FINITE ELEMENT ANALYSIS ON LATERAL TORSIONAL BUCKLING BEHAVIOUR OF I-BEAM WITH WEB OPENING

Fatimah De'nan, Fadzli Mohamed Nazri, Nor Salwani Hashim

School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus, 14300 Nibong Tebal, Penang, Malaysia. *Corresponding author email: wani_a02@yahoo.com

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ABSTRACT

Lateral torsional buckling may occur in an unrestrained beam. A beam is considered to be unrestrained when its compression flange is free to laterally rotate and laterally displace. In this paper, the finite element analysis is used to investigate the lateral torsional buckling behavior of I-beam with and without web opening. The analysis considers a wide range of practical distances between two openings as well as, various shapes and sizes of web opening. Analysis results show that the size of web opening has slightly effect on the buckling moment resistance. Furthermore, five shapes and three sizes of opening with 1.1m section length were used to find the optimum size and shapes of opening. It was concluded that, the optimum size is 0.5D due to the high values of the buckling moment compared with 0.6D and 0.7D. Meanwhile, model 2 is classified as the optimum model because the value of buckling moment is higher than model 1. It was noted that C-hexagon has the highest buckling moment compared to other web opening shapes. Besides that, the differences in buckling moment values decrease when the opening becomes larger in size such as square opening. However, I-beam without web opening has the highest buckling moments resistance compared to C-hexagon.

1. INTRODUCTION

In recent years, a great deal of design for both steel and composite beams with web openings. Among the benefits is the behavior of steel and composite beams is quite similar at web openings. It was clearly found that the stress and the deflection values are higher when the web opening is provided near to the support. Therefore, it is preferable to provide web openings in the predominant bending region. Besides that, by strengthening the plate with 70mm offset and thickness of the strengthening plate equal to the thickness of the flange, there is a reduction in stress ratio and deflection for the I section [1]. However, the behaviour of statically indeterminate castellated composite beams is more complex than that of simply supported beams [2]. This is because instability effects of the castellated composite beam are subjected to the negative moment regions where the bottom compression flange is unrestrained. The restrained distortional buckling mode is a torsional-distortional for shorter beam spans while for longer spans the buckling mode changes towards the lateral-distortional.

In 2006, an extensive research of open-web castellated and unaltered standard wide flange beam to economical design methods is presented [3]. It was found that open web beams lead to great savings in construction costs when beam spans exceed forty feet. Furthermore, by using castellated beam for large projects requiring more than one hundred beams, some savings may be achieved. However, the provision of these web openings has a significant effect on the stress distribution and deformation characteristics [4].

Recently, there are two known types of open web beams: castellated beams with hexagonal openings, and cellular beams with circular web openings. The recent increase in usage of castellated and cellular beams highlights the need for additional research. Castellated steel beam which is fabricated from standard hot-rolled I-section has a lot of advantages such as aesthetic architectural appearance, ease of services through the web openings, optimum self-weight-depth ratio, economic construction, larger section modulus, and greater bending rigidity. However, the castellation of the beam results in distinctive failure modes depending on geometry of the beams, size of web openings, web slenderness, type of loading, quality of welding and lateral restraint condition [5]. The potential failure modes comprise shear buckling of a web post, formation of flexure mechanism [6,7], lateral torsion buckling [8-10], formation of Vierendeel mechanism [11-14], rupture of welded joints in a web post and compression buckling of a web post [5,15]. Investigation of these failure modes was previously detailed by Kerdal and Nethercot [16]. However, very few tests were found in the literature on the distortional

buckling behaviour of castellated beams. These tests were carried out by Zirakian and Showkati [17] and provided useful information in the form of failure loads, failure modes, load-lateral deflection curves and load-strain curves that could be used in developing finite element models.

Cellular members are I-section steel members with evenly spaced round web openings. The main advantage of these members compared with beams without web openings of the same weight is the bending resistance and stiffness of cellular beams is considerably higher. Besides that, the weight of the cellular member is much lower than the beams spanning the same length. Additionally, cellular members have a lighter appearance, which may also be an advantage from an aesthetic point of view. These members are principally used for applications for strong-axis bending, but also applied in cases subjected to a combination of a compressive force and a bending moment [18].

