of compression portion of section about centroidal axis of entire section parallel to web, using full unreduced section.

$$I_{yc} = \frac{I_{yy}}{2}$$

S_f - Elastic section modulus of full unreduced section relative to extreme compression fibre.
K_y- Effective length factor for bending about y axis.
L_y -Unbraced length of member for bending about y axis.

3) Local buckling strength

The nominal flexural strength (resistance) M_{nl}, for local buckling shall be calculated in accordance with the following

a) For $\lambda_l \leq 0.776$

$$M_{nl} = M_{ne}$$

b) For $\lambda_l > 0.776$

$$M_{nl} = \left(1 - 0.15\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{crl}}{M_{ne}}\right)^{0.4} M_{ne}$$

Where,

$$\lambda_l = \sqrt{M_{ne}/M_{crl}}$$

M_{ne} = a value defined in session (2)
M_{crl} = critical elastic local buckling moment determined by following method.

$$f_{crl} = \frac{K\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2$$

Where,
E - Young's modulus
μ - poisons ratio
t - Thickness of element
W-plate width of element
K - Element (plate) buckling co-efficient
K = 4 for flange and web
K = 0.43 for lip
$M_{crl} = f_{crl}$ X S_e

4) Distortional buckling strength

The nominal flexural strength (resistance) M_{nd}, for distortional buckling shall be calculated in accordance with the following

a) For $\lambda_d \leq 0.673$

$$M_{nd} = M_y$$

b) For $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y$$

Where,

$$\lambda_d = \sqrt{M_y/M_{crd}}$$

M_y - a value defined in section (3)
M_{crd} - critical elastic distortional buckling moment determined by following method

$$M_{crd} = S_f \times F_d$$

S_f - Elastic section modulus of full unreduced section relative to extreme compression fiber.
F_d - Elastic distortional buckling stress

$$F_d = \beta K_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2$$

Where,
β - A value accounting for moment gradient, which is permitted to be conservatively taken as 1.0

B. Design as per Australian/New Zealand Specification of Cold formed steel (AS/NZ 4600:2005)

1) Nominal section moment capacity (M_s)

Based on initiation of yielding (M_s) = Z_e x f_y

2) Nominal member moment capacity (M_b)

The nominal member moment capacity (M_b) shall be lesser of nominal section moment capacity (M_s), and the values calculated by the following methods.

a) Local bucking moment of resistance

$$M_b = Z_c \times f_c$$

Where,
Z_c = effective section modulus calculated as a stress f_c in the extreme compression fibre.

$$f_c = M_c / Z_f$$

Z_f = full unreduced section modulus for extreme compression fibre.
M_c= critical moment calculated as following condition.

i. For $\lambda_b \leq 0.60$,
$M_c = M_y$

ii. For $0.60 < \lambda_b < 1.336$,

$$M_c = 1.11 M_y\left[1 - \left(\frac{10\lambda_b^2}{36}\right)\right]$$

iii. For $\lambda_b \geq 1.336$,

$$M_c = M_y\left(\frac{1}{\lambda_b^2}\right)$$

Where,
λ_b = non dimensional ratio used to determine critical moment

$$\lambda_b = \sqrt{\frac{M_y}{M_o}}$$

Where M_o = elastic buckling moment

$$M_o = \frac{C_b \pi^2 E d\, I_{yc}}{2 \times l^2}$$

b) Distortional buckling moment of resistance

$$M_b = Z_c \times f_c$$

Where,
Z_c = effective section modulus calculated as a stress f_c in the extreme compression fibre.

$$f_c = M_c / Z_f$$

Z_f = full unreduced section modulus for extreme compression fibre.
M_c = critical moment calculated as following condition.

i. For $\lambda_d \leq 0.674$,
$M_c = M_y$

ii. For $\lambda_d > 0.674$,

$$M_c = \frac{M_y}{\lambda_d}\left[1 - \frac{0.22}{\lambda_d}\right]$$

Where,
λ_d = non dimensional ratio used to determine critical moment

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V. FINITE ELEMENT ANALYSIS

The finite element method is a numerical analysis technique for obtaining approximate solutions to wide variety of Engineering problems. Most of the engineering problems today make it necessary to obtain approximate numerical solutions to problems rather than exact closed form solutions. The basic concept behind the finite element analysis is that structure is divided into a finite number of elements having finite dimensions and reducing the structure having infinite degrees of freedom to finite degrees of freedom. The original body of structure is then considered as an assemblage of these elements connected at a finite number of joints called Nodes or Nodal points. This method of analysis has an advantage of that it can take care of any boundary and loading conditions. An engineering problem can be solved in four phases.

A. *Preprocessing*

a) Solid Modelling

The geometric Modeling is done using ABAQUS 6.13. The connectivity between web and flanges for spot welding constrain is done. The dimensions of the created solid model are same as the dimensions of the specimen used in the experimental test. Fig 2 shows the Perspective view of the specimen.

