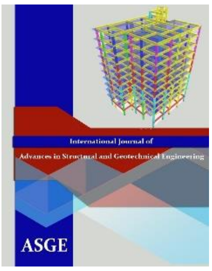

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REVIEW OF CASTELLATED BEAMS' ERECTION STABILITY CONSIDERATIONS AS PER AISC DESIGN GUIDE 31

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ABSTRACT

AISC Design Guide 31 proposes the same procedure used for calculating the lateral torsional buckling strength of plain webbed beams be used for castellated beams during both construction and utilization stages. The basis for these provisions is an experimental program which tested 24 samples of two different castellated beam sections with the aim of determining the critical unbraced length of these beams during construction . This paper examines the validity of the Design Guide's provisions by comparing it against the experimental results. It is found that the Design Guide's provisions and the experimental results conflict, with the Design Guide's provisions yielding highly unconservative results . This discrepancy in results arose from an arithmetic blunder in the calculations of the experimental program. An alternate procedure is presented to rectify the Design Guide's provisions.

Keywords: AISC Design Guide 31, Castellated Beam, Erection Loading, Elastic Lateral Torsional Stability

INTRODUCTION

Erection stability of a steel structure refers to its overall stability under loading conditions during erection (i.e. loads from wind and loads resulting from erection operations). Temporary supports may then be needed to preserve the site safety and prevent hinderance to the erection of the steel structure . The need and installation of such supports is put forward in several standards and is a task left primarily to erectors [1-2]. These temporary supports are required when any element in the structure has not reached a stable condition or acquired enough strength to support its self-weight and superimposed loads [3]. Guidelines for the design of such supports can also be found [3] .

An example of a structural element that may experience instability due to the absence of temporary supports during erection is a steel beam. During erection, the absence of floor decking or bracing corresponds to lengthier spans without a lateral support, increasing the beam's susceptibility to lateral torsional buckling instability at far lighter loads than design loads. Temporary bracings may then be needed to laterally support the beam in order to prevent lateral torsional buckling instability during erection. The need for these bracings can be assessed by identifying the erection loads along with the beam's span at each erection stage and then determining if these loads exceed the beam's capacity or not. If they do, an erection bracing may then be used to decrease the laterally unsupported span of the beam and increase its capacity. Equations for finding the lateral torsional buckling moments of prismatic sections such as a steel

I section can readily be found [1]. However, the problem tends to be more complex in the case of a non-prismatic section such as a castellated beam.

Castellated beams are beams with hexagonal web openings. They are very beneficial in industry due to their ability to allow the passage of service pipes, wires and ventilating ducts without sacrificing clear height or increasing floor height. The presence of web openings, however, alters the structural behavior of the beam. Atypical failure modes can be identified and the beam's resistance to the typical ones is affected. Considerable research was conducted to develop design guides that define the limit states for the design of beams with web openings in general and castellated beams in particular [4-5] .

Amongst the limit states to be investigated during the design of castellated beams is lateral torsional buckling and since the presence of web openings reduces the torsional rigidity of the beam, the beam's proneness to lateral buckling therefore increases. This is of utter importance during erection. However, a survey of literature revealed that only one study discussed the erection stability of castellated beams [6-16]. This study was carried out by T. Patrick Bradley as part of his master's thesis in Virginia Tech and was later cited in the *AISC Design Guide 31* as its premises for provisions regarding the castellated beam's erection stability [5-6].

AISC Design Guide 31 proposes the use of full gross-sectional properties in the ordinary equations of plain webbed beams when determining the erection bracing requirements of castellated beams [5]. This is somewhat counter-intuitive since majority of the literature suggested the use of tee or net section properties in the traditional equations of plain webbed beams to account for the influence, even if insignificant, of the web openings [9-16]. The reason behind this proposal is the conclusion put forward in Bradley's thesis that using the ordinary plain webbed beam equations with the gross sectional properties yields the closest results to the specimens tested in his experimental program.

Careful review of the thesis however reveals an arithmetic blunder in Bradley's calculations that led to former *AISC Design Guide 31*'s provision [5-6]. It follows that when applying the *AISC Design Guide 31*' s provisions and comparing the resulting loads to the experimental loads, the resulting loads are quite higher than the experimental ones, thus proving to be unconservative [5]. This paper presents the details of experimental program and provides an alternate solution that will amend the design guide's provisions.

