Numerical Study on Load Carrying Capacity Evaluation of Concrete-filled Square Steel Tubular Beams

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Synopsis
A numerical study of concrete-filled square steel tubular cantilever beams under a monotonic loading has been conducted in order to apply the beam to the members of civil engineering structures effectively and also economically. As a result, the obtained load carrying capacities using a nonlinear three dimensional finite element method have agreed sufficiently with the capacities from an existing design equation.

KEYWORDS: Concrete-filled square steel tubular cantilever beams, Large Breadth-Thickness Ratio, Load carrying capacities evaluation

1. Introduction

Recently, a concrete-filled steel tubular member, called CFT hereafter, becomes noticeable because of their excellent load carrying capacity and also deformability, so that a lot of study on the member was carried out. However, most of the studies were used the square steel tube with a ratio of breadth to thickness: \( \frac{B}{t} \) below 70 as a building column. The value of the ratio is regarded as being small for an infrastructure member.

Furthermore, existing design specifications of the CFT members for infrastructures in Japan have prescribed that, shear action should be carried only by the steel tube, while bending action should be carried accumulatively by the filled concrete and the tube. It is to ensure the safety of the members for no rational estimation of the concrete contribution under the shear action. Moreover, it also said that a reliable evaluation of the concrete contribution can draw more economical design of the CFT member with less steel amount.

Thus, the numerical study using a nonlinear three dimensional finite element method of the square CFT members with a large \( \frac{B}{t} \) has been conducted in order to examine their load carrying capacities evaluation.

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2. Numerical method

2.1 Numerical idealization

As shown in Fig.2-1, the CFT member to be analyzed herein had a square cross section with 200mm of edge length. The two parameters were a ratio of shear span length to effective depth: $a/d$, and $B/t$. A monotonic displacement increment was applied downward at a free end of the cantilever member, amount of the increments at a final loading stage was about $1/40$ of the member length by 100 steps. Additionally, a buckling isn’t made consideration in this analysis.

![Analytical model](image)

Fig.2-1 Analytical model

<table>
<thead>
<tr>
<th>Analytical parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a/d$ = 4.0, 3.0, 2.0, 1.5, 1.0, 0.5</td>
</tr>
<tr>
<td>$b/t$ = 200, 125, 62.5</td>
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<tr>
<td>($t=1.0, 1.6, 3.2$ mm)</td>
</tr>
</tbody>
</table>

2.2 Elasto-plastic constitutive relation

2.2.1 Elastic relation

The elastic relationship between stress and strain is according to the Hook’s law as Eq.(2.1).

\[
\begin{bmatrix}
\sigma_x \\
\sigma_y \\
\sigma_z \\
\tau_{xy} \\
\tau_{yz} \\
\tau_{xz}
\end{bmatrix} = \frac{E}{(1-2\nu)(1+\nu)} \begin{bmatrix}
1-\nu & \nu & \nu & 0 & 0 & 0 \\
\nu & 1-\nu & \nu & 0 & 0 & 0 \\
\nu & \nu & 1-\nu & 0 & 0 & 0 \\
0 & 0 & 0 & \frac{1-2\nu}{2} & 0 & 0 \\
0 & 0 & 0 & 0 & \frac{1-2\nu}{2} & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{1-2\nu}{2}
\end{bmatrix} \begin{bmatrix}
\varepsilon_x \\
\varepsilon_y \\
\varepsilon_z \\
\gamma_{xy} \\
\gamma_{yz} \\
\gamma_{xz}
\end{bmatrix}
\]

Where, $E$ and $\nu$ are Young’s Modulus and Poisson’s ratio, respectively.
2.2.2 Elasto-plastic relation

The relationship between stress and strain in plastic region obeys the following rules: First, the relationship of the steel tube obeys the von Mises’s yield criteria; Second, the relationship of the filled concrete under tri-axial loading obeys the Ottosen’s criteria; Last, the relationship between incremental stress and strain is according to an associated flow rule.

2.3 Materials

As shown in Tab. 2-1, typical material constants are given assuming concrete as an ordinary one, and steel as SS400 grade, Reference[1].

2.3.1 Concrete

Sold type finite elements were used for the filled concrete, divided into 1000 elements. As shown in Fig.2-2(a) of a relationship between stress and strain, Modified Ahmad model and Tension cut-off model were also employed in compressive and tensile region, respectively.

