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# Concrete–filled steel tube truss girders—a state-of-the-art review

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## Abstract

Steel–concrete composite construction is practised worldwide due to the strength and stiffness achieved with minimum use of materials. Among the various forms of steel–concrete composite construction in practice, concrete–filled tube (CFT) and specifically concrete–filled steel tube (CFST) is a well-received system. Concrete–filled steel tube (CFST) system comprises of an infill concrete core that withstands loading and prevents local buckling and steel at the outer circumference to effectively resist bending. Owing to the benefits of CFST in buildings viz., strength, ductility, constructability, and aesthetics, its usage has been extended to bridges in recent years. One such adaptation is CFST truss girders, where the concrete infill increases the compressive strength of the top chord and the tensile strength of the bottom chord preventing buckling and pinching action respectively. Since design methods for CFST composite girders are not yet established in detail, the use of CFSTs in girder bridge systems is limited. So far, the responses of this system have been studied under static and dynamic loading both experimentally and analytically. This paper collectively presents the behavior of such CFST truss girders with special emphasis on truss bridge girders based on the extensive research activities carried out in the last two decades. Potential areas of research in the future are also identified and summarized.

**Keywords:** Steel-concrete composite construction, Concrete–filled steel tube, Truss girders, Concrete–filled steel tube truss

## Introduction

Steel has high tensile strength, ductility, strength-to-weight ratio, and significant load carrying capacity but possesses low resistance to fire and buckling, both lateral and local. On the other hand, concrete under compression exhibits good strength but with time it is likely to undergo shrinkage and creep. When steel plates are combined with concrete, the resistance against buckling increases which has made steel-concrete composite structures competitive against their concrete counterparts. One principal form of steel–concrete composite structures is the concrete-filled steel tube (CFST) system. It is acknowledged that the compressive strength of concrete surpasses its tensile strength. Besides, this compressive strength is further enhanced under bi-axial or tri-axial restraint. In CFSTs, steel and concrete are used in a way that their natural and most prominent characteristics are taken full advantage of. The concrete infill also prevents or delays local buckling in the hollow steel tube and the steel encasing provides the necessary tri-axial restraint and adds strength, stiffness and ductility. It also eliminates the need for shuttering or formwork during construction which reduces the construction cost and time. The steel tubes are factory-made which improves their quality. Apart from being structurally efficient, these girders minimize construction wastage and time. These advantages have been widely exploited and have led to a great deal of research and the use of CFST in civil engineering structures.

From previous works of literature, it is noted that the circular cross-section provides better confinement to the concrete core, whereas local buckling and improper bonding are more likely to occur in square or rectangular cross-sections. However, the CFSTs with square and rectangular steel tubes are still used in construction as they facilitate easier beam-column or any member-to-member connections, have high cross-sectional bending stiffness and have better aesthetics. Other cross-sectional shapes such as polygons, round-ended rectangles, and ellipses have also been used for aesthetical purposes. In addition, concrete-filled double skin tubes (CFDST), concrete encased CFST, CFSTs with additional reinforcement and CFSTs stiffened with longitudinal stiffeners have also been used based on the requirement [1–5]. India, America, Europe, Japan, China and Australia have developed their own codes and specifications to address the design of steel-concrete composite members viz., IS: 11384–1985, IRC:22–2015, ANSI/AISC 360–16, EC4–1994, JGJ 138–2016, JSCE 2007. Specifically, DBJ/T13-51-2010 and AIJ guide–2008 cater to the needs of CFST structures. Apart from the design codes, industrial and local specifications regarding the CFST structures are also available. However, China is the pioneer in using such CFSTs for bridges. Over the last two decades, Chinese researchers and practicing engineers have made efforts to include CFSTs in arch, suspension, cable-stayed, and truss bridges.

An effort has been made in this paper to provide a detailed review of the application of CFST trusses and their role in bridges. The scope of this work is limited to CFST truss girders studied both experimentally and analytically under static loading. Effect of dynamic loading on CFST truss girders for, e.g., vehicular loading is not reviewed as only superficial research has been carried out so far. Use of CFST as bridge piers, ribs in arch bridges and towers in cable-stayed bridges is beyond the scope of this paper.

## Behavior of CFST truss girders

### *General*

A truss also referred to as an open web girder is a system with triangularly interconnected structural elements that are pin connected at the nodes. Trusses are efficient when used in long-span structures as they utilize minimal material when compared to the solid web. It reduces the self-weight of the structure without compromising on its structural rigidity and is economical to use. Angle sections, channel sections and hollow tube sections are conventionally used as truss members. Among these, the steel pipe structure has good geometrical characteristics. The even distribution of cross-section material around the centroid, and the large radius of gyration provide good torsional stiffness. But the limited load carrying capacity of the hollow pipe girders paved way for their replacement with concrete-filled steel tubular sections [4].

