EXPERIMENT ON MECHANICAL BEHAVIOUR OF CONCRETE-FILLED STEEL TUBULAR TRUSS ARCH

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SUMMARY

In order to study the mechanical behavior of concrete filled steel tubular (CFST) truss arch bridge, a testing of CFST truss arch structures had been conducted, and then their differences had been compared with CFST latticed columns and truss girders. The results indicate that the CFST truss arch has a preferable integrity before ultimate load appear. However, the failure mode of CFST truss arch ribs has a great difference with truss girders and latticed columns, even all of them have some similar mechanical behaviors. The plastic area will be developed along with arch axial direction besides cross-sectional direction, which will be redistributed for the internal force. After continuing to load, the load carrying capacity will decrease and diagonal web members will crack at ends, which show that the truss arch has the behavior of truss-girder. Nevertheless, the chords, welding joints and the areas of intersecting junction for the CFST arch ribs do not crack during the loading process, which are different to truss-girder. The arch presents a favourable performance of strength and ductility.

Keywords: *Concrete-filled steel tube, truss arch bridge, truss girder, laced column, experiment, mechanical behavior, failure mode.*

1. INTRODUCTION

In recent years, there're many FEM and experimental researches on CFST latticed column and truss girder [1-6], and their ultimate bearing capacity calculating methods have been investigated [7-12]. Researches show that CFST latticed column has favorable strength and ductility, which is similar to CFST solid single-pipe column. In addition, concrete can prevent the tube from local buckling, and the strength failure and integral instability of CFST laced columns are their mainly failure modes. The chords are the main load-carrying components, while web members and nodes of latticed columns are only construction measurement for benefiting to integrity of structure and can be negligible to bear the load. However, shear deformation of chords affects the mechanical performance of CFST latticed columns greatly and need consider its influence on the ultimate bearing capacity.

Researches [13, 14] show that there are distinct differences of mechanical behavior and failure mode between the CFST latticed columns and truss girder. The CFST latticed columns often performs the overall compressive instability, while CFST truss girder is usually the local failure (tension crack of web members, welding nodes failure). In addition, mechanical properties of CFST truss girder are associated with the type of layout of web members and its ultimate bearing capacity is also influenced by the welding nodes, but the web members and nodes don't have too much influence on CFST latticed columns.

Obviously, the mechanical behavior of CFST truss arch lies between latticed columns and truss girder, in another word, CFST truss arch has both the mechanical characteristics of truss beams and laced columns simultaneously. However, whether the performance of CFST truss arch is more similar to latticed column or truss girder, and what the differences among them, et al, are still lack of deeply study. Thus, further researches of mechanical properties and failure mode on CFST truss arch structure are necessary for the purpose of understanding the differences.

CFST arch bridge becomes an important type of bridge in China, whose total amount over 300 and the longest span reaches 530m at present. Among them, most are of truss type for the span is more than 150m [15] . Thus, in practical engineering, understanding the mechanical properties and failure mode of CFST truss arch as well as equivalent beam-column method for calculating the ultimate bearing capacity, are necessary to study.

Aiming at these problems, an experiment of CFST truss arch structure based on the standard prototype in preference [10] was carried out. The mechanical property and failure mode of CFST truss arch were analysed and compared with CFST laced columns and truss girders. Finally, some design recommends were presented. Due to the limited space, this paper only shows a part of testing results.

2. TESTING INTRODUCTION

2.1. Specimen Design

Two CFST truss arch structures have been build according to preference [10], for convenience, two truss arches are named A-0(mainly introduced in this paper) and A-1 respectively. Their computational span is 9m, and vertical height (rise) is 1.8m with rise-span ratio of 1/5. Arch axes are of quadratic parabola, and total number of internodes is 26. The main components, such as chords, laced tubes, straight (vertical) and diagonal web members, skewbacks and loading pads, are showed in Fig.1.

