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INVESTIGATION OF FACTORS INFLUENCE ON POST WEB BUCKLING OF CASTELLATED BEAMS WITH INTERMEDIATES PLATE

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ABSTRACT

This paper focuses with the investigation of the factors affecting the post web buckling of castellated beams with intermediate plate in which the analytical results of thirty castellated beams cases were summarized for this purposes study. This research aims to find the effect of main factors that controlled the buckling behavior of castellated beams. These factors include the cutting angle (45°, 60°, 90°), the increment plate height (0,50,100,150,200) mm, the element formulation (thin and thick shell), and finally the loading conditions (shear force, bending moment). The SAP 2000 software was applied to find the buckling load factor by using four nodes shell element to represent the castellated beam sections web and flange. The obtained results showed that the thick element type formulation gave higher values by range 10% to 15% according to the cutting angle and the ultimate buckling shear and moment forces decrease with increasing the intermediate plate height by a nonlinear amount so that the cutting angle 60 ° gives the higher shear section shear capacity and values 90° gives higher section moment capacity. Also, 12 regression models had been developed between the reduction buckling section capacity and the intermediate plate height ratio. In addition, the effect of the intermediate plate height changes the mode of buckling failure from combined tee-web failure to web portion. Finally, the plate height increases causing shifting down the domain of interaction diagram of shear force and bending moment without changing its shape.

Keywords: castellated beam, litska beam, web openings, buckling load factor, intermediate plate height, interaction diagram.

INTRODUCTION

The story of castellated beams was started in U.S.A. at 1910 by the Chicago Bridge and Iron Works. Later, Great Britain used these beams in 1930. After that, on 1939, British Patent number 498281 was the first a specification related to improvements in built-up structural members which granted to Geoffrey Murray Boyd in which it was called at that time the Boyd beam. The further development application of this beam, the cellular beams had been developed in the 1990's. Today castellated beams are frequently used around world as either secondary or main units in light to medium constructions for medium to long spans and these application ranging to the multi-storey buildings, commercial and industrial buildings, warehouses, and others (Amayreh *et al*, 2005).

The basic principle of fabrication castellated beams is through process by increasing the original beam section, usually made from a standard rolled shape of wide flange section and sometimes built up from welded plates, in such a manner creates a regular pattern of holes in the web of the original beam as illustrated in Figure-1.

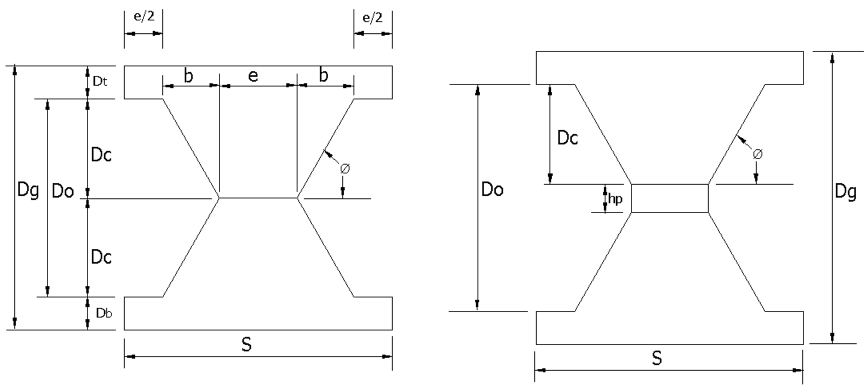

Figure-1.a Without plate **Figure-1**.b With plate

Figure-1. Castellated beam.

The production of a castellated beam involves main step of the fabrication beginning from separating a standard section into two halves by cutting the web in a regular alternating pattern followed by joined the two halves by welding after offsetting one portion so the high points of the cut beams come into contact to form a castellated beam. There are various openings shapes in these beams such as hexagonal (Honeycomb), circular (cellar), rectangle, diamond, sinusoidal. Another improvement of castellated is achieved by increased the web depth using welding rectangular plates that called the intermediate plate (sometimes referred as an increment plate) and in many references this beam is called Litska (Tsavdaridis *et al,* 2015).

The basic benefits of these structural members are to improve the flexural strength of the original section to save the cost of the structure in addition to reduce the weight of used material than the corresponding webbed sections. Also, castellated beams have been proved to be an efficient for reasonably loaded longer spans where the design is controlled by the moment capacity or the deflection because of their high strength-to-weight ratios and their lower maintenance and painting costs. In addition, the possible utilization of the web openings as passages for many services so that it can in some situations used instead built-up girders. Numerous examples had been shown the advantageous of these beams as given by (Tsavdaridis *et al,* 2015).