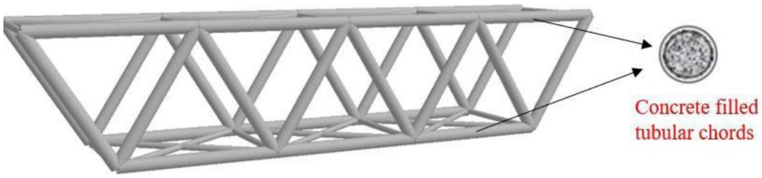

**Fig. 1** Diagrammatic representation of a CFST truss girder

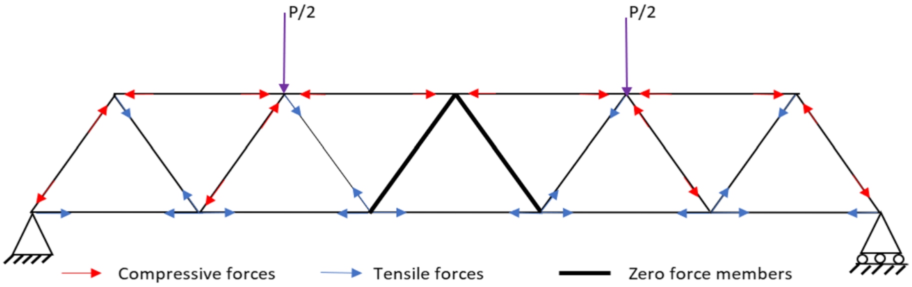

**Fig. 2** Force distribution in a typical truss girder

CFST trusses (shown in Fig. 1), usually consist of concrete-filled chords and hollow steel tube braces. They serve as the main load-bearing system in a truss girder bridge and hence, filling the compressive chords with concrete is an ideal solution to improve its axial strength taking into account the advantage of confinement effects. A bridge girder under dynamic load will be subjected to stress reversal. So, it is safe to infill concrete in both the compressive and tension chords. The bending and shear generated under loading are transferred as axial forces of chords and axial force of webs respectively [5] as shown in Fig. 2. The concrete infill increases the compressive strength of the top chord preventing it from buckling and at the same time improves the tensile strength of the bottom chord and strengthens it against pinching action, i.e., inward contraction due to Poisson's effect.

The CFST truss girder is a flexural member subjected to bending which is effectively resisted by the top and bottom chord members. Compared with hollow tubular trusses, both the stiffness and ductility are improved significantly for the CFST trusses because they are dominated by the tensile yielding of the bottom chord, whereas hollow tubular trusses fail by local buckling or joint failure before the full flexural strength is reached [6]. Warren and warren-vertical are the broadly analyzed CFST truss configurations. Warren truss girders are preferred to their counterparts since they have a simple configuration and higher flexural rigidity. Such CFST warren-vertical truss girders were flexure dominated when the brace-to-chord strength ratio ($\kappa$) is greater than or equal to 0.8 and the shear span-to-depth ratio is greater than or equal to 4.8 [6, 7].

The load vs mid-span deflection of all the tested CFST specimens exhibited 3 stages: elastic, yield, and strain hardening stages. The stiffness in the initial elastic stage ($K_e$) involves the contribution of both the steel and core concrete and hence is usually higher. During the yield stage, the decrease is followed by a slight increase in plastic stiffness ($K_p$) and strength in the hardening stage. However, by the time the specimen reaches $K_p$, its deflection crosses its serviceability limit.

Finite element analysis of CFST truss models involves modelling of materials, establishing confinement of concrete infill and contact interaction between the steel tube and concrete infill followed by loading and boundary conditions. For FE modelling, ABAQUS software is widely used. Brace-to-chord strength ratio, shear span-to-depth ratio, compressive strength of infill concrete and provision of RC slab over the truss have been varied to study their influence on the failure modes, load carrying capacities, overall deflections, stress and strain distributions. Both material and geometric non-linearities are taken into account to generate the finite element models. Accuracy of results and computational time are two key factors that need to be balanced. To achieve this the element type and mesh size of the infill should be carefully determined by convergence studies. A mesh convergence study is conducted to determine the proper dimension of meshes for both chords and braces. Both 3D solid shells and simplified 2D models can be used to study the behaviour of CFST trusses. Though a detailed 3D model can shed light on multiple factors like steel tube yielding, local buckling, fracture, concrete confinement, geometric imperfections and residual stresses due to welding, developing such models demands more time and effort [7, 8].

