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SEISMIC BEHAVIOR OF SQUARE CONCRETE-FILLED STEEL TUBULAR BEAM-COLUMNS STIFFENED BY INTERIOR TIE BARS

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SUMMARY

In this study, nineteen square CFT specimens with a common width-to-thickness ratio (B/t) of 70 and a common width (B) of 280mm were tested. The chosen B/t ratio does not satisfy the requirement of most design codes. Among these specimens, fifteen were tested under monotonic flexural load and the rest were tested under cyclic flexural load. The corresponding simulations were also carried out with fiber section method considering the effects of concrete's confinement and steel's buckling. It is observed that: (1) Stiffened CFT beam-columns have better flexural performance, in terms of moment capacity and curvature ductility, than un-stiffened ones. The improvement is more significant as the specimen is subjected to higher axial load; (2) The analytical model satisfactorily simulates the moment-curvature relationship of specimens with acceptable accuracy.

INTRODUCTION

In modern structural constructions, concrete-filled steel tubular (CFT) columns have gradually become an option in structural systems like buildings, bridges and so forth. CFT columns have become so widespread owing to their axially compressed nature making them superior to conventional reinforced concrete and steel structural systems in terms of stiffness, strength, ductility and energy absorption capacity. The steel tube not only takes axial load, but also provides confining pressure to the concrete core, while the concrete core takes axial load and prevents or delays local buckling of the steel tube. Furthermore, concrete-filled composite columns also have the advantage of requiring no formwork during construction, thus reducing construction costs.

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It has long been recognized that, as the tube width-to-thickness ratio becomes high, local buckling is more likely to occur on a square tube than on its circular counterpart. Accordingly, all current CFT design guidelines [1,2,3] enforce a lower value of upper limit to the width-to-thickness ratio of square tubes than that to circular tubes. This is the reason why it is less economical, and thus less desirable, to use square tubes in CFT construction. A scheme called “tie-bar stiffening” was consequently proposed by the authors [4,5] for improving the performance of square CFT members with high tube width-to-thickness ratios. This scheme is carried out by welding sets of stiffeners, each of which consists of four tie bars arranged at the tube corners, at certain cross-sections with equal spacing along the tube axis. Hypothetically, such arrangement could make more durable the square tubes, especially while under high axial load.

For the flexural behavior of CFT beam-columns, Sakino and Nakahara [6] summarized test results of sixty-seven square CFT beam-columns. For small B/t (like 22.8) specimens, the flexural ductility is very good. But for large B/t (like 63) specimens, the flexural ductility is very poor with large axial load ratio P/P_n (like 0.4).

The monotonic and cyclic flexural behaviors of the stiffened square concrete-filled steel tubular (CFT) beam-columns under constant axial load were investigated in this research work experimentally and numerically. The effectiveness of tie-bar stiffening scheme for the large B/t (70) square CFT beam-columns, in terms of moment capacity, flexural ductility and energy absorbing ability of the stiffened specimens, is to be verified experimentally. The fiber section method is also used to study the confining pressure of steel tube to core concrete and the local buckling behavior of steel tube through the comparison between test results and analytical results.

EXPERIMENTAL PROGRAM

Fabrication of specimens

The geometry of specimens tested in this study is as showing in Fig.1. The stiffening tie bars use steel deformed bars with diameter 10mm and yield strength 478MPa. Nineteen specimens, with a common width (B) of 280mm and a common width-to-thickness ratio (B/t) of 70, were fabricated for testing. The first character S or B in the specimen designation stands for square sections or stiffened square sections, respectively. The second character U or number denotes a specimen without stiffening or its stiffener spacing parameter, respectively. The last two digits denote axial load ratio (P/P_n) of the specimen. If the specimen name has a last character C, the specimen was under cyclic flexural loading. For example, the specimen B-3-0.2 means its stiffener’s spacing is B/3 and it has a constant axial load about 20% of nominal axial strength by Eurocode 4. The properties of specimens are summarized in Table 1. The steel tubes were assembled by using 2 cold formed U shape parts, on which tie bars were welded in advance at equal spacing, as shown in Fig. 2. Note that the four tie bars at the same level were welded at points dividing the tube width into three equal segments. It should be noted that, the width-to-thickness ratio (B/t) of 70 used in this study does not meet the Eurocode 4 and AISC-LRFD requirements.

Test setup

As shown in Fig. 3, we used a four-point bending test system to eliminate the influence of shear force to the specimen. The axial load was applied by an actuator with 2500kN capacity and ±250mm stroke. The lateral load was applied by an actuator with 1000kN capacity and ±500mm stroke. Two tilt meters with capacity ±10 degree were attached to the upper surface of rigid connection beams. Several displacement gages were used to measure the deflection of the specimen. The whole test setup is shown in Fig. 4.

