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Investigation of the performance of externally collared RC short columns via aspect ratio

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Abstract. This paper presents the experimental study of nine pieces of reinforced concrete (RC) short columns. RC short columns were tested with cyclic loading with displacement control under the influence of constant axial load with load index of 0.2. Three columns within the tested nine columns are reference columns which have the details of the reinforcement given in the modern regulations and six of them are 150 mm and 100 mm externally collared columns. In addition to the parameter of the collar spacing, aspect ratio (αs=2-1.5-1) is also considered as a parameter. The data obtained from experimental results have shown that externally collar contributes significantly to increasing the shear resistance of RC short columns and limiting the shear dominant behavior. It has been observed that the effectiveness of the externally collar increases with the decrease of the aspect ratio.

Keywords: RC short column; aspect ratio; collar; cyclic loading; stiffness; ductility; energy dissipation

1. Introduction

The shear dominant behavior of an RC carrier system component reveals that safety factor must be greater in the design of this component because the failure of the component displaying shear dominant behavior occurs instantaneously. This sudden failure causes a significant increase in the risk of loss of life and property. Considering the vertical carrier system elements, this critical shear behavior is most clearly seen in short columns. The short columns cannot meet the demand for floor displacement caused by the lateral load effect due to their shorter effective length than the other floor columns. This leads to the necessity of meeting the demand for displacement with the strength, so the shear dominant behavior develops. It is very important that the elements that display or have to display shear dominant behavior are designed and dimensioned to have sufficient strength, sufficient stiffness and/or ductility. The requirement given in almost all regulation in the design of RC short columns is that the reinforcement conditions of the junction region are maintained throughout the entire column. However, it is expressed in FEMA274 (1997) that this detailing is not sufficient in meeting the demand that a short column is subjected to and Japan-Kobe earthquake is given as an example for this situation. Therefore, it is seen as a need to search for new applications that could create alternatives to the details of the reinforcement in modern regulations.

In order to improve the shear behaviors of the columns from the outside applications, the wrapping applications of reinforced concrete jacketing, steel jacketing, FRP and its derivation came to the fore. Bett *et al.* (1988) tested three short columns with the aspect ratio of 1.5 under constant axial load and reversible lateral load. Shotcrete and RC jacketing were applied to the two columns. Rodriguez and Park (1994) applied RC jacketing with 100 mm thickness to full-scale columns with 350×350 cross-sectional dimensions. These two studies have resulted that the applications of RC jacketing increase the strength and stiffness of the columns. In the second study, it was stated that this rehabilitation technique is laborious and time-consuming. Priestley *et al.* (1994a, 1994b) examined the effectiveness of steel jacketing in strengthening columns with insufficient shear strength. In this study, 8 circular columns and 6 rectangular columns with the aspect ratio of 1.5 and 2 were tested. It has been stated that the steel jacketing very effectively increases the shear strength and bending ductility of the columns with insufficient shear strength. Xiao and Wu (2003) suggested the use of a partially rigid steel jacketing for the rehabilitation of square and rectangular columns. Zhou and Liu (2010) investigated a total of eight columns, 3 of which are placed in the circular pipe and the other 3 of which are placed in the square pipe. Liu *et al.* (2011) tested ten short column samples, two of which were control specimens and eight of which were strengthened by steel collar. The methods applied in the studies have shown that there is an increase in the stiffness and shear strength of short columns and there is also an increase in ductility and energy absorption values. Promis *et al.* (2009) conducted studies on seven samples reinforced with CFRP and GFRP wraps which were applied continuously or discontinuously. While fully wrapped specimens show rigid behavior, it has been reported that many cracks occur in the interstices between the bands

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which were wrapped with interval wrapping. It has been stated that the elastic energy capacity of the fully wrapped columns is considerably higher than the others, and 3-fold wrapped sample at frequent intervals has more ductility and energy absorption capacity than the others. Chen *et al.* (2010) carried out both experimental and theoretical investigations of RC short column subjected to horizontal forces under constant compressive loading. Eight column with section of 400×400 mm, height 400 mm and 500 mm and different type hoop were tested as an experimentally. As a result, the promotion of strength of specimens with closed type is evident only when the concrete compression strength less than 17.64 MPa, but exceed the stress formal structure behavior isn't evident. In addition to this, it stated that specimens with closed type hoop revealed better resistance in lateral drift than with conventional hoop type.