To incorporate material behavior, a five-stage stress–strain model for steel as proposed by Han et al. [8] is generally adopted. This model, considers the deformation of steel in elastic, elastic–plastic, plastic, hardening and fracture stages. In a CFST truss girder bridge deck, concrete in the slab is unconfined whereas the concrete in the tube is confined. For confined concrete the constitutive model proposed by Han et al. [8] and for unconfined concrete the stress-strain relationship as given in design codes are utilized. Particularly in ABAQUS concrete damaged plasticity (CDP) model is used to define the material behavior of concrete. Careful selection of concrete model should be made especially for hybrid trusses as the concrete inside the tubular members will be under passive confinement with improved strength and ductility whereas the concrete slab is unconfined. The interaction between the steel tube and concrete infill is assumed to be fully bonded [9]. Friction coefficient is the ratio of frictional force resisting movement to the normal force acting on the element. This is adopted to define the frictional behavior between the steel tube and concrete. A friction coefficient of zero indicates no contact between steel and concrete and fully bonded indicates the interface between the steel and concrete are never separated under load. As per coulomb's friction model adopted, the coefficient of friction for dry friction is assumed as 0.6 [10–12]. Also, in the work done by Baltay and Gjelsvik [13], the authors have suggested that the friction coefficient varies between 0.3 to 0.6 for concrete and mild steel interface. The connection in tube joints influences the strength and behavior of CFST trusses and tie constraint in ABAQUS is used to establish interaction between the chords and braces. Their behavior is studied under 3 connection types: hinge, semi-rigid, and rigid and is found that the elastoplastic stiffness of a truss structure using the hinge connection is greater than that of a truss using a rigid connection. Boundary and loading conditions are set similarly to the experimental setup. The models are hinge supported at one end and roller supported at the other i.e., restrained in all directions except for in-plane rotation and unrestricted displacement and rotation in all directions respectively [7, 10, 14, 15].

Various geometric and material parameters play a pivotal role in the behavior of CFST trusses and some of the important parameters are discussed below.

## Factors influencing the behavior of CFST trusses

### *Effect of geometrical properties of truss members*

AISC, EC3, and EC4 classify steel and composite sections based on slenderness ratio (D/t), to take account of the effect of local bucking. As a result, the cross-sectional dimensions of chord members are of much importance in understanding the flexural behavior of the tubular truss [6]. The bending moments in CFST trusses are primarily carried by the top and bottom chords. These chords also sustain secondary bending moments transmitted from the joints. However, the stresses and strains due to the secondary bending moments are negligible as compared to those produced by the overall truss bending [16]. The load carrying capacity increases with the increase of cross-section dimensions of the bottom chord. It was also observed that the tensile strain developed in the chords was much higher than that of the compressive strain, implying the importance of the strength of the bottom chord members. But there is an upper bound for the cross-section dimensions of the bottom chord, after which the flexural behavior of the CFSTs is determined by the strength of the braces. The optimum bottom chord to top chord ratio was recommended to be 1.5 [6, 7]. Confinement factor ($\xi$), which depends upon the cross-sectional area and material strengths is a measure of the flexural strength of CFST. The application of this factor in predicting flexural strength is also pivotal and can be found in studies conducted by Han et al. [6, 10, 17].

The bottom chord of the truss that is under tension can also be strengthened by adding steel reinforcement bars. But as only a slight increase in flexural strength was evident, increasing the wall thickness of the steel tube to enhance the tensile property of the lower chord is recommended alternatively. Increasing the diameter and wall thickness for the first and last braces alone, to avoid local buckling is also recommended because these braces transfer extra forces from supports [18]. The brace-to-chord strength ratio measures the influence of chord diameter ($D$), chord thickness ($t$), and brace diameter ($D_b$) on the strength of the truss girder. Strength decreases as the chord diameter and chord wall thickness decrease and the strength increases as the brace diameter increases [14]. For instance, for each 50 mm increment in chord diameter, 1mm increment in thickness of chord wall and 100 mm increment in height of truss, 6%, 9%, and 3.5% increases in load carrying capacities, respectively are observed in previous studies [19]. Curved Concrete filled steel tube (CCFST) truss members were also studied by Xu et al. The arch rise to span ratio becomes an important parameter affecting the strength of the girder. The flexural stiffness of a CCFST truss was found to be 3.5 times greater than hollow tubular curved truss. Care must be taken for design of joints in CCFST trusses as the failure of joints with a sudden loss of strength was evident [12, 15].

Although concrete-filled circular hollow sections (CFCHS) are popular in truss bridges due to the higher confinement efficiency, concrete-filled rectangular hollow sections (CFRHS) are also adopted as they are easy to stack, handle and weld on site. It is also easy to attach a deck slab on top of trusses with a rectangular top chord for composite sections. While RHS and CHS trusses have similar load carrying capacity and deformation, CFCHS have higher capacity and better deformability than CFRHS.

