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Study on the Eccentric Compression Performance of Stiffened Rectangular CFST

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Abstract: Concrete-filled steel tubular columns (CFST) have high bearing capacity, high stiffness and good ductility. The columns with rectangular section are more suitable for compression bending than round. However, its lateral binding force is small in the middle of the section, and local buckling is easy to occur. Stiffened Rectangular Concrete Filled Steel Tubular columns (SRCFST) are proposed to solve the problem that ordinary rectangular concrete filled steel tubular columns (RCFST) are prone to appear local buckling before reaching the limits of carrying capacity in 3 cases such as the thin steel tube wall, the larger diameter to thickness ratio or width to thickness ratio. SRCFST are RCFST added with stiffening ribs, stiffening hoops and binding bars. By its set, the lateral stiffness and the resistance ability of the local buckling of the steel tube are effectively enhanced, so the confinement effects of the steel tube to core concrete become stronger and more uniform. To compare the improved effects of SRCFST and RCFST with binding bars under eccentric compression, the paper compiles the solving procedure of the cross section grid unit method which can accurately solve the whole process curve of SRCFST under eccentric compression. The whole process curves show that the eccentric bearing capacity and ductility of SRCFST are obviously improved with respect to RCFST with binding bars.

1. Introduction

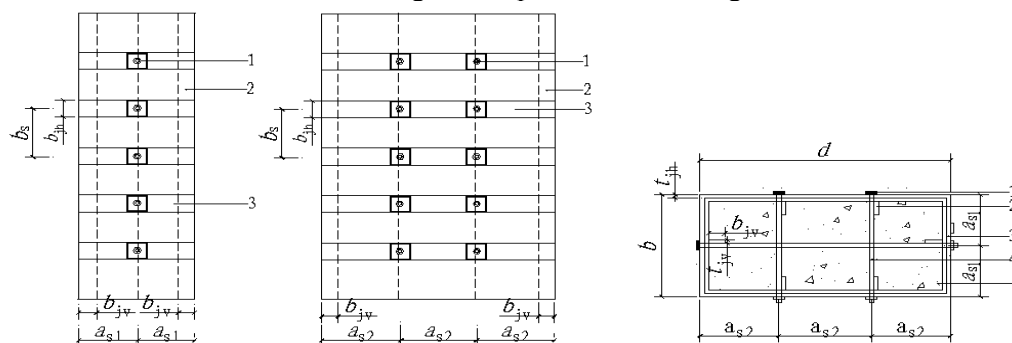
The binding effects of RCFST upon internally filled concrete, center in the corner. The lateral constraints depend on the out-of-the-plane rigidity of the steel tubular walls. Commonly used CFST have a relatively large width to thickness ratio. Thus the lateral binding is smaller. The deform ability of columns makes it difficult to achieve the desired effects of improved bearing capacity [1-3]. Measures such as increasing the thickness of steel tubular walls can improve the lateral binding force. Unfortunately it requires great amounts of steel consumption, and with the currently weak economy, this becomes an obstacle.



The scholars at home and abroad have undertaken numerous endeavors to improve the performance of RCFST placed under axial compression. The reference [4] studied the axial compression properties of RCFST with binding bars. Studies show that the set of binding bars is helpful in improving its axial-load carrying capacity and ductility. The reference [5-7] studied eccentric compression properties of RCFST with binding bars. The research shows that the set of binding bars is helpful in improving its eccentric-load carrying capacity and ductility. Susantha et al. [8] proposed a study of the buckling mode bearing capacity and ductility of stiffeners, in a structured form and at the midpoint of the surrounding sides of CFST. Zhou et al. calculated and compared the cost of ribbed RCFST to the cost of ordinary RCFST that has been placed under the typical conditions of axial and eccentric pressure [9]. Liang, Liu investigated the local buckling of CFST [10-11].

