Theoretical and Numerical Investigation of Cold Formed I Section Castellated Beam with Hexagonal Openings

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***Abstract* - The use of cold formed steel structures is increasing throughout the world as they are efficient in terms of stiffness and strength. The main aim of introducing the concept of cold formed thin section is to reduce the cost to strength ratio of the building components and to reduce the overall cost of construction besides making the structure light in weight. Castellated beams are becoming very popular now a day due to its advantageous structural applications. The advantage of using such beams is that it causes reduction in total weight of the structure and hence requires less quantity of steel .Investigation of cold formed I section castellated beam with hexagonal 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.**

Keywords- Castellated beam , Hexagonal Opening, I-Section , ABAQUS 6.13, AISI S 700:2007, AS/NZS 4600:2005

I. INTRODUCTION

Use of steel for structural purpose in structure is rapidly gaining interest these days. In steel structures the concept of pre-engineered building (PEB) is most popular due to its ease and simplicity in the construction. Pre-engineered buildings have very large spans but comparatively less loading. Generally, steel sections satisfy strength requirement, the difficulty is that, section have to satisfy serviceability requirement i.e. deflection criteria in safety check. This necessitates the use of beams with greater depth to satisfy this requirement. Use of castellated beams is the best solution to overcome this difficulty. The castellated or perforated web beam is the beam which has perforation or openings in its web portion. Generally, the openings are with hexagonal or square or circular in shapes.Use of castellated beam with hexagonal opening is very common in recent years because of the simplicity in its fabrication. Castellated beams are fabricated by cutting flange of a hot rolled steel I beam along its centerline and then welding the two halves so that the overall beam depth gets increased for more efficient structural performance against bending.

Hot rolled and cold formed members are the two main families of structural members in steel construction. Even though cold formed structural members are less familiar of the two, they have a growing importance relative to the traditional heavier hot-rolled structural members. Cold-formed steel members are made at room temperature using rolling or pressing thin flat steel sheet to get a desired shape that will support more load than the flat sheet itself.

II. REVIEW OF PREVIOUS STUDIES

A. Salokhe S.A and Patil P. S

It can be interpreted that the cold formed steel sections shows 17.37 % more load carrying capacity as compared to hot rolled sections and it gives 33% lesser lateral deformation as compared to hot rolled section. It also shows little variation in axial deflection of both cold formed steel section and hot rolled steel section. The stress distribution of hot rolled steel section is much uniform throughout the length, on the contrary cold formed steel section shows distinct variation in stress distribution. The finite element software ABAQUS gives results nearer to experimental results up to 0.6 % for load carrying capacity calculation. While comparing failure pattern, hot rolled steel member shows bending failure and cold formed steel shows distortional local buckling failure

B. Benediktas Dervinis and Audronis Kazimieras Kvedaras

An analysis of perforated beams with a hexagonal form of perforations by finite element method is accomplished in this paper. Some conclusions may be drawn:

1. A non-traditional method for selecting castellated beams has been proposed.
2. The proposed method may be adopted and used for design in future works.
3. The charts determining the behaviour of beam failure may be drawn.
4. Data received are only valid for beams with the condition mentioned above.
5. With some coefficients, the curves used in charts can be adopted for beams of different lengths.
6. It can be seen from calculation data that the higher the web, the more efficiently the beam material is used.

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However, the bigger the web slenderness, the more critical influence has local buckling on the beam's carrying capacity. That is why it is very important to find such dimensions of the beam which in the moment of failure would ensure maximal stresses in the section.

C. B.Anupriya and Dr. K. Jagadeesan

Studied 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).