2.3.2 Steel

Shell type finite elements were used for the steel tube divided into 400 elements. As shown in Fig.2-2(b), a bi-linear model with no strain hardening region were also employed.

2.3.3 Bond between concrete and steel

Based on the Reference [2], the bond characteristics between the concrete and the steel under low restriction was expressed as shown in Fig.2-3

<table>
<thead>
<tr>
<th>Tab.2-2 material constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ec(N/mm²)</td>
</tr>
<tr>
<td>2.5 × 10⁴</td>
</tr>
</tbody>
</table>

Fig.2-2 Stress-strain relationship

Fig.2-3 Bond stress-sliding relationship
3. Numerical Result

3.1 Failure modes

Fig. 3-1 shows six examples of the obtained numerical results at each maximum loading stage for a series CFT beams with the thinnest tube thickness of 1.0 mm relevant to $B/t = 200$ and various ratios of $a/d$. A vertical layout of the figures corresponding to a ratio of $a/d$ is as follows: The top is crack propagation of filled concrete; the second is Mise's stress distribution of the steel tube; the third is shearing stress distribution of the tube, and the bottom is longitudinal stress distribution of the tube. Furthermore, the front of each figure indicates the free end, so that the back is the fixed one.

From the figure, predominate bending action could be recognized for the longest beam of 800 in length relevant to $a/d$ of 4, because of low shearing stress of the web plates of the tube. The tendency was continued to be shown till the length decreased to 400. However, the yielding of the web due to shear obviously occurred when the length was 300 as $a/d$ of 1.5, which was regarded as a transient case of failure mode. As the length shortened, furthermore, the predominate action could change from bending to shear, so that the whole cross section of webs yielded due to shear when the length was short below 200. It should be remarked that the failure modes were distinguished naturally dependent upon $a/d$ as follows: the shear and bending failure occurred when $a/d$ was below 1.0 and over 2.0, respectively.

3.2 Shear load carrying capacities

The obtained load carrying capacities of the CFT beams in the shear failure were compared with the corresponding values from existing design specifications. The design values were comprised with the carrying capacity of the filled concrete and the steel tube as denoted in an accumulative form of Eq.(3.1), in which the former of $V_c$ was according to JSCE Standard Specifications for Concrete Structures (2007) as Eq.(3.2) and the latter of $V_s$ referred to that for Railways Composite Structures [3] as Eq.(3.3). Two of the design equations in Eq.(3.2) for $V_c$ were prepared, in which one is for an ordinary beam and the other is for so-called deep beam to expect a tied arch mechanism against shear action.

Fig. 3-2 shows the comparison of the obtained load carrying capacities with the design values in case of $a/d$ below 1.5 observed the transient failure. It could be found that all the obtained values were located between two of the design values independent upon $B/t$. Thus, the obtained values were over the design values for the ordinary beams but did not attained to another design values for the deep beams. As a result, it is necessary to make compressive stress flow in the filled concrete clear for revealing the shear load carrying mechanism of the CFT members.

$$V = V_c + V_s (= Pu) \tag{3.1}$$

$$\begin{align*}
V_c &= \beta_d \beta_p \beta_a f_{vc} b_w d \quad \text{(bar member)} \\
V_{cd} &= \beta_d \beta_p \beta_a f_{cd} b_w d \quad \text{(deep beam)} \\
V_s &= \frac{A_s f_{sy}}{\sqrt{3}} \tag{3.3}
\end{align*}$$
Concrete crack propagation

\[ \sigma_{eq} = \sigma_{sy} \]

Mises stress

\[ \sigma_{eq} = \sigma_{sy} \]

Mises stress

Shear stress

Shear stress

X-normal stress

X-normal stress

Fig. 3-1 Stress distribution of steel tube (cracked condition of filled concrete)
Concrete crack propagation

Mises stress

Shear stress

X-normal stress

Fig. 3-1 Stress distribution of steel tube (cracked condition of filled concrete) (cont.)
Fig. 3-1 Stress distribution of steel tube (cracked condition of filled concrete) (cont.)
4. Concluding Remarks

The concluding remarks can be drawn from the study as follows:

1) The obtained failure mode could be divided dependent on a ratio of shear span length to effective depth into three modes of bending, transient and shear.

2) The obtained load carrying capacities failed in the shear mode were over the accumulative design equation for ordinary beams, while the capacities did not attained to the another equation for deep beams.

3) It is necessary to make compressive stress flow in the filled concrete clear for revealing the shear load carrying mechanism of the CFT members.

5. References