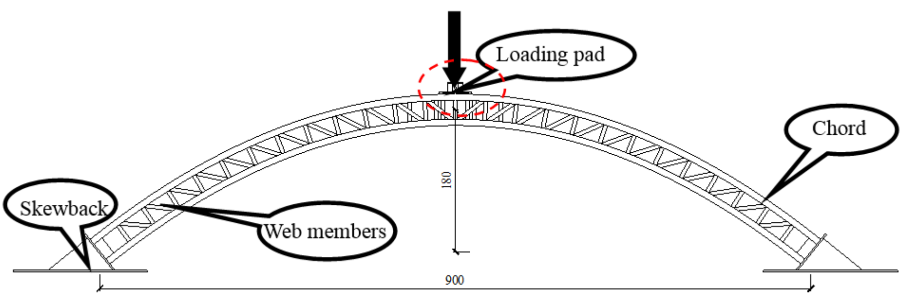


Fig. 1. *Plan view of CFST truss arch model (A-0, Units: cm).*

Cross-section of the truss arch is 0.4 m × 0.4 m that consists of chords, laced tubes, web members and diagonal web members just as shown in Fig. 2. The dimension of chord (column limb) is Φ89mm×4mm, the others (including laced tubes, vertical and diagonal web members, et. al) are Φ48mm×2mm. Main arch rib are composed of 4 chords that filled with concrete, for convenience, the four chord are named “top chord 1”, “top chord 2”, “bottom chord 1” and “bottom chord 2” (details in Fig. 2).

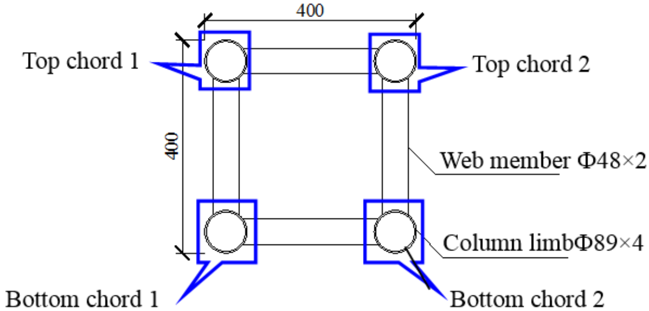


Fig. 2. *Cross-section of arch rib (unit: mm).*

Fig. 3. *Photos of CFST truss arch model.*

A loading pad was made for the purpose of distributing the concentrated force load to the whole section of the CFST truss arch model. Meanwhile, considering the CFST truss arch may bear a larger vertical load than truss girder generally, the web members near the concentrated force loading area were densified to avoid local punching failure (concave). Around the loading area between two internodes were welded 4 extra steel pipes (details of densification showed in Fig.1).

The steel pipe of CFST truss arch is Q235 with a measured elastic modulus of 2.1×10^5 MPa, Poisson's ratio v of 0.283, yield strength of 260 MPa, ultimate tensile strength 350MPa and yield strain of 1018 $\mu\varepsilon$, while the concrete is C30 with a measured elastic modulus E_c of 3.0×10^4 MPa, 28-day cubic compressive strength of 31.6 MPa and Poisson's ratio v of 0.168.

2.2. Sensors arrangement

Total of 9 controlling sections, such as span of 1L/6, 1L/4, 1L/3, 1L/2, 2L/3, 3L/4, 5L/6 of the arch rib, left and right springings, were selected as measuring areas. Section of 1L/4, 3L/4 and 1L/2 are usually called quarter spans and arch crown, respectively. Transverse and longitudinal strain gages were attached to every chord of controlling section, web members and horizontal laced tubes, where are sum of 96 strain gages. Meanwhile, top and bottom chord of 9 controlling sections were equipped with displacement meters to measure vertical and longitudinal displacement of arch as well as the relatively compressive deformation of web members between top chord and bottom chord. In addition, some transverse displacement meters are installed at the 1L/4, 1L/2 and 3L/4 sections to measure the lateral deformation. Every displacement meter was fixed with magnetic gauge stand, data of strain and displacement were acquired by DH3816 system. Furthermore, dial indicators were installed at the skewbacks to measure their slippages.