The disadvantages of castellated beams which results from the subsistence of the holes in the web in addition to the new web slenderness effect in which these two main effects cause increasing the stress at vicinity of voids and reducing the buckling resistance of the new section. Therefore, the structural behavior of castellated beams will be largely different from that of the plain webbed beams. These beams are represented externally as indeterminate structures which required a more complicated analysis methods and design methods so the simple analysis methods are not applicable here. In addition, the web post is a critical factor of these beams then consequently the castellated beams are not suitable for short heavily loaded spans or dynamic load effects are sever case (Altifilish *et al,* 1957).

Previous works

Various researches and studies had been carried out for analysis and design of castellated beams so in the following paragraphs some samples of these studies:

(Boyer, 1964) reviewed the Litzka process in Germany that developed this fabrication method to help making castellated beams more economical and increase their use. Boyer also selected several sections and compared them with their hot-rolled equivalent and found that the economic savings ranged from 11% to 22%.

(Hosain *et al*, 1973) analyzed castellated beams using plastic methods and found that this produced more realistic factors of safety than elastic methods. They found that the plastic method better utilized the reserve strength in the beam.

(Kerdal *et al*, 1984) conducted a series of ultimate strength tests and found that lateral buckling occurred in all the tests. The characterization of failure in the test specimens usually involved a combination of any of the six failure modes. The authors also found that the lateral bracing was required for the beams to reach their full strength.

(Knowles, 1991) presented a description steps leading up to the invention and the early attempts to devise methods of calculating the load carrying capacity and deflexion of castellated beams. Both elastic and plastic methods of analysis are examined, the basis and use of interactive design charts is explained, and the requirements of BS 5950 are outlined.

(Walid, 1995) conducted an experimental test program which incorporated to 14 castellated steel beams of thin webs in which the modes of failure and their corresponding loads were predicted based on a number of previous studies using a nonlinear finite element analysis of NASTRAN program. The results showed a good correspondence with the experimental buckling load and may be a good to conduct a more complete parametric study.

(Sevak, 1999) work an experimental study on the web-bucking behavior for 60 castellated beams in which both elastic and plastic methods of analysis were utilized to predict the failure modes of these beams. Interaction diagrams predicting formation of plastic mechanisms yielding of the horizontal weld length and elastic buckling analysis using the finite element method were correlated with a number of experimental test results from previous studies of the subjects.

(Mohebkhah *et al,* 2005) investigated the effect of elastic lateral bracing stiffness on the inelastic flexural-torsional buckling of simply supported castellated. It was found that for inelastic castellated beams, the effect of bracing initially is increased to some extent as the lateral unbraced length increases and then decreased until the beam behaves as an elastic beam meaning that the effect of bracing depends not only on the stiffness of the restraint but also on the modified slenderness of the beam bracing requirements for inelastic castellated beams.

(Wakchaure *et al,* 2012) used finite element software package ANSYS14 to analyze a castellated beam which fabricated from wide I beams section. The beams with increase in depth were compared with each other and with parent section for various parameters and for serviceability criteria. The results showed that, the castellated steel beam behaved satisfactorily with regards to serviceability requirements up to a maximum web opening depth of the 0.6 beam height.

(Ahmed, 2013) presented Element Free Galerkin (EFG) in combined with EFG/RSA method the Rotational Spring Analogy (RSA) to separating the planar and out-of-plane responses. Several illustrative examples are provided which highlight the efficiency and accuracy of the developed approach in comparison with detailed nonlinear finite element analysis performed using ADAPTIC, and which demonstrate general applicability to local buckling analysis of steel beams with web openings of various shapes and sizes.

(Morkhade *et al,* 2015) performed an experimental study on a prototype model of steel beams with web openings with different spacing to diameter ratio to investigate the failure modes and ultimate strength of these steel beams. Also, the beams were analyzed by the elasto-plastic finite element method by using general finite element analysis software ANSYS and the results were compared with those obtained experimentally. The test results indicate that the load-carrying capacity decreases with the increase in the openings area as well as position of the openings. The result of parametric study shows that, web openings reducing the ductility of this beam till 68%.