The performance and mode of failure of cellular beams varies depending on the geometric details of the beam. Various simple design methods have been presented for cellular beams to check the beam's resistance against web post buckling, Vierendeel mechanism and other failure modes. However, there is still no design method available for the beams with web openings. The only codified design guide was provided in the National Annex N of BS EN 1993-1-1 (BSI, 1998) which was superseded later on due to reliability concerns [19]. The composite use of cellular beams with a concrete slab has become increasingly popular within which the resulted section benefits from the concrete's compressive strength and steel's tensile strength. This composite action has also added to the complexities of implementing simple methods to design cellular composite beams [20]. It was observed that the long opening caused the tension (pull-out) forces in the shear connectors at the edge of the opening. This is due to the development of local composite action. It is recommended that the length:depth of unstiffened elongated openings should not exceed 2.5 [21]. Nowadays, it is common practice to provide web openings in beams for the passage of service ducts and pipings and also for inspection purposes in beams structures. The presence of web openings will affect the lateral torsional buckling behavior of the beam.

1.1 Lateral-torsional Buckling of the Beam

Non-composite castellated beams are more susceptible to lateral-torsional buckling than composite beams due to lack of lateral support to the compression flange [22]. These are prone to buckle laterally because of their relatively deeper and more slender section and due to the reduced torsional stiffness of the web. The lateral torsional buckling

behaviour of castellated beams is similar to that of plain webbed beams because the holes had a significant influence on lateral-torsional buckling behaviour [10].

Previous studies of the structural behavior of castellated beams conducted by Mohebbkhah [23] and Mohebbkhah and Showkati [25] are reviewed, and a number of different possible failure modes identified. Methods for predicting the loads at which each of these types of failure occurs are evaluated against the available experimental data and the limitations in a number of these analytical approaches is discussed. The modified slenderness of these beams appears to have a significant influence on the moment-gradient factor.

Lateral-torsional buckling can be avoided by properly spaced and designed lateral bracing. Bracing is usually assumed to be elastic and characterized by elastic stiffness. It is well known that an elastic lateral brace restricts partially the lateral buckling of slender beams and increase the elastic buckling moment. Accordingly, the effect of elastic lateral bracing stiffness on the inelastic flexural torsional buckling of simply supported castellated beams with an elastic lateral restraint under pure bending is investigated by previous researcher. The effect of bracing depends not only on the stiffness of the restraint but also on the modified slenderness of the beam [25].

Lateral buckling is the problem of stability loss and bearing capacity failure before plastification of cross-section. Radic and Markulak [8] have reported the stability of the castellated beams considering resistance to lateral buckling. The research results show that web openings of the castellated beams have little influence on determining of M_{cr} values of lateral buckling critical moments obtained on 3D FEM models, depending on lateral boundary conditions, give practically the same or little higher values (up to 10%) in relation to manual calculation method.

An existing design rule for this failure mode is investigated for purely elastic beams [17, 23, 24]. Even though this rule is shown to be quite accurate for most cases, it could sometimes give unsafe results for certain short length geometries. Besides that, the lateral torsional buckling moment of Elliptical Cellular beam using sectional properties at the web-post section as used in AISC standard gives unsafe results with approximately 5% error in elastic range and up to 20% error in inelastic and plastic range [26]. In reality, however, the steel will display elastic-plastic behaviour, causing these geometries to fail by plasticity of the cross-section rather than elastic instability. In this work [18], the design rule will be further examined by performing numerical simulations, taking into account these effects of plasticity, as well as imperfections and geometric nonlinearity. As expected, the failure of the short cellular beams is governed by plastic yielding instead of elastic buckling.

In another paper, prediction of the bending coefficient, C_b of unbraced castellated steel beam has been done. Unlike the usual cases when a uniform load is applied on the top flange of a simple castellated beam, the bending moment is less than the pure bending case, with a mean bending coefficient of 0.823 [9]. Elastic bending capacities of I-shaped and castellated steel beam under uniform loading on the top flange have a difference of between 4.9% and 8.6%, depending on the section properties. Besides that, the moment-gradient coefficient C_b is significantly influenced by the beam geometry, slenderness and web perforation configuration [25].