DESCRIPTION OF THE EXPERIMENTAL PROGRAM

The Test Specimens' Details and Loading Cases

Two castellated steel beam sections were used in the experiment. The beams were labelled CB24X26 and CB27X40. The letters CB refers to the initials of the word Castellated Beam, the first number refers to the beam's nominal depth and the second number refers to the beam's self-weight measured in pounds per foot. These labels will be used in this study. Each beam section was tested under the influence of its self-weight and concentrated upper flange loading at mid-span to account for the weight of an erector and his tools. The concentrated loading was applied both concentrically and eccentrically. Details of the tested beams are shown in Figures (1) and (2).

CB24X26 was tested at spans of 48 feet 4 inches , 44 feet 10 inches , 41 feet 4 inches, and 37 feet 6 inches while CB 27X40 was tested at spans of 51 feet 9 inches, 47 feet 3 inches, 44 feet 6 inches , and 42 feet 6 inches . The beams were first tested without any eccentricity and then with 1.5 inches and 2 inches eccentricity respectively.

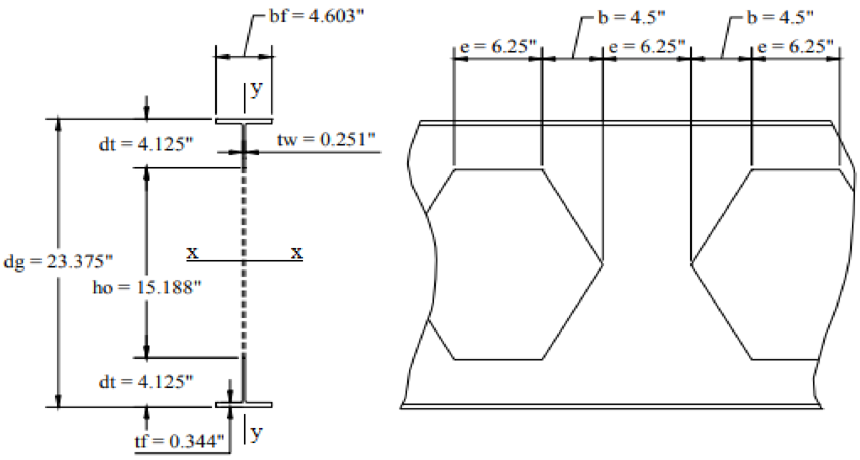

Fig. 1: Dimensions of CB24X26

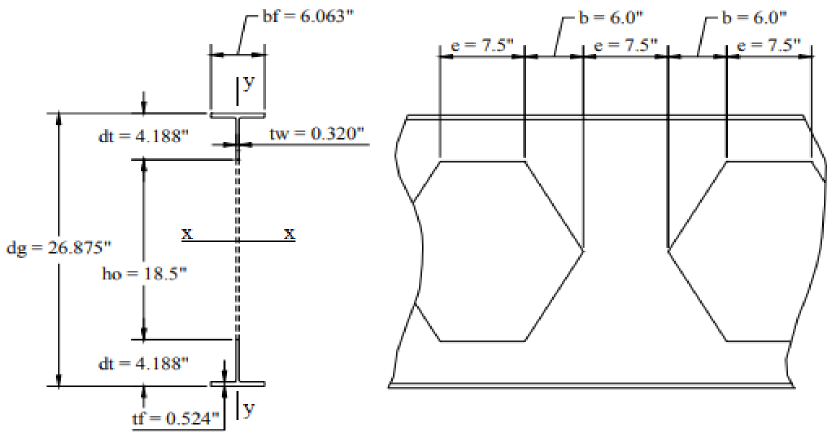

Fig. 2: Dimensions of CB27X40

Where; *bf* flange width
tf flange thickness
tw web thickness
dt depth of tee section
dg depth of section
ho *dg-2dt*
e, b dimensions defining castellation

Testing Procedure and Failure Criterion

The beams were placed into the test setup using a crane, and lab technician would then connect the beams to support columns mounted on a floor beam via double angle connections. The test setup and connection details are shown in Figures (3) and (4). Catch bracings was placed at mid-span and quarter points to prevent any safety hazards from the buckling and swaying of the beam during testing .

Each of the tested beams was considered unstable if not satisfying the following two requirements. After the lab technician connected the beam to the columns, he would then walk to the center of the beam to start its loading . The beam was loaded gradually by 10-pound plates as shown in Figure (5). The slightest deformation due to buckling of the beam would then deem the length of the beam critical. The span of the beam would then be shortened, and the process would be repeated at the new span. The testing was stopped when the length of the beam that would support the weight of the erector and his tools, a 300-pound central upper flange load without any eccentricity, was reached. Then, the beam was unloaded, and lab technician would then climb on the beam and try to walk from the support column to beam's center and back. If the beam started to sway, the lab technician would instantly stop , get off the beam and the beam's length was considered critical. The beam would further be shortened, and the process repeated until the beam could support the comfortable walk of the lab technician. This was considered an appropriate depiction of the sequence of erection encountered in the field.

