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Buckling of axially loaded castellated steel columns

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ABSTRACT

The majority of the existing literature on castellated members is focused on beams. Very little work has been done on the stability of castellated columns although they have been increasingly used in buildings in recent years. This paper presents a new analytical solution for calculating the critical buckling load of simply supported castellated columns when they buckle about the major axis. This analytical solution takes into account the influence of web shear deformations on the buckling of castellated columns and is derived using the stationary principle of potential energy. The formula derived for calculating the critical buckling load is demonstrated for a wide range of section dimensions using the data obtained from finite element analyses published by others. It was found that the influence of web shear deformations on the critical buckling loads of castellated columns increased with the cross-sectional area of a tee section and the depth of web opening, but decreased with the length and the web thickness of the column. It is shown that the inclusion of web shear deformations significantly reduces the buckling resistance of castellated columns. Neglecting the web shear deformations could overestimate the critical buckling load by up to 25%, even if a reduced second moment of area is used.

1. Introduction

Castellated beams have been used as structural members in structural steel frames [1]. An example is shown in Fig. 1. A castellated beam or column is fabricated from a standard steel I-shape by cutting the web on a half hexagonal line down the centre of the beam. The two halves are moved across by one spacing and then rejoined by welding [1]. This process increases the depth of the beam and hence the major axis bending strength and stiffness without adding additional materials. This allows castellated beams to be used in long span applications with light or moderate loading conditions in floors and roofs. The fabrication process creates openings on the web, which can be used to accommodate services. Despite the increase in the beam depth the overall building height can hence be reduced, compared with a solid web solution, where services are provided beneath the beam. This leads to savings in the cladding costs. Despite the increase in the fabrication costs caused by cutting and welding, the advantages outweigh the disadvantages.

Some design guidance on the strength and stiffness of castellated beams is given [1–3]. Due to the opening in the web, castellated beams are more susceptible to lateral-torsional buckling. Intensive research on the lateral stability [4–13] of castellated beams started in the early 1980s. Experimental investigations [4–6,9,13] were carried out and finite element methods [6–8,10–12] were used to predict the buckling behaviour of such beams and to compare the predictions with the results from the experiments [6,12]. The effects of slenderness on the moment-gradient factor [7] and of elastic lateral bracing stiffness on the lateral-torsional buckling [8,13] of simply supported castellated beams were studied using 3-D finite element models. The failure modes [4–6,9–11] and the interaction of the buckling modes [10] of castellated beams were investigated. It was found that the web opening of castellated beams had little influence on the lateral-torsional buckling [4] and the failure mode by lateral-torsional buckling of castellated beams was shown to be similar to that for solid web beams [5], while web distortional buckling was prone when an effective lateral brace was provided at the mid-span of the compression flange [9,13] and this type of failure reduced significantly the failure load [10] of slender castellated beams.

In recent years castellated members have also been widely used as columns in buildings [14]. The main benefit of using a castellated column is to increase its buckling resistance about the major axis. However, because of the openings in the web, castellated columns have complicated sectional properties, which make it extremely difficult to predict their buckling resistance analytically. Compared to a solid web column, the castellated column has weak web shear stiffness and thus the shear deformations are more pronounced when the column has a flexural buckling, which can significantly reduce the buckling capacity of the columns [14]. The effect of shear on the buckling capacity of built-up columns was reported by Gjelsvik [15], who showed that the columns exhibit reduced shear stiffness and this reduces their buckling capacity due to the increase in the lateral deflection. This indicates that the buckling theory taking into account shear deformations developed by Timoshenko and Gere [16] for solid web columns may not be suitable for castellated columns.

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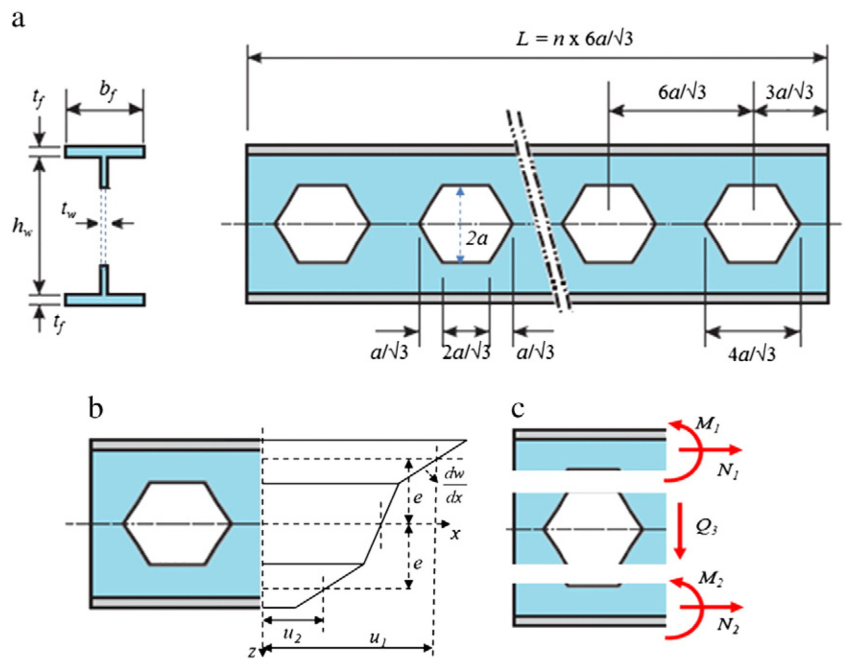

