FLEXURAL BEHAVIOR OF CONCRETE FILLED SQUARE STEEL TUBULAR BEAM-COLUMNS

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SUMMARY

A large number of concrete filled square tubular (square CFT) beam-columns were tested under a cyclic shearing force to clarify the seismic resistant performance. On the other hand, there are few experimental results under a cyclic uniform bending. Due to the large shearing capacity of the CFT beam-columns, the columns with normal proportion fail in flexural mode. Thus, it is important for estimating the capacity and ductility of the CFT columns to investigate their inelastic flexural behavior. The purpose of this study is to improve the understanding of the characteristics of square CFT beam-columns excluding the influence of shearing force. Tests are carried out on eleven CFT specimens subjected to monotonic and cyclic uniform bending under a constant gravity load. Important experimental parameters are; a) width-to-thickness ratio of steel tube, b) axial load ratio and c) deformation history. The strength of filled concrete and the tensile strength of steel tube are approximately 50MPa and 400MPa, respectively. The experimental failure loads and load-deformation hystereses are compared with those of the elasto-plastic analysis based on the proposed stress-strain relationships established for the confined concrete and for the locally-buckling steel tube. The analytical results show good agreement with the test results for all specimens. This implies that the proposed stress-strain relationships for square CFT beam-columns are useful to predict the characteristics of filled concrete and steel tube.

INTRODUCTION

It is known that concrete filled steel tubular (CFT) structures possess the efficient seismic resistant performance by extensive amount of beam-column tests. Most of the beam-column tests in Japan were conducted in the manner that specimens were subjected to shearing force under a constant axial load as shown in Fig.1 (a). To estimate the flexural behavior from this double curvature column, however, there are following uncertain factors.

1) The effect of shearing force is not clear.

2) The extra confinement caused by the rigid loading beam is not clear.

In order to clarify the moment-curvature characteristics, the above unsure effects should be removed. Therefore, we test eleven square CFT columns subjected to uniform bending moment under a constant gravity load as shown in Fig.1 (b). In this paper, we deal with the details and results of the test and investigate the flexural behavior of square CFT beam-columns based on the elasto-plastic analysis.

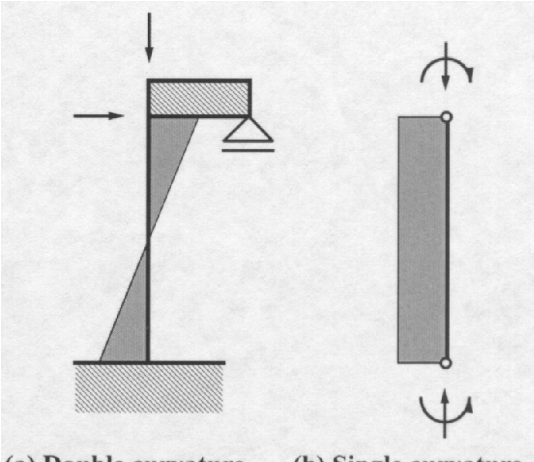

Fig.1: Loading conditions

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UNIFORM BENDING TESTS

All specimens are square columns of the sizes 200 x 200 x 600 mm as shown in Fig.2. Two 22 mm thick plates are welded to both ends of the steel tube for introducing the required axial force and bending moment. Twelve holes of 26 mm diameter in the end plates are used to fasten the specimens to loading beam. A steel tube is fabricated by welding four steel plates at the corners. Residual stress is removed by annealing after welding the end plates. Concrete is placed vertically in three portions.

The bending moment and the constant axial load are applied to the specimens by using an experimental apparatus shown in Fig.3. After the specimen is placed inside the bending frame shown in the figure, it is centered by controlling the locations of the cylindrical seats at the top and bottom. The bending moment is generated by two double-acting hydraulic jacks, of 200 mm stroke, each acting at an eccentric distance Lm from the center of the specimen. The axial load is applied through the 5MN Universal Testing Machine in Kyushu University and keep constant throughout the test.

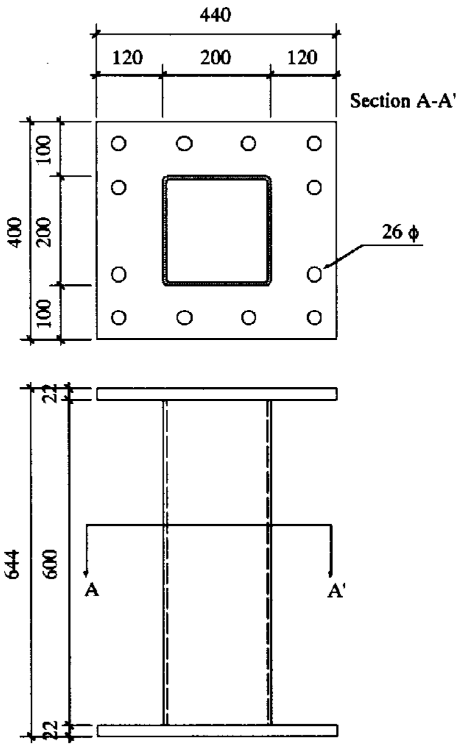


Fig.2: Test specimen
(Unit : mm)