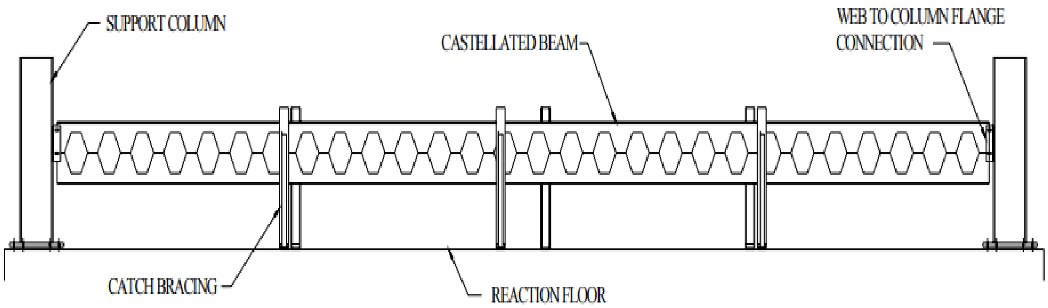

Fig. 3: Test Set-Up

Fig. 4: Connection Details

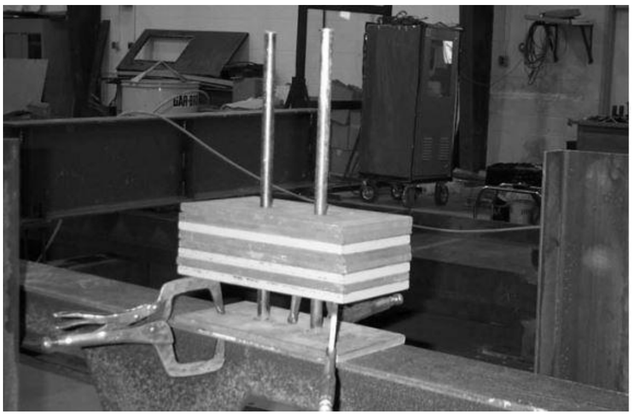

Fig. 5: Loading Plates

The buckling loads were recorded along with the beams' respective spans. However, it is mentioned that the values of the spans recorded are approximate values. The experimental results for two beams are shown in Tables (1) and (2). It follows that the critical length at which the two castellated beams should be braced in order to prevent any of the aforesaid erection stability issues are 37.5 ft for CB24X26 and 42.5 ft for CB27X40.

CALCULATION OF THEORITICAL LATERAL TORSIONAL BUCKLING LOADS ACCORDING TO AISC DESIGN GUIDE 31

To check for the limit state of global lateral torsional buckling of steel beam with compact web and compact flanges, AISC Specification Chapter F Section F2 [1] applies the following equations

$$M_n = F_{cr}\, S_x \leq M_p \tag{1}$$

Where; M_n the nominal flexural strength
S_x elastic section modulus taken about the x-axis
M_p plastic moment
F_{cr} calculated as shown in Equation (2)

$$F_{cr} = \frac{C_b\, \pi^2 E}{(\frac{L_b}{r_{ts}})^2} \sqrt{1 + 0.078 \frac{Jc}{S_x\, h_o} (\frac{L_b}{r_{ts}})} \tag{2}$$

Where; C_b lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced
E modulus of elasticity of steel
L_b length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section
r_{ts} calculated as shown in Equation (3)
J torsional constant
c for a doubly symmetric I-shape: c = 1
h_o distance between the flange centroids.

$$r_{ts^2} = \frac{\sqrt{I_Y C_W}}{S_x} \tag{3}$$

Where; I_y moment of inertia about the principal y-axis
C_w calculated as shown in Equation (4)

$$C_w = \frac{I_y {h_o}^2}{4} \tag{4}$$

The square root term in Equation (2) and the C_b were conservatively taken as 1 to account for the load location and eccentricity respectively [1] . The theoretical buckling loads are shown in Tables (1) and (2). Moreover, the deviance between the theory and experiment is also conveyed. It is clearly evident that the theoretical loads are significantly higher than the experimental ones for the majority of the tested beams. It follows that the theoretical critical unbraced lengths, i.e. (beams' spans that should be braced during erection) are 44.5 ft and 50.5 ft for CB24X26 and CB27X40 respectively which are also quite higher than the ones reported in the experiment and thus are unconservative.