COMPONENTS OF CASTELLATED BEAMS

The castellated beams can be made of different web opening shapes and sizes, but all these shapes do not alter the main terminology which used for their definition as shown in Figure-1 and below give the description of the main terms of them as follows (Kerdal *et al*, 1984):

- Web Post Section A_w= Cross-section of the castellated beam where the section is assumed to be a solid cross-section and equal to= $t_w \times D_s$
- Weld Length e= Length of the horizontal cut on the root beam and in many references is termed as the throat width, i.e. the length of the portion of the web that is included with the flanges.
- Throat Depth D_t= Height of the portion of the web that connects to the flanges to form the tee section.
- Hole Height D_o= Height of the portion of the web at the center of hole end equals to twice the cutting depth.
- Expansion Percentage λ= Percentage change in depth of the section from the root (original) beam to the fabricated section and equal to=D_g/D_N

Modes of failure

In recent times, a lot of research work has been carried out for analysis and design of castellated. Although, there is no in general accepted design method for al castellated beam types due to the complexity in geometry which causes complex modes of failure. Basically, the modes of failure are classified into two types which are the overall member mode failure and the local member mode failures so that the total numbers of two modes types are different from reference to another. As an example, (Wakchaure *et al,* 2012) work involved five and six modes of failure whereas (Amayreh *et al,* 2005) indicate to seven modes while (Krzysztof *et al,* 2015) and (Ajim *et al*, 2016) includes eight modes. These modes of failure of castellated beam can be classified into two main categories which given as follows:

A. Global section failure

The following four modes are associated to the entire beam sections failure, these are:

a - Formation of Flexure Mechanism,
b - Formation of Vierendeel mechanism,
c - Formation of Lateral-Torsional Buckling,
d - Formation of Distortional Buckling

B. Local Section Failure:

The next four modes are related to partial section failure which namely known as web portion failures which are:

a - Shear Bucking of a Web Post,
b -Compression buckling of web post,
c -Lateral Torsional buckling of the web post
d -Rupture of the Welded Joint in a Web Post.

The reason is related to the section modes failure are reduced in composite structures and even in the industrial structures because the bracing is usually provided at intermediate points along the length of castellated beams which results reducing the non braced length of free flange and lead to decreasing the risk possibility of flange buckling that represent the major and sensible stage of entire section buckling. Since this paper will focuses on the web buckling modes only which present the most critical issue in castellated beams situations. Since, the illustration of first type of above modes is beyond scope of the present study which will focuses on the web post modes failure related to the buckling modes of web post failure. The existing design models for beams with web openings are based on two patterns known the first as a simplified analytical modeling and the second as a detailed numerical modeling (Ahmed , 2013).

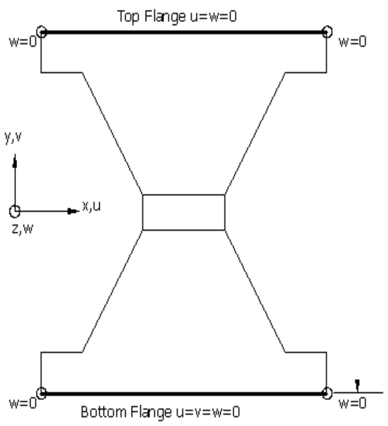

Figure-2.a One cell geometry

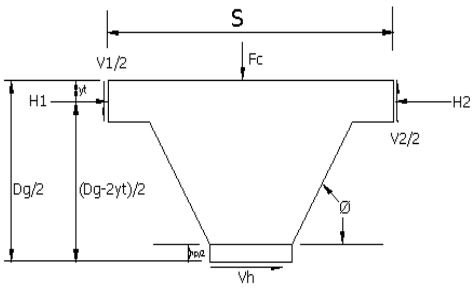

Figure-2.b Free body diagram

SCOPE OF RESEARCH

In spite of the progressive development of castellated beams analysis, design, and fabrication methods and techniques but the existing methods of valuation are still restricted to simplified models which changeable to numerical results from finite element analysis which lead to found for extra reliable analysis tools in determining the structural response of these beams. Since the web post buckling of castellated beam can be result from the effect of two different stresses on both edges of opening which are one edge in tension stress and other in compression and this buckling led to twist the web post along web height. So that the present study focuses in the investigation of the main factors that control the buckling behavior of web post castellated beams with intermediate plate which applicable to medium and long spans by considering the buckling failure model based on elastic local buckling effects. For this propose, a parametric study has been made to investigate the most parameters which effecting the web posts buckling failure modes of castellated beams with an intermediate plate. A finite element model of this beam had been created and an extensive parametric study was employed to achieve this goal. Also, a attention will give to the reduction of buckling loads by comparing with the associated beams without presence an intermediate plate. These parameters include the load pattern type, the web opening shape and geometry, load type, and the intermediate plate height.

