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Resistance of rectangular concrete-filled tubular (CFT) sections to the axial load and combined axial compression and bending

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ABSTRACT

This paper describes the development of the direct strength method (DSM) for concrete-filled tubular (CFT) sections. The axial and flexural strength of CFT sections with local buckling are proposed based on previous test results. Although Eurocode4 does not allow the use of slender steel skins for CFT sections, the limit of the width-to-thickness ratio for the steel skin has recently been extended to slender sections in AISC specifications. A simple formula for the axial and flexural strength of CFT sections for the DSM is proposed to account for the local buckling of a thin steel skin and for the enhanced compressive strength of concrete from the confining effect of the steel skin. The squash load predicted by the proposed formula is compared with test results and those predicted by AISC specifications and Eurocode4. A formula for strength interactions of CFT members under combined compression and flexure is proposed and is compared with test results. The comparison confirmed that the formula for axial and flexural strength and that for strength interactions can conservatively predict the resistance of CFT columns to the axial load and combined compression and bending.

1. Introduction

Concrete-filled tubular (CFT) sections have advantages such as high strength, excellent ductility, and large energy dissipation capacity. CFT sections are used as compressive and flexural members for high-rise buildings and long-span bridges. Since the steel skin confines the outward deformation of the filled-in concrete and the concrete resists the inward deformation of the steel skin, both steel and concrete enhance the strength of CFT sections. However, the steel skin of CFT sections is susceptible to elastic or inelastic local buckling under compression and/or bending. However, there is substantial post-buckling strength in the local mode, and this should be accounted for in estimating the strength of the steel skin. An increase in the compressive strength of concrete from tri-axial confinement by the locally buckled steel skin should also be considered in estimating the ultimate strength of CFT sections.

Eurocode4 (2004) [1] has provisions in which nominal cube strength for the compressive strength of concrete and nominal yield strength for the compact steel skin are adopted with partial safety factors, and some additional amount of concrete strength can be considered for circular CFT columns. AISC specifications (2010) [2] allow for the use of slender steel sections. According to the slenderness of the steel skin, the strength of the steel skin ranges from the yield stress for compact sections to the elastic local buckling stress for slender sections, and concrete strength concordantly ranges from0.85F_c for compact sections to 0.70F_c for slender sections for rectangular sections and 0.95F_c to 0.70F_c for circular sections. Strength interaction curves for compact sections in EC4 and AISC specifications are quite similar. However, the AISC recommends the use of interaction curves for steel sections because of a lack of research.

Since the steel skins for CFT columns are usually thin, they are subject to local buckling [3]. The local buckling of the steel skin which occurs before the collapse of filled-in concrete can affect the ultimate strength of CFT sections [4]. However, most tests have been focused on the behavior and stability of compact CFT sections under axial loading and on the use of high-strength steel and concrete. A few studies have examined the stability of CFT columns under combined compression and bending [5–7]. Uy [5] argued that EC4 produced unconservative resistance for some high-strength steel CFT columns under combined compression and bending and suggested a stress distribution different from that in EC4 as a solution. This paper proposes a set of squash load equations for circular and rectangular CFT columns to account for the local buckling of the steel skin based on previous compression test results for CFT sections. The predicted squash load of CFT

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columns is compared with test results and those predicted by existing specifications. In addition, this paper also proposes a simple formula for flexural strength for the direct strength method (DSM) based on the sectional slenderness of the steel skin of CFT sections.. All strength curves use the elastic local buckling stress of the steel skin, which can be computed by a rigorous analysis program or theoretical equations, and a limiting strength formula based on various test results. This paper proposes and compares strength interaction equations with test results under eccentric loading. The comparison confirmed that the proposed formula for axial and flexural strength and interaction equations can conservatively predict the strength of CFT columns.

2. Resistance of CFT columns to axial compression

2.1. General

In general, because of the enhancement of the local buckling strength of the steel skin by filled-in concrete, thin steel skins are often used for CFT section columns. Therefore, the local and distortional buckling shown in Fig. 1(a)–(c) can occur with overall buckling in rectangular CFT section columns in compression or combined compression and bending. The distortional buckling mode for large-scale box section columns in Fig. 1(c) can occur when the bond strength between concrete and longitudinal stiffeners is not sufficient to resist the resulting force from the outward expansion of concrete. In practice, distortional buckling is rarely found in structures. If a flat stiffener of sufficient length or T-shaped stiffener is used, then this type of buckling does not occur. For the accurate estimation of the member strength of those CFT columns with local buckling, there is a need for a rational design strength formula that can account for the local buckling of the steel skin and its effect on concrete strength.