Therefore, to improve the performance of CFRHS sections, stiffening with Perfobond Leister (PBL) stiffeners is proposed. This very effectively confines the bending deflection of the chord and improves the rigidity and fatigue resistance of truss joints. In such a girder, the infilled concrete improves the truss strength under compression and the PBL under tension [20–23].

### *Influence of infill concrete on failure modes*

The failure modes of a CHS truss are chord face failure, chord sidewall failure under the compression brace member, chord shear failure, punching shear failure, brace failure and local buckling failure. Joints with higher resistance fail via punching shear whereas joints with reduced resistance fail via chord face failure [16]. Filling chords with concrete modifies joint failure mode. CFT truss girders usually fail by tensile fracture of the bottom chord or joint failure. These failure modes are determined from the condition if either buckling of compressive braces or yielding of the braces occurs before the maximum tensile strain in the bottom chord wall reaches 0.003. If the condition satisfied joint failure governs, otherwise, tensile fracture of the bottom chord governs the failure mode [10]. But increasing the concrete compressive strength ($f_{\mathrm{cu}}$) in the bottom and top chords has null to negligible influence on the strength of CFT trusses. Partially filling the chords may be effective only for long span trusses as it reduces the dead weight considerably but does not have much influence on short span trusses [14, 24].

### *Influence of shear span*

Shear span is the distance between the point of application of load and its reaction force. The shear force is constant throughout a single shear span. The shear span is equal to span/2 for a 3-point loading and span/3 for 4-point loading. Trusses with smaller shear spans exhibited shear failure. Under pure bending, trusses with higher shear spans had more closely spaced loading points and thus failed in a ductile manner. The shear span-to-depth ratio (a/H) is altered by changing the truss height (H) or the span length (L). It is observed that the stiffness and lateral load capacity of the truss increase with increasing truss height or decreasing span length. Therefore, the lateral load carrying capacity decreases as the shear span-to-depth ratio (a/H) increases [6, 14].

### *Influence of concrete deck slab*

For CFST trusses with concrete deck slabs, both the infill concrete and concrete slab improve the flexural performance notably. Traditionally, two methods are followed to predict the flexural strength of composite beams, the plastic stress distribution method (referred to as the plastic method) and the strain compatibility method. Unlike the second method which requires complicated calculations, the plastic method is both simple and gives reasonably accurate results. The plastic method recommended by ANSI/AISC 360 and EC4 for composite beams has been previously adopted to study the flexural behavior theoretically [6, 10]. There is a lack of studies on CFST composite truss beams with concrete slabs in the negative moment region. Forms of concrete slabs and shear studs are found to have little influence on the ultimate bearing capacity of composite truss beams, but they significantly affect the structural stiffness, structural ductility, failure mode and cracking resistance of concrete slabs of composite truss beams [20].

Conventional shear studs, perfobond strips of different forms, channel-shaped connectors, slip releasing studs and even micro-expansive concrete have been used so far to create a bond between the steel truss and concrete slab. However, slip at the slab-girder interface is prevalent as load increases. A maximum slip was observed between the midspan and the boundary support because the concentrated force in this region increased the interfacial friction. In addition, the relative slip of the specimens using slip releasing shear studs was significantly larger than that of the specimens with conventional shear studs, indicating that the slip-releasing shear studs reduce the degree of the shear connection between the upper truss chord and deck slab [20]. The spacing between the shear connectors also influences the slip resistance. When the shear connectors are widely spaced, debonding at the interface is evident. Such CFST girders with concrete slab were termed as 'Hybrid CFST trusses' by Han et al. and were found to be well restrained from local buckling as the top chord was embedded in the concrete slab. Such girders exhibited good ductility. Vertical cracks are initiated in the midspan of the slab followed by inclined cracks in the shear span. FE study showed that the hybrid CFST trusses had 16–28% higher flexural strength than their counterparts [6, 10].

### *Behavior of stainless-steel trusses*

Stainless steel is the only alternative used to mild steel in CFST trusses so far. It is a highly versatile material with less weight but superior strength, ductility and corrosion resistance. Due to these benefits, concrete-filled stainless steel stub columns have been studied in detail [25, 26]. Taking inspiration from these studies, concrete-filled stainless-steel tubular (CFSST) trusses were developed and studied. The location of concrete infill in trusses viz., infilled top chord, infilled bottom chord, infilled top and bottom chords, and hollow chords, has a significant effect on the flexural rigidities of CFSST trusses, especially in the elastic stage. The specimens with both the top and bottom chord filled show the best ductility and flexural rigidity followed by specimens with either top or bottom chord members filled. The typical failure modes of CFSST trusses are slightly different from CFSTs. The surface plasticity or punching shear of the chord members was the main cause of CFSST failures. Specimens with only the top chord filled and both top and bottom chord filled fail by bending of the top chord and the weld fracture. Whereas surface plasticity of the top chord members is observed in hollow stainless steel tubular trusses and those filled only in the bottom chord [27].