FINITE ELEMENT MODELING

According to the many studies and researches, there are two basic models in the formulation of castellated beams. In first one (Kaveh, 2014), the overall beam length is considered including the external loads, stiffeners, and boundary conditions and this model is usually used for overall section failure while in the second model only a part of beam length is considered that involve one o more cell (each cell has one hole) and this model is applicable to the web portion failure types (Ahmed, 2013) in which the required boundary conditions are satisfied. Here, the present study interest's bout the buckling behavior of web post, therefore the second model is modified to include the intermediate plate effect. Figure-2 shows the formulation of one cell in addition its depicts the body diagram of upper cell. From Figure-2, the equilibrium of the horizontal, vertical, and moment forces yields the following equations:

$H_1 = M_1/(Dg-2\times y_t)$, $H_2=M_2/(Dg-2\times y_t)$, $V_h=H_2-H1$, $V_2=2F_c+V_1$, $V_h=(V1+v2)\times S/(Dg-2\times y_t)$ where:
H_1=Axial force on first cell face due to mending moment on the face (M_1), H_2= Axial force on second cell face due to mending moment on the face (M_2), y_t=Distance from flange top to the center of the axial force H_1 and H_2, V_h=Horizontal web shear force acting at the mid height of the intermediate plate, V_1= Axial force on first cell face, V_2=Axial force on second cell face, and F_c= External vertical loads if present.

RESULTS AND DISCUSSIONS

To satisfy the target study, a constant model length (S) is used and four parameters that mainly controlled the behavior of these beams were investigated here. The geometry of parapet beam satisfies the plastic section according BS 5950 (Knowles, 1985) which given as follows:

Nominal Depth 600 mm, cutting depth 175 mm, model length 950mm, expansion ratio 1.58, flange width 250 mm, flange thickness 18 mm, web thickness 16 mm.

The angle of cutting are (ϕ) (45°,60°,90°), the intermediate plate height (0,50,100,150,200) mm, and load type (shear, moment), and the element type (thin, thick).

For FEM modeling the finite element software SAP2000 Advanced Ver. 14.00 was selected using thin and thick four node shell element that has the membrane and the bending capabilities for nonlinear analysis of 3D structural model. A bi-linear stress-strain curve is adapted in the material modeling of steel grade A992F50 of yield stress and the modulus of elasticity 345 MPa, 200000 MPa respectively, also the density of the finite element mesh was chosen about 3 to 6 cm. The obtained results were summarized in Table 1 to 2 and the results of Table-1 are drawing in Figures 3.a and 3.b.

Table-1. The critical shear forces and critical bending moments of cutting angles 45°, 60°, 90° and different intermediate heights for thin shell element formulation.

Intermediate plate height (mm)	**Cutting angles**					
	45°		**60°**		**90°**	
	Vocr (kN)	**Mocr** (kNm)	**Vocr** (kN)	**Mocr** (kNm)	**Vocr** (kN)	**Mocr** (kNm)
0	994.4	1063.2	983.7	1265.3	833.6	1214.1
50	810.3	843.1	954.1	1080.5	813.3	1114.2
100	772.8	805.8	854.0	1041.2	780.0	1098.0
150	674.7	697.7	778.0	920.3	730.0	986.1
200	535.5	550.9	690.0	739.1	703.0	879.7

ARPN Journal of Engineering and Applied Sciences

Table-2. The critical shear forces and critical bending moments of cutting angles 45°, 60°, 90° and different intermediate heights for thick shell element formulation.

Intermediate plate height (mm)	**Cutting angles**					
	45°		**60°**		**90°**	
	Vocr (kN)	**Mocr** (kNm)	**Vocr** (kN)	**Mocr** (kNm)	**Vocr** (kN)	**Mocr** (kNm)
0	1149.4	1238.7	1114.0	1376.4	938.5	1456.2
50	983.8	1035.7	1088.0	1302.0	922.0	1387.7
100	881.1	923.5	1008.2	1203.5	877.0	1333.0
150	761.5	797.1	912.0	1059.0	803.6	1222.3
200	615.3	634.7	808.0	870.2	760.0	1122.0

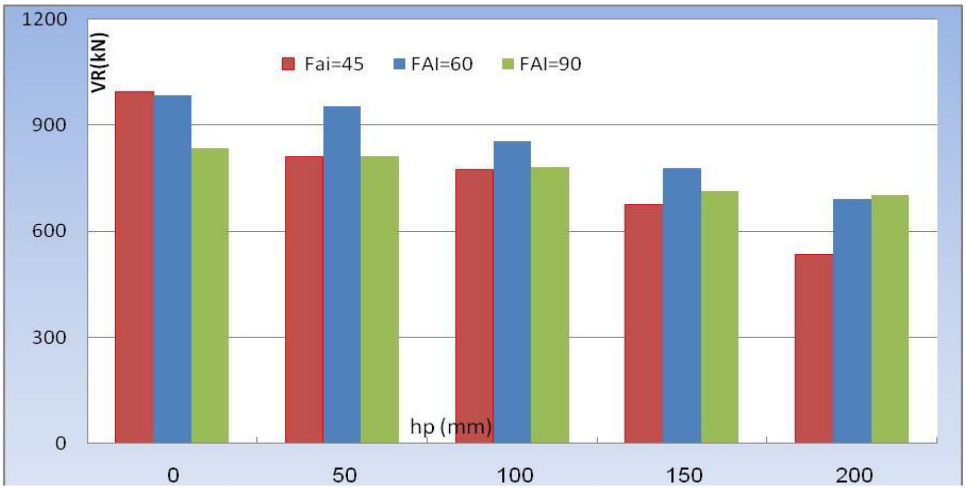

Figure-3.a. Variation of buckling shear with three cutting angles for increasing the intermediate plate height.

