EARTHQUAKE-RESISTING PERFORMANCE OF R/C COLUMNS
CONFINED BY SQUARE STEEL TUBES

(Part 3 Effects of shear span ratio of column)

正方形鋼管横補強鉄筋コンクリート柱の耐震性能

(その3 柱のせん断スパン比が耐震性能に及ぼす影響)

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As the third part of an integrated theoretical and experimental study of earthquake-resisting performance of square tubed concrete columns, this paper deals with effect of shear span ratio (a/D) of columns on the seismic behavior as well as confining effectiveness of square steel tubes. Fourteen reinforced concrete columns having two kinds of shear span ratios of 1.0 and 1.5 were confined by square steel tubes with five kinds of wall thickness. These columns were tested under reversed cyclic lateral force while subjected to constant axial load. It is shown that as confined in square steel tubes having same wall thickness, the column that has shear span ratio of 1.5 behaved in a much more ductile manner than the short column whose shear span ratio is 1.0. To make the short columns under axial load with $n=0.33$ and $n=0.67$ ductile enough, use of thick steel tubes with $B/t=43$ and $B/t=28$ were needed, respectively. Experimental results also indicate that the ultimate drift ratio of the short column with $a/D=1.0$ was about 0.01rad smaller than that of the column having a/D larger than 1.5 when the steel tube with same wall thickness and the concrete having same compressive strength were used.

Keywords: earthquake-resisting performance, square steel tube confinement, width-to-wall thickness ratio of steel tube, shear span ratio, ultimate drift ratio, axial load ratio
耐震性能、正方形鋼管横拘束、鋼管の幅厚比、せん断スパン比、限界部材角、軸力比

1. INTRODUCTION

Confinement of concrete by steel tube is one of the effective methods for upgrading earthquake-resistant capacity of reinforced concrete columns. As compared with conventional transverse hoops or spirals, the steel tube, when used as transverse confiner, has a significant advantage of preventing spalling of the shell concrete, which generally reduces effective area of column section, and hence load-carrying capacity of the column in addition to causing bond-splitting failure. Another advantage of the steel tube is that it can work as form for column and save materials for formwork. Because of these advantages, confinement by steel tube has recently gained its wide use in strengthening existing reinforced concrete columns and in enhancing ductility of high strength concrete (HSC) columns.

In order to establish a rational design method for the reinforced concrete columns confined by square steel tube (referred to as tubed columns in this study), the authors have conducted an integrated experimental and theoretical work to study the following important issues related with seismic behavior of the square tubed columns.

- 1) Seismic performance of the tubed column subjected to high axial compression.
- 2) Earthquake-resisting performance of the tubed column with large longitudinal reinforcement ratio.
- 3) Effect of wall thickness of steel tubes on the earthquake-resisting performance of the tubed column
- 4) Effect of shear span ratio on the seismic performance of the tubed column.
- 5) Estimation method for the ultimate load-carrying and the deformation capacities of the tubed column.

The authors' previous works [1,2] have experimentally studied the first three issues listed above for the tubed columns having shear span ratio (a/D) of 2.0. As the third phase of the integrated study, this paper focuses on the fourth issue with emphasis on the following two aspects: 1) effect of shear span ratio on the seismic behavior of square tubed columns, and 2) effect of shear span ratio on the confinement effectiveness of steel tubes. To achieve these objectives, fourteen 250mm square concrete columns having shear span ratios of 1.5 and 1.0 were fabricated and tested under reversed cyclic lateral force while subjected to constant axial compression. The test columns were confined in square steel tubes with five kinds of B/t ratio ranging from 28 through 117. The other experimental variable in the tests was axial load ratio n with values of 0.33, 0.50, and 0.67.

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Table 1 Properties and main results of the 2D-series columns