3. PROCESS OF LOADING

The test speicmen were loaded at mid span(single-point) symmetrically by using double-acting hydraulic jack in the tests. The double-acting hydraulic jack has a maximum loading capacity of 200 t and working stroke of 200 mm, whose really reactive force of loading value will be monitored by a 150 t calibrated sensor. Anti-laterally overturning devices were installed near the loading area. The device that can avoid overturn effectively, consisted of two diagonal straight steel pipes, lateral bracings and adjustable bolts. In fact, there was a small gap (5 mm) between model and anti-roll devices to see if the model has lateral displacement just shown in Fig. 4.

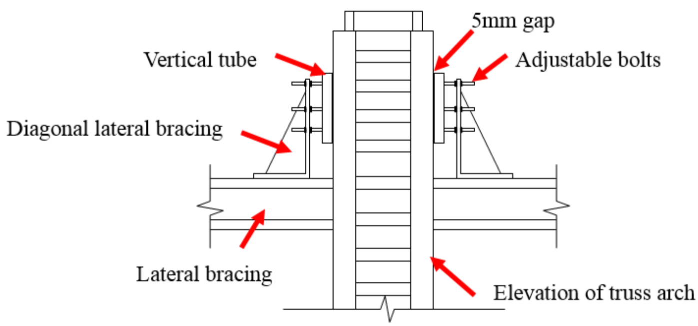


Fig. 4. *Anti-laterally overturning device.*

Step loading method was used during the loading process wiht 180 to 300 secends per step. After the observed data was of stabilization, the data of strain gages, displacement meters and dial indicators were acquired immediately. In addition, experimental phenomena was observed during the test. When the arch was of elastic loading period, the step loading is 5kN per level. While the arch reached elastic-plastic, it changed to 3 kN per level. After 75% of ultimate load value, it changed to 2 kN per level, and then continuing finally to failure. During the tests, monitoring and comparison of curves of loading and displacement with FEM were carried out.

4. ANALYSES OF TEST RESULTS

4.1. Analysis of fully loading process

Deformation of CFST truss arch was small at the preliminary stage of loading. A certain amount of deformation was observed along with the increase of load, and the shape of structure was similar to “M”, which is called “reversed arch”. Points of inflection located at around 1L/3 and 2L/3 spans. However, “reversed arch” was not obvious as the loading is not too big.

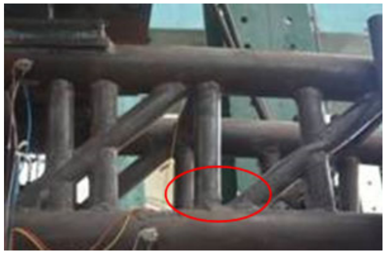
The arch structure remained in the elastic condition when the load was less than 250 kN, after that it changed into elastic-plastic condition. When the load reached 410 kN (about 80% of ultimate load), there was an obviously downward displacement at mid-span of the truss arch. The appearance of “reversed arch” were distinct when the load was 470 kN. In addition, the end of vertical web members of internodes No.9 and No.17 (near 1L/3 and 2L/3) buckled firstly and had a plastic deformation.

When the load reached 490 kN (about 96% of ultimate load), the web members of internodes of No. 13 and No. 14 close to the mid-span loading pad bulged slightly, which is shown in Fig. 4a. However, the deformation of top and bottom chord were of accordance, there were no local buckling or punching shear failure, which means the CFST truss arch has a good integrity.

When the load reached 500 kN, the straight web members of four internodes (No. 8, No. 10, No. 16 and No. 18) bulged as well, which shows that the plastic area will be developed along with arch axial direction and will induce re-distribution of internal force. In addition, the skewbacks didn’t observe the slippage during the loading process, which means the skewbacks are fixed. By the way, the gap between the model and anti-lateral device still exists.

When the load reached 510 kN (maximum value), the end of vertical web member buckled distinctly and the arch structure cannot steadily bear the load and begin to decrease. After continue to loading, the bearing capacity declined slightly to 496 kN, but the deformation increased significantly. The vertical web members of four internodes near the loading place, the sections of 1L/3 and 2L/3 buckled and bulged remarkably. While continuing to load, the joints of diagonal web members at mid-span, 1L/3 and 2L/3 were crack, just as shown in Fig. 4b and Fig. 4c. Finally, many web members were out of working and chords were of instable failure under compression and bending.