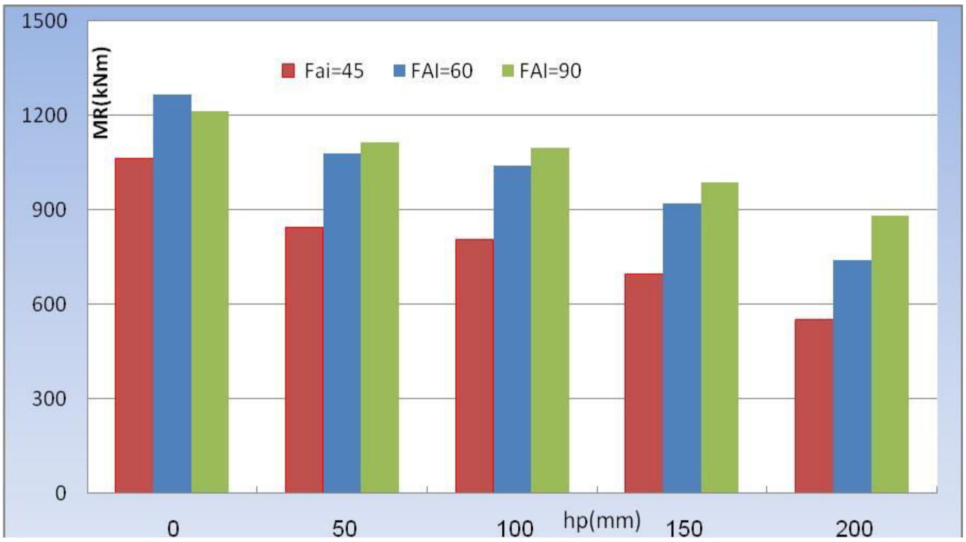

Figure-3.b. Variation of buckling moment with three cutting angles for increasing the intermediate plate height

From Figure-3a, it is found that the cutting angle value 60° gives the higher buckling shear force by an average values than sections of cutting angles 45° and 90° by about (12%, 11%) for thin shell and (12 %, 15%) for thick shell respectively leading to the overall average is 12% and 13% for 45° and 90°. In other side, Figure-3.b shows that the buckling bending moment, the cuing angle 90° gives the higher values than values 45° and 60° by

VOL. 13, NO. 24, DECEMBER 2018 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

(34%, 5%) for thin shell and (41 %, 12%) for thick shell respectively leading to the overall average is (37% and 8%) for 45° and 60°.

Unfortunately, no analytical method is provide to compared the obtained results with them, also other methods such as (Biodgett, 1967) method which based on Olander's Wedge method is not corrected for comparing because Biodgett method is applicable to the beams without intermediate plate in addition is not valid for high cutting angles reaching 90°. Also, the design aids were limited to small castellated depths as the work of Zaarour experiments which sensibly involved a small beam depth, 350 to 400 mm, with the intermediate plate height equals to 50 mm (Walid, 1999).

The second point discussed here is the effect of formulation element type. From Table 1and 2, it is so clear that the thick element type formulation gives higher values of the shear and moment forces than the thin formulation regardless of the other factors values, this effect can be attributed to the present of shear strain in thick shell formulation which causes an increase in the overall element stiffness that leading to increase the buckling resistance.

It is so obviously that the providing of the intermediate plate causing a reducing of the critical buckling shear and moment as seen in Tables 1 and 2, the main reason is associated to the increasing of the unsupported web post height which has an equivalent nature to the unsupported column length in the column stability theory.

But the important matter is how this force reduction can take form as the increasing the plate height. (Sevak, 1995) gave a relationship for which the reduction buckling shear force is decreasing with increasing unsupported web post height in similar manner as the column buckling stability equation. For further and deep discussion, three terms are introduced herein that defined as follows:

VR= Ratio between the buckling shear force in present the intermediate plate over the buckling shear force without present the intermediate plate, MR= Ratio between the buckling moment in present intermediate plate over the buckling moment without present the intermediate plate, Rhp= Ratio of the intermediate plate height over the gross height of castellated beam.