Notation (1)	Details of specimen					Measured results			Predicted results							
	f'_c	N	n	t	B/t	V_{max}	R_{max}	R_u	V_{mu}	$\frac{V_{max}}{V_{mu}}$	V_{su}	$\frac{V_{max}}{V_{su}}$	V_{un}	$\frac{V_{max}}{V_{un}}$	$\frac{V_{su}}{V_{mu}}$	$\frac{V_{su}}{V_{un}}$
	(MPa)	(kN)	(=N/No)	(mm)		(kN)	(rad)	(rad)	(kN)		(kN)		(kN)			
T2-33-2D	30.3	632	0.334	2.17	117	364	0.010	0.017	390	0.93	304	1.20	326	1.12	0.78	0.93
T3-33-2D	30.4	634	0.334	3.11	82	406	0.015	0.022	395	1.03	316	1.28	327	1.24	0.80	0.97
T4-33-2D	29.8	621	0.333	4.29	60	390	0.015	0.025	398	0.98	334	1.17	323	1.21	0.84	1.03
T6-33-2D	29.3	611	0.334	6.09	43	396	0.015	0.036	413	0.96	360	1.10	321	1.23	0.87	1.12
T9-33-2D	27.8	579	0.333	9.46	28	410	0.035	0.050	436	0.94	382	1.07	311	1.32	0.88	1.23
T2-50-2D	35.6	1112	0.500	2.17	117	389	0.010	0.013	463	0.84	358	1.09	345	1.13	0.77	1.04
T4-50-2D	36.7	1145	0.499	4.29	60	418	0.015	0.019	500	0.84	397	1.05	351	1.19	0.79	1.13
T3-67-2D	37.2	1550	0.667	3.11	82	447	0.010	0.011	476	0.94	410	1.09	296	1.51	0.86	1.39
T6-67-2D	35.6	1484	0.667	6.09	43	472	0.015	0.017	563	0.84	448	1.05	289	1.63	0.80	1.55
T9-67-2D	36.0	1499	0.666	9.46	28	546	0.020	0.025	676	0.81	480	1.14	291	1.88	0.71	1.65

 f'_c : compressive strength of concrete cylinder

N: axial load applied

B/t: width to wall thickness ratio of steel tube

 V_{max} : measured maximum lateral force R_{max} : drift ratio of column at V_{max} R_u : drift ratio of column at which the lateral load dropped to 95% of the V_{max} V_{mu} : lateral load corresponding to the ultimate moment calculated by authors' method [4] V_{su} : ultimate shear strength calculated by the modified Ohno-Arakawa formula [3] V_{un} : lateral load corresponding to the ultimate moment calculated by the ACI code [5]

2. EXPERIMENT OF TUBED COLUMNS WITH SHEAR SPAN RATIO OF 1.0

2.1 TEST SPECIMENS

In the tubed columns with ordinary proportion, the flexural deformation is predominant and the steel tube provides confinement effect mainly through constraining the lateral expansion of the compressed concrete due to axial load and bending moment. Therefore, as the axial load applied is low, which means the compressed portion in column section is small; theoretically the thin steel tube can be expected to assure enough ductility to the column. This theoretical consideration has been verified in authors' previous work [2]. However, in the case of short columns, because the shear deformation becomes predominant and the column is apt to fail as a whole, the steel tube also has to restrict the lateral dilatation due to widened shear cracks. Consequently, it is reasonable to anticipate that confinement effectiveness of the steel tubes may vary with shear span ratio of the column.

To investigate effect of shear span ratio on the seismic performance of the tubed columns as well as the confinement effectiveness of steel tubes, ten 250x250x500mm prismatic columns were made and tested under reversed cyclic lateral load and constant axial compression. The test columns have shear span ratio of 1.0 and are referred to as 2D-series specimens hereafter simply because the clear height of each column is twice the size of the depth of concrete section. These specimens were confined by square steel tubes having inner width of 250mm and five kinds of wall thickness, 2.17mm through 9.46mm. In the same manner as the test columns reported in Ref. 2, the longitudinal bars in each column consisted of twelve 12.7mm diameter deformed bars (D13, $f_y=318\text{MPa}$) uniformly distributed around the column perimeter to give a steel ratio of 2.44%. Fig.1 shows reinforcement details of the 2D-series columns along with the 3D-series columns described in the next section. It can be seen from Fig. 1 that clearances of 10mm were provided between the loading stub and the steel tube at both ends of the column. This was done for ensuring that the steel tube provides only a confining effect to the concrete, rather than a direct resistance to the axial stress induced by the applied axial load as well as bending moment.

Experimental variables among the 2D-series specimens were: 1) the wall thickness of steel tube, expressed in terms of width-to-wall thickness ratio B/t, and 2) the magnitude of the axial load, represented in terms of axial load ratio $n(=N/A_c f'_c)$, where N is the applied axial load, A_c is the concrete core area, and f'_c is the compressive strength of concrete cylinder. Table 1 lists outlines of the 2D-series specimens along with their measured and predicted main results. The B/t ratio of the used steel tubes varied from 28 through 117, while the axial load ratios had values of 0.33, 0.50 and 0.67. These two experimental variables as well as their varying ranges were the same as adopted for the tubed columns having shear span ratio of 2.0 [2]. The targeted compressive strength of concrete was 30MPa.