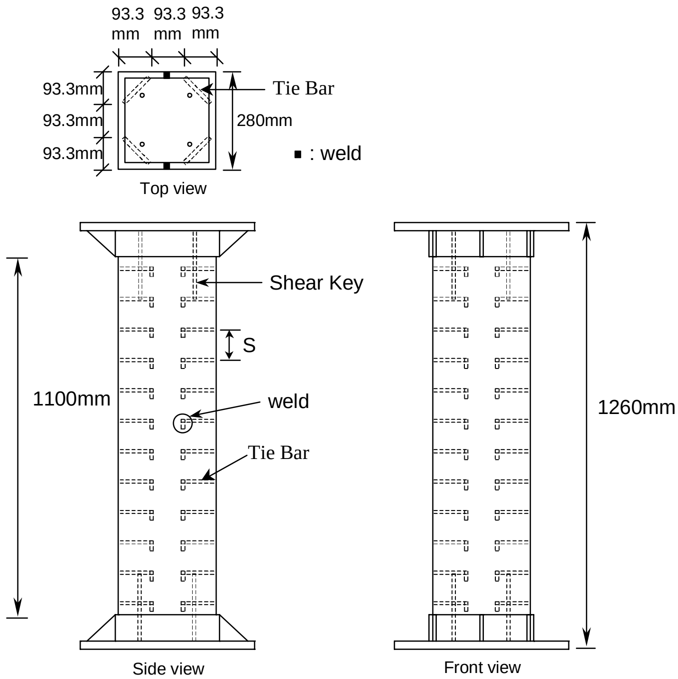

Fig. 1 Schematic diagram of stiffened specimens (S=B/3 or B/5)

Fig. 2 Sectional view of tie bar stiffened U shape parts

Table 1 Properties of specimens

Specimen	f_y (MPa)	f_c' (MPa)	P (kN)	P_n (kN)	$M_{u,\exp}$ (kN-m)	$M_{n,EC4}$ (kN-m)	$M_{u,\exp}/M_{n,EC4}$	f_r (%)	ε_b	K
S-U-0.0	292	26.8	0	3270	172	160	1.08	4.0	0.010	--
B-3-0.0	292	26.8	0	3270	179	160	1.12	6.0	0.011	--
B-5-0.0	317	35.4	0	4020	212	175	1.21	0.0	&	
S-U-0.2	317	35.4	802	4020	226	225	1.00	0.0	0.005	-0.102
B-3-0.2	317	35.4	801	4020	237	225	1.06	2.0	0.020	--
B-5-0.2	317	35.4	806	4020	254	225	1.13	8.0	0.017	--
S-U-0.3	292	26.8	1035	3270	200	201	1.00	0.0	0.005	-0.062
B-3-0.3	317	35.4	1201	4020	256	233	1.10	4.0	0.008	-0.052
B-5-0.3	317	35.4	1201	4020	277	233	1.19	12.0	0.015	--
S-U-0.4	292	26.8	1378	3270	201	195	1.03	2.0	0.005	-0.143
B-3-0.4	317	35.4	1602	4020	240	231	1.04	2.0	0.007	-0.082
B-5-0.4	317	35.4	1603	4020	280	231	1.21	8.0	0.014	--
S-U-0.5	292	26.8	1725	3270	186	178	1.04	2.5	0.005	-0.159
B-3-0.5	317	35.4	2004	4020	240	217	1.11	2.5	0.010	-0.129
B-5-0.5	317	35.4	2004	4020	276	217	1.27	7.0	0.022	--
B-3-0.0-C	317	35.4	0	4020	175	175	1.00	0.0	0.003	-0.159
B-5-0.0-C	317	35.4	0	4020	183	175	1.05	5.0	0.003	-0.264
B-3-0.4-C	317	35.4	1378	4020	220	234	0.94	0.0	0.006	-0.159
B-5-0.4-C	317	35.4	1378	4020	247	234	1.06	2.0	0.006	-0.099

Note: f_y : yield stress of steel tube; $f_c^{'}$: cylinder strength of concrete; P : applied axial load; P_n : uniaxial load strength predicted by EC4; $M_{u,\exp}$: experimental bending moment strength; $M_{n,EC4}$: bending moment strength predicted by EC4 under applied axial load; f_r : confining pressure ratio; ε_b : buckling strain; K : descending modulus ratio; and symbol & means strain hardening occurred in steel tube .

