Performance of concrete-filled steel tube truss girders strength by adding reinforcement

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Abstract: Concrete filled steel tube (CFST) truss girder usually consists of CFST chords and hollow braces. The performance of steel tube truss girders filled with self-compacting concrete was investigated in this study. A total of eight CFST truss girders (Warren-vertical truss) specimens were tested. The main parameters were the concrete compressive strength and adding a reinforced bar in the concrete core of the bottom chord. Two of specimens without reinforcement were suggested as reference specimens. This study shows the load-deflection curves at the midspan, overall deflections, ultimate loads, flexural strength and failure modes of the tested specimens. The Proposed design equation found in the literature was used to predict the flexural strength of CFST trusses. The failure mode includes: weld fracture and shredding around joints and local buckling of the diagonal braces. Results show that the increase of concrete compressive strength caused a slight increase of the CFST truss strength by about (2.8%-10.34%), also the results showed that the best addition to the core concrete of the bottom chord was the circular steel tube compared with the addition of the steel bars.

Key words: Concrete filled steel tube (CFST), Truss girders, Warren-vertical truss, Self-compacting concrete, Load deflection, Design equation.

1. Introduction
Concrete Filled Steel Tube (CFST) truss girder consists of chords filled with concrete and hollow braces, the existence of concrete in chords has many benefits, the concrete can delay the local buckling of the steel tube while the steel tube restricts the concrete. This leads to increase load carrying-capacity, flexural stiffness and ductility. The steel tube work as formwork for concrete and this leads to reduced working hands, construction cost and time [1-4]. Concrete-filled sections affected by vibrations less than hollow sections [5, 6]. These advantages have led to the wide use of CFST members in civil engineering structures. Han et al.[7] explained the evolution and applications of CFST members, comparison between CFST column with hollow steel column and reinforced concrete RC column, the load-carrying ability of CFST column was greater than the sum a load of the hollow steel column and RC column.

CFST members used in different types of structures around the world. For instance, they have been used in high rise buildings[8, 9], electric pole[7], girders in roof system[10] and main members in bridges, as shown in Figure 1.

The first research on the CFST truss girder was done in 2000 by Zhang et al[11]. Since then, a few studies investigated the behavior of CFST truss girders. Chen et al.[12] tested experimentally three truss girders, the type of trusses was Pratt, first truss specimen was hollow, the top chord filled with concrete only in the second truss specimen, while the top and bottom chords of the third truss specimen were...
filled with concrete. The test results shows that the existence of concrete in chords increased the stiffness and the strength of the CFST truss girders. Huang et al.[13] studied the effect of truss type on the performance of CFST truss girder. Three types of trusses were tested in this study, the first type was Warren-vertical truss (has vertical braces along with diagonal braces), the second type was Pratt truss, and the third type was Warren truss. The results showed that the Warren-vertical CFST truss girder had great performance, followed by the Pratt truss and then the Warren truss. The Warren-vertical and Pratt trusses failure mode was punching shear failure of a tensile brace at the brace to chord joint area, followed by local buckling of the compression brace at the same joint. Meanwhile the Warren truss failure mode was local buckling of the compression braces.

Xu et al. [14] studied the performance of curved CFST trusses. eight specimens were tested, four of these specimens were curved CFST trusses, two specimens were straight CFST trusses, and the last two specimens were curved hollow steel tube trusses. Han et al. [15] tested six CFST trusses without slabs and four hybrid CFST trusses with slabs and two hollow steel tube trusses. The effect of shear span to depth ratio, concrete strength of the core, concrete strength of the slab and the angle between the chords and the diagonal brace were studied. These studies reveal that the strength and stiffness of CFST trusses significantly increases compared to hollow steel tube trusses and the CFST trusses have two types of failure modes, the first is a tensile fracture of bottom chord and the second is a shear failure. Zhou et al. [16] studied the behavior of CFST truss girders by tested four specimens with circular section, the first specimen (CH) has hollow steel section chords, the second (CT) has only the top chord filled with concrete, the third (CB) has only the bottom chord filled with concrete and the fourth specimen (CA) has top and bottom chords filled with concrete. This study indicated that the best load carrying capacity obtained from specimen (CA) and was 180 kN, while the maximum load obtained from specimens (CH), (CB), and (CT) were 85, 92.2, and 145 kN, respectively. From these results, we can see increasing in ultimate load by 111.76% in the specimen filled with concrete (CA) compared to the empty specimen (CH).