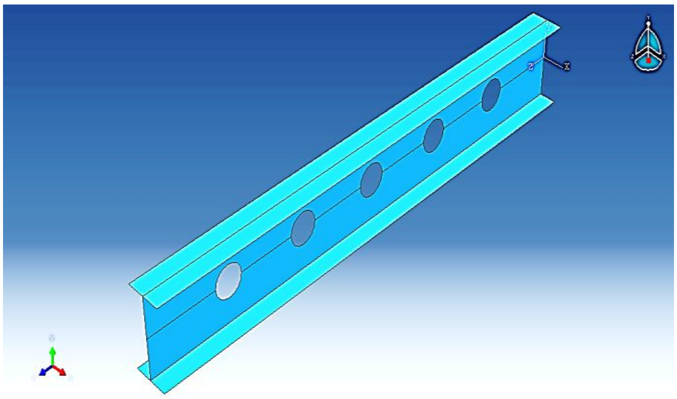

Fig . 2. Perspective view of the specimen

b) Element Type

The type of element chosen for finite element model idealization plays an important role in the prediction of actual behavior of the structure. From the finite element behavior study it is finalized that element 3D DEFORMABLE SHELL is used. Each element are created by individual parts then assembled together.

c) Material Properties

The elastic properties of the material were assigned to the created model of castellated cold formed steel beam. The value of Young's modulus 'E' is given as 2 x 10^5 N/mm^2. The Poisson's ratio is given as = 0.33. The yield stress of the material is 250 Mpa. Thickness of section is assigned to 2 mm.

d) Meshing

The construction of a 3D Finite element model usually requires a variety of mesh generation techniques. In our case global meshing size of 25mm meshing is done. Depending upon the range of fine and coarse meshing the computer time varies to run the process. This figure represents the modeling of the specimen number 1 with meshing size 25 mm and the parts are connected using tie constraint. Fig 3 shows the perspective view of the specimen with meshing.

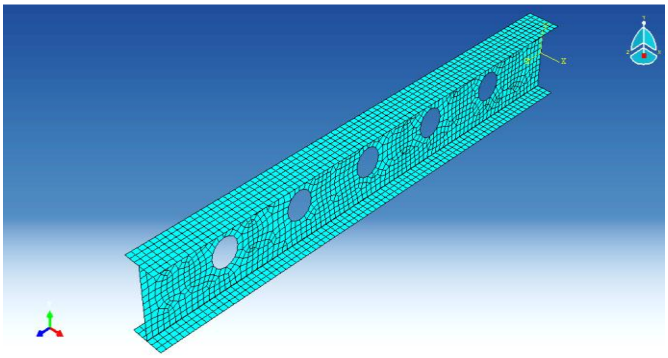

Fig. 3. Perspective View of specimen with meshing

e) Interaction of Elements

Top and bottom flange and web with circular openings are created by separate parts; those parts are welded together by means of tie constraint. The nodes are selected and tie connections are applied.

f) Applying Boundary Condition

Boundary conditions imposed on a finite element solid model is usually given in ABAQUS by specifying the nodal point index and then restraining the necessary displacement component. Here in our problem the castellated beam is analyzed by simply supported end condition. So that displacement components U_x, U_y, and U_z are restrained at one end and displacement components U_x and U_y are restrained at another end.

g) Applying Loads

Loads can be applied to the finite element model in various forms such as applying loads to the key points, lines, areas, elements and at the nodes. For our problem the analysis is carried out for the two points loading on castellated beam. Loads are applied at one third from the both end of the span of beam.

B. *Linear Analysis*

Linear analysis is based on the following assumptions that stress and strain follows Hooke's Law (i.e. linear relationship between stress and strain), deformations are covered by small deflection theory (i.e. small geometric difference between the initial and deformed shape) and other material properties are constant. In this stage problem is subjected to static linear analysis. The errors and warnings are identified at this stage. After nullifying those errors the solution process gets completed and the various deformations are studied. Fig 4 shows the distortional failure of specimen.

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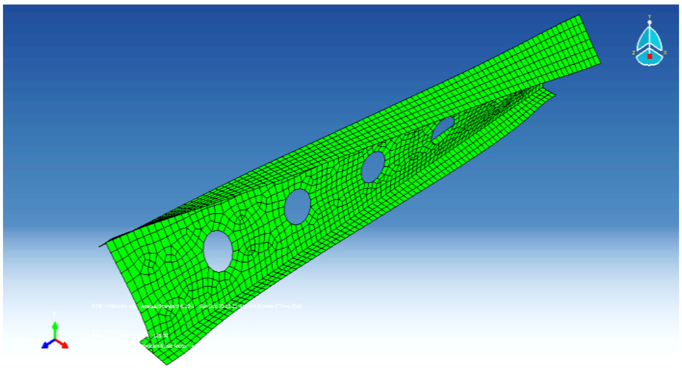

Fig. 4. Distortional failure of specimen

C. Non-Linear Analysis

Nonlinear analysis is based on the following assumptions that stress and strain does not follow Hooke's Law (i.e. nonlinear relationship between stress-strain due to material plasticity), deformations are covered by large deflection theory (i.e. large geometric difference between the initial and deformed shape) and material properties that are temperature dependent.

