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Flexural behaviour of curved concrete filled steel tubular trusses

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ABSTRACT

A series of tests were conducted on curved concrete filled steel tubular (CCFST) trusses with curved CFST chords and hollow braces subjected to bending. A total of 8 specimens, including 4 CCFST trusses, 2 straight CFST trusses (referred to as CFST trusses) and 2 curved hollow tubular trusses were tested to study the effect of the rise-to-span ratio and infill concrete on the flexural performance of CCFST trusses. Different failure modes were observed: bending failure for CFST trusses, bending-shear failure for CCFST trusses and local buckling failure for hollow tubular trusses. The experimental result showed that the stiffness and load-carrying capacity of CCFST trusses were larger than those of CFST trusses, but the CCFST trusses experienced a joint failure with relatively low ductility. Meanwhile, the infill concrete provided support to the steel tubular chords and increased the stiffness, load-carrying capacity and ductility. A finite element analysis (FEA) model of the CCFST trusses was developed and verified by the test results. The model was then used to investigate the mechanical behaviour of the truss by full range analysis. A simplified model was proposed to predict the elastic stiffness considering both the flexural and the shear deformation. The load-carrying capacity of the CCFST structure was also discussed.

1. Introduction

Curved concrete filled steel tubular (CCFST) truss is a type of truss curved in shape and has tubular chords filled with concrete. Fig. 1 shows a schematic view of its application in large span structures. Compared with the truss with hollow tubular chords, buckling behaviour both in and out of the loading plane could be improved because the concrete filled steel tubular (CFST) chords can provide larger flexural stiffness. Local buckling of the chord, especially around the joint, can be prevented or delayed due to the support from the core concrete to the steel tube and thus can increase the strength and ductility of the structures. Also, the infill concrete could increase the fire resistance of the structures. Owing to the above benefits, CCFST trusses have been the interests of more and more structural engineers.

It is well known that CFST members have an excellent performance in compression with high strength and good ductility. Meanwhile, the trusses have been widely used due to the effective force transferring mechanism. At present, a large number of studies have been conducted on CFST members [1–5] and steel tubular trusses [6,7]. Previous research by Gjelsvik [8] on built-up columns (trusses under compression) showed that the effect of shear deformation in a built-up column is much greater than that in a solid column because of the weaker shear stiffness. Research was also conducted on composite trusses composed of steel trusses and concrete slabs. Brattland and Kennedy [9] reported flexural tests of two full-scale composite trusses, where ductile failure due to the tension of the bottom chord was observed. In the 1990s, due to the increasing application of composite trusses, the SCI publication [10] was developed to guide the design of these structures.

Kawano et al. [11,12] studied the deformation capacity of 2-chord CFST trusses under monotonic and cyclic loading. Fong et al. [13] conducted tests of two trusses composed of CFST members and hollow tubular members to investigate the buckling behaviour of CFST trusses. Han et al. [14] reported a series of tests on 3-chord CCFST members subjected to axial compression, where formulas to predict the axial load-carrying capacity were proposed. Till now, however, the behaviour of CCFST trusses under bending or combined bending and shear has seldom been reported yet.

Although there are several types of cross sections for CCFST trusses, only 3-chord CCFST trusses will be investigated in this paper. An experimental programme was developed, including 4 CCFST trusses, 2 straight CFST trusses (referred to as CFST truss) and 2 hollow tubular trusses. Four-point bending tests were conducted to investigate the flexural behaviour. Failure modes and the developments of deflection and strain were analysed. Finite element analysis was carried out. A simplified model was then proposed to predict the elastic stiffness and discussion of the load-carrying capacity was made.

2. Experimental programme

2.1. Test specimens

All truss specimens were designed as the well-known Warren trusses which are commonly used in engineering practice, as shown in Fig. 2. The design guide published by CIDECT [15] was used for the

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Nomenclature

A_c	Cross-sectional area of concrete in a chord
A_d	Cross-sectional area of a diagonal brace
A_s	Cross-sectional area of steel in a chord
b	Effective width of a truss section
d	Overall diameter of a steel tube
E_c	Elastic modulus of concrete
E_s	Elastic modulus of steel
$(EI)_c$	Predicted flexural stiffness of a truss
$(EI)_e$	Measured flexural stiffness of a truss
$(EI)_{ec}$	Summation of flexural stiffness of all the CFST chords
f	Arch rise of a curved truss
f_y	Yield strength of steel
f_u	Tensile strength of steel
h	Effective height of a truss section
K_c	Predicted elastic stiffness of a truss
K_e	Stiffness of the CCFST truss in elastic stage
K_p	Stiffness of the CCFST truss in hardening stage
l	Effective span of a curved truss
m	Shear-to-span ratio ($m = l/4\,h$)
M	Bending moment
M_u	Ultimate bending moment
P	Vertical load
P_{max}	Vertical peak load
P_y	Vertical yield load
P_u	Vertical ultimate load
P_{u1}	Predicted load-carrying capacity when steel reaches its yield strength
P_{u2}	Predicted load-carrying capacity when steel reaches its tensile strength
P_{us}	Predicted load-carrying capacity due to shear
Q	Shear force
r	Ratio of shear deflection ($r = w_2/w_1$)
R	Initial radius of a curved truss
t	Thickness of a steel tube
u_m	Mid-span deflection of a truss
ω	Deflection of a truss
ω_1	Deflection due to flexure
ω_2	Deflection due to shear
ε	Strain
ϕ_m	Mid-span curvature of a truss
γ_s	Flexibility coefficient of shear (in N^{-1})
μ_s	Poisson's ratio of steel

design of the specimens. Elastic analysis was conducted based on the assumption that all joints are pin connected.