From this experiment, it can be seen that the arch structure works as ideal in-plane loading because of the puny out-plane’s displacement. There was no welding node failure before the load reached ultimate load, which illustrates the structure shows a good integrity. The end of vertical web members will be buckled and bulged firstly, and then the node of diagonal web members will be cracked, which shows some mechanical behavior of truss-girder for the CFST truss arch. However, none of the chord is cracked and failed instability, and the CFST truss arch has good ductility property. In another word, its performance also has a little similar to the CFST laced column [16-18].



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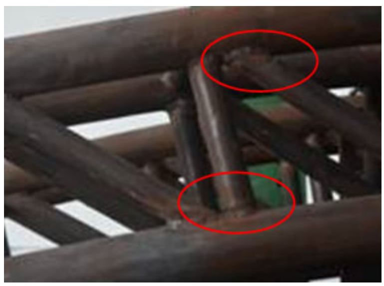
b)



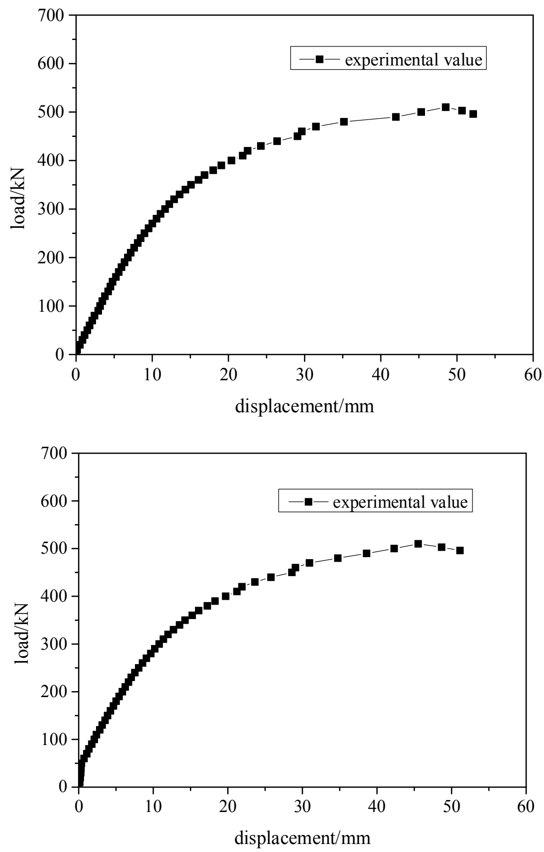
c)

Fig. 4. *Failure photos of web braces of A-0:*
a) straight web members near mid-span bulged at ends,
b) diagonal web members near mid-span cracked at ends,
c) bulge and crack at end of straight and diagonal web member near 1L/3.

4.2. Load displacement curves

Load-average vertical displacement curves of mid-span section and quarters span are shown in Fig. 5 and Fig. 6, respectively. Fig. 5a and Fig. 6a are the average values of bottom chords while Fig. 5b and Fig. 6b are the average values of top chords, respectively.

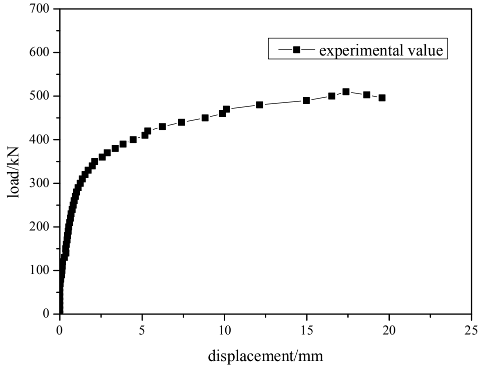
Figure 5 and 6 indicate that the relationships between load and displacement exhibit a linear variation when the load is lower than 250 kN. Slope of curves will decrease continually along with the increase of loading, then the relationship between loading and displacement exhibits nonlinearity, which means the truss arch enters into elastic-plastic state. When the load reaches 410 kN, displacement increases rapidly as the load increases. As the maximum load is 510 kN, measured data show the maximum average vertical displacement at mid-span are 51.1 mm (top chords) and 52.1mm (bottom chords) at this moment, which reaches about 2.89% of rise. It can be seen that vertical displacement at mid-span of top chords is lower than that of bottom chords of 1.0 mm. Nevertheless, vertical displacement at quarter-span of top chords is larger than that of bottom chords of 1.0 mm, as 20.6 mm and 19.6 mm respectively.