Frames to measure deformations are also illustrated in the figure. An average curvature f through the main gauge at the middle of the specimen is measured by a couple of the transducers. The main gauge length is 400 mm. The lateral deformation at the center of the specimen is measured for the sake of estimating the additional moment introduced by the axial load. Two types of electronic wire strain gauges, single and double at 90-degree, are placed on the tube surface. Eight single gauges are used to set the specimens at the center in the loading frame and eight double gauges are used to observe the yield point in bi-axial stress state of the steel tube. The experimental variables and test results are presented in Table 1. Major parameters investigated in the test include:

a) width-to-thickness ratio of steel tube (B/t = 34, 47, 98)
b) axial load ratio (N/N0 = 0.2, 0.4)
c) deformation histories (monotonic, cyclic)

In the table, ssy is the yield stress of steel tube and csB is the compressive strength of concrete. The yield stress of steel tube is taken in standard tensile test coupons known as type A-1 that are cut from unformed steel and annealed. The compressive strength of concrete is taken by testing cylinders of which diameter are 100 mm and height are 200 mm. The cylinders are pick out from untested specimen. The concrete is normal weight concrete with a slump of 120 mm and a maximum aggregate size of 20 mm. Deformation histories for uniform bending tests are shown in Fig.4, where solid and doted lines show monotonic and cyclic loading patterns, respectively. The deformation history is controlled by f monitored in the loading procedure. In this figure, the average curvature f is multiplied by the depth D of the steel tube as a non-dimensional value. For monotonic loading

Table 1: Experimental variables and test results

Specimen	B (mm)	t (mm)	B/t	$_s\sigma_y$ (MPa)	$_c\sigma_B$ (MPa)	N (kN)	N/N_0	M_{exp} (kN-m)	M_{exp}/M_{cal}
BRA4-6-5-02	200	5.93	33.7	320	47.6	570	0.18	143	0.991
BRA4-6-5-04	200	5.93	33.7	320	47.6	1140	0.36	144	1.001
BRA4-4-5-02	200	4.25	47.1	211	47.6	426	0.17	87.7	1.026
BRA4-4-5-04	200	4.25	47.1	211	47.6	851	0.35	95.7	1.026
BRA4-2-5-02	200	2.04	98.0	253	47.6	380	0.17	62.7	1.001
BRA4-2-5-04	200	2.04	98.0	253	47.6	761	0.34	69.1	0.932
BRA4-6-5-02-C	200	5.93	33.7	320	47.6	570	0.18	147	1.016
BRA4-6-5-04-C	200	5.93	33.7	320	47.6	1140	0.36	142	0.987
BRA4-4-5-04-C	200	4.25	47.1	211	47.6	851	0.35	91.9	0.984
BRA4-2-5-02-C	200	2.04	98.0	253	47.6	380	0.17	63.5	1.015
BRA4-2-5-04-C	200	2.04	98.0	253	47.6	761	0.34	71.5	0.965

B : width of steel tube, t : wall thickness of steel tube, $_s\sigma_y$: yield stress of steel tube, $_c\sigma_B$: strength of concrete cylinder,
N : applied axial load, N_0 : nominal squash load,M_{exp} : maximum experimental momet, M_{cal} : full plastic momet

procedure, the peak value of fD is set as 3.5% which is the maximum applicable deformation by this loading apparatus. The peak fD of the cyclic loading pattern increase stepwise by 0.5%, after three successive cycles up to fD of 2.0%.

In Table 1, all specimens are designated by a seven or eight symbol code such as "BRA4-6-5-02" or "BRA4-2-5-04-C". The names of the specimens added a letter "C" show "Cyclic loaded ones". The letters "B","R" and "A" represent "Bending test", "Rectangular steel tube" and "Annealed steel tube", respectively. The four numerals are tensile stress of steel tube (sst=400MPa), wall-thickness of steel tube (t=6,4,2mm), compressive strength of concrete (csB=50MPa) and axial load ratio (N/N0=0.2, 0.4). N0 is nominal squash load of the CFT specimen and given as