D. Sagade A.V. and Auti V. A

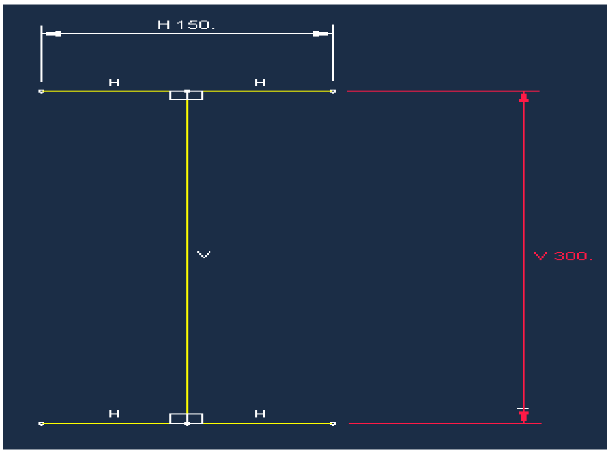
The authors have experimentally studied the behavior of simply supported castellated beams under two point loading (four point bending) by varying the depth of hexagonal openings (and hence the overall depth). Modes of failure of the castellated were examined for different depths of openings. From the 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.

III. SPECIMEN SPECIFICATIONS

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

TABLE I: Details of the Specimen

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



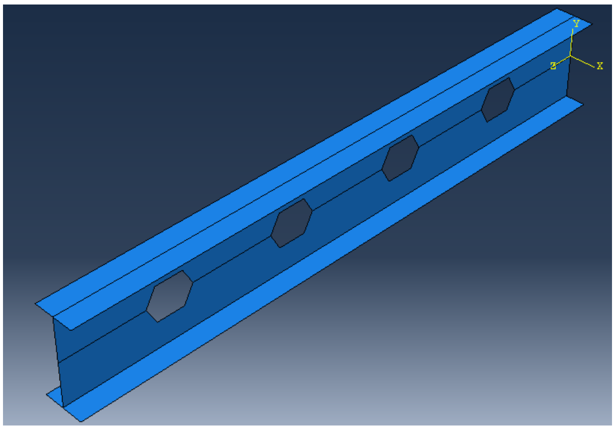


Fig -I: Details of the specimen

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 $2.78\, M_y \geq M_{cre} \geq 0.56\, M_y$

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

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

$M_{ne} = M_{cre}$

Where,

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

Where,

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

3. Local buckling strength

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

- For $\lambda_l \leq 0.776$

  $M_{nl} = M_{ne}$

- 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 6.1.2

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 6.1.3

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.

$$f_c = M_c / Z_f$$

Where,

Z_f = full unreduced section modulus for extreme compression fibre

M_c= critical moment calculated as following condition.

a) For $\lambda_b \leq 0.60$

$M_c = M_y$

b) For $0.60 < \lambda_b < 1.336$

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

c) 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.

a) For $\lambda_d \leq 0.674$

$$M_c = M_y$$

b) 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

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.

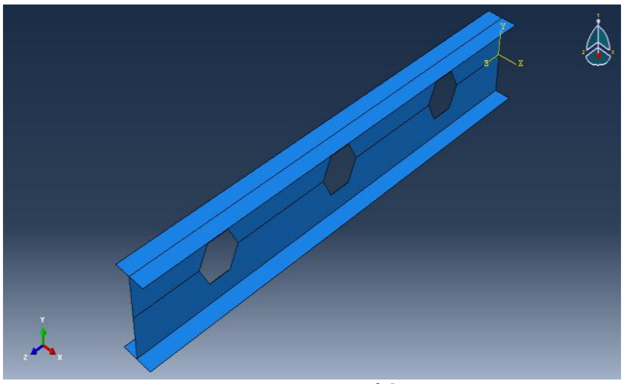

Fig -2: Perspective view of the specimen

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.