Fig.1 Reinforcement details and loading condition of the specimens

The ready-mixed concrete, with a slump of about 180mm and maximum aggregate size of 13mm, were used to fabricate the 2D-series specimens as well as the 3D-series specimens. Compressive strength of concrete cylinder for each column at the testing age is given in Table 1. The mechanical properties of the steels used in this experiment are the same as those used in authors' previous work [2] and hence can be found there.

2.2 LOADING CONDITION AND INSTRUMENTATION

The loading apparatus described in Ref. 2 was used to apply designed loading history to the columns. For each specimen, after the axial load being applied by means of an universal testing machine with 5MN capacity, the reversed cyclic lateral load was applied through one 500kN hydraulic jack to deform the column in a double curvature shape. The designed loading pattern for the lateral load was displacement-controlled type with alternating drift reversals and the loading program comprised the following loading steps; 1) the peak drift ratios, defined as ratio of the lateral displacement Δ to the clear height h ($=500\text{mm}$) of the column, were increased stepwise from 0.005 rad to 0.02 rad with increment of 0.005 rad after three complete cycles at each drift ratio level, and 2) beyond $R=0.02$ rad the peak drift ratios were increased up to 0.05 rad or until failure with increment of 0.005 rad after one cycle at each drift ratio level.

Deflections were measured by displacement transducers (D.T.). In addition to the lateral deflection measurements, the average axial deformation of each specimen was also measured. The measuring instrumentation for the deflections was identical to that used in the tests described in Ref.2. Strains in the longitudinal bars as well as the confining steel tube were measured with strain gauges. All readings from both the strain gauges and the D.T.'s were fed into a computer-controlled data acquisition system, and stored into floppy disk for later analysis.

2.3 OBSERVED BEHAVIOR AND TEST RESULTS

Fig. 2 shows crack patterns of the 2D-series specimens. The crack patterns were observed by cutting and removing steel tube after the tests. Values of drift ratio given in Fig. 2 represent the maximum drift ratios experienced by each specimen. As can be seen from Fig.2, the axial load level greatly influenced the crack patterns of short columns. For the columns subjected to axial load of $n=0.33$, shear cracks were dominant regardless of the wall thickness of steel tubes. As the axial load ratio became higher than 0.50, however, the damages in short columns were primarily caused by splitting cracks rather than shear cracks. The damaged region extended along the column height for all the 2D-series specimens regardless of the axial load level, but use of the thicker steel tube could effectively prevent widening of the cracks, and mitigate damages in the short columns.

The experimental hysteretic lateral load V – drift ratio R relationships and the axial deformation δ – drift ratio R curves for the 2D-series specimens are plotted in Fig. 3. Broken curves in the V – R hysteretic loops as well as the δ – R curves express the testing stages when the confining steel tube contacted with the loading stubs due to accumulation of the axial shortenings. The experimental maximum lateral load $V_{\max}$ as well as the corresponding drift ratio $R_{\max}$ are given in Table 1 along with the ultimate drift ratio R_u , at which the lateral load dropped to 95% of the $V_{\max}$. The measured $V_{\max}$ given in Table 1 represents the average of the maximum lateral forces measured in both loading directions.

In addition, three kinds of lines are superimposed on the measured V – R loop of each specimen. The solid lines with solid squares and open squares represent the theoretical ultimate lateral forces V_{mu} and V_{un} , respectively, and the broken lines express the ultimate shear strength obtained by the modified Ohno-Arakawa formula [3]. The V_{mu} corresponds to the ultimate flexural strength calculated using the method proposed by the first two authors [4], and the V_{un} is based on the ACI ideal flexural strength [5]. As described in Ref. 4, authors' calculation method is different from the ACI method in that the proposed method can take into account not only confinement effect by the steel tube, but also extra confinement effect by the stiff loading stubs. For the specimen T9-67-2D, the theoretical V_{mu} line can't be seen in Fig. 3 only because its value is beyond 600kN.