The effect of defined terms are depicted in Table-3 which displays the results comparison of web buckling load between two element formulation types and it can seen from this table that the cutting angle has less considerable effect for its values (45°,60°). In addition, Table-4 displays the reduction in buckling shear and moment terms as the intermediate plate height ratio increased using the thin and thick shell formulation so this effect can be seen in Figure-4 which exhibited different relationships between critical shear and moment ratio with the plate height ratio using thin formulation.

Table-3. The ratio of web buckling loads between thin and thick shell formulation.

Intermediate plate height (mm)	**Cutting angles**					
	45°		**60°**		**90°**	
	RV	**RM**	**RV**	**RM**	**RV**	**RM**
0	0.865	0.858	0.883	0.919	0.888	0.834
50	0.824	0.814	0.877	0.830	0.882	0.803
100	0.877	0.873	0.847	0.865	0.889	0.824
150	0.886	0.875	0.853	0.869	0.890	0.807
200	0.870	0.868	0.854	0.849	0.925	0.784
Average	0.864	0.858	0.863	0.867	0.895	0.810

Table-4. The ratio of web buckling forces ratio with the plate height ratio using thin and thick formulation.

Element type	**Intermediate plate ratio** (Rhp)	**Cutting angles**					
		45°		**60°**		**90°**	
		VR	**MR**	**VR**	**MR**	**VR**	**MR**
Thin Shell	0.000	1	1	1	1	1	1
	0.050	0.815	0.793	0.970	0.854	0.976	0.918
	0.095	0.777	0.758	0.868	0.823	0.936	0.904
	0.136	0.679	0.656	0.791	0.727	0.858	0.812
	0.174	0.539	0.518	0.701	0.584	0.843	0.725
Thick Shell	0.000	1	1	1	1	1	1

ARPN Journal of Engineering and Applied Sciences

	0.050	0.856	0.836	0.977	0.946	0.982	0.953
	0.095	0.767	0.746	0.905	0.874	0.934	0.915
	0.136	0.662	0.644	0.819	0.769	0.856	0.839
	0.174	0.535	0.512	0.725	0.632	0.810	0.771

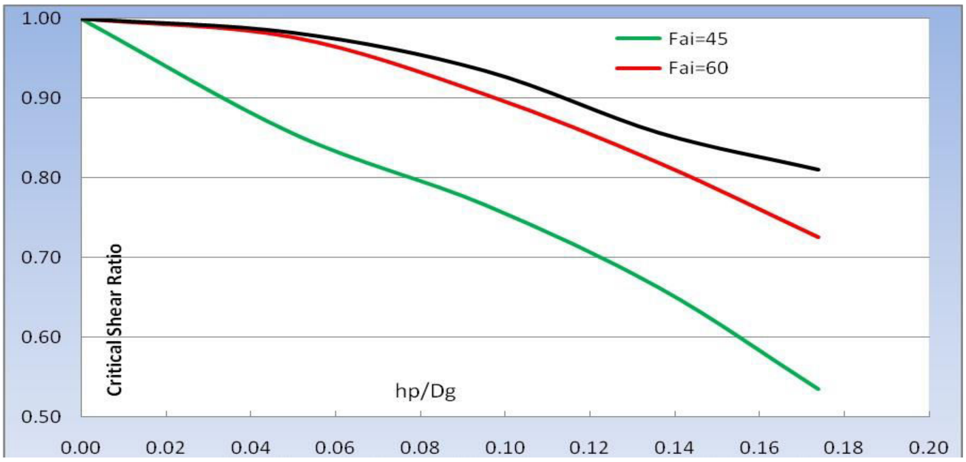

Figure-4.a. Relation of critical shear ratio with the plate height ratio using thin formulation.

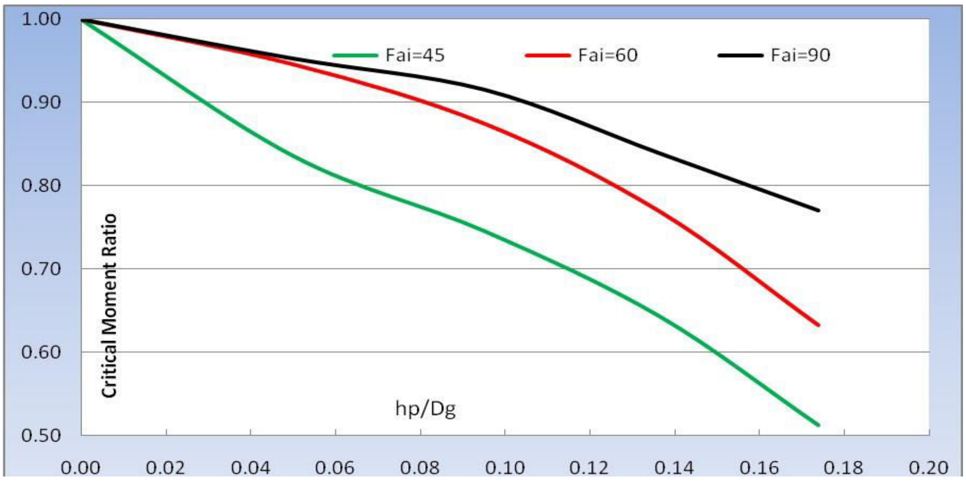

Figure-4.b. Relation of critical moment ratio with the plate height ratio using thin formulation.