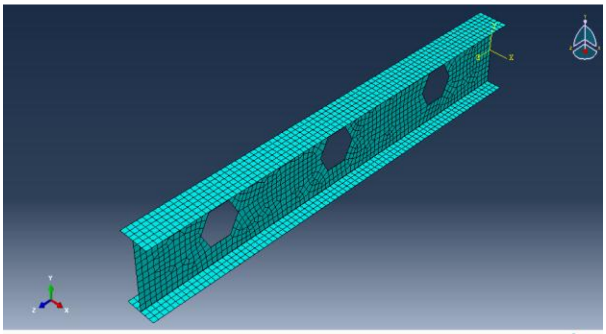

Fig -3: Perspective View of specimen with meshing

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.

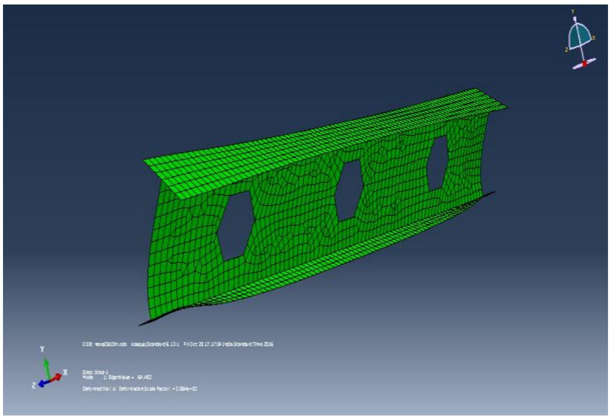

Fig -4: Distortional failure of specimen

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 processing 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 hexagonal openings in the web are estimated by theoretical investigations and numerical analyses were discussed here.

A. Comparision of Theoretical Results

The ultimate moment M_u obtained by the two code books AISI S-100:2007and AS/NZS 4600:2005 were compared in Table 2. It shows that the moment values obtained by AS/NZS 4600:2005 is higher compared to the other

TABLE II: Comparison of Theoretical Results

Specimen	AISI S-100:2007 M_u (kN-m)	AS/NZS 4600:2005 M_u (kN-m)
1	9.579	11.268
2	9.579	10.203
3	9.579	11.352
4	7.725	8.954
5	7.725	8.035
6	7.725	8.984
7	5.962	6.817
8	5.962	6.111
9	5.962	6.905

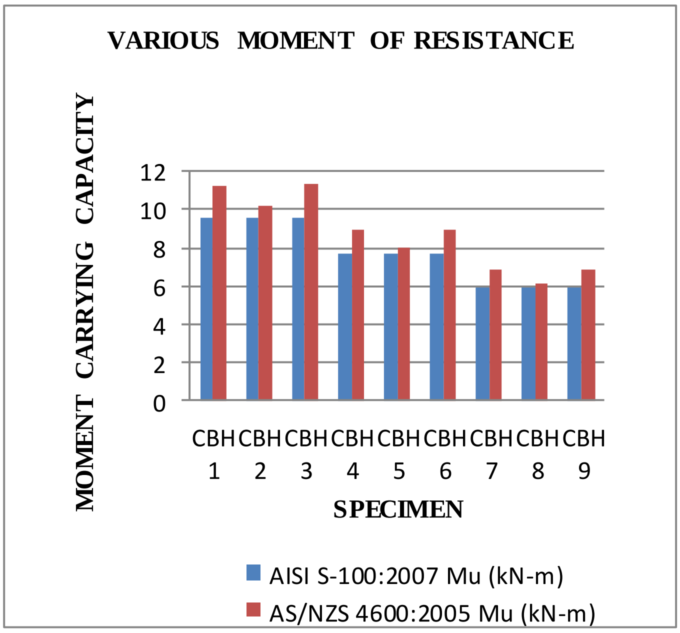

Chart 1: Various Moment of Resistance Obtained By AISI S-100:2007and AS/NZS 4600:2005

B. Comparisons of Theoretical results and Numerical results

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.