Any reason causing a variation in stiffness of the assembly being analyzed is potentially a source of non-linearity and therefore requires a non-linear analysis to be captured. It is widely accepted that the three main sources of non-linearity are;

- Plasticity of material (variation of the material Young's modulus will cause the stiffness of the structure to change).
- Large displacements (Stiffness varies as a result of large geometric difference between the initial and deformed shape).
- Contact: if two parts or bodies of the assembly come into contact, or lose contact, or the extent of their contact patch changes, then the stiffness of the assembly also varies.

D. Post Processing

Post processor helps us to view the results obtained from the analysis. The results obtained as nodal solution may be viewed in the tables form or contour plots. These plots are very much useful for us to identify the results such as displacements stresses and strains and also their maximum and minimum values.

VI. COMPARISON OF RESULTS

The moment carrying capacities of castellated beam I section with cellular openings in the web are estimated by theoretical investigations and numerical analyses were discussed here. The ultimate moment M_u obtained by the two code books AISI S-100:2007 and AS/NZS 4600:2005 were compared with the ultimate moment obtained from the numerical analyses by ABAQUS. Table 2 shows the results of Theoretical and Numerical investigation.

TABLE II. RESULTHS OF THEORETICAL AND NUMERICAL INVESTIGATION

Sp. No.	M_{AISI} KNm	$M_{AUS/NZ}$ kNm	M_{ABAQUS} kNm	$\frac{M_{ABAQUS}}{M_{AISI}}$	$\frac{M_{ABAQUS}}{M_{AUS/NZ}}$
1	9.579	10.620	14.649	1.529	1.379
2	9.579	11.352	14.551	1.519	1.282
3	9.579	11.889	14.598	1.524	1.228
4	7.725	9.488	13.715	1.775	1.446
5	7.725	8.824	12.160	1.574	1.378
6	7.725	9.376	13.238	1.714	1.412
7	5.962	8.354	10.582	1.775	1.267
8	5.962	6.496	12.423	2.084	1.912
9	5.962	7.089	11.872	1.991	1.675
Mean				1.701	1.442
Standard Deviation				0.500	0.473

M_{AISI}– AISI S100-2007
$M_{AUS/NZS}$– AS/NZS 4600:2005
M_{ABAQUS}- Numerical Analysis by ABAQUS 6.13

The load carrying capacities of cold formed castellated beam with cellular openings are estimated by theoretical investigations and numerical analyses were discussed here. Chart 1 show the various moment of resistance obtained by the numerical and theoretical analysis.

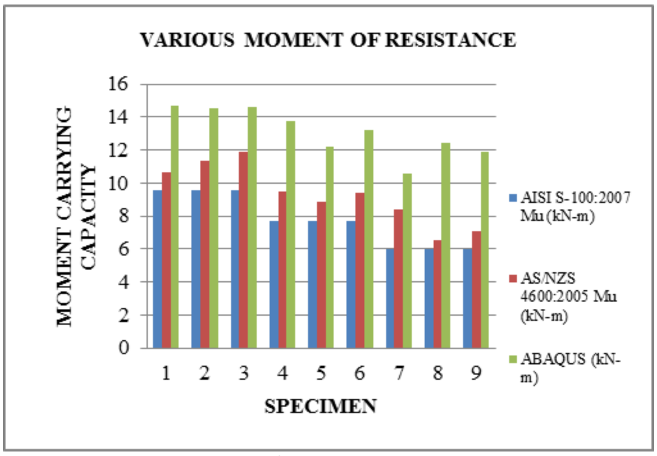

Chart. 1. Various moment of resistance obtained by the numerical and theoretical analysis.

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VII. CONCLUSION

- The ratio of strength predicted using Numerical to Theoretical AISI-S100:2007 for all beams put together was found to have mean 1.701.
- The ratio of strength predicted using Numerical to Theoretical AS/NZS 4600:2005 for all beams put together was found to have mean 1.442.
- It also shows that the standard deviation which was obtained holds good between M_{ABAQUS}/M_{AISI} and $M_{ABAQUS}/M_{AUS/NZS}$ ratios.
- Within the parametric study, it was observed that the theoretical investigation AISI S-100:2007 and AS/NZS 4600:2005 holds in good agreement with numerical investigation.
- Comparing the specimens with openings of 0.4 times the overall depth of the beam (i.e., Specimen number 1, 2 and 3 with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that specimen number 1 gives the result.
- Comparing the specimens with openings of 0.5 times the overall depth of the beam (i.e., Specimen number 4, 5 and 6 with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that specimen number 4 gives the result.
- Comparing the specimens with openings of 0.6 times the overall depth of the beam (i.e., Specimen number 7, 8 and 9 with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that specimen number 8 gives the result.
- In overall comparison of all the nine specimens, Specimen number 1 (i.e., openings with 0.4 times the overall depth of the beam and spacing of 0.8 times the overall depth of the beam) gives the better result.

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