Fig. 1. Definitions of notations for (a) geometry where a is the half depth of a hexagon, (b) deformations, and (c) internal forces of a castellated member with hexagonal holes.

The above survey of the literature shows that the majority of the existing literature on castellated members dealt with the research into castellated beams using experiments and/or numerical studies. Very little work has been found on castellated columns. The stability of castellated columns was studied by El-Sawy et al. [14] using finite element methods. Their solution takes into account shear and flexural deformations for the calculation of the buckling capacity. However, their study used finite element methods and only numerical solutions were provided. To the best of the authors' knowledge, no analytical work is available to predict the buckling capacity of castellated columns.

In this paper an analytical solution using the energy method is presented to determine the buckling capacities of castellated columns. A simple close-form solution for determining the critical buckling load of simply supported castellated columns of doubly-symmetric sections, subject to axial compression load is developed. The critical buckling load derived is demonstrated using the results from the finite element analysis published in the literature.

2. Analytical study

The classical bending theory of beams, based on Bernoulli's hypothesis that the plane normal cross sections of a beam remain plane and normal to the deflected centroidal axis of the beam during deflection, ignores the deformation caused by shear forces. When a column buckles, however, the axial load causes not only bending moments in the cross sections, but also shear forces. This is particularly so in castellated columns because the web is flexible in shear. The deformations due to shear forces in castellated columns can be taken into account by using either the generalized form of the classical bending theory called Timoshenko beam theory [16] or the bending theory of sandwich beams [17]. In the former the assumption that the plane cross sections remain normal to the deflected beam axis is relaxed, that is, the slope of the deflected beam axis is no longer required to be equal to the rotation of the cross section. The difference of these two rotations is defined as the shear angle, which is produced by shear forces that are normal to the deflected beam axis. In the latter the outer layers of the sandwich beam are assumed to deform according to Bernoulli's hypothesis and the cross section of the middle layer behaves as a shear wall. However, the rotation of the middle layer due to shear forces does not need to be equal to the slope of the deflected beam axis.

In addition to the shear deformation, another difficult problem that arises in castellated members is the second moment of area that varies periodically from that of an "I-section" shaped beam (i.e. with no openings) to that of a "two-tee-section" shaped beam (i.e. with openings). This unique nature makes the castellated beam more like a sandwich beam, in which the two tee sections behave as the outer layers of the sandwich beam to take the bending moment, whereas the discontinuous parts of the web behave as the middle layer of the sandwich beam to take shear forces.

Consider a doubly-symmetric section castellated member shown in Fig. 1a, in which the flange width and thickness are b_f and t_f, the web depth and thickness are h_w and t_w, and the half depth of hexagons is a. The distance between the centroids of the top and bottom tee sections is $2e$ as shown in Fig. 1b. Let $u_1(\mathrm{x})$ and $u_2(\mathrm{x})$ be the axial displacements of the centroids of the top and bottom tee sections, and $w(\mathrm{x})$ be the transverse displacement of the section (i.e. all points on the section have the same transverse displacement). According to the displacement assumptions shown in Fig. 1b, the axial displacement at any point at the section with distance x from the origin can be expressed as follows:

For the top tee section, $-(h_w/2+t_f)\le z\le -a$

$$u(x,z)=u_1(x)-(z+e)\frac{dw}{dx}. \tag{1}$$

For the bottom tee section, $a\le z\le (h_w/2+t_f)$

$$u(x,z)=u_2(x)-(z-e)\frac{dw}{dx}. \tag{2}$$

For the middle part between the two tee sections, $-a\le z\le a$

$$u(x,z)=\frac{u_1(x)+u_2(x)}{2}-\frac{z}{a}\left(\frac{u_1(x)-u_2(x)}{2}-(e-a)\frac{dw}{dx}\right). \tag{3}$$

The axial strains in the two tee sections can be obtained using the strain-displacement relation as follows: