Thin-Walled Structures 111 (2017) 93–102

Contents lists available at ScienceDirect

Thin–Walled Structures

journal homepage: www.elsevier.com/locate/tws

Full length article

Resistance of circular concrete-filled tubular sections to combined axial compression and bending

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ARTICLE INFO

Keywords:
Direct strength method (DSM)
Circular concrete-filled tubular (CFT) sections
Resistance
Axial load
Combined axial compression and bending
Simplified strength interaction equations

ABSTRACT

This paper describes strength formulae and simplified strength interaction curves for the direct strength method (DSM) for circular concrete-filled tubular (CFT) sections. A simple axial strength formula and a flexural strength formula for circular CFT sections are proposed to account for the post-local buckling strength of the circular steel skin and the increase in concrete compressive strength caused by confinement of the steel skin. The squash load predicted by the proposed strength formula is compared with test results in the literature, and those predicted by AISC specifications and Eurocode4. A simplified strength interaction curve for circular CFT members under combined axial compression and bending is proposed and compared with test results. The comparison confirms that the proposed axial and flexural strength formulae and simplified strength interaction curves can be used to conservatively predict the resistance of circular CFT columns to an axial load, and combined axial compression and bending.

1. Introduction

Circular concrete-filled tubular (CFT) sections have been commonly used as structural members for high-rise buildings, bridges, and tower structures in recent years. Generally, circular CFT sections show significantly increased local buckling strength in comparison with circular hollow steel (CHS) sections, and significantly higherstrength than the total sum of the individual strengths of the steel skin and filled-in concrete. When a circular hollow steel section contains concrete infill, it inhibits the outward deformation of filled-in concrete. The restraining effect of circular steel skin is much stronger than that of rectangular steel skin dueto the hoop tension in the circular steel skin, which significantly enhances the strength of the concrete. However, since a state of tri-axial stressof the steel skin reducing the hoop tension stress at yield, in accordance with the von Mises' criterion, and a local instability of the steel skin have the effect of loweringthe confining effect, the effect cannot be estimated theoretically. Although the elastic local buckling stress of a circular steel section is proportional to the ratio of the thickness to the diameter and is much higher than that of other shaped steel sections, a practical circular steel skin of CFT sections is susceptible to elastic or inelastic local buckling under compression. Moreover, there is a significant post-local buckling strength for circular steel skins, and this should be accounted for in estimating the ultimate strength of the steel skin of circular CFT sections. A reasonable estimation of the increments in the compressive strength of filled-in concrete from lateral confinement by the locally buckled circular steel skin should also be considered properly in predicting the ultimate strength of circular CFT sections.

The local buckling of steel skin and the enhancement of filled-in concrete strength have already been considered in current specifications [1,2]. Although circular steel skin can be treated as a shell buckling problem [3], the local and post-local buckling strength of circular steel skin can be determined in a manner similar to the calculations used for rectangular steel skin [4]. Eurocode4 (2004) (EC4) [1] has provisions for nominal strength in which the nominal concrete cube strength and nominal yield strength for steel are used with partial material factors, and an additional increase in concrete strength enhanced by hoop tension can be accounted for in circular CFT columns. AISC specifications (2010) [2] allow for the use of slender steel sections and provide detailed strength formulae. According to the slenderness of the steel skin in the AISC specifications, the strength of the steel skin ranges from the yield stress for compact sections to the elastic local buckling stress for slender sections. Concrete strength for circular sections ranges from $0.95F_c$ for compact sections to $0.70F_c$ for slender sections. However, EC4 [1] does not provides detailed procedures to account for the influence of the local buckling of slender sections with diameter-to-thickness ratios exceeding the maximum limit ($=90\cdot235/F_y$). In EC4, the design concrete strength is taken as $1.0F_c/1.5$ for sections within the maximum limit of D/t. EC4 states that the effects of local buckling should be considered in design for slender sections

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http://dx.doi.org/10.1016/j.tws.2016.11.014
Received 18 May 2016; Received in revised form 17 November 2016; Accepted 21 November 2016
0263-8231/

exceeding the maximum D/t limit.

Referring to the strength interaction curves of compression and bending, those for compact composite sections in the EC4 and AISC specifications are quite similar. The AISC specifications [2] recommends that the interaction curves for steel members rather than those for composite members should be applied to noncompact and slender composite sections because of a lack of research but the EC4 does not have any tangible statements for those sections. Studies of circular CFT columns have been performed experimentally and numerically over several decades. However, most experimental studies have focused on the behavior and stability of compact circular CFT sections under axial loading, and on the use of high-strength steel and concrete [5–13]. A few studies have examined the stability of circular CFT columns under bending and combined axial compression and bending [14–19]. The present paper proposes a simplified strength interaction curve and the adoption of a set of squash load equations for the direct strength method (DSM) for circular CFT columns to account for the local buckling and post-local buckling strength of circular hollow steel skin, based on previous compression test results for circular CFT sections. The predicted squash loads of circular CFT columns are compared with test results in the literature [5–15] and those predicted by existing specifications [1,2]. In addition, the paper also proposes a simple formula for flexural strength for the DSM, based on the sectional slenderness of the steel skin of circular CFT sections. All strength formulae for the DSM use the elastic local buckling stress of the CHS skin, which can be computed by using a rigorous analysis program or theoretical equation, and a design strength formula based on test results. In addition, the paper proposes and compares simplified strength interaction curves with test results under eccentric loading for circular CFT sections [16–18]. The comparison confirms that the proposed formula for axial and flexural strength, and the simplified strength interaction equations, can conservatively predict the strength of circular CFT columns.

