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Mechanical behaviour of concrete-filled CHS connections subjected to in-plane bending

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ABSTRACT

This paper presents an investigation on the mechanical behaviour of concrete-filled circular hollow section (CHS) connections. Four large-scale connection specimens were tested to failure under in-plane bending. The main test parameters included chord-wall slenderness and diameter ratio between brace and chord. The main failure mode observed from the tests was chord-wall punching shear failure. Meanwhile the chord-wall deformation was also investigated to determine the governing limit state. Complementary finite element (FE) methodology was validated against the experimental findings and the validated FE models were used to further study the mechanical behaviour and the strength design of the connections. The modified Mohr-Coulomb criterion (MMC) was introduced into the finite element models to capture steel fracture failure. Based on both experimental and numerical investigations, a theoretical analytical model with the consideration of the inner concrete was established and an ultimate strength design equation was proposed accordingly to predict the punching shear strength for concrete-filled CHS connections under in-plane bending.

1. Introduction

Concrete-filled steel tubular (CFST) structures are widely adopted in buildings, bridges, marine infrastructures and transmission towers. Inner concrete in a CFST member would significantly improve its local buckling resistance [1–3] especially for those with large diameter-to-thickness ratios. In transmission towers, poles and arch bridges [4], hollow or CFST chords and braces are commonly connected through welding. Previous research on the axial compressive [5–7] and tensile performance [6–9] of T-type concrete-filled CHS connections demonstrated that the capacity and radial stuffiness could be significantly improved when compared with the hollow section connections counterpart. These experimental investigation also showed that the typical failure modes of composite connections were brace failure [5,6] and crushing of inner concrete [5,7] in the case of axial compression and punching shear in the case of axial tension [6–9]. The inner concrete also prevents the inward tube-wall deformation thus enhancing the radial stiffness of the chord member, though not providing the direct tensile or compressive strength resistance to the external force. It can make full use of chord-wall material when the strength of brace is adequate as the connections fail at material level. The hollow horizontal branches welded to the CFST transmission poles will also support the gravity load of the electrical conductors and the ice-load, which will introduce in-plane bending to those connecting areas. However, limited studies [6] have been reported on the performance of in-plane bending loaded concrete-filled or grouted circular hollow section (CHS) connections.

In addition, current design code, AISC 360-10 [10], and guideline, CIDECT-1 [11], for tubular connections are primarily for hollow structural section (HSS) which mainly includes square hollow section (SHS) connections, rectangular hollow section (RHS) connections and CHS connections. By limiting the ratio of chord diameter to thickness (D/t) and selecting adequate material qualities and suitable welding procedures, the failure modes of plain steel HSS connections subjected to in-plane bending can be categorized into 1) chord punching shear (which is unlikely to occur without sufficient rotation capacity [12]) and 2) chord plastification. The analytical model currently adopted in design codes and guidelines for punching shear failure assumes a full plastification of the punching shear area [12] with due account on the influence of chord-to-brace angle [11,13], whilst the design equations for chord plastification in CIDECT-1 [11] are based on the modified function of Gerstein [14] and the analysis by van der Vegte et al. [15,16] and Qian et al. [17]. As for the concrete-filled connections, the design procedure given in CIDECT-3 [18] is based on the

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Notation

The following symbols are used in this paper:

A	parameters of material strain hardening
c_1	friction coefficient in Mohr-Coulomb model
c_2	shear stress resistance in Mohr-Coulomb model
D	outer diameter of the chord member
D_b	outer diameter of the brace member
e_{con}	flow potential eccentricity in the concrete damaged plasticity model
E_s	Young's modulus of steel
E_c	Young's modulus of concrete
f_{bo}, f_{co}	initial equibiaxial compressive yield stress and initial uniaxial compressive yield stress in the concrete damaged plasticity model
f_c	concrete compressive design strength
f_{cu}	concrete compressive cube strength
$f_{u,v}$	ultimate shear strength of steel
$f_{v,cmax}$	the maximum shear stress on the punching shear face at the compression side
$f_{v,tmax}$	the maximum shear stress on the punching shear face at the tension side
f_τ	shear stress distribution on the punching shear face
F_y	yield strength of steel
F_u	ultimate strength of steel
$F_{V,t}$	the total shear force on the tension side of the failure face
K_c	ratio of the second stress invariant on the tensile meridian to that on the compressive meridian in the concrete damaged plasticity model
M_{AISC}	ultimate bending strength calculated from AISC 360-10
M_u	design ultimate bending strength calculated using the proposed equation
M_{Exp}	ultimate moment load obtained from experimental results under in-plane bending
M_{FEA}	ultimate bending strength obtained from FEA
M_{u_Weld}	design ultimate bending strength calculated using the proposed equation considering the weld width
n	parameters of material strain hardening
Q_f	coefficient specified in the AISC 360-10
S_t	the total area of the punching shear failure face at the tension side
Z	internal lever arm of the moment section
t	chord-wall thickness
t_b	brace-wall thickness
T_{eq}	equivalent thickness
$\bar{u}^{pl}$	effective plastic displacement
u_f^{pl}	effective plastic displacement at the failure point
ε_f	elongation (tensile strain) after fracture based on a gauge length of 50 mm
$\bar{\varepsilon}_f$	the equivalent plastic strain at the point of fracture
$\bar{\varepsilon}_p$	the equivalent plastic strain
Δ_B	measured chord-wall deformation at point B in Fig. 1
β	ratio of brace outer diameter to chord outer diameter
γ	ratio of chord outer radius to chord thickness
τ	ratio of brace thickness to chord thickness
θ	the angle parameter defined in Fig. 7(b)
$\bar{\theta}$	Lode angle parameter (normalized Lode angle)
θ_0, θ_1, θ_2	the position of $f_{v,cmax}$, neutral axis and $f_{v,tmax}$ expressed by the angle parameter in Fig. 7(b), respectively
μ	viscosity parameter in the concrete damaged plasticity model
μ_{eq}	a coefficient related to S_t, D_b and t
Φ_s	dilation angle measured in the p-q plane in the concrete damaged plasticity model
η	stress triaxiality.

investigations of Packer and Fear [19] and Packer [7], applicable for concrete-filled SHS and RHS connections. Their investigations are based on Yield-line Theory. The design provisions for concrete-filled CHS connections subjected to in-plane bending is currently scarce. Therefore, this paper aims to develop design provisions for such connections through experimental, numerical and analytical studies. Based on the generated structural performance data and analytical studies, design equations are proposed.

2. Experimental investigation

2.1. Test specimens

Totally four specimens were tested under in-plane bending. The measured geometries (D, t, D_b, t_b) are shown in Table 1. The steel CHS tubes filled with self-compacting concrete were used for chord members, whilst the brace members were fabricated from plain CHS tubes. The nominal lengths of the chord and brace members were kept constant in all the test specimens and were 2000 mm and 630 mm, respectively. Fillet weld was used to connect the brace and chord in this test, as shown in Fig. 1.

The steel mechanical properties (yield strength – F_y, ultimate strength – F_u, elastic modulus – E_s and% of elongation at fracture – ε_f) were determined from tensile coupon tests, as summarized in Table 2. The 150-mm cubic compressive strength (f_{cu}) and elastic modulus (E_c) of inner concrete after 28 days were 56.3 MPa and 34,100 MPa, respectively. A 20-mm steel plate and stiffeners were welded on the top of brace member for MTS installation. Four connection specimens were tested in two consecutive days on the 28th day and 29th day after casting the concrete.

The test specimens were labeled as the order of the connection type, the chord diameter, the chord-wall thickness, the brace

Table 1
Measured geometries, ultimate strengths and failure modes of the test specimens.

Specimens	Chord			Brace			β	M_{Exp} kN·m	Failure mode
	D mm	t mm	L mm	D_b mm	t_b mm	L_b mm			
T-300-4-133-6	298.8	4.09	2002.1	134.2	6.08	630.2	0.449	50.72	CPS
T-300-5-133-6	301.3	5.04	2001.8	133.9	6.09	630.4	0.444	52.98	CPS
T-240-4-203-8	238.8	3.93	2001.3	201.9	8.07	630.1	0.845	108.90	CPS
T-240-5-203-8	241.2	4.95	2000.9	204.1	8.06	630.1	0.846	124.93	CPS

Note: D: Outer diameter of the chord, t: Wall thickness of the chord, L: length of the chord, D_b: Outer diameter of the brace, t_b: Wall thickness of the brace L_b: length of the brace, β: the chord diameter ratio D_b/D, and CPS: Chord punching shear.