Wenjin and Bruno[17] studied the performance of welded joints of circular hollow steel and CFST truss girder. The existence of infill concrete in the top and bottom chords increased the joint rigidity and the strength of the whole truss, the governing failure mode was the punching shear failure. Wenjin et al.[18] investigated the behavior of prestressed CFST truss girder, the prestressed was achieved by using high strength strands. A total of five specimens were tested in order to investigate the effect of the shear span to depth ratio and prestress level. The results indicate that the strength and stiffness of the prestress CFST truss girders are increasing by increasing the ratios of shear span to depth and prestress level. When the shear span to depth ratio and prestress level ratio were increasing the failure mode was a tensile fracture of the bottom chord and when decreased the failure mode was a joint shear failure. Wenjin et al.[19] studied experimentally and numerically the performance of Warren-vertical truss girders. The test results showed that the CFST had two failure modes, tensile fracture of the bottom chord (flexural dominated) and shear failure in the joint (shear dominated). They recommended that the CFST trusses must be designed as a flexural dominated, moreover, they recommended that (i) the compression strength concrete in a top chord should be not lesser the bottom chord by 1.1%, (ii) shear span to depth ratio should be greater than 4.8, (iii) brace to chord ratio should be greater than 0.8.

The truss load carrying-capacity mainly depends on the strength of the bottom chord, this is due to the lack of concrete effect in the tensile area compared to the compression zone. Therefore, many researchers have tried to strengthen the tensile properties. So we seek in this study to strengthen the bottom chord by adding a steel reinforcement (four bars with diameter 8mm, one bar with diameter 16mm, and circular steel tube with diameter 50mm and thickness 1.3mm) embedded in the concrete core of the bottom chord, where the concrete used was self-compacting concrete for being suitable in such types of structures. Eight CFST truss girders were tested experimentally. The test parameter is the concrete grade and reinforcement type in bottom chord, while the concrete grade in the top chord is constant for all eight specimens.
2. Experimental work

Eight CFST truss girders (Warren-Vertical truss) were tested. The test parameters were the concrete strength and adding steel reinforcement bars and circular steel tube embedded in the concrete core at the bottom chord, as shown in Table 1. The first letter of the nomenclature used in Table 1 (R0) indicate that the bottom chord has no reinforcing, (R4) represent the concrete in the bottom chord reinforced by 4 steel bars of 8mm diameter, (R1) means the concrete in the bottom chord reinforced by one bar of 16mm diameter, (RΦ) indicate the concrete in the bottom chord reinforced by circular steel tube of 50mm diameter and 1.3mm wall thickness, the second part (T50) indicates the concrete compressive strength at the top chord is 50 MPa, the third part (B50) indicates the value of concrete compressive strength at the bottom chord. The concrete grade demonstrates the concrete cube compression strength at 28 days in MPa. All the three types of reinforcement used (4 bars 8mm, 1 bar 16mm, and circular tube Φ 50mm) approximate have the same area 200 mm² and adding 14 % ratio to the original section.

The section of the steel tube used was square and the geometric properties of all specimens were the same: chord width (B) was 100mm, chord thickness (t) was 3.8mm, brace width (Bφ) was 80mm, brace

Figure 1. CFST members in bridges[7].
thickness ($t_p$) was 2.8mm. Truss height was 500mm, overall length was 2660mm and the angle between the chords and diagonal braces was 54° as shown in Figure 2. The width and thickness for the vertical braces near support were increased to 100mm and 3.8mm, respectively, to avoid the local buckling due to concentrated reaction from the supports.

The fabrication of specimens was done by spaced the top and bottom chords to obtain the desired truss height. Then, welded the braces to the chords as shown in Figure 3. Then the specimens were placed vertically and filled with self compacting concrete, after 28day the specimens were tested.

The concrete used in this work was self-compacting concrete (SCC). The mix proportions of the SCC were given in Table 2. The fresh concrete tests were presented in Table 3. The concrete properties were measured according to European Guidelines [20].

The steel properties were shown in Table 4 measured according to ASTM 370-05a[21].