All these studies have shown that simple measures, such as setting binding bars and/or stiffeners (stiffening ribs, stiffening hoops), can significantly enhance the restriction from the steel tubes to concrete. Confinement effects also significantly improve, effectively delaying the local buckling of steel, and consequently improving the bearing capacity and ductility of present members.

Based on the findings above, this paper proposes that upon a foundation of RCFST with binding bars, SRCFST should be set with stiffening ribs, stiffening hoops and binding bars. It does not significantly increase the amount of steel needed, while delaying and preventing the usage of local-buckling steel. It improves the utilization of steel strength and constraints capacity flexibility available on the core, concrete-strength steel plate, shown in Fig.1.



Front elevation Lateral elevation A-A cross section
 a_s -longitudinal spacing of binding bars b_s -horizontal spacing binding bars b_{jh} -width of the stiffening hoops t_{jh} -thickness of the stiffening hoops b_{jv} -width of the stiffening ribs t_{jv} -thickness of the stiffening ribs

1-nut 2-stiffening rib 3-stiffening hoop 4-binding bar 5-concrete

Fig.1 SRCFST specimens

Binding bars uniformly transfer constraint force upon steel plates. This uniformity causes binding bars to administer linear constraint and avoid the over concentration of stress, which, in turn, divides the plates into segments along the longitudinal and sectional length direction. The division reduces the steel plates' height-to-width and width-to-thickness ratios, thus greatly reducing the semi-buckling wavelength. The set of stiffening ribs slow local buckling between the binding bars that are found along the vertical portion of the observed component. This process allows for the full display of lateral strength exerted by the binding strips.

2. Sectional grid element method

As is illustrated in Fig.2, a sectional grid element method can be effectively employed to the whole process analysis of SRCFST that have been found under eccentric compression.

Sectional grid element method divides the specimen cross-section into various grids, and assumes that the strains of each grid are consistent while stress is evenly distributed. A numerical analysis is dependent upon the following assumptions:

- (1) Sectional strain is linear;

- (2) Any relative slip between steel and concrete is not considered;
- (3) Tensile strength of concrete is not considered;
- (4) Specimen ends are hinged and the deflection curve shows a half sine-wave curve;

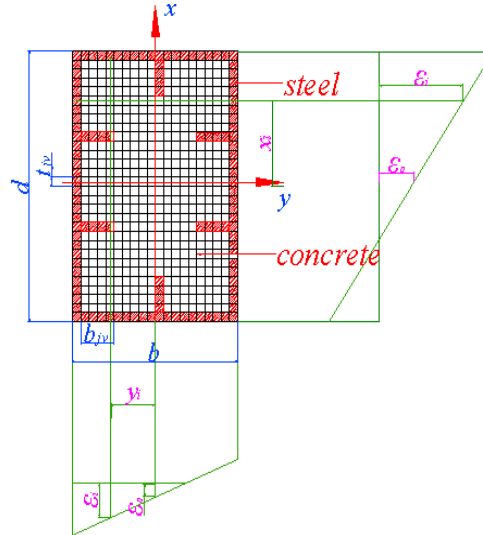


Fig.2 Sectional grid unit of stiffened CFST column

From (4), curvature shows as follows when the mid-span deflection is u_m .

$$\phi = \frac{\pi^2}{L^2} u_m$$

When eccentric compression occurs along the x direction, a centroidal strain of cross-sectional units may be found:

$$\varepsilon_i = \varepsilon_0 + \phi x_i$$

When eccentric compression occurs along the y direction, a centroidal strain of cross-sectional units may be found:

$$\varepsilon_i = \varepsilon_0 + \phi y_i$$

In the formula,

ε_0 represents the centroidal strain.

x_i displays calculated centroidal coordinates of the unit.

ε_i is determined by the steel stress σ_{ai} (σ_{aj}), and the concrete stress σ_{ci} .