When subjected to axial load with $n=0.33$, the short column exhibited more ductile behavior as the steel tube became thicker. The specimens T2-33-2D ($B/t=117$), T3-33-2D ($B/t=82$), and T4-33-2D ($B/t=60$) showed large strength degradation from $R=0.02\text{rad}$, 0.025 , and 0.035rad , respectively, but all of them remained gravity sustaining capacity till the end of tests, and the axial shortenings were very small. This means that the strength degradation was mainly induced by the widened shear cracks at large deformation. For the specimens T6-33-2D ($B/t=43$) and T9-33-2D ($B/t=28$), though they didn't develop their ultimate flexural strength, both of them exhibited very stable cyclic responses till the end of tests. Moreover, the longitudinal bars in all of the 2D-series specimens under axial load with $n=0.33$ reached the yield stress both in tension and in compression before and/or at the peak lateral loads. Combining the above observations with the crack patterns, it can be said that the failure pattern for the short tubed columns under axial load with $n=0.33$ is shear failure after flexure yielding.

On the other hand, under high axial load with n larger than 0.50, the short tubed columns exhibited behavior of shear crushing failure. The strength degradation in the specimens confined by steel tubes with $B/t > 43$ was abrupt and accompanied with accumulation of the axial shortenings, but use of thick steel tube with $B/t < 43$ could enhance both ductility and ultimate strength, although the short column couldn't develop its' ultimate flexural strength. No

Fig. 2 Crack patterns of the 2D-series specimens viewed from web side

Fig. 3 The measured cyclic responses of the 2D-series specimens

rebars in these specimens reached the yield stress in tension till the end of tests.

These observations imply that confinement by square steel tube is hard to prevent the concrete short columns with $a/D=1.0$ from shear failure. To make the short columns under axial load with $n=0.33$ and $n=0.67$ ductile enough, use of steel tubes with $B/t=43$ and $B/t=28$ are necessary, respectively.

To predict the ultimate shear strength of the tubed short columns, the modified Ohno-Arakawa formula [3] was adopted in this study. It can be seen from Table 1 (column (13)) that the modified Ohno-Arakawa formula gives a reasonable lower limit prediction of the shear strength for the tubed columns failing in shear although the formula was originally proposed for the columns confined by conventional hoops. The ratio of the V_{max} and the V_{su} had a mean value of 1.12 and a standard deviation of 0.07. The measured and calculated results shown in Fig. 3 and Table 1 also indicate the importance of taking into account confinement effect of the steel tube on the ultimate flexural strength as predicting failure mode for the tubed columns. As shown in Table 1 (columns (16) and (17)), the calculated flexural strengths for unconfined columns are much smaller than the shear strengths, which means that all of the short columns are supposed to fail in flexure rather than in shear from the viewpoint of ultimate strength. This is obviously contrary to the observed results.

Table 2 Properties and main results of the 3D-series columns

Notation	Details of specimen					Measured results			Predicted results							
	f'_c (MPa)	N (kN)	n (=N/No)	t (mm)	B/t	V_{max} (kN)	R_{max} (rad)	R_u (rad)	V_{mu} (kN)	V_{max} V_{mu}	V_{su} (kN)	$\frac{V_{max}}{V_{su}}$ $\frac{V_{mu}}{V_{su}}$	V_{un} (kN)	$\frac{V_{max}}{V_{un}}$ $\frac{V_{mu}}{V_{un}}$	$\frac{V_{su}}{V_{mu}}$	$\frac{V_{su}}{V_{un}}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
T3-33-3D	34.6	721	0.333	3.11	82	300	0.014	0.032	284	1.06	283	1.06	235	1.28	1.00	1.20
T4-33-3D	34.9	728	0.334	4.29	60	298	0.028	0.042	291	1.02	305	0.98	236	1.26	1.05	1.29
T6-33-3D	34.8	726	0.334	6.09	43	311	0.035	0.050	302	1.03	333	0.93	236	1.32	1.10	1.41
T9-67-3D	34.8	726	0.334	9.46	28	314	0.035	0.060	326	0.96	363	0.87	236	1.33	1.11	1.54

3. EXPERIMENT OF TUBED COLUMNS WITH SHEAR SPAN RATIO OF 1.5

3.1 TEST SPECIMENS

In order to obtain more experimental information about the effect of shear span ratio on the seismic behavior of tubed columns, four specimens with shear span ratio of 1.5 were fabricated and tested under the same loading condition as of the 2D-series specimens. As shown in Fig.1, the four specimens had the same 250mm x250mm square section as the 2D-series specimens, but a different column height that was 750mm. Since the column height was three times the size of the section depth, these specimens are referred to as 3D-series specimens. The experimental variable among the 3D-series specimens was wall thickness of the steel tubes only. All of the four specimens were subjected to axial load with a designed axial load ratio $n=0.33$. The testing conditions for the 3D-series columns, such as materials properties, loading apparatus as well as instrumentation, were the same as for the 2D-series specimens. Details of the 3D-series specimens are tabulated in Table 2.