$$N_0 = {}_cA\cdot{}_c\sigma_B + {}_sA\cdot{}_s\sigma_y \quad (1)$$

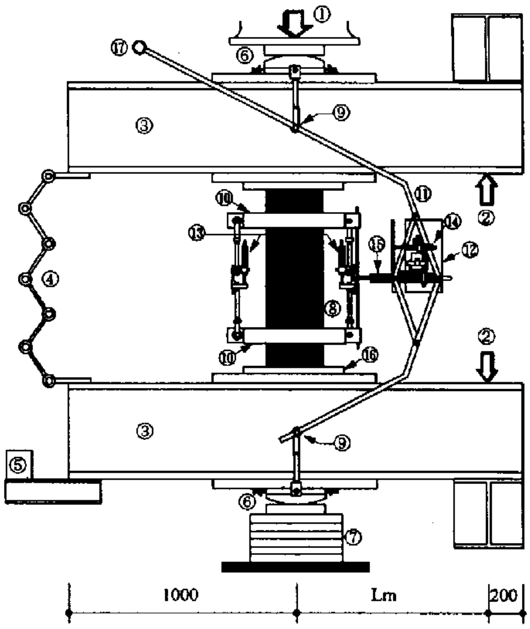
where cA and sA show the sectional areas of concrete and steel tube, respectively. Mexp stands for the experimental maximum bending moment including additional moment introduced by the axial load. The calculated moment Mcal is defined by equation (2).

$$M_{cal} = {}_cM_u + {}_sM_u \quad (2)$$

where cMu and sMu are the full plastic moments of the filled concrete and steel tube and are given by equations (3) and (4), respectively.

$${}_cM_u = \frac{1}{2}\left(d-X_n\right)b\cdot X_n\cdot{}_c\sigma_B \quad (3)$$

$${}_sM_u = Bt(D-t)\,{}_s\sigma_y + 2t\left(d-X_n\right)X_n\cdot{}_s\sigma_y \quad (4)$$



① Universal Testing Machine with Capacity of 5MN
② Hydraulic Jacks
③ Loading Beams
④ Twist Preventer
⑤ Counter Weight
⑥ Cylindrical Seats
⑦ Steel Plates
⑧ Specimen
⑨ Rotation Centers
⑩⑪⑫ Measuring Frame
⑬⑭⑮ Transducers
⑯ End Plate
⑰ Counter Weight for Measuring Frame

Fig.3: Loading apparatus and measuring frames (Unit : mm)

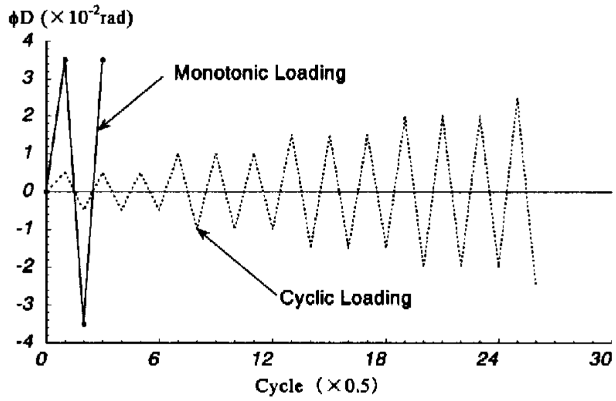


Fig.4: Deformation histories

where B and D are outside width and depth of steel tube, b and d are inner width and depth of steel tube, respectively. Xn is the distance from the extreme compression edge of filled concrete to the neutral axis of the section and is given as

$$X_n = \frac{N + 2td\cdot{}_s\sigma_y}{b\cdot{}_c\sigma_B - 4t\cdot{}_s\sigma_y} \quad (5)$$

where N is applied axial force. In Table 1, it is clear that Mcal can predict Mexp except for the specimens with large width-to-thickness ratio of 98.0.

ANALYTICAL PROCEDURE

To compare with the experimental results, moment-curvature relationships are obtained by the numerical analysis using the proposed stress-strain relationships. The method of the analysis is based on the strain-compatibility solution and the following assumptions are used to generate the moment-curvature curve of square CFT section subjected to uniform bending under a constant gravity load;
1) the section remains in the same shape,
2) a linear strain distribution is assumed,
3) tensile stress of concrete is neglected,
4) a hysteric stress-strain model shown in Fig.5 is used for concrete fiber,
5) a hysteric stress-strain model shown in Fig.6 is used for steel tube fiber.

The compressive stress-strain relationships had been already proposed by authors (Nakahara et al., 1998) for the confined concrete and for the locally-buckling steel tube, which were developed on the bases of the experimental results of axial compressive loaded stub columns conducted in the fifth phase of U.S.-Japan Cooperative Earthquake Research Program. The test results of concentrically loaded stub columns were summarized in our previous paper (Sakino et al., 1998). Since these stress-strain relationship models are for concentrically loaded stub columns, the models have to be modified to take into the effect of strain gradient in column section. The modification of the stress-strain relationships are done on their slope of the descending branch which is reduced by multiplying a coefficient of 2/3 in this paper. These are shown as the envelop curves of compressive stress-strain relationships illustrated in Fig.5 and Fig.6.