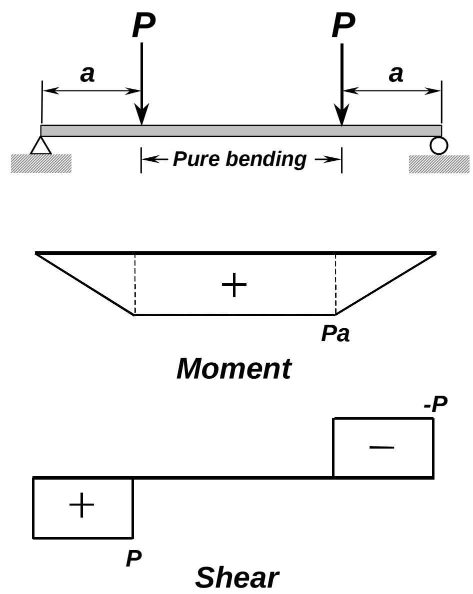

Fig. 3 Four-point bending test system

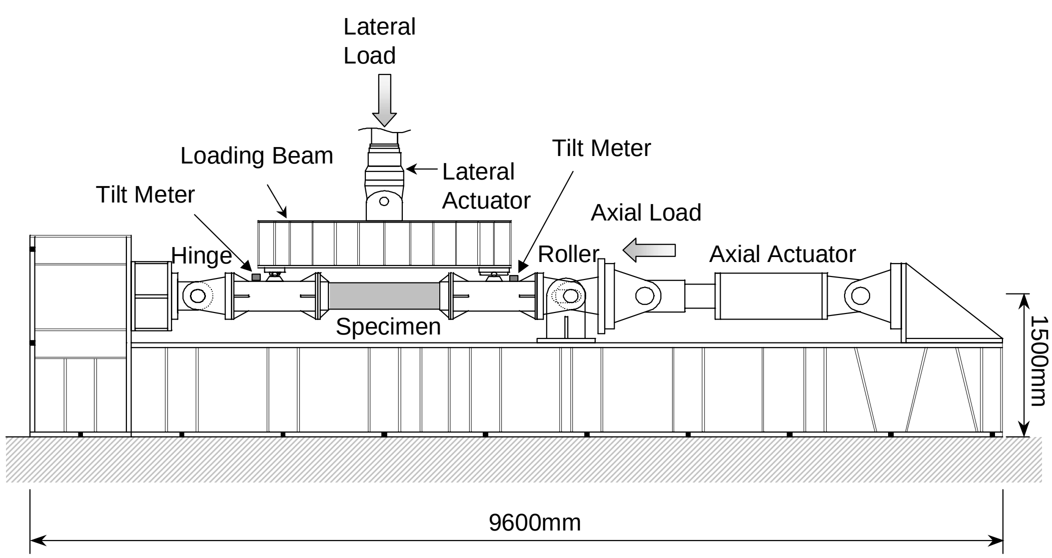

Fig. 4 Test setup

The axial load was applied before the flexural load. The axial actuator was under force control to keep a constant axial load during the testing. The lateral actuator was under displacement control with a displacement rate 0.125mm/sec to give a quasi-static loading condition. From the load cell reading of lateral actuator and the length of moment arm, we can calculate the moment applied by the lateral load. From the axial load and the lateral deflection at the middle of the specimen, the secondary moment due to the axial load can be computed to get the total moment resistance M of the specimen. The average curvature of the specimen φ can be calculated from the rotation angle of the two rigid connection beams, as measured by the tilt meters.

ANALYTICAL MODEL

Fiber section method

The cross section of the specimen is divided into many small elements. Each element, also called fiber, is parallel to the neutral axis and has its area and position. According to the assumption that plane sections before bending remain plane during bending, the strain of each fiber can be calculated from the position of the neutral axis and the curvature of the section. According to the strain and area of each fiber, the stress-strain curves of concrete and steel can be used to determine the axial load and moment taken by the section. If the calculated axial load equals the applied axial load, the curvature and moment of the section are achieved.