3.2 OBSERVED BEHAVIOR AND TEST RESULTS

The crack patterns, observed after the tests, are shown in Fig. 4. As can be seen from Fig. 4, for the 3D-series specimens, the flexural cracks were dominant within the end regions with length of less than 1.0D. Like the columns with shear span ratio of 2.0 (4D-series specimens) [2], range of the damaged regions in the 3D-series specimens was shortened as the wall thickness increased. Substantial damage was observed at the upper end of the specimen T3-33-3D. This was due to rupture of the welding portion of steel tube that occurred during the push loading to drift ratio $R=0.045$ rad.

The experimental V-R as well as the δ -R cyclic responses for the 3D-series specimens are shown in Fig. 5. The solid and broken lines superimposed on Fig. 5 have the same meaning as those in Fig. 3. As apparent in Fig. 5, all specimens showed very stable performance until drift ratio of 0.04rad and reached their theoretical ultimate flexural strength V_{mu} . While abrupt strength degradation occurred in the specimen T3-33-3D at $R=0.045$ rad, this degradation was caused by

Fig. 4 Crack patterns of the 3D-series specimens viewed from web side

Fig. 5 The measured cyclic responses of the 3D-series specimens

Fig. 6 Comparison of the envelope curves (effect of shear span ratio)

Fig. 7 Comparison of the ultimate drift ratio (effect of shear span ratio)

rupture of the welding portion of the steel tube, and the column still maintain its axial load-carrying capacity till the end of test. Combining these observations with the crack patterns shown in Fig. 4, it can be said that confinement provided by thin steel tube with $B/t=82$ could make the column having shear span ratio of 1.5 fail in flexure and assure the column sufficient ductility.

It also can be seen from Table 2 (Column (11)) that the theoretical ultimate flexural strength V_{mu} obtained by authors' method agreed satisfactorily well with the measured maximum load V_{max} . The ratio of the V_{max} and the V_{mu} had a mean value of 1.02 and a standard deviation of 0.03. On the other hand, ignorance of confinement effect of the steel tube underestimated the ultimate lateral load of the tubed columns failing in flexure by 26% - 33%.

4. DISCUSSION ON EFFECT OF SHEAR SPAN RATIO

To assess the influence of shear span ratio on the seismic performance of tubed columns, envelope curves of the V-R responses in the push direction are compared in Fig. 6 for the specimens subjected to axial load with $n = 0.33$. For comparison, the envelope curves of the 4D-series specimens reported in Ref. 2 are also plotted in Fig. 6. As shown in Fig. 6, although the axial load was not high, thick steel tube with B/t ratio less than 43 was needed to assure sufficient ductility to the short columns having $a/D = 1.0$. On the other hand, thin steel tube with $B/t = 82$ could make the columns with a/D larger than 1.5 sufficiently ductile.

Effects of the shear span ratio can be more clearly observed from comparisons shown in Fig. 7. Fig. 7 displays relationships between the measured ultimate drift ratio R_u and the B/t ratio of steel tubes. As mentioned in the footnote for Table 1, R_u , an index of deformation capacity of a tubed column, is defined as the drift ratio at which the lateral force of the column dropped to 95% of its measured maximum lateral load V_{max} . It can be seen from Fig. 7 that for specimens made of concrete with designed compressive strength $f_c = 30\text{MPa}$ and subjected to axial load with $n = 0.33$, decreasing shear span ratio of the column from 1.5 to 1.0 reduced the ultimate drift ratio of tubed columns by 0.011rad on average, while difference between the ultimate drift ratios of the 4D-series columns and of the 3D-series columns is very little. Also true is this observation for the columns subjected to axial load with $n = 0.67$ (see Fig. 7(a) and (c)). The ultimate drift ratio of the specimen T4-67-4D deviated from the trend line of the 4D-series specimens by about 0.012rad, because the concrete used in the specimen T4-67-4D had much higher compressive strength ($f_c = 51.1\text{MPa}$) than the concrete ($f_c = 33.0\text{MPa}$) used in the other 4D-series specimens. For the specimens subjected to axial load with $n = 0.50$, the ultimate drift ratios of the 4D-series columns exhibited smaller values than those of the 2D-series columns (see Fig. 7(b)). This phenomenon may be attributed to the use of high strength concrete in the 4D-series specimens, whose concrete strength was 49.1MPa and was about 13.0MPa higher than that of the concrete used in the 2D-series specimens. Therefore, if taking into account the effect of concrete strength on the ultimate deformation capacity, the ultimate drift ratios of the 4D-series specimens under axial load of $n = 0.50$ would show higher values than the 2D-series columns.

