Evaluation of Bending Capacity and Deformation Performance of Concrete Filled Steel Tube Member with Small Shear-span Ratio

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Concrete filled steel tube (CFT) members have been applied to the columns of rigid frame structures. However, the failure mode, the bending capacity and deformation performance of CFT members with small shear-span ratio are not clear because there have been few studies on them. In this study, cyclic loading tests were conducted on short-column CFT members with constant vertical loads. As a result, it was confirmed that they show a flexural failure mode, but the existing design method overestimates their bending capacity. Therefore, a new evaluation method was proposed for their bending capacity and deformation performance.

Keywords: CFT member, shear-span ratio, bending capacity, deformation performance, stress-strain curve

1. Introduction

In the construction of railroad structures, there are many examples of concrete filled steel tube members (hereinafter called CFT members) with a circular cross section being applied to columns of viaducts built in station or similar areas, not only because CFT members are characterized by higher deformation performance and larger load carrying capacity than what could be expected from its section size, due to the composite action of a steel tube and concrete, but also because it offers excellent workability. Normal viaduct columns possess values of between 3.0 and 6.0 for the shear-span ratio $L_a/D$, which is the ratio between a shear span $L_a$ and the external diameter $D$ of the column. “Design Standards for Railway Structures and Commentary - Seismic Design” (hereinafter called the Seismic Standard) [1] specifies a method for estimating the bending capacity and deformation performance on this type of circular CFT member. Nonetheless there are many cases where the adoption of a CFT member with a shear-span ratio smaller than 3.0 (hereinafter called a short-column CFT member) is unavoidable for columns to be added to end sections of a viaduct or set on the upper layer of a two-layer rigid-frame viaduct.

A CFT member would suffer a flexural failure mode if its shear-span ratio rose to 3.0 or more, but with a decrease in its shear-span ratio, shear failure is possible as well as a reduction in deformation performance and bending capacity, due to its shear behavior [2, 3]. On the other hand, there are only a very limited number of studies on short-column CFT members centering on their circular cross section [4]. In its CFT guideline [3], the Architectural Institute of Japan presents the results of a study based on monotonic loading tests on short-column CFT members subjected to uniform bending moments. However, the failure mode, the bending capacity and deformation performance are still unclear in the case of CFT members, such as column members subjected to linear bending distribution and cyclic bending loads.

For this reason, in this study alternating load tests were conducted using a cantilever-column shaped test specimen with a low shear-span ratio of the type adopted for a railway viaducts, in order to propose a method for quantitatively evaluating the deformation performance as well as the failure mode and bending capacity of short-column CFT members characterized by a circular cross section. Studies were conducted to find a method to estimate the bending capacity and deformation performance of a “short-column CFT member.”

2. Overview of the alternating load test

2.1 Test specimens

Table 1 shows the specifications of the test specimens. The test specimen K-1 has a standard shape with a shear-span ratio of 3.0. It was used for comparison with a short column. The test specimen K-2 is a short-column characterized by a shear-span ratio of 1.7, and is almost identical to the test specimen K-1 except for its shear-span ratio. Test specimen K-3 is a short-column with a thin-walled section and a diameter thickness ratio of 117.1. It is identical to the test specimen K-2 except for its diameter thickness ratio. Based on these comparisons, the focus of the study was placed on differences between test specimens in terms
of the shear-span ratio and the diameter thickness ratio.

Incidentally, the shear-span ratio value of 1.7 was selected in reference to a range of values estimated from the column of a railway viaduct and the restrictions due to the capacity of the test equipment. The diameter thickness ratio was also set for both test specimens K-1 and K-2 at a value that is common in actual structures, while that for K-3 was set approximately at the maximum value for a thin-walled section which could be applied to actual structures.

2.2  Loading condition

As indicated in Fig. 1, the adopted loading method used a displacement control to provide positive-negative alternating displacement in the horizontal direction at the top of the test specimen under constant axial force in the vertical direction. Horizontal displacement was increased in increments of $\delta y$, $2\delta y$, $3\delta y$ and so on, through integral multiplication of $\delta y$, the yield displacement. Three cycles of alternating load was applied at each increment.