### *Behavior of multiplanar trusses*

The truss girders studied are mostly multiplanar (3D) tubular trusses that can be designed with the help of guidelines given in the CIDECT code and Chinese code GB50017-2003. Apart from the cross-section dimensions of brace and chord members and infilled concrete strengths, the strength and behavior of concrete-filled multi-planar inverse-triangular CHS truss are mainly influenced by the truss span-to-height ratio (L/H), truss span to joint spacing ratio (L/S) and truss height to width ratio (H/W) [7]. The 3D Inverse-Triangular truss is the most ductile, while the 3D square truss and the 3D trapezoid truss have lower ductility comparatively, followed by the least ductile 3D triangular truss [28].

Though the failure modes of concrete-filled multiplanar tubular trusses are classified broadly as an overall flexural failure and local shear failure, they are further sub categorized by Feng et al. [7] into plastification of diagonal braces at truss end (PDBTE), the plastification of the bottom chord, connecting braces and joints at truss midspan (PCBJTM), and the local buckling of straight braces at truss end (LBSBTE). The failure mode PDBTE usually occurs for specimens with a small span-to-height ratio (L/H) and small span-to-joint spacing ratio (L/S). This results in larger bending rigidity at the mid-span and smaller bending rigidity at the ends. Trusses subjected to PDBTE failure possess reduced load carrying capacity. PCBJTM failure usually occurs for specimens with a large truss span-to-height ratio (L/H) and large truss span-to-joint spacing ratio (L/S), which results in the uniform distribution of overall bending rigidity. The concrete infill in the top chord enhances the load carrying capacity of top chords and connecting joints, which makes the empty bottom chord and braces at the mid-span of the truss to fail by plastification. LBSBTE usually occurs for specimens with a small truss span-to-height ratio (L/H) and small truss span-to-joint spacing ratio (L/S) as well as a small truss height-to-width ratio (H/W), which results in the larger overall bending rigidity [7]. By comparing the failure modes of multiplanar trusses of different configurations it is seen that triangular trusses fail by the end support failure of the top chord member and the surface plasticity of the bottom chord member. On the other hand, square and trapezoidal multiplanar truss fails by the local buckling of vertical braces, surface plasticity and shear failure of the bottom chord and weld fracture around tubular joints at the bottom chord. Regardless of the different types of failures, practically overall flexural failure should be targeted rather than premature local shear failure [28]. The various modes of failure observed in Hybrid CFST trusses and CFST and CFSST truss members are shown in Fig. 3.

## Future scope

Detailed studies on the experimental and finite element analysis of CFST trusses carried out worldwide on CFST trusses in bridges are summarized in Table 1. The gap found in the research is classified under 3 domains:

(a) Material: Effects of cold-formed steel tube and stainless-steel tube trusses have been studied by a few researchers. A number of literatures are available on the compressive behavior of CFSST columns under static loading and impact loading. However, very little research is found on the behavior of CFSST truss girders under pure bending. Though stainless steel has high ductility, durability, fatigue, and fire resistance its application is limited in long-span structures due to its high initial material and fabrication cost. Welding of joints in stainless steel tubes also requires careful supervision as the material can retain more heat and distorts or warps leaving a non-aesthetic finish. Using life-cyclic cost analysis will encourage the use of CFSSTs in the future [25, 26]. Similarly, researchers face challenges using cold-formed steel sections because of their high cost as well as the difficulty in connecting them. Unlike hot rolled steel sections where shear studs are easily welded to the top of the girders, cold-formed steel requires pre-drilled holes or the use of other connectors like channel sections. Studies on the flexural behavior of lightweight concrete-filled steel