2. Resistance of circular CFT columns to axial compression

2.1. General

In general, because of the enhancement of the local buckling strength of circular steel skin by filled-in concrete, thin circular steel skins with or without longitudinal stiffeners are often used in CFT section columns to carry heavy axial loads with eccentricity in large-scale structures. Therefore, the local buckling shown in Fig. 1 usually occurs in circular CFT section columns. Unusually, the distortional buckling can occur for longitudinally stiffened circular sections in which the bond strength between the concrete and longitudinal stiffeners in a compression zone is not sufficient to resist the resulting force occurring at the stiffeners from the outward expansion of the concrete under compression and/or bending. In practice, distortional buckling is rarely found in circular CFT columns. If a flat stiffener of sufficient length or a T-shaped stiffener is used in a circular CFT column, then this type of distortional buckling cannot occur and is therefore negligible in the practical design. For an accurate estimation of the member strength of circular CFT columns with local buckling, there is a need for a rational design strength formula that can account for the post-local buckling strength of the steel skin and its effects on the strength of filled-in concrete.

2.2. Current design specifications

Even if the magnitude of the concrete compressive strength of a compact concrete-filled tubular stub column is defined in a slightly different manner in Eurocode4 (2004) [1] and AISC (2005) [2], the squash load is generally given by

$$P_o = F_y A_s + K_c f_c A_c \tag{1}$$

where F_y is the steel yield stress, A_s is the steel cross sectional area, f_c is the concrete compressive strength, A_c is the concrete cross sectional area, and the factor K_c for circular CFT sections is taken as 0.95 in the AISC specifications [2] and 1.0 in the EC4 [1]. In EC4, account may be taken of an increase in the concrete compressive strength caused by the confinement of steel skin, as follows:

$$K_c = 1 + \eta_c \frac{t}{d}\frac{F_y}{f_c} \tag{2a}$$

$$\eta_c = \eta_{10}\left(1 - \frac{10e}{d}\right) \tag{2b}$$

$$\eta_{10} = 4.9 - 18.5\lambda + 17\lambda^2 \tag{2c}$$

where e is the eccentricity of loading, d is the exterior diameter, and λ is the slenderness in the plane of bending. Linear interpolation is permitted for various load eccentricities $e(\leq d/10)$ with the basic value η_{10} depending on the relative slenderness λ. The compact limit of the diameter-to-thickness ratio d/t is defined as $90(235/F_y)$ in the EC4 and $0.15E/F_y$ in the AISC specifications (2005).

Regarding slender and noncompact sections, it is mentioned in the EC4 that the effect of the local buckling of steel skin for which the diameter-to-thickness ratio exceeds the compact limit for the resistance of CFT columns should be considered. However, any tangibly detailed method has not been provided. Recently, AISC specifications (2010) [2] allow the use of noncompact and slender steel skins for CFT column and flexural members. The strength formulae for circular CFT columns are given as follows.

For noncompact sections,

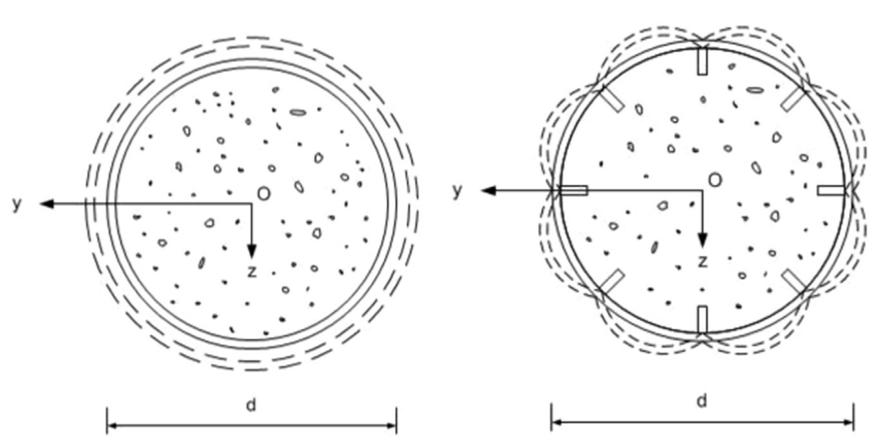

Fig. 1. Local buckling modes of circular CFT columns.