Sun stress-strain relationship (Sun, 1991) is used for the unloading and reloading branches of concrete. Meng-Ohi-Takanashi stress-strain relationship (Meng et al., 1992) is used for the unloading and reloading branches of steel tube. In Fig.5, the maximum stress of the filled concrete csp is evaluate as crUÅEcsB, where crU is the reduction factor for introducing the scale effect of concrete and derived from the regression analysis on the test results obtained by Blancks et al (Blancks et al., 1935). Though the scale effect were discussed only for circular columns in their paper, we applied this method by replacing the filled concrete in the square CFT column into the circular columns with the same sectional area. The common point (ceun , csne) is introduced to explain the strength deterioration caused by cyclic loading. The value of csne is given by

$$ {}_c\sigma_{ne} = 0.7\,{}_c\sigma_{un} + 0.3\,{}_c\sigma_{ro} \tag{6} $$

where csun and csro are stresses of unloading point and reloading point, respectively.

In Fig.6, compressive and tensile skeleton parts are defined separately. The feature of the Meng model is to make the opposite skeleton adjustable to fit the test results. According to the plastic strain sep, the target point is

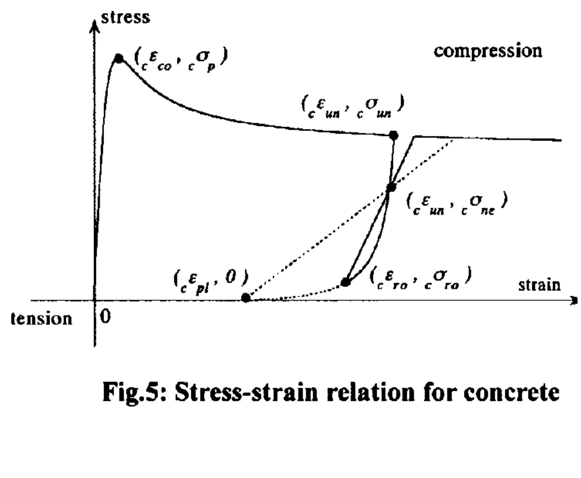


Fig.5: Stress-strain relation for concrete

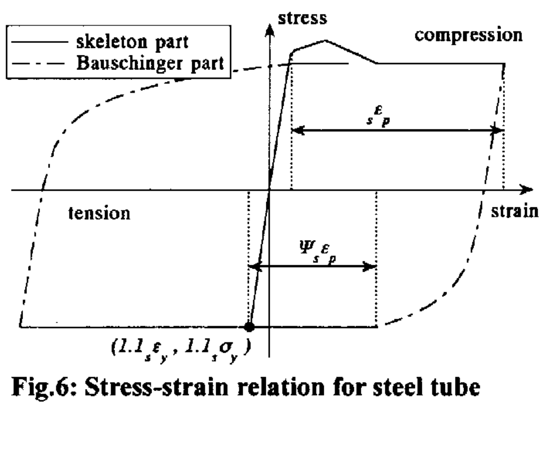
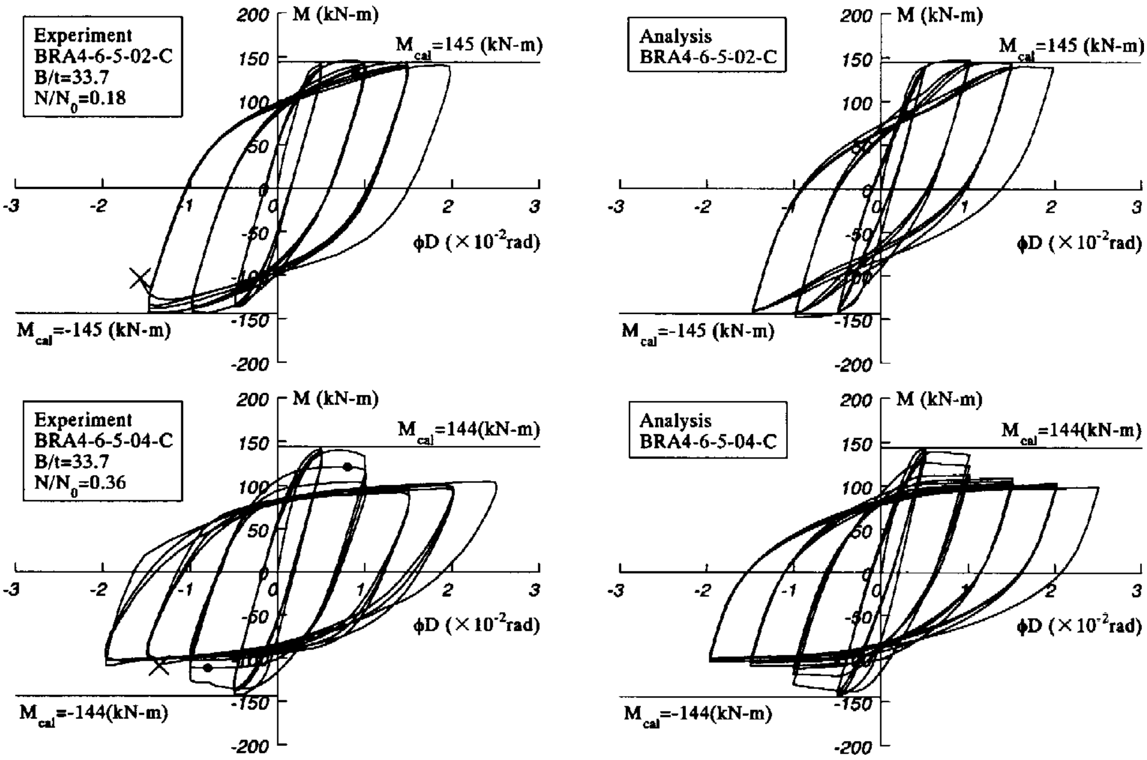
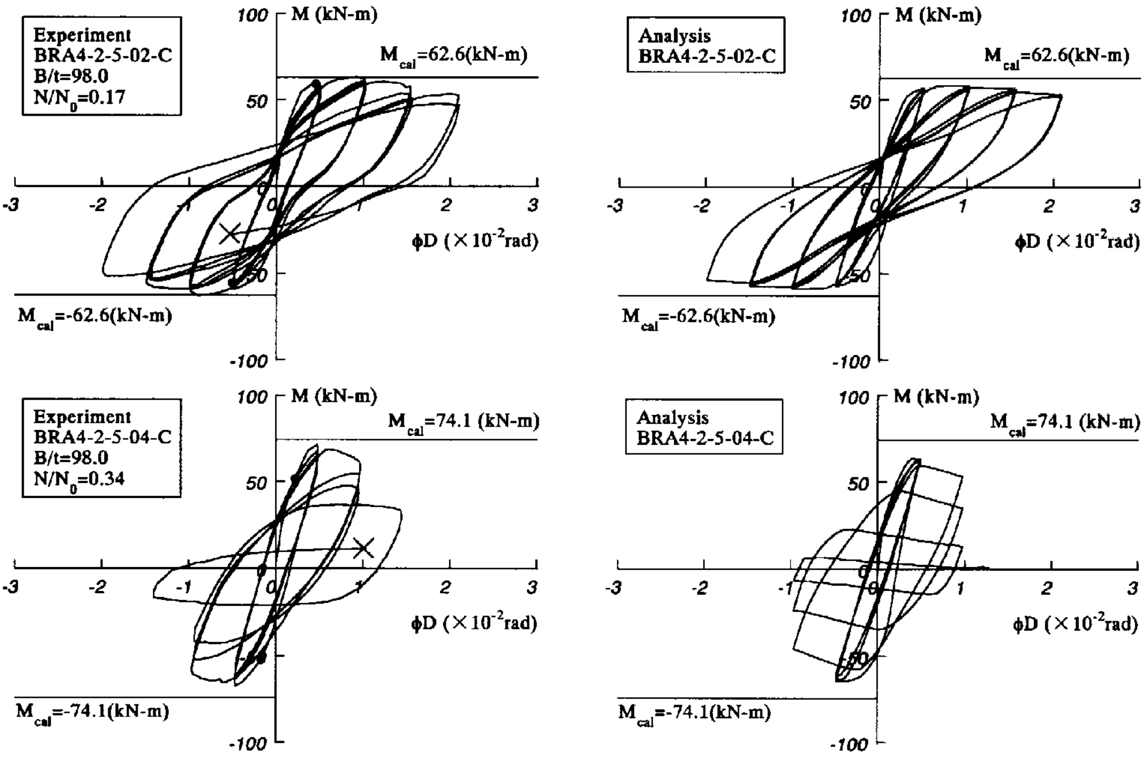