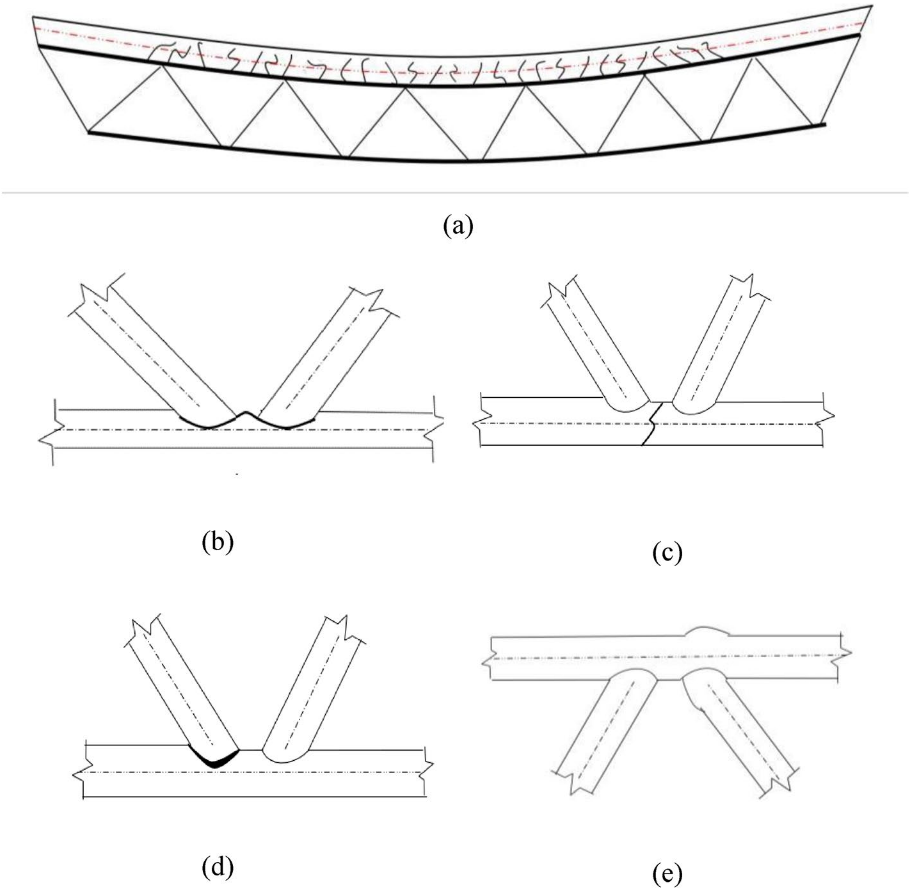

**Fig. 3** Common failure modes. **a** Crack distribution in a CFST truss girder deck. **b** Surface plasticity. **c** Chord fracture. **d** Weld fracture. **e** Local buckling of compression members

tubes (LWCFST) and truss girders and self-compacting concrete-filled steel tubes (SCCFST) and truss girders are scarce. Few studies compared the performance of LWCFST or SCCFST members relative to that of normal weight CFST members. However, further research is needed to fill up the gaps in this field [29, 30].

(b) Geometry: Bridge decks with CFST truss girders analyzed so far have straight geometry. This may not be the case practically. Practical space constraints demand the use of curved in-plane or skewed bridge decks. The load distribution of such decks is much more complicated and needs to be analyzed, e.g., Li et al. [31] studied the famous Ganhaizi bridge deck which is curved in its plane. Due to the unique bend and twist mechanism and development of centrifugal forces, curved bridges usually have more complex dynamic features than straight ones under moving vehicles. Skewed bridge decks with any form of CFST are not studied to the best of the authors' knowledge. The knowledge of the dynamic behavior of the curved CFST bridge subjected to moving vehicles is also limited. Such studies must be taken up in the future to improve the versatility of CFST in bridges. Similarly, research on non-prismatic CFST sections like inclined sections and tapered sections is also scarce.

**Table 1** Summary of detailed studies on CFST truss girders

| Sno | Author, year, country | Model configuration | Model dimension (m) | Materials used | Parameters varied | Response studied | Structural codes referred | Software used | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Fong et al.,2011, China * MERGEFORMAT [24] * MERGEFORMAT [35] * MERGEFORMAT [36] * MERGEFORMAT [37] * MERGEFORMAT [38] | Triangular CHS and CFST truss with square tubes | L = 2 H = 1 | *Outer tube:* Carbon steel *Infill:* Normal concrete | Presence of concrete infill | Load – deflection behaviour, Comparison between test result and codal provisions | EC 3, EC4, CoPHK, AS5100 | NIDA | CFST member resistances are predicted using conventional effective length and 2nd order analysis methods. |
| 2 | Wang et al. 2012, China [39] | Inverted triangular truss with circular tubes | L = 5.6<br>H = 0.59 | *Outer tube:* Carbon steel *Infill:* Self Consolidating Concrete | None | Failure mode, Load deflection relation, Strain distribution of: composite beam, reinforcement in slab and steel truss | Not specified | NA[a] | Under monotonic loading, the specimens showed good ductility and torsional resistance. |
| 3 | Xu et al. 2013, China [12] | Warren truss with circular and curved CFST | L = 4.8 | *Outer tube:* Carbon steel *Infill:* Normal concrete | Rise to span ratio, Type of chord | Load vs midspan deflection, Stress and strain distribution | Not specified | ABAQUS | FE analysis of curved CFST taking into account geometric and material non-linearity. |
| 4 | Chen et al. 2015, China [28] * MERGEFORMAT [40, 41] | Pratt Triangular, Inverse Triangle, Square and Trapezoid trusses with circular tubes | L = 3.13<br>Leff = 3<br>W = H = 0.4 | *Outer tube:* Carbon steel *Infill:* Normal concrete | Truss configurations | Failure mode, Load carrying capacity, Flexural rigidity, Overall deflection, Strain intensity | CIDECT, GB50017-2003 | NA[a] | Comparison among the 4 truss shapes showed, inverse triangular truss has optimum rigidity, ductility and efficiency. |
| 5 | Feng et al. 2015, China [7] | Inverse triangular Pratt truss with circular tubes | L = 3<br>W = H = 0.4 | *Outer tube:* Carbon steel *Infill:* normal concrete | Presence of concrete filling | Truss span to height, truss span to joint spacing and truss height to width ratios | Not specified | ANSYS | Extensive FE analysis with 256 models to predict economical configuration. |