The above observations imply that shear span ratio influences the seismic behavior of the tubed columns mainly through changing failure pattern of the columns. For the tubed columns that had shear span ratio larger than 1.5, influence of shear span ratio on both seismic performance of the columns and confinement effectiveness of the steel tubes is very, if any, little, since the columns all failed in flexure. On the other hand, although very thick steel tube with $B/t=28$ had been used, all of the short tubed columns having $a/D=1.0$ failed in shear, which consequently reduced the ultimate drift ratio by about 0.01rad as compared with the longer columns when the steel tubes with the same wall thickness were used.

5 CONCLUDING REMARKS

As the third phase of a systematic study on the seismic performance of reinforced concrete columns confined by square steel tube, fourteen square columns having shear span ratios of 1.0 and 1.5 were fabricated and tested under reversed cyclic lateral load and constant axial compression. These specimens were confined by square steel tubes with five kinds of width-to-wall thickness ratios, 28 through 117, to provide experimental information about effect of shear span ratio on the seismic performance of the square tubed columns. The following conclusions can be drawn from the test results.

1) While confinement by the square steel tubes seems to be hard to prevent the concrete short columns with $a/D=1.0$ from shear failure, seismic response of the short column became ductile as the steel tubes became thicker. To make the short columns under axial load with $n=0.33$ and $n=0.67$ ductile enough, use of steel tubes with $B/t=43$ and $B/t=28$ are needed, respectively, as compared with respective necessary B/t ratios of 83 and 43 for the columns having a/D larger than 1.5

2) Shear span ratio of column influences the seismic behavior of the tubed columns mainly through changing their failure mode. For the tubed columns that had shear span ratio larger than 1.5, influence of shear span ratio on both seismic performance of the columns and confinement effectiveness of the steel tubes was very, if any, little, since the columns all failed in flexure. On the other hand, although very thick steel tube with $B/t=28$ had been used, all of the short tubed columns having $a/D=1.0$ failed in shear, which consequently reduced the ultimate drift ratio by about 0.01rad as compared with the longer columns when the steel tubes with the same wall thickness were used

3) For the short tubed columns that failed in shear, the modified Ohno-Arakawa formula originally proposed for the reinforced concrete columns confined by conventional hoops can give a reasonable lower limit prediction of the shear strength of them. The ratio of the measured ultimate loads V_{max} and the calculated shear strengths V_{su} of the 2D-series specimens had a mean value of 1.12 and a standard deviation of 0.07.

4) The confinement model and calculation method proposed by the first two authors in Ref. 4 could quite accurately predict the ultimate flexural strength of the tubed columns failing in flexure. The ratio of the measured ultimate loads V_{max} and the theoretical flexural strength V_{mu} had a mean value of 1.02 and a standard deviation of 0.03 for the 3D-series columns. Therefore, combining the authors' method [4] with the modified Ohno-Arakawa formula [3] would enable one to predict failure mode, in terms of flexure failure and shear failure, of square tubed columns.