The internal moment M_{in} and the internal axial force N_{in} are available as follows:

$$N_{in} = \sum_{j=1}^m \sum_{i=1}^n (\sigma_{ai} \Delta A_{ai} + \sigma_{ci} \Delta A_{ci})$$

$$M_{in} = \sum_{j=1}^m \sum_{i=1}^n (\sigma_{ai} x_i \Delta A_{ai} + \sigma_{ci} x_i \Delta A_{ci})$$

where σ_{ai} (σ_{aj}) is the longitudinal stress of steel;

σ_{ci} is the longitudinal stress of concrete;

m represents the division number of steel and concrete unit along the x direction;

n represents the division number of steel and concrete unit along the y direction.

Initial eccentric specimens should meet this formula:

$$M_{in} / N_{in} = e_0 + u_m$$

While changing u_m , ε_0 is automatically adjusted to meet the balance.

The specific steps involved in the process:

- 1) Dividing the sectional grid cells according to Fig.2;
- 2) The centroidal strain ε_i , found on each grid cell, is derived by designating the centroidal sectional strain ε_0 . Sub-routine 2 is set by the constitutive relation of concrete and steel. The centroidal strains σ_{ci} and σ_{si} of each stress unit for concrete, steel and stiffeners can be calculated.
- 3) Compiling sub-routine 1 for the calculation of the sectional internal-axial force and movement. The output results meet $M_{in} / N_{in} = e_0 + u_m$; conversely ε_0 constantly adjusts. Then steps 2) and 3) are repeated until $M_{in} / N_{in} = e_0 + u_m$ is reached.

By changing u_m , the N - w curve can be created (axial force-deflection in the middle of the column), and the M - ϕ curve (moment-curvature curve) can be created while repeating the above steps 2) and 3). The peak values of the curves respectively indicate the ultimate bearing capacity N_{max} and the ultimate bending moment M_{max} of specimens in the initial conditions of eccentricity.

3. Eccentric compression bearing capacity of SCFST only with binding bars and SRCFST

Table 1 Eccentric characteristics of SCFST only with binding bars and SRCFST

Model	b_{jv} (mm)	t_{jv} (mm)	b_{jh} (mm)	t_{jh} (mm)	N_{max} (MN)	W_{max} (mm)	M_{max} (kN·m)	ϕ_{max} (10^{-4})
RDB	0	0	0	0	2.5157	4	137.6250	0.219
NZWH-1	10	8	8	10	2.8035	5	159.1304	0.439
NZWH-4	16	5	5	16	2.8497	6	163.7827	0.439
NZWH-6	20	4	4	20	2.8759	6	165.6798	0.439

It can be seen from table 1:

The eccentric bearing capacity and ductility of SRCFST are obviously improved with respect to SCFST with binding bars. With the width-to-thickness ratio of the stiffening hoops decreasing, or the width-to-thickness ratio of the stiffening ribs increasing, SRCFST experience an increasing peak eccentric bearing capacity and an increasing ultimate moment.

4. Conclusion

This paper established a analytical model by taking the sectional grid units to accurately calculate, communicated with data, and discussed the eccentric bearing capacity of SRCFST. The main conclusions are withdrawn from the study as follows:

- (1) Binding bars uniformly transfer constraint force upon steel plates. This uniformity causes binding bars to administer linear constraint and avoid the over concentration of stress, which, in turn, divides the plates into segments along the longitudinal and sectional length direction.
- (2) Sectional grid element method divides the specimen cross-section of SRCFST into various grids, it can be effectively employed to the analysis of SRCFST that have been found under eccentric compression.
- (3) The optimized stiffening ribs and stiffening hoops can effectively improve the eccentric performance of SRCFST. The peak capacity and the ultimate moment of SRCFST significantly increase with respect to RCFST with only binding bars installed.
- (4) With the increase of the width to thickness ratio of the stiffening ribs or the decrease of the stiffening hoops, the peak eccentric bearing capacity and ultimate moment of the specimens are both increasing accordingly, but the corresponding ultimate deflection and sectional curvatures changed little.

Acknowledgments

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