| Table 1  Specifications of test specimens |
| Test specimens | K-1 | K-2 | K-3 |
| Diameter $D$ (mm) | 269.0 | 269.5 | 269.9 |
| Thickness $t$ (mm) | 4.2 | 4.2 | 2.3 |
| Material strength | Compressive strength of concrete $f'_c$ (N/mm$^2$) | 21.9 | 23.2 | 24.9 |
| | Yield strength of steel tube $f_{sy}$ (N/mm$^2$) | 299.8 | 299.8 | 358.9 |
| Shear span $L_a$ (mm) | 810 | 459 | 459 |
| Axial force $N'$ (kN) | 425 | 440 | 374 |
| Diameter thickness ratio $D/t$ | 64 | 64 | 117 |
| Shear span diameter ratio $L_a/D$ | 3.0 | 1.7 | 1.7 |
| Axial force ratio $N'/N'_y$ | 0.21 | 0.21 | 0.20 |
| Radius thickness ratio parameter $R_t$ | 0.08 | 0.08 | 0.17 |
| Slenderness ratio parameter $\lambda$ | 0.22 | 0.12 | 0.13 |

$R_t$ : Radius thickness ratio parameter  \[ R_t = \frac{1.65(f_{sy}/E_s)}{(r/t)} \]

$\lambda$ : Slenderness ratio parameter  \[ \lambda = \frac{N'_y}{N'_{cr}} \]

$E_s$ : Young's modulus of steel tube,  
$r$ : Radius of steel tube,

$N'_y$ : Fully plastic axial force,  
$N'_{cr}$ : Elastic buckling axial force.

As indicated in Fig. 2, the yield displacement $\delta y$ was defined as the displacement observed when the point of the steel tube positioned at 45 degrees on the tensile force side with respect to the loading direction, reached the yield strain. It was so defined in order to remain consistent with the yield point of the skeleton curve (Fig. 3) used for modeling a CFT member as shown in the Seismic Standard [1].
3. Results of the loading tests

3.1 Damage found to test specimens

On each test specimen, flexural yielding occurred prior to shear yielding. Subsequently, local buckling occurred due to flexural compression at the base of the steel tube when the displacement reached about \(5\delta_y\), and around the time when local buckling appeared, the test specimen was under maximum load. Furthermore, as load application continued, local buckling progressed until the load gradually decreased. Finally, a crack developed at the top of the area where local buckling had occurred on the steel tube; accordingly, the load decreased rapidly. At this point loading was stopped. The generated crack appeared to be the result of low cycle fatigue.

3.2 Failure mode of the short-column CFT member

Figure 4 shows local buckling of the steel tube on test specimen K-2 and the damaged condition of its internal concrete after application of loading. The damage was similar to that to test specimen K-3.

As shown in Fig. 4 (a), local buckling appeared on the steel tube due to flexural compression, and which developed as loads were applied. In addition, as shown in Fig. 4 (b), cracks, exfoliation, and other defects were found in the internal concrete around the base of the column after loading. No visible damage, such as shear cracks, was found except in this base area. This failure mode was similar to that of test specimen K-1 which had a shear-span ratio of 3.0. Consequently, this confirmed that it was typical flexural failure [5] of a CFT member.

3.3 Shear strain of the steel tube

Figure 5 shows results of shear strain measurements on the steel tubes used in test specimens K-1 and K-2, respectively. Specifically a triaxial gauge was attached to the side surface of the steel tube as shown in Fig. 2. Figure 5 shows the result at a yield point \((1\delta_y)\) and for \(5\delta_y\) when the height of the local buckling of the steel tube had reached around 10 mm. The time point \(5\delta_y\) corresponds approximately to “point N” on the skeleton curve in Fig. 3 where 90% of the maximum load is maintained.

Figure 5 clearly shows that the difference in shear strain between K-1 and K-2 both at the yield time and at \(5\delta_y\) corresponds approximately to the horizontal load ratio. Incidentally, it is posited that a rapid increase in strain at the base of the steel tube at \(5\delta_y\) was caused by local buckling in both the test specimens.

3.4 Relationship between load and displacement

Figure 6 shows the relationship between the bending moment \(M\) at the base of the test specimen and the angle \(\theta\) of rotation of the specimen which is obtained by dividing horizontal displacement at the column head by the shear span. This figure also gives calculated values which will be explained later. Figure 6 indicates that the load on test specimens K-2 and K-3, each in the form of a short column, decreases at an almost constant gradient, without rapid decrease, after reaching the maximum bending moment and the crack appears on the steel tube, similar to what happened with test specimen K-1 which has a standard shear-span ratio of 3.0. Accordingly, it can be concluded that it has excellent ductility.

The only difference between K-1 and K-2 is the shear span, while the relationship which is observed between the bending moment and the angle of rotation of the member is almost identical. Taking the observation into consideration, it is postulated that in a CFT member characterized by a shear-span ratio of 1.7, the shear effect has almost no influence on the relationship between the bending moment and the angle of rotation of the member even in the domain where the load decreases.