a) b)

Fig. 5. *Load-average vertical displacement curves at mid-span section of A-0 arch:*
a) bottom chords at mid-span,
b) top chords at mid-spa.

The difference of deformation between top and bottom chords can be seen as relative deformation of web members, in another word, this deformation under vertical load is mainly causes for the buckling of the ends of the web members. In addition, the value of deformation of mid-span is same as quarter-span (1/4L), but in the different direction. The compressive deformation of vertical web member, is about 1mm (= bottom chords - top chords), and its length is 220 mm, thus the micro strain of steel is $(1/220)\times10^6$, which is far larger than the yield strain of steel. Therefore, the web members of these sections mentioned above should be buckled and bulged.



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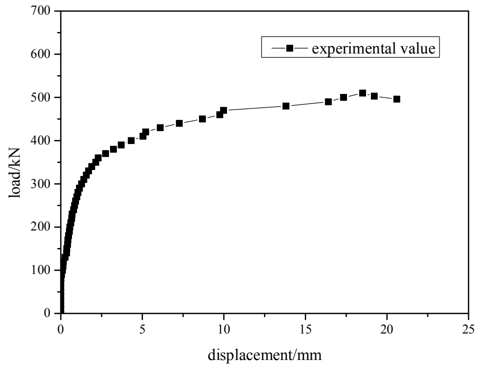

b)

Fig. 6. *Load-average vertical displacement curves at quarter span section of A-0 arch:*
a) bottom chord at 1/4 sections,
b) top chord at 1/4 sections.

Curves of vertical deformation along the span direction under the corresponding load are shown in Fig. 7. It can be seen from the figure 7 that downward displacement of mid-span is lower than 10.0mm when the load is about 250kN(49% of ultimate load) and the displacement of span of 1L/6, 1L/4, 1L/3, 2L/3, 5L/6 and 3L/4 almost can not be observed. With the load larger than 250 kN, arch structure enters into nonlinear state. Downward displacement of mid-span is about 17.1 mm and increases of deformation of left springing to 1L/3 and right springing to 2L/3 with the load reaching 410 kN. There is obvious upward deformation exist in left springing to 1L/3 and right springing to 2L/3, which is 10mm (maximum) with a 460 kN. After continue loading, "M" shape can be seen distinctly, just as shown in Fig. 7. In general, the deformation of spans of 1L/3 and 2L/3 is nearly zero (highlight in Fig. 7 using red-line cycles) during the whole loading process, only reaches 2.4mm at 2L/3 and 1.2mm at 1L/3 even at ultimate loading. The maximum downward displacement of CFST truss arch appears at vault while the maximum upward displacement appears at quartile sections. Fig. 8 shows the photo of failure mode of model.

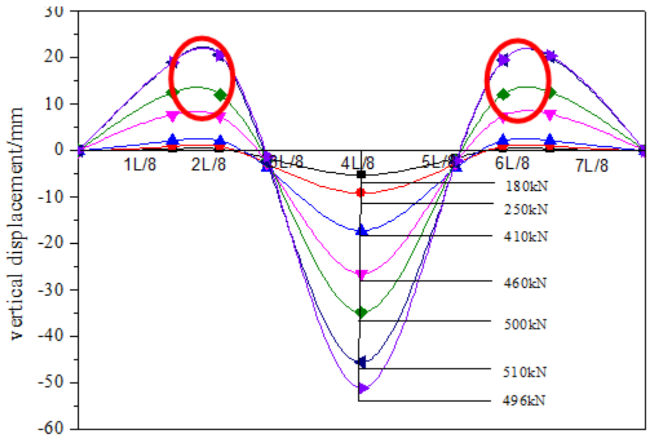

Fig. 7. Load-vertical deformation curves of A-0.