**Table 1** (continued)

| Sno | Author, year, country | Model configuration | Model dimension (m) | Materials used | Parameters varied | Response studied | Structural codes referred | Software used | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| 6 | Han et al. 2015, China[6, 36, 40, 42] | Warren truss with circular tubes | L = 0.5<br>H = 0.375<br>W = 0.432 | *Outer tube:* Carbon steel *Infill:* self Consolidating concrete | Shear span ratio, Angle between brace and chord, Profile of bottom chord, presence of concrete infill and deck slab | Failure mode, flexural behaviour, strain distribution | CIDECT, ANSI/AISC 360-10, EC4 | NA$^a$ | Based on existing codes, simplified models for flexural strength and stiffness of truss are suggested. |
| 7 | Zhou et al. 2017, China [27] | Warren truss with circular tubes | L = 3.3<br>Leff = 3<br>W = 0.1<br>H = 0.5 | *Outer tube:* Stainless steel *Infill:* normal concrete | Compressive strength of concrete and addition of reinforcement | Failure mode, load carrying capacity, load vs strain curves, overall deflection | Not specified | NA$^a$ | CFST top and bottom chords – Good flexural rigidity and load carrying capacity, CFST top chord – best ductility |
| 8 | Hou et al. 2017, China [10, 36, 40, 42] | Warren truss with circular tubes | L = 0.5<br>H = 0.375<br>W = 0.432 | *Outer tube:* Carbon steel *Infill:* self Consolidating concrete | Presence of deck slab, dimensions of truss members, compressive strength of concrete | Failure mode, load carrying capacity, overall deflection | CIDECT, ANSI/AISC 360-16, EC4 | ABAQUS | CFST chords with deck slab gives more ductile failure. Predicted, analytical and experimental flexural strengths were in good agreement |
| 9 | Huang 2018, China * MERGEFORMAT [16] * MERGEFORMAT [35] | Warren, Warren vertical, Pratt and N type truss | L = 2.88<br>H = 0.4 | *Outer tube:* carbon steel *Infill:* normal concrete | Different web arrangements, position of concrete infill | Failure mode, joint resistance | EC 3 | NA$^a$ | Joint resistance of truss evaluated using formulae for checking CHS joints as per EC 3, AWS D1.1 |
| 10 | Huang, 2018, China * MERGEFORMAT [14] * MERGEFORMAT [42] | Warren Vertical with circular tubes | L = 6<br>H = 0.5 | *Outer tube:* Carbon steel *Infill:* self consolidating concrete | Compressive strength of concrete, brace to chord ratio, shear span to depth ratio | Fundamental behavior, failure modes | ANSI/AISC 360-16 | ABAQUS | Based on experimental and FE analysis, design equations were proposed to estimate flexural strength of warren vertical truss girders. |