Test specimen K-3 has smaller maximum bending capacity and poorer deformation performance than K-2. It is assumed that the cause of this condition is the small thickness of the steel tube and the consequent decrease in the confined effect. A similar trend was observed in a CFT member that had a shear-span ratio of approximately 3.0 [5].
to calculate, with greater precision, the bending capacity and deformation performance of a CFT member that has a shear-span ratio of 3.0. On the other hand, Fig. 6(b) and 6(c) show that the deformation performance of a short-column CFT member with a shear-span ratio of 1.7 fits with experimental values, but its bending capacity tends to be overestimated regardless of its diameter-thickness ratio.

4.2 Discussion on the overestimation of the bending capacity

This chapter discusses possible causes of the overestimation of the bending capacity produced using the current method, from the viewpoint of applicability of the current method to short-column CFT members.

When calculating the bending capacity $M_m$, as shown in Fig. 3, the formation size of the failure zone at the end part of a member is taken into consideration, as shown in Fig. 7. Hence, a section shifted upward by an equivalent plastic hinge length $L_p$ is regarded as a failure section for the purposes of calculating the bending capacity $M_u$ of this section. After obtaining $M_u$, $M_m$ is calculated with (1).

$$M_m = \frac{L_a}{L_a - L_p} M_u$$

(1)

Where $L_a$: Shear span

$L_p$: Calculated in (2) using the plastic hinge length [7]

$$L_p = D \left\{ 1.5 \left( N' / N'_y \right)^2 + 0.5 \right\}$$

(2)

where, $N'$: Applied axial force; $N'_y$: Full plastic axial force of a CFT member.

In addition, (3) is adopted while increasing strain $\varepsilon'_{cu}$ at the compressive edge to more than 0.0035, a value commonly used for an RC member, taking into account the confined effect caused by the steel tube, though conventionally a stress-strain curve [8] like the one observed in an RC member is used for presenting material characteristics of in-filled concrete that are applied to the calculation of $M_u$.

$$\varepsilon'_{cu} = 1.474 \left( f_{y}/E_s \right) \cdot 100 / (D/t) + 0.006$$

(3)

Where, $f_{y}$: yield strength of the steel tube; $E_s$: elastic coefficient of the steel tube; $D$: external diameter of the steel tube; and $t$: steel tube thickness.

Figure 8 shows the result of comparing the bending capacity obtained using the current method and experimental values from data obtained in tests and test data presented in [5]. Figure 8 also illustrates that there is a tendency to overestimate the bending capacity, using the current method, as the shear-span ratio falls.

Figure 6(b) and 6(c) and Fig. 8 show that the test specimens K-2 and K-3 with different radius thickness ratios have similar ratios of a calculation value of the bending capacity provided by the current method to an experimental
value. Therefore, we consider that the test specimens show the tendency stated above, where regardless of the values of their radius thickness ratios.

As described above, it was found that applying (1) from the current method to a short-column CFT member would result in overestimation of the bending capacity.

5. Proposal of a calculation method of the bending capacity of short-column CFT member

Current calculations of bending capacity, do not take into account an increase in strength in the stress-strain relationship, though it does consider an increase in strain at the compressive edge generated by the confined effect. In reality however, the strength of concrete strength increases because of the confined effect, too. Additionally, it was found that there were issues yet to addressed when applying (1) to short-column CFT members.

Therefore, instead of using (1), studies were made into a method for calculating the bending capacity of a CFT member using a stress-strain curve where an increase in concrete strength due to the confined effect is taken into consideration.

The following equations (hereinafter called the proposed method) proposed by Mander et al. [9, 10] were used to obtain the stress-strain curve of the concrete. This equation has been already examined with reference to an RC member where horizontal constraining reinforcements are arranged relatively densely. There are many examples of studies on its application to a structure that is horizonally constrained by a circular steel tube [11]. Here stress is calculated using (4).

\[ \sigma'_{c} = \frac{f'_{cc} \cdot x \cdot r}{r+1+x'} \]  

(4)

Where

\[ x = \frac{\varepsilon'_{cc}}{\varepsilon_{cc}} \]  

(5)

\[ r = \frac{E_{cc}}{E_{cc} - E_{sec}} \]  

(6)

\[ f'_{cc}: Compressive strength of concrete horizontally constrained that can be calculated using (7). \]

\[ f'_{cc} = f_{c} \left( 2.254 \left[ \frac{7.94 \cdot f_{c}}{f_{c}^{'}} - 2 \right] - 2.54 \right) \]  