2. THEORETICAL STUDY

The eigenvalue buckling analysis was applied to relatively stiff structures to estimate the maximum load prior to structural instability or collapse. From three eigenvalues of each model (see Table 1), the first mode was selected. This is because, the lower value of the eigenvalue, the more probability of the mode failure to occur. The selected eigenvalues then are multiplied with applied load, P to get buckling loads, P_b . The lowest value of eigenvalue, e was selected because the probability of the failure mode to occur was highest. Therefore, for this model $e = 7.25215$. Therefore, buckling load, P_b equaled to:

$$P_b = e \times P \quad (1)$$

Where, P_b = Buckling load, e = Eigenvalue and P = Applied load
In this case, $P = 10$ kN. Therefore, $P_b = 7.25215 \times 10$ kN
= 72.5215 kN

Table 1 Eigenvalue results

Mode	Eigenvalue	Load Factor
1	7.25215	7.25215
2	22.4855	22.4855
3	42.6129	42.6129

From the buckling loads, critical buckling moments, M_{cr} for I-beam with c-hexagonal web opening with span 1.1 m were;

$$M_{cr} = (72.52 \times 1.1) / 4 \\ = 19.943 \text{ kNm} \quad (2)$$

3. FINITE ELEMENT ANALYSIS USING LUSAS SOFTWARE

Numerical study with finite element method was used to study structural behaviour of I-beam steel section (I-beam) with and without web opening. A parametrical study of structural behavior was discussed here. Finite element program, LUSAS was adopted throughout the analysis. In this paper, all models are assumed to buckle under perfect conditions, where there is no initial imperfectness and eccentric load. The buckling moments were then compared with result obtained from the manual calculation. Eigenvalue analysis of LUSAS Modeller was used to determine the buckling loads.

The models were analysed with higher-order elements (QTS8). The main objective in the eigenvalue buckling analysis is to obtain the critical buckling load. The resulted eigenvalues are the load factors was multiplied with applied load in order to obtain critical buckling load. The eigenvalue buckling analysis in LUSAS Modeller gives both local and global buckling modes.

3.1 Convergence Study

The mesh convergence was established by increased the mesh density in each part of the model as tabulated in Table 2. From Figure 1, it was observed that the increment in displacement becomes smaller from model 1 to model 8. The results clearly indicate that a convergence solution is obtained when the number of elements is 2228. Therefore, further increment of meshing density is unnecessary. Elements size 20 (Model 2) was used in subsequent analyses.

Table 2: Data of maximum nodal displacements with different mesh sizes

Model	Size of elements	Number of elements	Number of nodes	Displacement (result) mm
1	10	8230	25171	0.2958
2	20	2228	6929	0.2943
3	30	1002	3169	0.2929
4	40	450	1469	0.2918
5	50	320	1057	0.2911
6	60	249	824	0.2897
7	70	202	677	0.2895
8	80	179	598	0.2883

Figure 1: Graph of maximum nodal displacement at mid span against number of element

3.2 Process of Analysis

Two types of 200×100×8×6mm model, five shapes and three sizes of web opening was used. There are two types of model namely model 1 and model 2. For the model 1, the distance between two openings is equal to 150 mm center to center and 200 mm center to center for model 2. Meanwhile, the edge length is 50 mm for model 1 and 100 mm for model 2. Figure 2 shows the details summarize of the model.

Figure 2: Types of model

It is known that the shear and flexural failures of typical perforated sections are controlled mainly by the size (i.e. depth, do) of the web openings [13,14]. In this study, there are three sizes of opening; the first size of opening is equal to 0.5D which the diameter of the opening is 100 mm. The second size of opening is equal to 0.6D which the diameter of the opening is 120 mm. The third size of opening is equal to 0.7D which the diameter of the opening is 140 mm where D is the height of the web. The size of web opening used also comply the existing design of web openings. The basis regarding the spacing-to-diameter ratio ($S/Do = 1.08-1.5$) is taken from BS 5950 Part-1 (2000). To meet the model 1 and model 2 requirement for spacing-to-diameter ratio, therefore 1.1m length was used.