These relationships are given in Table-5 which depicts an important result that the cutting angle has a same quality of relationship for each value but with a different parameters quantity. Using two tangential lines the following important results are found: For cutting angles 60° and 90°, the linear proportionality limits between buckling force re-education value (shear and moment) and the plate height ratio equals to 0.08 for thin and 0.10 for thick shell formulation respectively and beyond this value a nonlinear appearance in the specific curve is increased.

As mentioned in the web post stress distribution in (Walid, 1999), the web post section of castellated beam is subjected to two different stress types that the first one is an axi- symmetric compression stress, the path of the compression stress depends on the geometry of beam hole, while the second is an axi-symmetric tensile stress and the action of these combined stresses will cause a crippling in web post so the applicable of stability column theory is questionable for these beams and leading to the predication with considering the intermediate plate height increasing the complex of the analyses of such beams.

Final remarks for the increasing the intermediate plate height is the mode of failures is changed from combined tee-web failure to buckling web failure as shown in Figure-5 and this results can be attributed to the increasing web post unsupported height.

The final point discussed here is the interaction between the sheer force and bending moment. The pure shear force is found into two cases in which the first case is the simply supported region while the second case at the pin connection joint. In other side, the pure moment is found at any zone of zero shear force (sometimes arising from any suddenly applying moments such as beam-girder connection).

VOL. 13, NO. 24, DECEMBER 2018 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

Table-5. The summary details of gotten relationships.

Description of relationship	Item	Cutting angle	Relation type	Regression model	R^2
Relationship between critical shear ratio (VR) with intermediate plate height ratio (hp/Dg) using thin shell formulation	1	45	Linear	VR=-2.4364x(hp/Dg)+0.9838	0.9630
	2	60	Polynomial second order	VR=-5.7657x(hp/Dg)²-0.7707 x(hp/Dg)+1.0060	0.9920
	3	90	Polynomial second order	VR=-2.0650x(hp/Dg)²-0.6322 x(hp/Dg)+1.0048	0.9563
Relationship between critical moment ratio (MR) with intermediate plate height ratio (hp/Dg) for cutting angle 45° using thin shell formulation	4	45	Polynomial second order	VR=-17.713x(hp/Dg)²+1.7363 x(hp/Dg)+0.7511	0.9998
	5	60	Polynomial second order	VR=-18.946x(hp/Dg)²+2.0648 x(hp/Dg)+0.7981	1.0000
	6	90	Polynomial second order	VR=-13.047x(hp/Dg)²+1.3044 x(hp/Dg)+0.8882	0.9893
Relationship between critical shear ratio (VR) with intermediate plate height ratio (hp/Dg) using thick shell formulation	7	45	Linear	VR=-2.5803x(hp/Dg)+0.9992	0.9934
	8	60	Polynomial second order	VR=-7.775x(hp/Dg)²-0.2579 x(hp/Dg)+1.0026	0.9985
	9	90	Polynomial second order	VR=-4.8434x(hp/Dg)²-0.3128 x(hp/Dg)+1.0036	0.9872
Relationship between critical moment ratio (MR) with intermediate plate height ratio (hp/Dg) for cutting angle 45° using thick shell formulation	10	45	Linear	MR=-2.6874x(hp/Dg)+0.9923	0.9921
	11	60	Polynomial second order	MR=-9.7355x(hp/Dg)²-0.3830 x(hp/Dg)+0.9969	0.9986
	12	90	Polynomial second order	MR=-4.5110x(hp/Dg)²-0.5528 x(hp/Dg)+0.9978	0.9958

In most structural application the shear force and bending moment are applied together with different magnitude. Therefore for two threes force type actions (shear and moment) some models were developed and here the model suggests by is used which had given as following:

$$\left(\frac{M}{M_{cr}}\right)^n + \left(\frac{V}{V_{cr}}\right)^n = 1$$

Where M_{cr} and V_{cr} corresponding to pure bending moment and shear conditions respectively. The parameter n represent a constant in which many researches had been found a value equals to 2 to be a very suitable to the FEM results. Figure-6 shows the interaction curve for cutting angle 45° in which the other results have the same pattern so that they are not showing here the abbreviated summary and not to repeated the same meaning idea.