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和文要約

1. 序

鋼管横補強柱の設計を合理的に行うためには以下の問題点を明らかにする必要がある。

- 1) 高軸力を受ける鋼管横補強柱の耐震性能
- 2) 大量の主筋を有する鋼管横補強柱の耐震性能
- 3) 鋼管横補強柱の耐震性能に及ぼす鋼管板厚の影響
- 4) 鋼管横補強柱の耐震性能に及ぼすせん断スパン比 a/D の影響
- 5) 鋼管横補強柱の終局耐力および変形能力の評価法

第1と第2報においては a/D が 2.0 の柱を用いて、上述した問題の内、其々1)と2)及び3)の問題について検討した。本報においては4)の問題を取上げ、14体の試験体を用いて実験的に検討する。

2. せん断スパン比 1.0 の鋼管横補強柱の実験

2.1 実験概要

通常の形状比を有する柱の場合、曲げ性状が支配的であることから、鋼管の拘束効果は曲げ圧縮部の横方向膨らみを拘束する事により生じる。このことについては、本報第2報において明らかにしている。しかしながら、短柱になるとせん断性状が支配的となり、柱の破壊現象が柱全長に渡って生じ易くなるため、鋼管は柱のせん断ひび割れの進展による横方向膨らみを拘束する必要がある。以上より、鋼管の拘束効果は柱の a/D によって異なる事が予想される。

鋼管横補強柱の耐震性能に及ぼす a/D の影響を明らかにするために、250mm 角の正方形柱 10 体（以下 2D シリーズと呼ぶ）を製作し、一定軸力下における繰返し曲げせん断実験を行った。主筋比は本論第2報に用いた試験体と同じで、2.44%である。試験体の形状および配筋詳細を、図1に示す。2D シリーズ実験における実験変数は、鋼管板厚と鋼管の外幅の比である鋼管幅厚比（5種類、 $B/t=28 \sim 118$ ）と軸力比（ $n=0.33, 0.50, 0.67$ ）である。表1に、2D シリーズの試験体の性質を、耐力・変形に関する主な実験および解析結果とともに示す。各試験体のコンクリート圧縮強度を表1に示す。鋼管の力学的性質については第2報を参照されたい。試験体には逆対称の繰返し水平変形を与え、加力・測定方法は前報²⁾と同じである。

2.2 実験結果

図2に2D シリーズ試験体のひび割れ図を示す。軸力比 n が 0.33 の試験体は、鋼管板厚には関係なくせん断ひび割れが支配的である。しかしながら、 n が 0.50 より大きくなると、せん断ひび割れよりもむしろ縦割裂ひび割れが顕著となる。

実験より得られた水平力 V と部材角 R の関係、軸方向伸縮量 δ と R の関係を図3に示す。実験中に観察された最大水平力 V_{max} とその時の部材角 R_{max} 、水平耐力が最大耐力の95%に低下した時の限界部材角 R_u の値を表1に示している。V-R 関係図には、耐力計算値を示す3本の線が示されている。■を結んだ線は著者等により提案⁴⁾された曲げ耐力計算値 V_{mu} を、□を結んだ線は ACI 規準⁵⁾ による曲げ耐力計算値 V_{un} を、点線は修正大野・荒川式³⁾ によるせん断耐力計算値 V_{su} を示している。著者等の曲げ耐力算定式は、鋼管の拘束効果のみならずスタブの拘束による耐力上昇も考慮に入れている。

n が 0.33 の短柱は鋼管板厚が厚くなるほど靱性に富む挙動を示す。鋼管幅厚比が大きい（117、82、60）場合は、部材角が大き（それぞれ、0.02rad, 0.025rad, 0.035rad）なると耐力劣化が顕著になるが、軸力支持能力は最後まで保持していた。鋼管の幅厚比が小さい（43、28）場合は、拘束効果を考慮に入れた曲げ耐力には達していないが、大変形時まで非常に安定した挙動を示す。なお、すべての試験体において、主筋が引張り圧縮とも最大耐力時あるいはそれ以前に降伏ひずみに達していた。以上のことから試験体の破壊モードは、曲げ降伏先行後のせん断破壊であるということが出来る。

一方、 n が 0.50 以上の高軸力を受ける短柱の場合は、せん断圧縮破壊を生じた。幅厚比が 43 以上の鋼管で横補強された柱の場合は、耐力の低下は急激で、軸方向縮みの累積を伴った。一方、幅厚比 43 以下の鋼管を用いれば、拘束効果を考慮に入れた曲げ耐力