Stress-strain relationship of confined concrete

Concrete in the CFT member is confined by the steel tube. Mander *et al.* [7,8] use a confining pressure to affect the shape of the stress-strain curve of confined concrete. As shown in Fig.5, the compressive stress is given by

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \tag{1}$$

where

$$f'_{cc} = f'_c \left[-1.254 + 2.254 \sqrt{1 + \frac{7.94 f_l}{f'_c}} - 2 \frac{f_l}{f'_c} \right] \tag{2}$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \tag{3}$$

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_c} - 1 \right) \right] \tag{4}$$

$$r = \frac{E_c}{E_c - E_{\sec}} \tag{5}$$

$$E_{\sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \tag{6}$$

and where ε_c is the compressive strain corresponding to stress f_c; f'_{cc} = peak compressive stress; f'_c = concrete compressive cylinder strength; ε_{cc} = compressive strain corresponding to f'_{cc}; ε_{co} = compressive strain corresponding to f'_c; E_c = tangent modulus of elasticity of concrete; and f_l is the confining pressure applied by steel tube. A value of ε_{co} = 0.002 was used in this study. In this model, the behavior of concrete is a function of the applied confining pressure and a parameter f_r, which is defined as $f_l / f'_c \times 100\%$, is used to specify the effect of confining pressure.

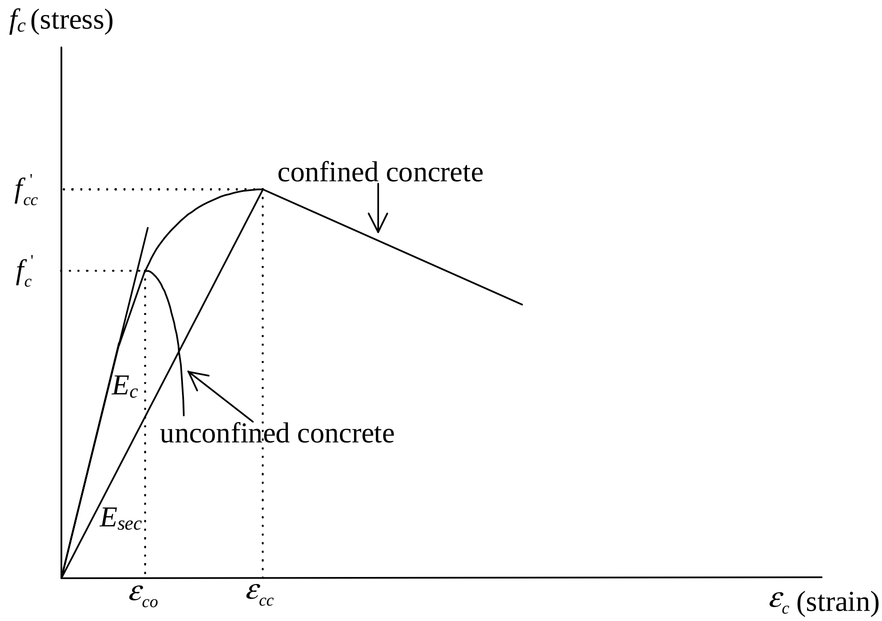

Fig. 5 Stress-strain relationship of concrete suggested by Mander *et al*. [7,8]

Stress-strain relationship of steel tube

The steel tube can be divided into the tension and compression parts. The tension part uses the elastic-perfectly plastic steel model or strength hardening model. In order to count the local buckling behavior, the stress-strain curve as shown in Fig. 6 for steel in the compression part uses the model suggested by Varma *et al*. [9]. Two parameters ε_b and K are used to reflect the local buckling effect of steel tube. ε_b is the buckling strain. K is the ratio of post-buckling modulus ratio which is defined as E_{sd}/E_s .

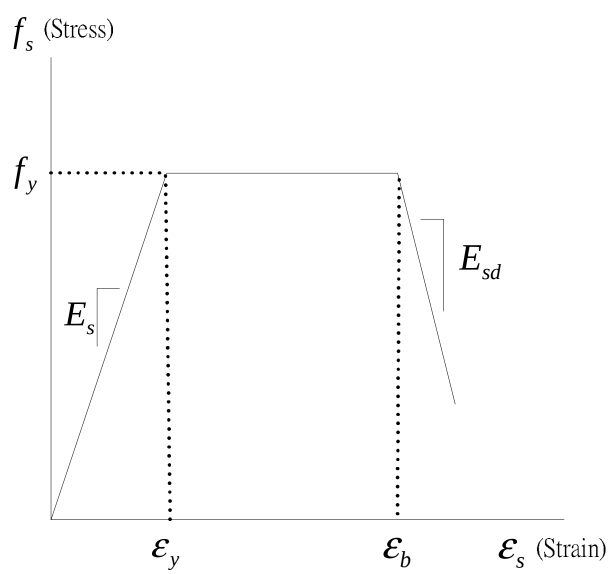

Fig. 6 Stress-strain relationship of steel in compression

EXPERIMENTAL AND ANALYTICAL RESULTS

In the earlier study [5] about monotonic tests of CFT specimens, we have the conclusions: (1) The moment capacities predicted by EC4 are more accurate than the values predicted by AIJ for unstiffened specimens, especially when the axial load is high; and (2) the strength capacity and ductility of square CFT beam-columns can be improved by the proposed tie-bar stiffening scheme. Comparison between the experimental moment capacity $M_{u,\exp}$ of each specimen and the corresponding nominal value $M_{n,EC4}$ predicted by EC4 is summarized in Table 1.