On the other hand, some researchers (Bikce 2011, Kocak 2013, Cagatay *et al*. 2010) have made case-studies to evaluate the effect of shear critical columns on the behavior of structural systems. In these studies, the effect of infill walls on the behavior of short columns was investigated. Cagatay stated that the addition of infill wall of 20% of the span length is recommended to reduce the short column effect by about 67%. Bikce emphasized that in cases where the width of the band window is less than 60% of the internal width of the frame, almost the total shear force of the short columns is transferred to the reinforced concrete infill which behaves like a coupled shear wall. As a common statement in these studies, it is stated that the shear critical elements are formed due to architectural purposes.

2. Research significance

Conducting short column experiments is very difficult. In order to be able to capture the behavior, the aspect ratio needs to be at a very low level. This situation leads to a considerable increase in lateral load levels. Previous studies have shown that the applications of steel jacketing and collar contribute to short column behavior.

However, in a large amount of these studies, faulty columns are referred to as a control sample. In this study, reference columns are perfect elements that are designed and dimensioned according to the requirements of modern regulations. Therefore, it is investigated whether the proposed method can create alternatives as a rehabilitation method to the design requirements in current regulations. Full-scale columns were used in the study. The range of the proposed collar application was changed and the winding effect was examined. In addition to this, by changing the aspect ratio, the effectiveness of shear dominant behavior was investigated and the effects of the methods on the behavior were revealed.

3. Test columns and experimental setup

3.1 Specimen details

Nine columns with a cross-sectional dimension of 400×400 were used as a test column. Three of these are reference column reinforced according to winding region conditions. The shear reinforcements of the reference columns are Ø10/100 mm. The shear reinforcements of the other six collared columns are Ø10/150 mm. Collar spacing of three of these collared columns is s_2=150 mm, and collar spacing for the other three columns is s_2=100 mm. Longitudinal reinforcement of all columns is 8Ø16 and the material properties used in production are the same. The lateral load was applied to the test columns with levels of 800 mm, 600 mm and 400 mm from the foundation. Therefore, the aspect ratios (M/Vd) of the test columns are 2, 1, 5 and 1 respectively. Thus, in the columns tested, the effectiveness of the application was evaluated both in terms of spacing and aspect ratio. The details of the column reinforcement, the representation of the collar application and an image of the experimental setup are given in Figs. 1-3. The column notation is made as SCα - X - s(d):

SC: Short Column

α: is the lateral load height from upper limit of the foundation which defines aspect ratio

X: Suggested method or reference

Ref: Reference

C: Collar

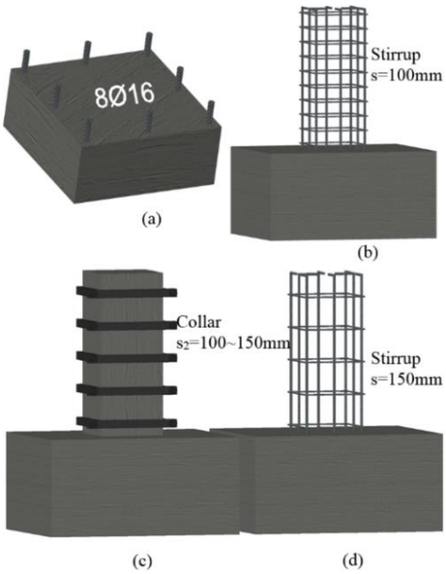

Fig. 1(a) Cross section (b) Ref. Column (c) Collared Column

3.2 Test variables

Aspect ratio is the most important factor affecting the orientation of the columns to shear behavior. In addition to this, the columns must have sufficient shear strength in order to exhibit the expected performance in shear behavior. In shear strength, reinforcement details play an important role as well as the strength of concrete.