Fig.6: Stress-strain relation for steel tube



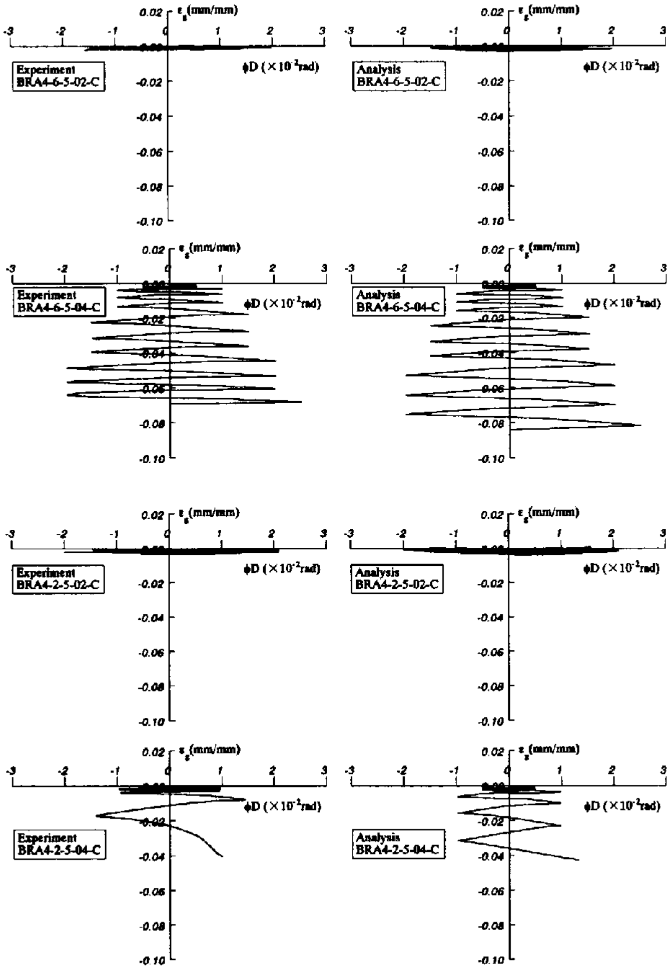
(a) Test **(b) Analysis**

Fig.7: M-ϕD relations for cyclic loaded specimens (B/t=34)



(a) Test **(b) Analysis**

Fig.7: M-ϕD relations for cyclic loaded specimens (B/t=98)



(a) Test **(b) Analysis**

Fig.8: ε_g-ϕD relations for cyclic loaded specimens (B/t=34,98)

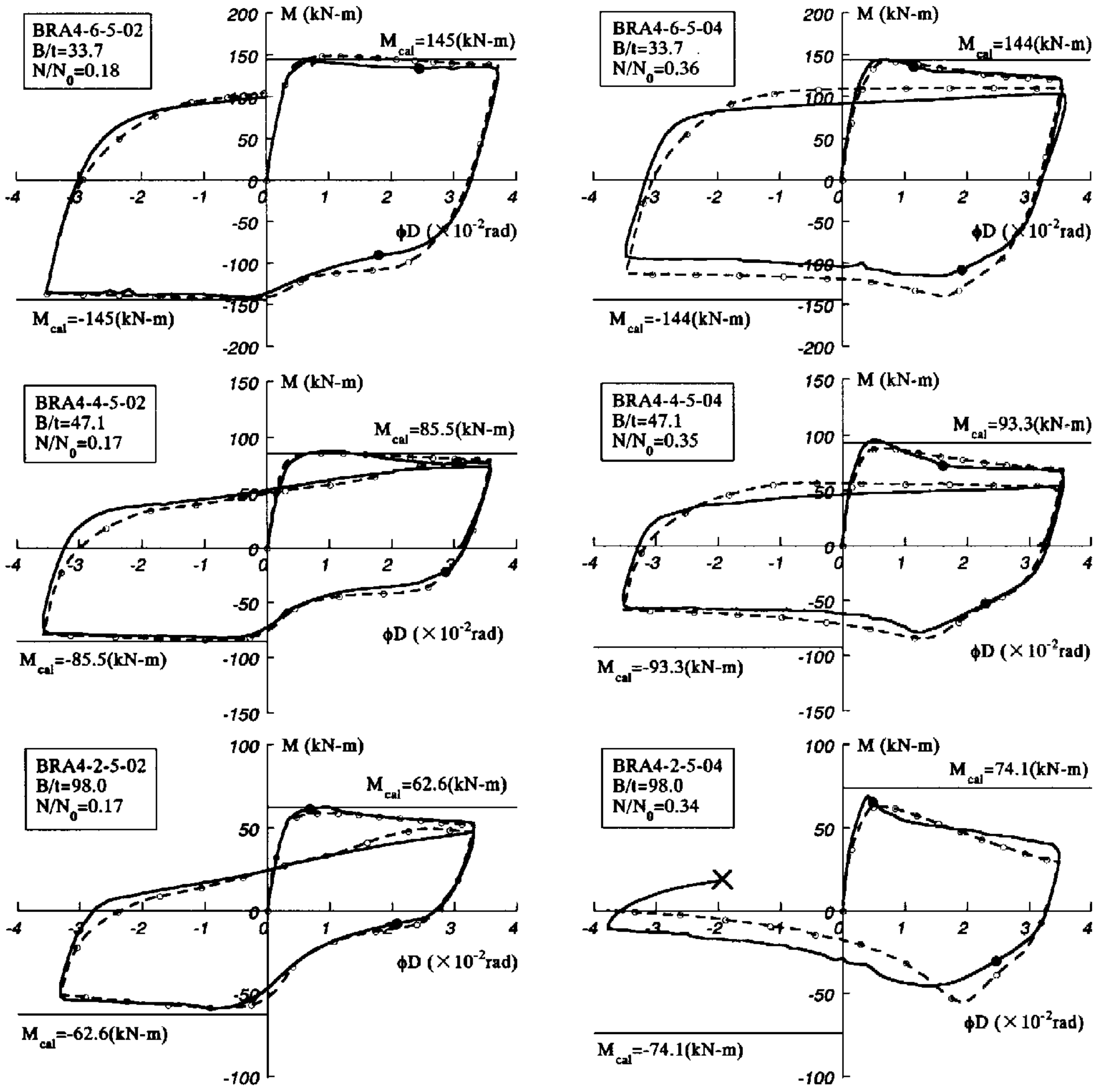
moved by multiplying a coefficient Y. In this case, the values of Y are 0.6 and 0.8 for the analysis of cyclic loaded columns and monotonic loaded columns. The compressive skeleton part is expressed by Nakahara model which be able to follow the behaviors in three different types; strain hardening type; type of buckling at yield point; elastic buckling type. On the other hand, the tensile skeleton is assumed to be an elastic-perfectly plastic model in which the yield point is taken as 1.1ssy. The increasing the yield point is due to the bi-axial tensile stress state of steel tube obtained by the strain measured from double wire strain gauges and von Mises yield criterion. The Bauschinger part of the curve is expressed by the Ramberg-Osgood function; the coefficient defining the round of the curve is of 4.5 to fit the experimental results.