### Table 1. Specimens properties.

| Specimen Nomenclature | fcu (MPa) Top chord | fcu (MPa) Bottom chord | reinforcement at the core of the bottom chord |
|------------------------|---------------------|------------------------|---------------------------------------------|
| R0T50B30               | 50                  | 30                     | Non.                                        |
| R0T50B50               | 50                  | 50                     | Non.                                        |
| R4T50B30               | 50                  | 30                     | 4 Ø 8                                       |
| R4T50B50               | 50                  | 50                     | 4 Ø 8                                       |
| R1T50B30               | 50                  | 30                     | 1 Ø 16                                      |
| R1T50B50               | 50                  | 50                     | 1 Ø 16                                      |
| RØT50B30               | 50                  | 30                     | Circular tube Ø= 50mm, $t_p$= 1.3mm         |
| RØT50B50               | 50                  | 50                     | Circular tube Ø= 50mm, $t_p$= 1.3mm         |

### Table 2. Mix proportions of concrete.

| Material                          | Concrete grade |
|-----------------------------------|----------------|
|                                  | C30            | C50            |
| Cement (kg/m³)                    | 300            | 450            |
| Sand (kg/m³)                      | 670            | 670            |
| Gravel (kg/m³)                    | 730            | 730            |
| Limestone powder (kg/m³)          | 110            | 90             |
| SP %                              | 1              | 1.5            |
| Water (kg/m³)                     | 210            | 189            |
| W/C ratio                         | 0.7            | 0.42           |
| fcu at 28 (MPa)                   | 29.23          | 48.41          |

### Table 3. Test results of fresh concrete.

| Test                  | Test result | Specification[20] |
|-----------------------|-------------|-------------------|
| Slump Flow (mm)       | 760         | 730               | (600 – 800) mm    |
| T50 Slump (Sec)       | 3           | 3.5               | (2 – 5) Sec       |
| L Box                 | 0.9         | 0.85              | (0.8 – 1)         |
| V Funnel (Sec)        | 9.4         | 10.1              | (8 – 12) Sec      |
Table 4. Properties of steel.

| Member                | Yield stress MPa | Ultimate stress MPa |
|-----------------------|------------------|---------------------|
| Steel tube 100mm      | 335              | 371.24              |
| Steel tube 80mm       | 390              | 437.59              |
| Steel tube Ø50mm      | 440.57           | 518.32              |
| Steel bar Ø8mm        | 410              | 589.37              |
| Steel bar Ø16mm       | 480              | 545.54              |

Figure 2. Details of specimens, (A) without reinforcement (B) reinforced by 4 Ø8mm (C) reinforced by 1 Ø16mm (D) reinforced by circular tube.
Figure 3. Specimens fabricated and casting.

Figure 4 shows the instrumentation, the specimens were simply supported and subjected to a concentrated load at the Midspan by a hydraulic jack machine with a capacity of 1000 kN. Four lateral supports two in each side to provide lateral stability to the specimen. Three linear variable displacement transducers (LVDTs) were used to measure the vertical deflection.
3. Test results

Table 5 shows the specimen ultimate load and its corresponding midspan deflection. When the concrete strength is identical to the specimens the ultimate load of specimens R0T50B50, R1T50B50, R4T50B50 and RΦT50B50 were 430.7, 474.9, 469.4, and 518.7 kN, respectively. While the ultimate load of specimens R0T50B30, R1T50B30, R4T50B30 and RΦT50B30 were 410.2, 430.4, 456.6, and 502.3 kN, respectively. As shown the ultimate load variation effected by changing the concrete compressive strength in the bottom chord of specimens R0T50B50, R1T50B50, R4T50B50 and RΦT50B50 as compared to R0T50B30, R1T50B30, R4T50B30 and RΦT50B30 were increased by about 4.99%, 10.34%, 2.8% and 3.26% respectively. Meanwhile the ultimate load variation effected by changing the reinforcing type in the bottom chord of specimens R1T50B30, R4T50B30 and RΦT50B30 as compared to R0T50B30, R1T50B30, R4T50B30 and RΦT50B30 were increased by about 4.92%, 11.31% and 22.45%, respectively, and increased by about 10.27%, 8.98% and 20.43% for the specimens R1T50B50, R4T50B50 and RΦT50B50 as compared to specimen R0T50B50. From these results, it can be concluded that the addition of a circular steel tube is the best choice added to the concrete on the bottom chord compared with the addition of steel bars because of the confinement between the concrete and the circular steel tube. The specimen stiffness were determined by calculating the slope of the elastic zone. The displacement ductility index can be estimated as the ratio of the displacement corresponding to 85% of the maximum load on the post-peak portion of the curve to the displacement corresponding to the first yield displacement of a specimen as shown in Figure 5[22]. It is seen that with increasing the concrete compressive strength at the bottom chord, the ductility ratio increases.