2.2. Current design specifications

Even if the magnitude of concrete compressive strength is defined a slightly different manner in current design specifications such as the AISC (2005) [8] and Eurocode4 (2004) [1], the ultimate strength of a concrete-filled composite stub column is generally given by

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

where F_y = steel yield stress; A_s = steel area; f_c = concrete compressive strength; A_c = concrete area; and the factor K_c is taken as 0.85 in AISC specifications and 1.0 in EC4. The compact limit of the width-to-thickness ratio is $52\sqrt{235/F_y}$ in EC4 and $2.26\sqrt{E/F_y}$ in AISC specifications (2005).

Recently, AISC specifications (2010) [2] have extended the width-to-thickness ratio limit to the slender sections. The strength formula is

For compact sections

$$\frac{b}{t} \le 2.26\sqrt{E/F_y} \quad P_{n0} = P_p = F_y A_s + 0.85 f_c A_c \tag{2}$$

For noncompact sections

$$2.26\sqrt{E/F_y} < \frac{b}{t} \le 3.0\sqrt{E/F_y}$$
$$P_{n0} = P_p - \frac{\left(b/t - 2.26\sqrt{E/F_y}\right)^2}{\left(0.74\sqrt{E/F_y}\right)^2}(p_P - p_y) \tag{3}$$

where

$$P_y = F_y A_s + 0.70 f_c A_c$$

For slender sections

$$3.0\sqrt{E/F_y} < \frac{b}{t} \le 5.0\sqrt{E/F_y} \quad P_{n0} = P_{cr} = F_{cr} A_s + 0.70 f_c A_c \tag{4}$$

where

$$F_{cr} = \frac{9E}{(b/t)^2}$$

The steel design stress formula accounts for the enhancement of the local buckling stress from filled-in concrete based on the CFT column tests [9], which is approximately 2.5 times that for rectangular hollow tubular sections.

2.3. Strength formula for the DSM

A squash load equation for circular and rectangular CFT section stub columns for the DSM has been proposed based on various test results in Kwon et al. [4]. The equation proposed adopted a single formula for both circular and rectangular CFT section columns. However, since the axial resistance of rectangular sections based on the elastic local buckling stress is very different from that of circular sections and thus produces too conservative estimates for rectangular CFT sections, it needs to be expressed in a different formula for the better estimation of axial strength.

A formula for the axial resistance of rectangular CFT stub columns can be given by

$$P_{no} = \varphi_s F_{sd} \cdot A_s + \varphi_c C_2 \cdot f_c \cdot A_c \tag{5}$$

where F_{sd} = steel design stress; A_s = steel area; f_c = concrete compressive strength; A_c = concrete area; A_{sr} = steel reinforcement bar area; F_{yr} = yield stress of steel bar; and φ_s and φ_c are material factors for steel and concrete and taken as 0.95 and 0.65, respectively. The format of the resistance formula is quite similar to that in EC4. The portion of the reinforcement steel bar for axial resistance is omitted in Eq. (5) for simplicity.

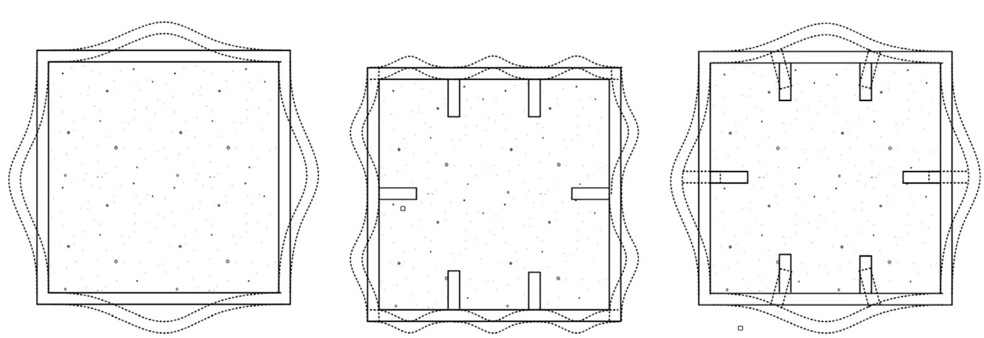

Fig. 1. Buckling modes of rectangular CFT columns.