(7)

\[ f'_{cc}: Compressive strength of concrete without the confinement. \]

\[ f': Lateral confinement stress that can be calculated with (8). \]

\[ f' = \frac{2 \cdot t \cdot f_{fy} R}{D - 2t} \]  

(8)

\[ \varepsilon'_{cc}: Compressive strain of concrete. \]

\[ \varepsilon'_{cc}: Strain of horizontally constrained concrete at the time of maximum strength (f'_{cc}), which can be calculated using (9). \]

\[ \varepsilon_{cc} = 0.002 \left[ 1 + 5 \left( \frac{f'_{cc}}{f_{c}} - 1 \right) \right] \]  

(9)

\[ E_{c}: Elastic coefficient of concrete that can be calculated with (10). \]

\[ E_{c} = 5000 \sqrt{f_{c}^{'}} \]  

(10)

\[ E_{sec}: Secant modulus of concrete that can be calculated with (11). \]

\[ E_{sec} = \frac{f_{c}^{'}}{\varepsilon'_{cc}} \]  

(11)

Additionally, the bending capacity \( M_{c} \) can be calculated using the above stated stress-strain relationship, adopting \( \varepsilon'_{cc} \) indicated in (9) as the compressive strain of concrete, and assuming a state of being held plane, based on the Seismic Standard [1].

6. Verification of the applicability of the proposed method

6.1 Short-column CFT members

Figure 6(b) and 6(c) showing test specimens K-2 and K-3, allows the comparison of values calculated using the proposed method and experimental values. The angle of rotation of a member was calculated in a similar manner to the Seismic Standard [1], except that a value formulated separately [12] was used for the rotation angle \( \theta_{m} \) of the plastic hinge portion observed at the time when the bending capacity was \( M_{m} \).

Figure 6 indicates that the proposed method corrects the tendency to overestimate bending capacity of a short-column CFT member, resulting in roughly the same values as the experimental values. It also provided almost the same deformation performance as the current method, offering roughly the same performance values as the experimental values.

6.2 CFT members other than short columns

The bending capacity and deformation performance of CFT members other than short columns was calculated, using test data obtained in the present tests and data from existing literature [5]. Figure 9, with the short column member included therein, indicates the results of the comparison of calculated values with experimental values, with a focus on the bending capacity \( M_{m} \), the angle \( \theta_{m} \) of rotation of a member at this value of bending capacity, and the angle \( \theta_{n} \) of rotation of a member at the point where bending capacity falls to 90%. The calculated values include both the values obtained by the use of the proposed method and those by the current method.

Figure 9(a) illustrates that the proposed method produces calculation results for bending capacity \( M_{m} \) which are closer to experimental values and with less variation than the current method. Figure 8 shows the problem of divergence between calculated and experimental values for bending capacity has been solved. Though Fig. 9(b) and 9(c) show some dispersion in the calculation results of the angle of rotation of a member, the precision of the calculation is better than with the current method. Results also confirmed that the precision in calculation of the yield bending capacity and the angle of rotation of a member at
the time of yielding is almost at the same level in both the cases whether using the proposed method or the current method.

As explained above, verifications were made to ensure that it was possible to calculate the bending capacity and deformation performance of CFT members with greater accuracy not just for short columns but for other members as well, when using the proposed method.

7. Conclusions

In this study, alternative load tests were conducted using cantilever columns with a view to proposing a quantitative evaluation method of the bending capacity and deformation performance of short-column CFT members with circular cross sections. Accordingly, their failure modes were examined and methods were investigated for calculating their bending capacity and deformation performance. In sum, this study achieved the following results:

(1) Short-column CFT member (with a shear-span ratio of 1.7) showed flexural failure accompanied by ductility, similar to when the shear-span ratio is 3.0.

(2) Bending capacity and deformation performance can be evaluated without reference to shearing until the shear-span ratio reaches 1.7.

(3) The present method is appropriate for calculating the deformation performance of a short-column CFT member, but it overestimates the bending capacity. It is considered that the cause is a way of the application of (1) to a short column.

(4) The bending capacity and deformation performance of a short-column CFT member were calculated using the proposed method based on a concrete stress-strain curve where the confined effect of a steel tube and other conditions were taken into consideration. As a result, it was confirmed that the proposed calculation method assured greater precision.

(5) Confirmation was obtained that the proposed method can calculate the bending capacity and deformation performance of CFT members other than short columns with a higher or equal degree of accuracy to the current method.

The contents of this paper will be described in the Design Standards for Railway Structures and Commentary (Steel-Concrete Hybrid Structures).

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