COMPARISON BETWEEN EXPERIMENT AND ANALYSIS

Because we do not have enough space to describe everything about load-deformation curves, we will introduce examples which are excluded the specimen named as " BRA4-4-5-04-C". Relationships between the bending moment M and the non-dimensional curvature fD are shown in Fig.7, and relationships between the strain at the centroid of the section eg and the non-dimensional curvature fD are shown in Fig.8. In these figures, (a) is experimental results and (b) is analytical results of cyclic loaded beam-columns. M-fD and e_g-fD relationships of monotonic loaded specimens are shown in Fig.9 and Fig.10. In these figures, solid lines and doted lines show test results and analytical results, respectively. In figures 7 and 9, M_{cal} is shown in order to be compared with the maximum bending moments obtained by experiment and analysis. The mark "•" stands for that the occurrence of local buckling are observed on the compression flange of steel tube, and the mark "X" shows fracture of the specimen due to breaking at the welded joint or failing in sustaining the axial load. Tests are stopped at the fracture of the specimens.

From the M-fD and e_g-fD relationships, it can be seen that the flexural behavior of square CFT beam-columns is significantly dominated by the axial load ratio. For lower axial loaded specimens, M-fD relations show very stable manners, and the shrinkages of the specimens do not increase in e_g-fD relations. While for higher axial loaded specimens, they show brittle behaviors in the M-fD, and the shrinkages are divergent. For the former, the influence of the B/t ratio is not observed apparently, and the envelop curves of M-fD relations of cyclic loaded specimens are almost same to the behavior of monotonic loaded columns. This implies that the specimens with stable behavior are unaffected by the deformation history. For the latter, the bending moment fall down abruptly with an increase in the B/t ratio of the steel tube. The specimen "BRA4-6-5-04-C" decreases the resisting capacity after peak moment, but strength deteriorations seem to converge after fD of 1.0 x 10^{-2} (rad) while it shrink continuously. The phenomenon is typical for CFT member and is observed also in the another tests under a cyclic shearing force.

Comparing experimental and analytical results, it is observed that the analysis well traces the test results in the M-fD and e_g-fD relationships in the manner of the strength deterioration due to the cyclic loading, and the convergence-divergence of the shrinkages due to the amount of the constant axial load. It is considered that proposed hysteric stress-strain models for filled concrete and locally-buckling steel tube are useful for predicting the cyclic behaviors of square CFT beam-columns tested in the current study.



(a) Test **(b) Analysis**

Fig.9: M-ϕD relations for monotonic loaded specimens

CONCLUSIONS

In total, eleven square CFT beam-columns were tested under uniform bending moment and a constant axial load and an numerical analysis was conducted for predicting the experimental behaviors. The following conclusions were reached from the tests and analysis of the single curvature CFT beam-columns.

1)The maximum bending moment obtained by the test were predicted by the full plastic moment accurately, except for the specimens with large width-to-thickness ratio of 98.0.