CALCULATION OF THEORITICAL LATERAL TORSIONAL BUCKLING LOADS ACCORDING TO ALTERNATE SOLUTIONS

Many approximations are followed in the previous proposed solution to account for both the tipping effect due to the upper flange loading and load eccentricity. This solution is considered to be overly-conservative for the case of an ordinary plain webbed steel beam under the same loading conditions. However, this is apparently not the case for castellated steel beams. Several alternatives have been presented by Bradley in his thesis [6] (i.e. using the tee section properties and weighted average section properties in the plain webbed equations) . Due to his miscalculations however, his judgement was flawed. Both of the solutions are shown in Tables (1) and (2). It is found that the most conservative solution is the one that utilizes the tee section properties and thus, should be followed.

Table 1: Buckling loads for CB24X26 obtained from experimental program and design rules according to AISC Design Guide 31

CB 24x26	Test Length (ft)	P_{Exp} (lb.)	$P_{AISC\ Gross}$(lb.)	$P_{AISC\ Tee}$(lb.)	$P_{AISC\ W.A.}$(lb.)	$\frac{P_{Exp}}{P_{Gross}}$	$\frac{P_{Exp}}{P_{Tee}}$	$\frac{P_{Exp}}{P_{W.A.}}$
Ecc.(in) = 0	37.50	300	975.20	Self wt.	546.57	0.31	-	0.55
	41.30	260	558.06		237.20	0.47		1.10
	44.80	220	275.46		24.07	0.80		9.14
	48.30	170	56.65		Self wt.	3.00		-
Ecc.(in) = 1.5	37.50	260	Loads obtained are similar for different eccentricities.			0.27		0.48
	41.30	190				0.34		0.80
	44.80	150				0.54		6.23
	48.30	100				1.77		-
Ecc.(in) = 2	37.50	200				0.21		0.37
	41.30	150				0.27		0.63
	44.80	120				0.44		4.99
	48.30	80				1.41		-

Table 2: Buckling loads for CB27X40 obtained from experimental program and design rules according to AISC Design Guide 31

CB 27x40	Test Length (ft)	P_{Exp} (lb.)	$P_{AISC\ Gross}$(lb.)	$P_{AISC\ Tee}$(lb.)	$P_{AISC\ W.A.}$(lb.)	$\frac{P_{Exp}}{P_{Gross}}$	$\frac{P_{Exp}}{P_{Tee}}$	$\frac{P_{Exp}}{P_{W.A.}}$
Ecc.(in) = 0	42.5	300	1370.84	Self wt.	719.30	0.22	-	0.42
	44.5	270	1044.65		477.08	0.26		0.57
	47.3	120	665.02		192.38	0.18		0.62
	51.8	self wt.	190.57		169.27	-		-
Ecc.(in) = 1.5	42.5	250	Loads obtained are similar for different eccentricities.			0.18		0.35
	44.5	210				0.2		0.44
	47.3	60				0.09		0.31
	51.8	self wt.				-		-
Ecc.(in) = 2	42.5	190				0.14		0.26
	44.5	160				0.15		0.34
	47.3	40				0.06		0.21
	51.8	self wt.				-		-

It is important to note that the difference in the solution is due to the dramatic decrease in the warping constant. The warping constant is calculated for a tee stem using the following equation

$$C_{w_{tee}} = \frac{1}{36}\left(\frac{{b_f}^3{t_f}^3}{4} + h^3{t_w}^3\right) \tag{6}$$

and then multiplied by 2 to account for the warping capacity of the whole section. It is also worth noting that Bradley used far more complex equations derived by Galambos to account for the effect of load location [6] . This equation involves far more computations and gave more conservative results than the ordinary plain webbed equations using tee section properties and thus for the sake of practicality, it was not included in this study.

To further clarify the deviation between the theory and the experiment, the theoretical beam capacities , obtained using the different solutions, along with the experimental ones are plotted on each of the following curves. A negative load indicates the beam will buckle under its self-weight.