を発揮するまでには至らないが、変形能力及び終局耐力を顕著に上昇させることが出来た。柱の主筋は引張降伏ひずみに達することとはなかった。

また、表1より分かるように、修正大野・荒川式は、鋼管横補強柱のせん断耐力実験値の下限値を比較的精度よく算定できる。

3. せん断スパン比 1.5 の鋼管横補強柱の実験

a/D が 1.5 の試験体 4 体（3D シリーズと呼ぶ）について、2D シリーズと同様な実験を行った。実験変数は鋼管板厚のみで、 n はすべて 0.33 の一定値とした。材料の性質等は前節のそれと同じである。図4に3D シリーズ試験体のひび割れ図を示す。曲げひび割れが支配的で、ひび割れは材端において長さ 1.0D より小さい範囲に生じている。 a/D が 2.0 の試験体²⁾ の場合と同様に、損傷を受けている領域の長さは、鋼管板厚が厚くなるほど短くなる。

実験より得られた V-R と δ -R 関係を図5に示す。実線および破線の意味は図3のそれと同じである。図5より明らかなように、全試験体とも $R=0.04\text{rad}$ に至るまで安定した挙動を示しており、曲げ耐力 V_{mu} を発揮している。T3・33・3D 試験体のみは $R=0.045\text{rad}$ 時に耐力劣化をおこしているが、これは鋼管溶接部にキレツが生じたためである。以上述べたことより、 a/D が 1.5 で n が 0.33 の場合は、幅厚比が 82 の薄肉鋼管による横補強でも、柱の破壊モードは曲げ破壊となり柱に十分な変形能力を与えることが出来る。

4. せん断スパン比の影響に関する考察

鋼管横補強柱の耐震性能に及ぼす a/D の影響を評価するために、V-R 関係の正荷重時における包絡線の比較を、 n が 0.33 の場合について図6に示す。比較のため a/D が 2.0 の試験体²⁾ の実験結果についても示している。図6から分るように、 a/D が 1.0 の場合は、 n がそれほど大きくない場合でも、柱に十分な靱性を期待するためには幅厚比が 43 以下の鋼管で横補強する必要がある。一方、 a/D が 1.5 より大きい柱の場合は、幅厚比が 82 の薄肉鋼管による横補強でも、十分な靱性を期待することが出来る。

a/D の影響は図7においても、より顕著に見ることが出来る。図7は、前節で述べた限界部材角 R_u の実験値と幅厚比の関係を示したものである。図7aより、コンクリートの圧縮強度が 30MPa で n が 0.33 の柱の場合、 a/D が 1.0 の短柱の R_u は、1.5 と 2.0 の柱に比較すると、いずれの幅厚比の場合も、平均的にみて 0.011rad 程度小さいが、 a/D が 1.5 と 2.0 の柱では R_u の値は殆ど変わらないことが分る。 n が 0.67 の場合についても同様のことが言える（図7b参照）。

5. 結論

本研究においては、 a/D が 1.0 と 1.5 の柱試験体 14 体について、繰返し曲げせん断実験を行い、以下の結論が得られた。

- 1) 本研究の実験条件の下では、 a/D が 1.0 の短柱のせん断破壊を鋼管横補強により完全に防ぐことは困難であったが、厚肉の鋼管を用いれば短柱の靱性を改善することはできた。靱性改善のために必要な鋼管の幅厚比は軸力比 n に依存し、 n が 0.33 と 0.67 の場合の必要幅厚比はそれぞれ 43、28 であった。
- 2) 鋼管横補強柱の耐震性能に及ぼす a/D の影響は、破壊モードの違う場合に顕著となる。 a/D が 1.5 以上の鋼管横補強柱は、いずれも曲げ破壊であったので、 a/D の影響は非常に小さい。 a/D が 1.0 の柱は、幅厚比が 28 の厚肉鋼管を用いてもせん断破壊を防ぐことは出来なかったため、 a/D が 1.5 以上の曲げ破壊する柱に比較すると、他の条件が同じであれば柱の R_u は約 0.01rad 程小さくなる。
- 3) せん断破壊する鋼管横補強柱のせん断耐力下限値は、修正大野・荒川式により推定できる。せん断耐力実験値と計算値の比は平均値が 1.12、標準偏差が 0.07 であった。
- 4) 曲げ破壊する鋼管横補強柱の耐力は、著者等により提案⁴⁾された拘束効果を考慮に入れた算定式により精度よく推定できる。 a/D が 1.5 の試験体の耐力実験値と計算値の比は平均値が 1.02、標準偏差が 0.03 であった。従って、著者等により提案された曲げ破壊する柱の水平耐力算定式と修正大野・荒川式によるせん断耐力算定式を用いれば鋼管横補強柱の破壊モードの推定が可能となる。

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