M_{AISI-} AISI S100-2007 $M_{AUS/NZS-}$ AS/NZS 4600:2005

M_{ABAQUS}- Numerical Analysis by ABAQUS 6.13

TABLE-III: Results of Theoretical and Numerical investigation

S no	M_{AISI} kNm	$M_{AUS/NZ}$ KNm	M_{ABAQUS} kNm	M_{ABAQUS}/M_{AISI}	$M_{ABAQUS}/M_{AUS/NZ}$
1	9.579	11.268	13.535	1.412	1.201
2	9.579	10.203	12.543	1.309	1.326
3	9.579	11.352	14.487	1.875	1.512
4	7.725	8.954	12.442	1.610	1.389
5	7.725	8.035	11.897	1.540	1.480
6	7.725	8.984	12.735	1.648	1.417
7	5.962	6.817	10.735	1.800	1.574
8	5.962	6.111	11.132	1.867	1.821
9	5.962	6.905	10.660	1.649	1.543
MEAN				1.701	1.473
STANDARD DEVIATION				0.472	0.351

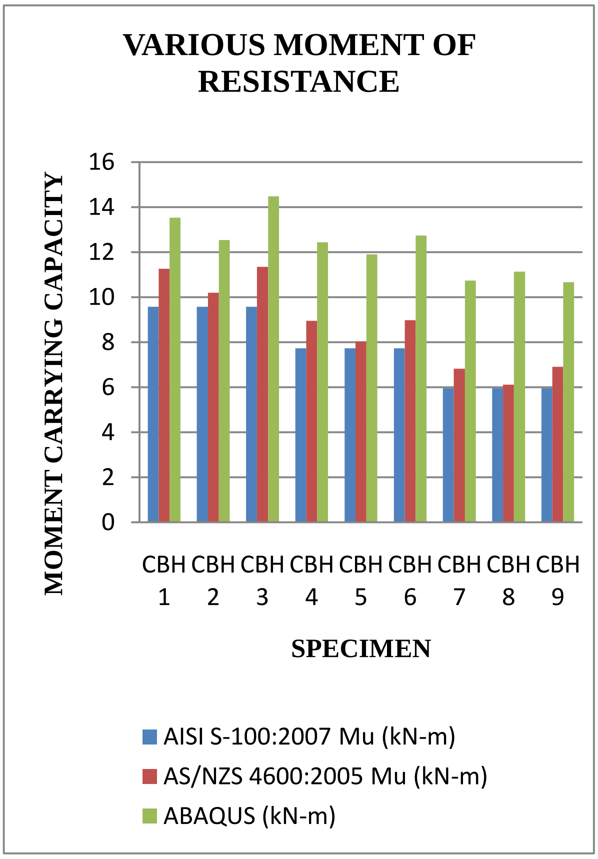

Chart 2: Various Moment of Resistance Obtained by Theoretical and Numerical investigation

VII. CONCLUSION

- The theoretical and numerical results obtained from ABAQUS 6.13 shows that the specimen with minimum opening and maximum spacing between the hexagonal openings provide better moment of resistance when compared to other specimens.
- The moment of resistance value obatained from AS/NZS 4600:2005 is higher when compared to AISI S100-2007 values due to the fact that length is not a factor for calculation as per AISI S100-2007.
- ABAQUS 6.13 provides greater moment of resistance when compared with the values from the AISI S100-2007 and AS/NZS 4600:2005 codes.
- 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 1.473.
- It also shows that the standard deviation which was obtained holds good between

MABAQUS / MAISI and
ABAQUS / MAUS/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., CBH 1, CBH 2and CBH 3 with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that CBH 3 shows the result.
- Comparing the specimens with openings of 0.5 times the overall depth of the beam (i.e., CBH 4, CBH 5 and CBH 6with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that CBH 6 shows the result.
- Comparing the specimens with openings of 0.6 times the overall depth of the beam (i.e., CBH 7, CBH 8 and CBH 9 with spacing of 0.8, 1.0 and 1.5 times the overall depth of beam respectively); it shows that CBH 8 shows the result.
- In overall comparison of all the nine specimens, CBH 3 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) shows the better result.

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