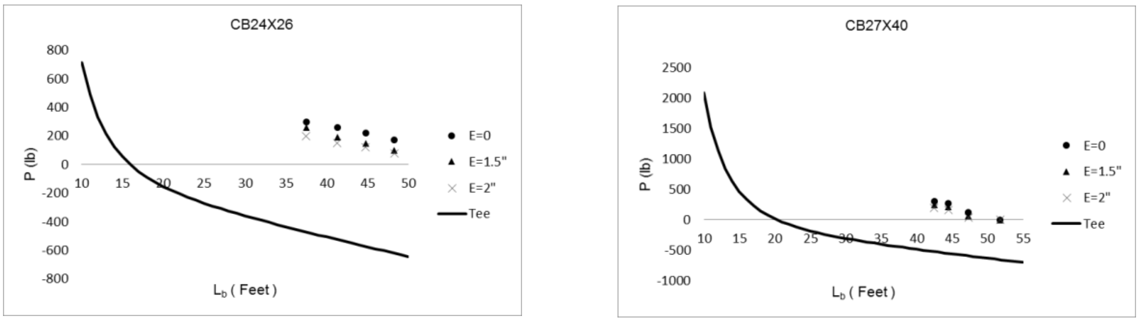

Fig. 6: $P_{AISC\ TEE}$ & P_{Exp}

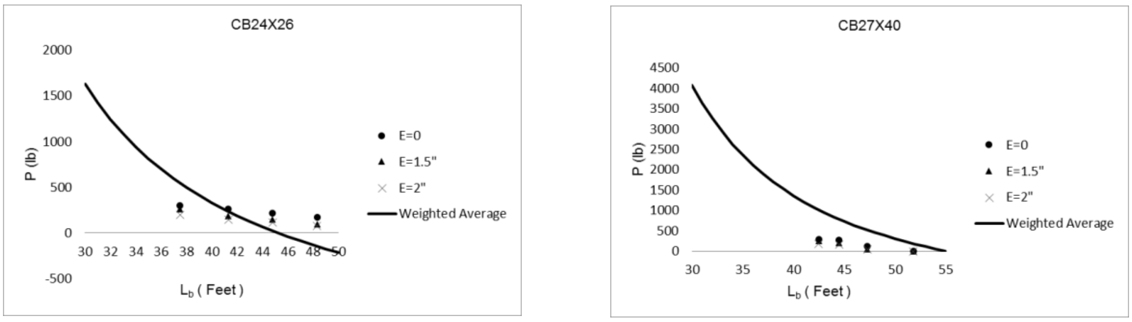

Fig. 7: $P_{AISC\ W.A.}$ & P_{Exp}

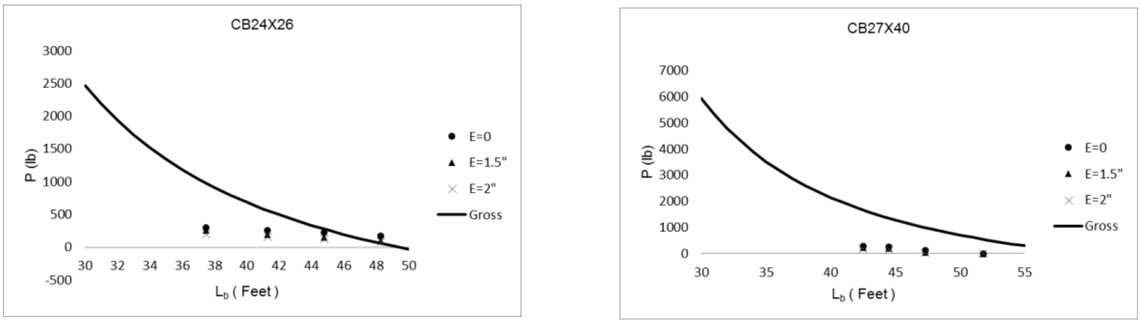

Fig. 8: $P_{AISC\ Gross}$ & P_{Exp}

CONCLUSIONS

When dealing with erection stability of castellated beams, the tipping effect due the load location (i.e. upper flange loading) which is always associated with the erection loads must be considered. In addition, the effect of the load eccentricity on the lateral torsional buckling capacity must be taken notice of since the loads during erection are almost never concentric. The swaying action due to the movement of any iron worker on the beam must also be taken into account. This is essentially not a limit of strength but rather a serviceability consideration which also must be carefully addressed when designing for the erection bracing.

The only study found in the literature, dealing with the erection stability of castellated beams, was presented. The experimental lateral torsional buckling results were compared with the theoretical results according to the recommendations found in *AISC Design Guide 31*. The experimental results were found to be way unconservative and several alternate solutions were compared. The conservative solution was found to be the one utilizing the properties of two tee stems in the ordinary plain webbed equations. While this is highly approximate, this solution is simple and practical, thus proving to be of usefulness to the site engineer.

The greatest deviation between the theory and experiment occurred when using the gross section properties and simply put, ignoring the influence of the holes and treating the beams as plain webbed ones. This implies that ,although the holes by themselves do not decrease the torsional rigidity of the beam significantly, the effect of load location and load eccentricity are far more pronounced in the case of elastic castellated beams with relatively lengthy unsupported spans. A more conclusive finite element study should be conducted to further provide an insight into the castellated beam's lateral torsional buckling behavior under the formerly presented conditions .

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