**Table 1** (continued)

| Sno | Author, year, country | Model configuration | Model dimension (m) | Materials used | Parameters varied | Response studied | Structural codes referred | Software used | Remarks |
|---|---|---|---|---|---|---|---|---|---|
| 11 | Li et al., 2019, China * MERGEFORMAT [31] * MERGEFORMAT [43] | 3D truss (similar to truss used in Ganhaizi bridge) with circular tubes | $L = 1044.7$<br>$W = 24.5$ | *Outer tube:* carbon steel<br>*Infill:* micro expansive concrete | Vehicular traffic | Dynamic behaviour characteristics and vehicular ride comfort | JTG D60-2015 | ANSYS | 3D vehicle and bridge coupled vibration analysis model for curved bridges is validated |
| 12 | Thejeel, Shallal, 2019, Iraq * MERGEFORMAT [30] | Warren vertical truss with square tubes | $L = 2.6$<br>$H = 0.5$ | *Outer tube:* carbon steel *Infill:* self consolidating concrete | Concrete compressive strength, usage of reinforcement in chord | Load-deflection at midspan, peak load, flexural strength, failure shapes | Not specified | NA[a] | Design equations are also used to predict flexural strength of CFST truss |
| 13 | Alneban, Shallal, 2020, Iraq[1] | Warren vertical truss with square tubes | $L = 2.66$<br>$H = 0.5$ | *Outer tube:* carbon steel *Infill:* self consolidating concrete | Compressive strength of concrete, presence of deck slab, location of concrete infill, shear connector spacing | Deflection,<br>Load capacity,<br>Failure mode, Slip between slab and CFST | Not specified | NA[a] | Experimental results showed location of concrete infill in the tube had significant effect on ultimate load and failure mode |
| 14 | Liu et al., 2020, China * MERGEFORMAT [20] | Warren truss with Perfobond stiffened rectangular tubes | $L = 6.6$<br>$W = 0.94$ | *Outer tube:* carbon steel *Infill:* self consolidating concrete | Different forms of concrete deck slab, types of shear connectors | Load-deflection response, Crack development,<br>Relative slip, Strain distribution | Not specified | NA[a] | Theoretical formulae derived to predict cracking moment, crackwidth under SLS and flexural capacity under ULS |
| 15 | Chen et al., 2020, China * MERGEFORMAT [17] * MERGEFORMAT [36] * MERGEFORMAT [42] * MERGEFORMAT [44] | Warren truss with circular tubes | $L = 0.5$<br>$H = 0.375$<br>$W = 0.432$ | *Outer tube:* carbon steel *Infill:* normal concrete | Cross-sectional confinement factor | Flexural strength evaluated with reliability analysis | ANSI/AISC 360-16, EC 4, JGJ/T D65-06 2015 | ABAQUS, MATLAB | Structural reliability of the model is further checked in MATLAB using First Order Reliability Method. |

[a] Only experimental studies

(c) Loading: Static loading is the commonly studied types of loads for CFST trusses. But there is a paucity in the study of such composite bridges against wind loading and their aerodynamic stability. When such girders are recommended for use in long-span bridges it is better to study their response to wind. Nakamura undertook aerodynamic stability studies on CFST girders in long span cable stayed bridges. Wind tunnel testing was also conducted and satisfactory results were obtained [32, 33]. For dynamic studies under vehicular loading, vehicle bridge coupled models were developed to study the local and global vehicle impacts and riding comfort. This research is limited to the global and local dynamic impacts and vehicle riding comfort under varying road surface conditions [31, 34]. This has to be extended to get a clear picture of the bridge deck's behavior under different vehicular loads and load combinations. Vehicular loading being the predominant dynamic load acting on the bridge girders, its effect is usually studied by varying the positions of the vehicular load across and along the span of the bridge in addition to considering the vehicle velocity, road surface roughness, curvature, and skew angles. Also, traffic loading varies from country to country and hence the response of CFST in bridge decks to such loads will enable the revision of design codes too. Mitigation measures under extreme loading conditions should be suggested and evaluated for their effectiveness.

Comparing the advancements made by China, Japan, and the USA in the research and implication of CFST in bridges, the rest of the world lies far behind in terms of codal provisions and practical research.

## Conclusions

Research works conducted globally on CFST in bridges under static loading show that the load carrying capacity of CFST members fared better than the summation of the capacities of individual steel and reinforced concrete members. Thus, the CFST structure can be treated as an alternative system to the steel or reinforced concrete system. Though the CFST structure offers several structural benefits such as favorable strength, fire resistance, ductility, and energy absorption capacities some questions on the feasibility of the CFST system should also be thoroughly evaluated for its broad application. Strong codal provisions highlighting the design requirements of CFST members are also a need of the hour.

### Abbreviations

| | |
|---|---|
| CFT | Concrete-filled tube |
| CFST | Concrete-filled steel tube |
| CFDST | Concrete-filled double skin tubes |
| CCFST | Curved concrete filled steel tube |
| CFRHS | Concrete-filled rectangular hollow sections |
| CFCHS | Concrete-filled circular hollow sections |
| PBL | Perfobond Leister |
| LWCFST | Lightweight concrete-filled steel tubes |
| CFSST | Concrete-filled stainless-steel tubular |

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## Declarations

### Competing interests

The authors declare that they have no competing interests.

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