Five types of opening were adopted in this study, which are c-hexagon, octagon, hexagon, circle, square and I-beam without web opening as shown in Figure 3 (a) to (f) respectively.

Figure 3: Types of opening

4. RESULTS AND DISCUSSION

The numerical analysis consists of two main models of I-beam with different type of web opening and without web opening. In order to study the buckling behaviour, five types of web opening models were derived by varying the shape of the web opening. Figure 4 show the deformation and contours results of I-beam with c-hexagonal opening based on eigenvalue number. Eigenvalue mode 1 was selected to determine the buckling load. This eigenvalue 1 was prone to occur compared to eigenvalue 2 and 3. Buckling moments of I-beam with and without web opening under 10kN loads for 1.1m section length is shown in Table 3. Figure 5 shows the graph for buckling moments, M_{cr} versus types of opening for I-beam with different types of web opening and without web opening.

Figure 4: Deformation of I-beam with c-hexagonal opening based on eigenvalue number

Table 3: Buckling moments, M_{cr} of I-beam with different shapes of web opening and without web opening under 10 kN loads for 1.1 m section length

Shape of Web Opening	Model 1 (section with 50 mm edge and 150 mm centre to centre distance)			Model 2 (section with 100 mm edge and 200 mm centre to centre distance)		
	5D	0.6D	0.7D	5D	6D	7D
C-hexagon	19.943	18.180	15.172	21.403	20.578	19.539
Hexagon	19.118	16.998	13.676	20.996	19.976	18.741
Octagon	18.752	16.030	11.083	20.826	19.671	18.189
Circular	18.755	16.184	11.770	20.826	19.690	18.244
Square	16.701	13.032	7.598	19.811	18.164	16.189
I-beam	23.078			23.078		

Analysis results show that the size of web opening has slightly effect on the buckling moment values. Significant reduction is seen in buckling moments values for model 1 and model 2 when the opening size become bigger from 0.5D to 0.7D. The optimum size is 0.5D due to the high values of the buckling moment compared with 0.6D and 0.7D. This is due to the 0.5D has lower opening area compared to others web opening size.

Meanwhile, model 2 is classified as the optimum model because the value of buckling moment is higher than model 1. For model 1, seven openings were provided throughout the span. Meanwhile for model 2, only five openings were introduced. Similarity decreasing in buckling values was observed in both model when the opening size become bigger. About 11-21% reduction in buckling moments compared with I-beam was observed. It shows that the opening diameter is a significant factor that affecting buckling moment. However, for overall analysis, the buckling moments for I-beam was higher than I-beam with web opening. This is true according to the fact where lateral buckling generally resisted by the flanges. Therefore, any elimination of web material did not much affect to buckling behavior.

Figure 5: Buckling moments, M_{cr} versus types of opening for I-beam with different types of web opening and without web opening

In term of web opening, it was noted that C-hexagon has the highest buckling moment compared to other web opening shapes. Besides that, the differences in buckling moment values decreased when the opening larger in size such as square opening. As showed in Figure 5, the buckling moments decreased from c-hexagon to square opening. However, I-beam without web opening has the highest buckling moments compared to C-hexagon.

5. CONCLUSIONS

In the current analysis, the finite element analysis is used to investigate the lateral torsional buckling behavior of I-beam with and without web opening. Analysis results show that the size of web opening has slightly effect on the buckling moment values. Furthermore, five shapes and three sizes of opening with 1.1m section length were used to find the optimum size and shapes of opening. The optimum size in this study is 0.5D due to the high values of the buckling moment compared with 0.6D and 0.7D. Meanwhile, model 2 can be classified as the optimum model because the value of buckling moment is higher than model 1. This means when the number of opening is lesser, the buckling moments become higher. For the model 1, there are 7 opening throughout the 1.1-meter length. However, for model 2, there are 5 opening in a 1.1-meter span. This is maybe because the edge distance was higher in model 2 compared to model 1. Therefore, spacing-to-diameter ratio (S/Do) significantly affected the lateral torsional buckling of each model. It was noted that C-hexagon has the highest buckling moment compared to other web opening shapes. Besides that, the differences in buckling moment values decrease when the opening becomes larger in size. However, I-beam without web opening has the highest buckling moments compared to C-hexagon.

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