In this study, aspect ratio and details of the shear reinforcement ratio were considered as variable parameters to investigate the performance of RC short columns. (See 3.1). Thus, the failure pattern of RC columns can be significantly affected by the shear-span ratio (Jin *et al*. 2017). Apart from the variable parameters, the concrete strengths of the columns, the characteristic properties and

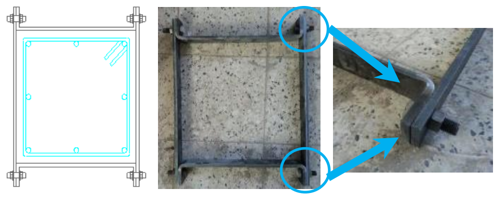

Fig. 2 Externally collar

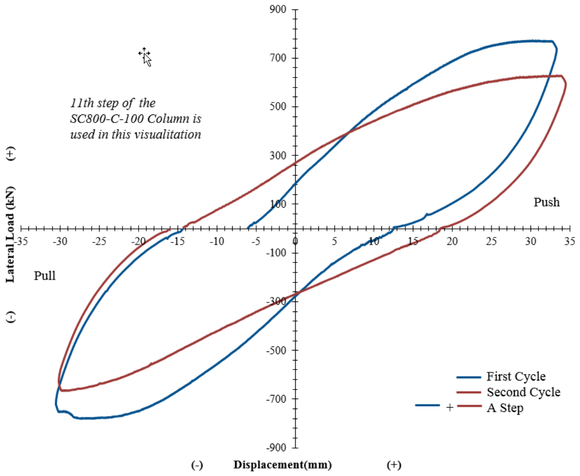

Fig. 3 Cycle and step display on the graph

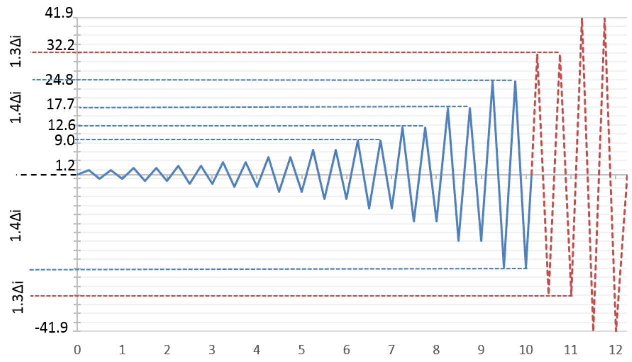

Fig. 4 The applied lateral loading procedure

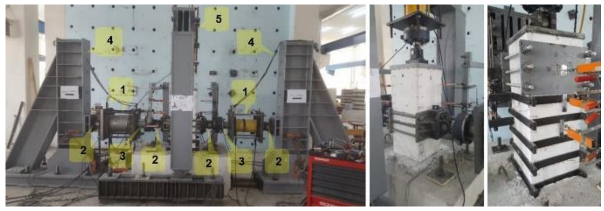

Fig. 5 Experimental setup and test columns (1-Hydraulic jack, 2-Hinge, 3-Loadcell, 4-Lateral loading wall, 5-Axial loading setup)

diameters of the reinforcements, the foundation dimensions, the cross-sectional dimensions of the columns and the longitudinal reinforcement ratios are the same. Axial load level is also kept constant.

3.3 Test setup

Externally applied collars consist of 4 separate parts. On two edges, the long flat parts, and the other two edges, which were curved like a "U", were connected by bolts at two edge corner points. In this way the wrap is formed. (Fig. 2). The experimental loading setup consists of two lateral loading walls and an axial loading setup with hydraulic jacks, connecting pieces, load cells, joints and measuring equipment (LVDT, Encoder, strain-gauge, etc.). The reason for choosing two lateral loading walls in experimental studies is that the lateral load level is especially high in columns with an aspect ratio of 1. When the lateral load level is more than a certain level, sudden break in the fasteners may happen due to the shear effect during the pulling is done. An image of the experimental setup and images of the ref column and collared column in the experimental setup are shown in Fig. 5.

3.4 Loading procedure

RC short columns were subjected to axial load using the 0.2 load index during the test. The average concrete pressure strength is 28.4 MPa. For short columns with a cross section of 400×400 mm, $0.2A_cf_c'\approx 910\ kN$ axial load was applied to all columns during the experiments.