Figure 6 and Figure 7 shows the load midspan deflection of the specimens effected by changing the concrete compressive strength and effected by changing the reinforcement, respectively. The
performance of specimens in the load deflection curve almost the same and can summarized as follows, the curve remained approximate linear when the applied load was less than 0.75Pu (Pu is the peak load). When the applied load from 0.75Pu to 0.9Pu the steel tube at the bottom chord starting to yield, while the steel tube at the top chord stayed mostly elastic. When the applied load reached the peak load the specimens failed. From Figure 6 it can be concluded that the strength of CFST truss not affected by increase concrete compressive strength in the bottom chord. However, Figure 7 shows that the strength of CFST truss is effected by adding steel bars or circular steel tube in the bottom chord.

| Specimen   | Ultimate load Pu (kN) | Deflection at Ultimate load (mm) | Stiffness (kN/mm) | Ductility ratio |
|------------|-----------------------|----------------------------------|-------------------|-----------------|
| R0T50B30   | 410.2                 | 20.34                            | 31.74             | 1.98            |
| R0T50B50   | 430.7                 | 22.45                            | 47.72             | 3.16            |
| R4T50B30   | 456.6                 | 16.59                            | 42.74             | 2.25            |
| R4T50B50   | 469.4                 | 15.19                            | 48.35             | 2.59            |
| R1T50B30   | 430.4                 | 16.99                            | 46.77             | 2.33            |
| R1T50B50   | 474.9                 | 24.38                            | 39.29             | 2.92            |
| RΦT50B30   | 502.3                 | 21.04                            | 44.28             | 1.87            |
| RΦT50B50   | 518.7                 | 19.29                            | 53.25             | 2.67            |

**Figure 5.** Definition of displacement–ductility ratio.
For each specimen, the deflections were measured at three locations (mid, one-third, and one-sixth of the span), the deflections of the second half span assumed equal to the deflections of the first half span. Figure 8 shows the deflections along the span of the specimen and compared with the half-sine curve in the dished line at different load levels. As shown when the applied load was below 50% of the peak load the deflections of the test specimens and the half-sine are almost identical, however, if the
applied load was above 50% of the peak load the deflections of the specimens are smaller than the half-sine except for the deflection at the midspan this is because the half-sine curve was depended on the value of the midspan deflection. These results were agreed with those obtained by Huang et al.[19] and Xu et al.[14]. After overriding the ultimate load, the deformations became high and this affected in the joints, some of these joints had failed in the welding zone completely due to tension loads, while the other joints suffered from buckling due to compression loads. The shapes of failure are shown in Figure 9, as clear the failure mode in Figure 9-A was weld fracture at the bottom side of the inclined brace, Figure 9-B shows failed the inclined braces in joint and local buckling at the upper side also support failure at the bottom side, Figure 9-C shows the inclined braces failed by weld fracture and local buckling at the bottom side, while Figure 9-D shows the inclined braces failed by joint failure and surface plasticity at the upper side and shredding of the vertical brace at the bottom side.
4. Design equation
The bending moments in CFST trusses were majorly carrying by the top and bottom chords. These chords additionally sustained secondary bending moments transmitted out from the joints. The stresses due to the secondary bending moment have been a little compared with the stresses due to the bending moment from the chords and can be neglected [18]. Therefore, the flexural strength of CFST trusses is calculated by top chord compressive strength or by bottom chord tensile strength. The failure in the
bottom chord occurs before the top chord (if the chords sections are identical), and the reason that the tensile strength at the bottom chord carried by the steel tube only and the core concrete prevent the local buckling, meanwhile the top chord carried a compressive strength by the steel tube and the concrete. Therefore, Han predicted Eqs.(1) and (2) [23] to find the CFST truss flexural strength depending on bottom chord tensile strength.

\[
M_{up} = T * h 
\]

\[
T = (1.1 - 0.4\alpha) f_y A_s 
\]

Where \(M_{up}\): predicted moment, \(h\): truss height center to center between chords, \(T\): bottom chord tensile strength, \(\alpha\): steel ratio = \(A_s/A_c\), \(A_s\): bottom chord steel tube cross section area of, \(A_c\): cross section area of the concrete and \(f_y\): bottom chord steel tube yield strength.