Fig. 8. Photo of failure mode of model.

5. COMPARISONS ON FAILURE MODE

5.1. Comparison of failure mode on CFST truss arch and truss girder

Based on experimental test on flexural property of CFST truss girder in [13], diagonal web members are of shear and tension failure under the tension and compression of top and bottom chords when the arch is loaded vertically at mid-span. Meanwhile, bottom chords are of tensile breakage. However, vertical web members are of low stress.

After comparison, it is found that nodes failure happens in CFST truss arch because of the shear and tensile failure of diagonal web members, which causes the arch structure losing bearing capacity. Thus, failure mode of CFST truss arch is similar with that of CFST truss girder.

However, there are some distinctly differences between CFST truss arch and CFST truss girder. For truss girder is bending structure with a large bending moment at mid-span. To balance this bending moment, top and bottom chords of truss girder are of large compressive and tensile stress, which cause the shear and shear failure of diagonal web members at nodes. For the truss arch structure, the failure of diagonal web members happens at descending branch far beyond the deformation corresponding to the ultimate bearing capacity due to continuing loading.

Meanwhile, truss arch is mainly of compressive stress, with a low bending moment the whole arch, which can bear much more vertical load than truss girder. The vertical web members are of large vertical load, which cause a compressive buckling of vertical web members before the shear failure of diagonal web members.

In addition, in comparison with hollow steel pipe truss girder, the filled concrete in the chords of CFST truss arch can prevent the tube from pitting damage (concave) caused by the loaded of vertical web members. In consideration of vertical web members of CFST truss arch would be taken up a large vertical force, the quality of concrete at nodes of chords should be paid attention in practical engineering.

5.2. Comparison of failure mode on CFST truss arch and latticed columns

Failure mode of CFST latticed columns is generally of strength and compressive bending instable failure (integral failure), which is similar with the failure mode of CFST truss arch. However, truss arch structure will be of local buckling (the end of web members) firstly and then integral failure after tensile crack of diagonal web members and bulge of vertical web members, which is not similar with latticed columns.

Another difference, as for building structure, bearing mode of CFST latticed columns is of compressive bending and web members are of low stress, layout of web members has little influence on ultimate bearing capacity of CFST latticed columns [6]. In generally, web members of CFST latticed columns are designed according to structural confirmation and measurements. However, for the CFST truss arch, web members need to bear a large load (like vertical force by suspender), and the web members are the important mechanical member. Thus, in practical engineering, stress of web members should be calculated and checked besides design of structural confirmation and measurements.

6. CONCLUSIONS

(1) There is a distinctly different failure mode between CFST truss arch structure and CFST latticed columns as well as CFST truss girder. The web members of internodes of CFST truss arch structure will bear large vertical load, which will induce a plastic deformation and buckling. The plastic area will be developed along span (arch axial) direction and more plastic deformation of vertical web members will be appeared, which will induce redistribution of internal force. Local buckling and bulge will happen eventually at ends of vertical web members from arch crown to quarter spans as for the arch belongs to truss conformation at the present of ultimate load. After that, the ends of diagonal web members are of shear failure and show the mechanical characteristic of truss girder. Finally, chords are of compressive bending instable failure, with an “M” shape of arch structure.

(2) The chords of CFST truss arch are mainly born by compressive stress except quarter sections (low tensile stress) during the loading process. Web members of CFST truss arch are of failure before the chords, and vertical web members are of buckling before

the diagonal web members, which show the mechanical characteristic of truss. Relative to the compressive buckling of vertical web members, failure of diagonal web members is shear and tension at nodes. Nevertheless, the chords and welding joints of CFST arch rib do not crack during the loading process, and the areas of intersecting junction do not fail. The arch has a favorable performance of strength and ductility.

(3) The calculation of CFST truss arch is different to single arch because the local buckling of web members will weaken the full play of its ultimate bearing capacity. Thus, influence of local buckling on integral ultimate bearing capacity of truss arch rib should be considered when using equivalent beam-column method.

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