Cyclic flexural test results

There were four specimens B-3-0.0-C, B-5-0.0-C, B-3-0.4-C and B-5-0.4C subjected to cyclic flexural load. Their moment-curvature hysteretic responses are shown in Figs. 7 to 10.

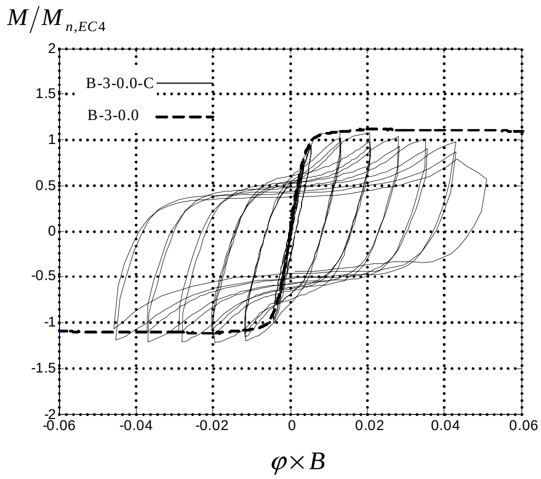

Fig. 7 Moment-curvature hysteretic response of B-3-0.0-C comparing with B-3-0.0

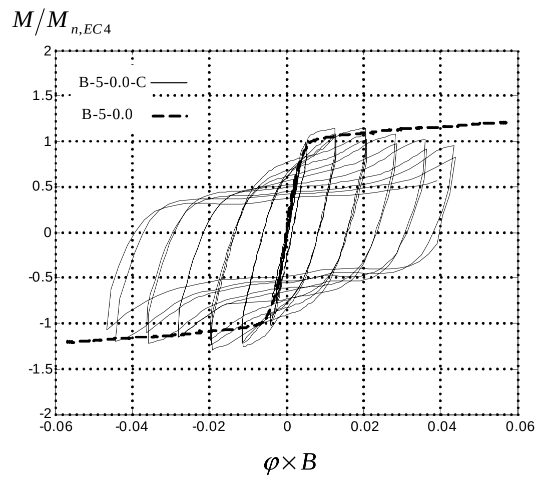

Fig. 8 Moment-curvature hysteretic response of B-5-0.0-C comparing with B-5-0.0

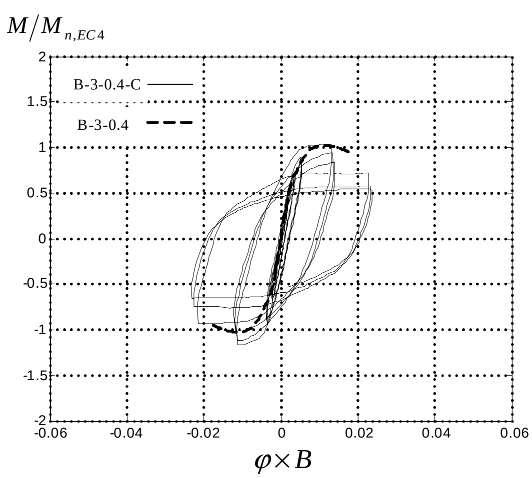

Fig. 9 Moment-curvature hysteretic response of B-3-0.4-C comparing with B-3-0.4

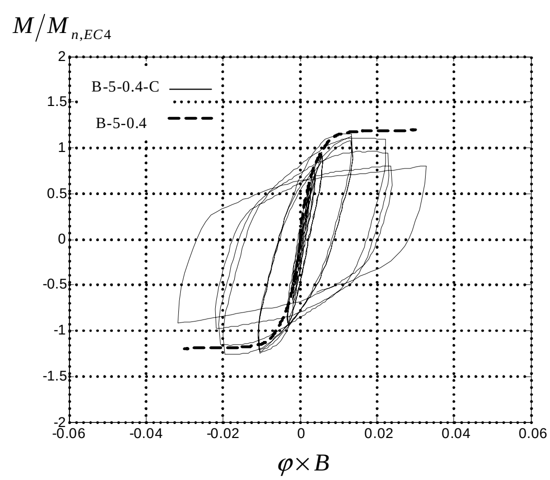

Fig. 10 Moment-curvature hysteretic response of B-5-0.4-C comparing with B-5-0.4

As shown in Figs 7 to 10, the moment-curvature curve of monotonic test is close to the envelope of hysteretic curve of cyclic test. Due to the low-cycle fatigue phenomenon, the strength decay of the cyclic test specimen is more serious than the monotonic test specimen. The specimens with smaller spacing of stiffeners have better strength, ductility and energy absorbing ability.