2)The flexural behavior of square CFT beam-columns was significantly dominated by the axial load ratio.

3)The ductility was almost unaffected by the width-to-thickness ratio for lower axial load levels (N/No=0.2). On the other hand, the effect was clear under higher axial load levels (N/No=0.4).

4) From the moment-curvature relationships, monotonic loading test results approximated the envelope curve of cyclic test results under lower axial load ratio.

5) Both monotonic and cyclic behavior of the CFT beam-columns were predicted by the elasto-plastic analysis using the proposed stress-strain curve models for the filled concrete and the locally-buckling steel tube.

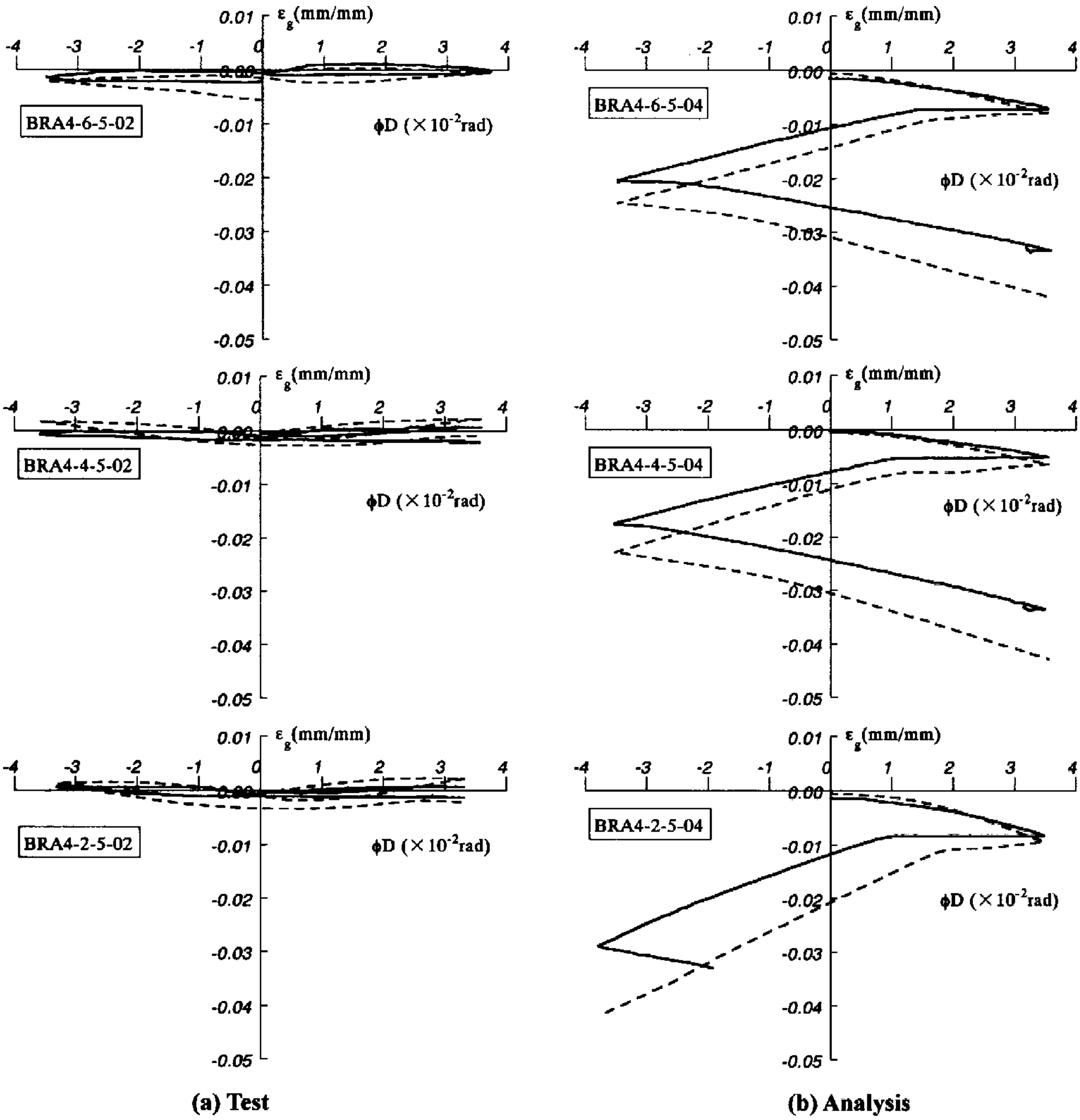


Fig.10: ε_g-ϕD Relations for Monotonic Loaded Specimens

using the proposed stress-strain curve models for the filled concrete and the locally-buckling steel tube.

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