Straight CFST trusses were designed first as reference specimens. The design procedure involved the determination of chords, braces and joints. Specimens were expected to experience a flexural failure in a ductile way. To achieve ductile failure, the flexural strength was proposed to be dominated by the tensile capacity of the bottom chord. Owing to the advantages of CFST in compression, the diameter of top chords (89 mm) was much smaller than that of the bottom chord (140 mm). Braces were then designed to bear the shear force. An effective length factor 0.9 was used for checking braces in compression. In the joint design, the K-joints were expected to be strong enough to prevent possible early failure during loading. All joints between braces and chords were gapped joints with positive eccentricity and were connected by fillet welds around the entire perimeter of the braces. The welds were designed to transfer axial force and in-plane moment which are stronger than the corresponding braces. The joint eccentricity was introduced to ensure that the gap of the adjacent braces was large enough to accommodate fillet welds. For joints in the top chords, the eccentricity was 60 mm. The induced secondary moment was considered in designing the top chords and braces. But for joints in the bottom chord, the eccentricity was less than 10 mm, and the effect of joint eccentricity was not considered.

Fig. 2 gives the elevation and cross section view of a CCFST truss specimen, where P is the vertical load applied on the specimen; f and l are the arch rise and effective span of the truss, respectively. The curved specimen was a circular arc in shape defined by the rise-to-span ratio f/l (also called height-to-span ratio sometimes). Based on the design of the above mentioned CFST trusses with $f/l = 0$, the f/l ratio was changed to obtain a CCFST truss. The member sizes of the CCFST truss, however, were not changed. The angle between two diagonal braces in the cross-section (2α, as indicated in Fig. 2) is 60°, and the angle between the diagonal braces in the elevation plane (2β, as indicated in Fig. 2) is also 60°. For comparison purposes, two curved hollow tubular truss specimens with a f/l ratio of 0.1 were included in the test programme. Table 1 gives detailed dimensions of all the specimens, in which specimen designations starting with T and TH represent trusses with CFST chords and steel tube chords, respectively. The section dimension of the truss ($b = 432$ mm, $h = 375$ mm), the profile of the top chord tube (Φ89 mm × 3 mm), bottom chord tube (Φ140 mm × 4 mm), diagonal brace (Φ76 mm × 3 mm) and transverse brace (Φ30 mm × 3 mm) were applied for all the trusses. The main parameters considered were the rise-to-span ratio and type of chords. Three different rise-to-span ratios ($f/l = 0, 0.1, 0.2$) were chosen to investigate the difference between CFST trusses and CCFST trusses. Two other curved hollow tubular trusses with a f/l ratio of 0.1 were tested to study the influence of concrete inside the steel tube chords. Duplicate specimens were prepared to guarantee the reliability of test results.

2.2. Material properties

Electric Resistance Welded (ERW) tubes were used to fabricate the trusses. The curved chord was bended from a 6 m straight tube with its two ends cut flat. Three chords were then welded to a steel end plate of 20 mm thick on both sides. Holes were cut on one of the steel plates if required, which were used for pouring concrete. After the fabrication of the steel truss, the specimen was vertically placed and concrete was poured into each chord except those hollow specimens. After a curing age of two weeks, the holes in the end plate were repaired by welding small steel plates. The measured overall diameter (d), thickness (t), yield strength (f_y), tensile strength (f_u), elastic modulus (E_s) and Poisson's ratio (μ_s) of the steel tubes are shown in Table 2. The measured stress versus strain curves of steel are given in Fig. 3.

Self-consolidating concrete (SCC) was used as the core concrete with a designed cubic strength (f_{cu}) of 60 MPa. The mix proportions of SCC were: water 173 kg/m^3, cement 380 kg/m^3, fly ash 170 kg/m^3, fine aggregate 840 kg/m^3, coarse aggregate 840 kg/m^3, and superplasticizer 2.5% of the cementing material. Properties of fresh concrete before pouring were tested, and the obtained results were as follows: slump flow 251 mm, slump flow distance 623 mm, and interior temperature 18 °C which was 1 °C higher than the environment temperature. At the time of tests, the measured average cubic compression strength was 69.9 N/mm^2 and the elastic modulus was 36,400 N/mm^2.

2.3. Loading and measurement

The loading and measuring devices of the test setup are shown in Figs. 2 and 4. Hinged support on the left side and sliding support on the right side were adopted. The specimen was tested under four-point bending. Vertical load (P) was applied at the mid-span by a loading jack with a maximum capacity of 5000 kN. Lateral restraint was provided at quarter points to eliminate lateral deflection. Prior to testing, yield load (P_y) of the specimen was predicted by finite element (FE) analysis. The computed P_y was used to determine the testing