Analytical results

The fiber section method was used to simulate the moment-curvature relationships of all specimens. Three parameters f_r, ε_b and K were adjusted to match the simulated results with the experimental results. By choosing adequate values for these parameters, the confining effect on core concrete and local buckling on steel tube can be simulated. Fig. 11 shows the comparison between the analytical and experimental results and the employed parameters for specimens under axial load ratio of 0, 0.2 and 0.4.

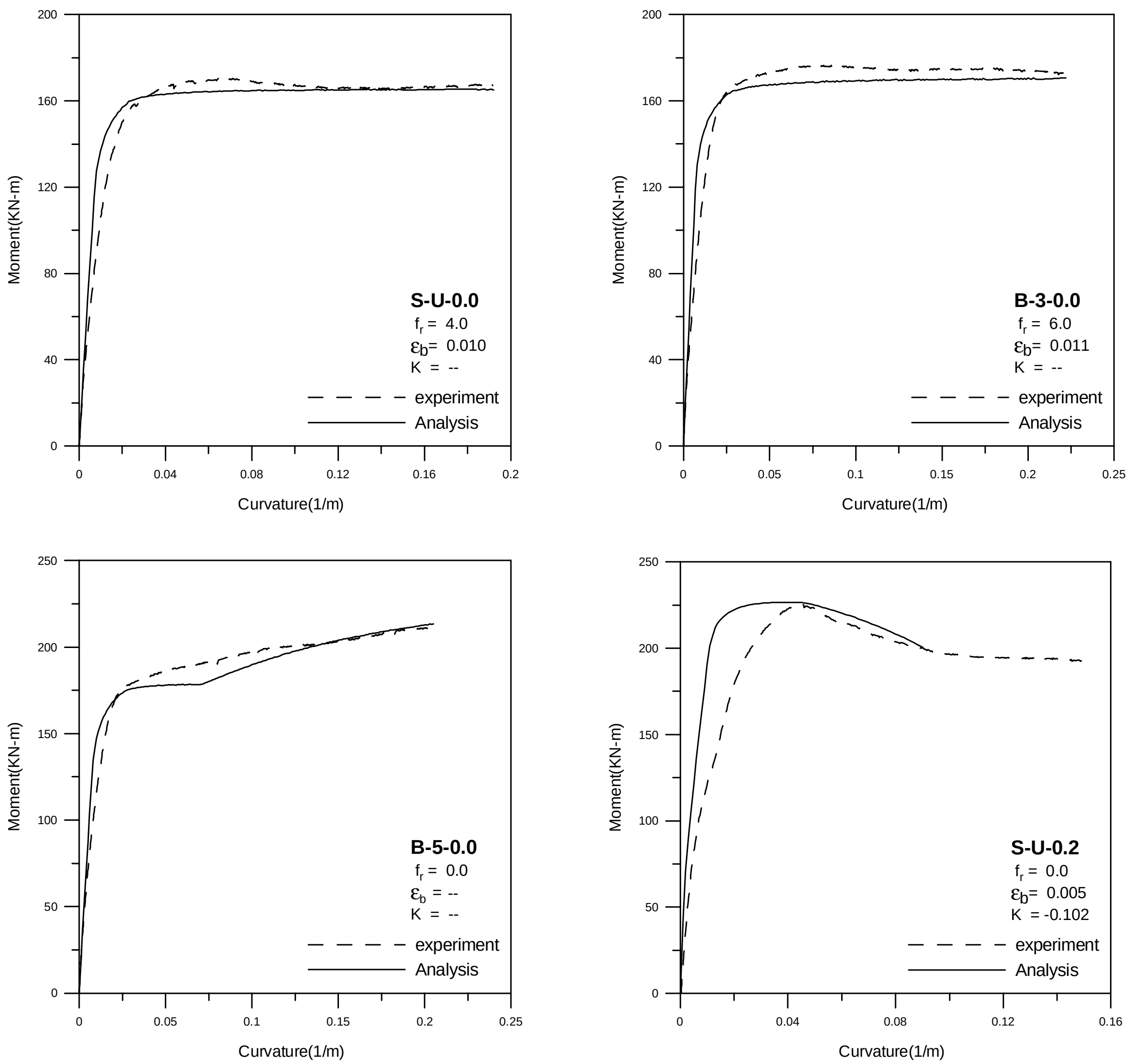

Fig. 11 Moment-curvature relationships

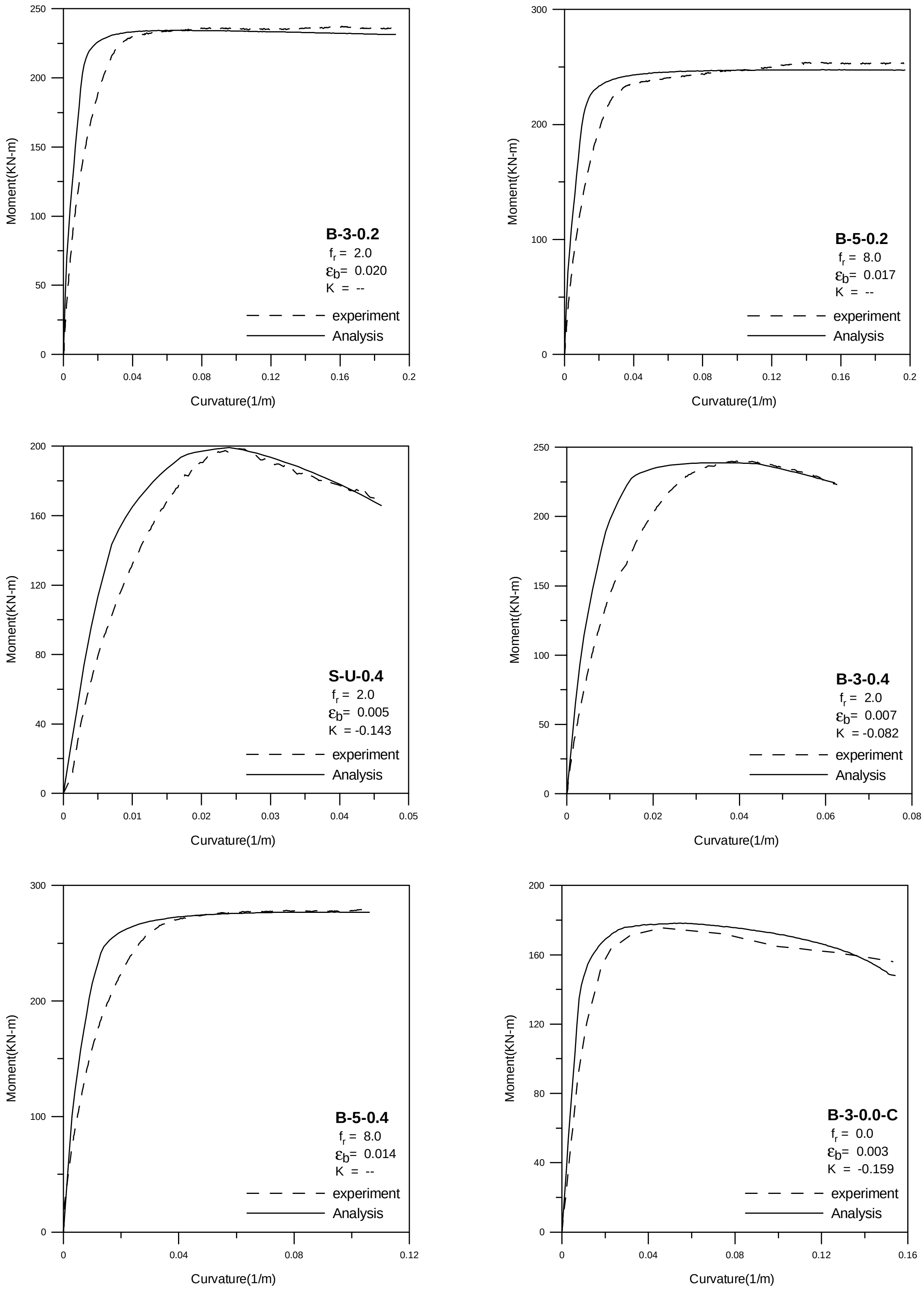

Fig. 11 Moment-curvature relationships (cont.)