The predicted design Eqs. (1) and (2) were used to find the theoretical flexural strength of all the specimens tested in this study and flexural strength obtained from other studies. Figure 10 and Table 6 show the flexural strength obtained from experimental test and from predicted equation. As shown, predicted equation gives reasonable results within 20%. The predicted equation has some defect because the predicted tensile force in Eq. (2) when tested a column under direct tension so the behavior in direct tension different the behavior of the bottom chord in truss girders. Moreover, this equation must be modulated to take into account the existence of any extra section with the basic section, for instance, there is steel reinforcement in the concrete as in this study.

Figure 10. Comparison between tests and predicted flexural strength, according to predicted equation with other studies.
Table 6. The flexural strength comparison between tests and predicted, according to predicted equation with other studies.

| Researchers          | Specimen | Section | $h$ (mm) | $A_s$ (mm$^2$) | $A_c$ (mm$^2$) | Steel properties MPa | Flexural strength kN.m |
|----------------------|----------|---------|----------|----------------|-----------------|-----------------------|------------------------|
| Huang et al.[17]     | W2       | circular| 399      | 1006          | 11450           | $f_y$ 181.01          | $f_u$ 230.04           |
|                      | AW2      |         |          |                |                 |                       | 205.02                 |
|                      | P2       |         |          |                |                 |                       | 212.4                  |
|                      | AP2      |         |          |                |                 |                       | 202.2                  |
|                      | T8       |         |          |                |                 |                       | 207                    |
| Han et al.[15]       | T8S      | circular| 375      | 1733           | 13714           | $M_{exp}$ 170.1       | $M_{equ}$ 209.8        |
|                      | T6       |         |          |                |                 |                       | 175.2                  |
| Zhou et al.[16]      | CA       | circular| 400      | 628            | 7238            | $M_{exp}$ 96.75       | $M_{equ}$ 91.52        |
| Huang et al.[19]     | H5C50C30 | circular| 500      | 1479.5         | 6687            | $M_{exp}$ 346.3       | $M_{equ}$ 327.77       |
|                      | H5C50C50 |         |          |                |                 |                       | 352.1                  |
|                      | H5C70C70 |         |          |                |                 |                       | 374.4                  |
|                      | R0T50B30 |         |          |                |                 |                       | 252.3                  |
|                      | R0T50B50 |         |          |                |                 |                       | 264.8                  |
|                      | R1T50B30 |         |          |                | 1375            | $M_{exp}$ 335         | $M_{equ}$ 190.8        |
|                      | R1T50B50 |         |          | 8538           | 371.24          |                       | 264.7                  |
|                      | R4T50B30 |         |          | 335            |                 |                       | 292.1                  |
|                      | R4T50B50 |         |          | 8337           |                 |                       | 280.82                 |
|                      | RΦT50B30 |         |          | 1576           | 8334            | $M_{exp}$ 335         | $M_{equ}$ 216.33       |
|                      | RΦT50B50 |         |          | 1576           |                 |                       | 308.95                 |

5. Conclusion
Performance of steel tube truss girders filled with reinforced self-compacting concrete was studied in this paper. Some main conclusions can be summarized from the test results as follows:

1- A slight increase in the load carrying ability and strength of CFST truss girders by increase the concrete compressive strength.

2- The presence of steel reinforcement increased the strength of specimens R1T50B30, R4T50B30 and RΦT50B30 as compared to specimen R0T50B30 by about 4.92%, 11.31% and 22.44% respectively. Meanwhile, increased for the specimens R1T50B50, R4T50B50 and RΦT50B50 as compared to R0T50B50 by about 10.27%, 8.98% and 20.43%. The addition of steel reinforcement in the core concrete of the bottom chord gives a higher strength from the reference specimen.

3- The addition of steel bars inside the core concrete of the bottom chord does not give the desired benefit while the best addition was using a steel tube inside the core concrete of the bottom chord.

4- The predicted design equation is acceptable within certain limits and we need many studies to prove its effectiveness or amendment.

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