From Figure-6 it is so clear that the including the intermediate plate height reducing (shifting down) the domain of interaction diagram without changing its shape and this result can be attributed to the reason mentioned above in the discussion.

CONCLUSIONS

a) The thick element type formulation gives higher values of the shear and moment section capacity forces than the thin formulation by amount ranging from10% to 15% according to the cutting angle.

b) The ultimate shear and moment forces of castellated beam decrease with increasing the intermediate plate height by a nonlinear amount in which the angle of cutting equals to 60 ° gives the higher section shear capacity than values 45° and 90° by an overall average value equals to 12% and 13% respectively while for bending moment the cutting angle 90° gives the higher section capacity than values 45° and 60° by an overall average value equals to 37% and 8% respectively.

c) 12 Regression Models have been developed in this study between reduction buckling section capacity and the intermediate plate height ratio with very good correlation factor.

d) The increasing the intermediate plate height changes the mode of buckling failure from combined tee-web failure to web portion.

e) As increasing plate height causes shifting down the domain of interaction diagram without changing its shape

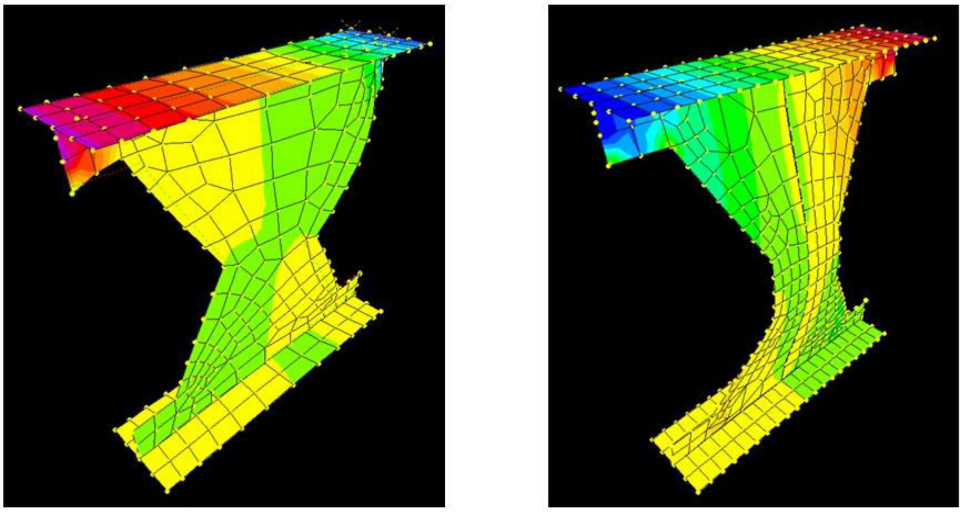

Figure-5. Lateral deflection with and without intermediate plate.

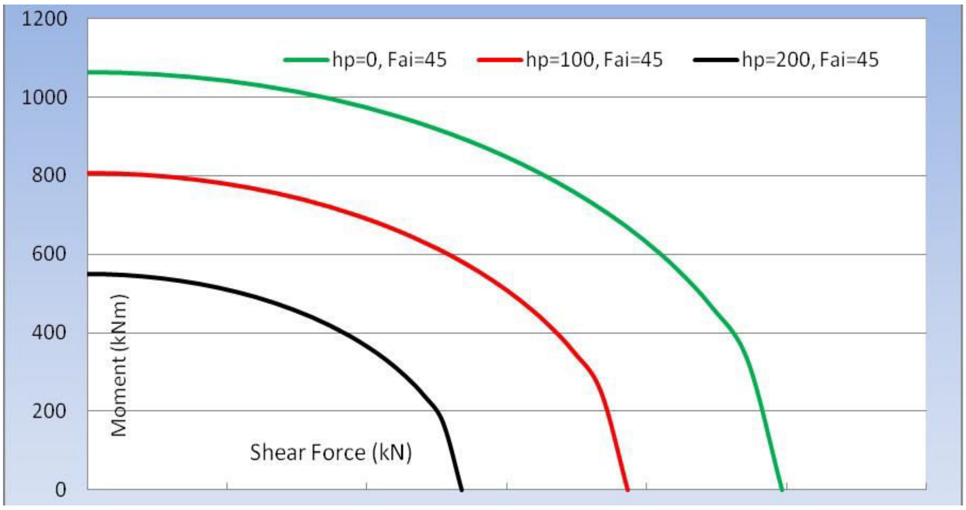

Figure-6. Interaction diagram of order 2 between shear force and bending moment.

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