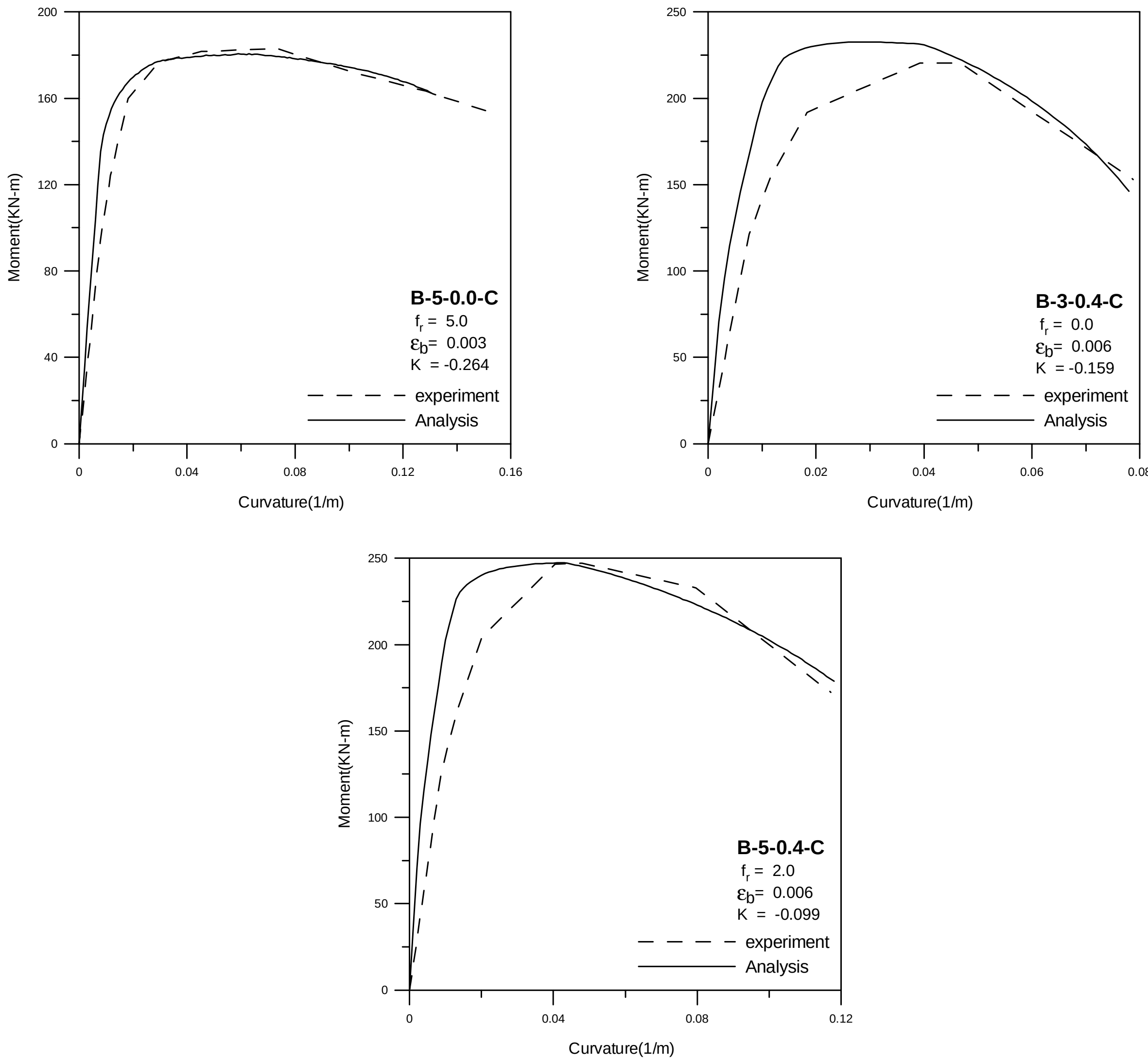

Fig. 11 Moment-curvature relationships (cont.)

The three parameters of each specimen are also shown in Table 1. A well-stiffened specimen has more large values of f_r , ε_b and K . Under low axial load condition, local buckling in steel tube did not easily occur, especially for more densely stiffened specimens. Strength hardening in steel tube occurred in the densely stiffened and low axial load specimen B-5-0.0.

CONCLUSIONS

1. Test results indicate that adequate application of the proposed tie-bar stiffening scheme for square CFTs can effectively improve the confinement of the tube to the concrete core, and substantially delay the occurrence of local buckling of the tube.
2. The strength capacity, ductility and energy absorbing ability of square CFT beam-columns can be improved by the proposed tie-bar stiffening scheme.
3. The analytical model satisfactorily simulates the moment-curvature relationship of all specimens with acceptable accuracy.

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