Double repetitive lateral loads were applied to the columns with the controlled displacement. During the experiment, the lateral loading conditions of RC short columns were carried out following the same procedure including the reference columns. Testing the columns by following the same loading procedure helps to minimize the errors that may occur in energy dissipation data. The lateral loading procedure was implemented following the quasi-static load application for the displacement-controlled loading of the structural components specified in FEMA461. Large damage may occur at small displacement values because of the brittle behavior of the short columns. For this reason, it is more appropriate to continue the experiment with small displacement increments. Thus, both behaviors are observed more clearly and sudden failures that may occur after a few cycles are avoided.

Tracking of start-push-pull-start path of the loading is a loop. Each cycle is repeated twice according to the procedure. Therefore, 2 cycles repeated one after another constitute 1 step. (See Fig. 3). At least 10 steps (20 Cycles) were also applied according to the procedure. If the column has not reached the lateral load capacity after 10 steps have been completed, or if the peak load has remained at a level close to the peak load, the lateral displacement repeated in the last step is increased by 30% for each new step. In Fig. 4, the lateral loading procedure is given by lateral displacement values applied at 800 mm level. Displacements in the columns that loaded at 400 mm and 600 mm were applied in accordance with this procedure depending on the height of the load from the foundation level.

4. Results and discussions

In experimental studies, a cyclic pushover test was performed under cyclic loading of 9 pieces of RC short columns with aspect ratios of 1, 1.5 and 2. Using the

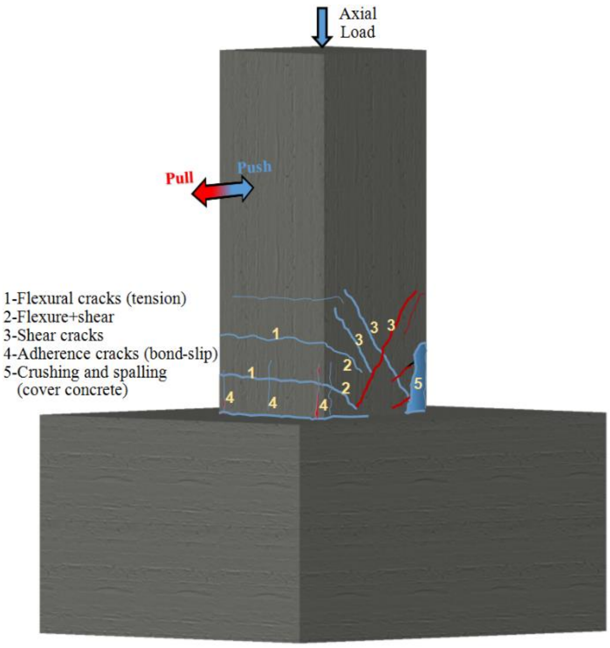

Fig. 6 Crack development characteristic of test columns

Fig. 7 Deformed collar

experimental data, the performance of the columns was numerically evaluated with hysteretic behavior, strength envelopes, ductility and stiffness findings. Moreover, dominance effect on behavior is examined using the shear strength envelopes.

4.1 Crack development

A reinforced concrete structure is weakened or damaged by a combination of stress reversals and high stress excursion. Consequently, any damage criterion should include not only the maximum response, but also the effect of repeated cyclic loadings (Park and Ang 1985). Under monotonic loadings, brittle failure modes such as shear and bond failures can be avoided through careful detailing of the members, and the ultimate flexural capacity can be accurately evaluated (Burns *et al.* 1962). However, under repeated cyclic loadings, it is difficult to ensure that such brittle failure modes will not occur. According to evidence from past strong earthquakes, reinforced concrete columns are susceptible to diagonal tension cracking that frequently leads to a brittle shear failure (Park *et al.* 2011). Priestley *et al.* 1994, reported the shear-strength degradation and early shear failure is attributed to the development of diagonal tension cracks in the plastic hinge regions. Some researchers have stated that the shear crack angle changes with axial load index (Ou and Kurniawan 2015).

The crack characteristics observed in RC short columns tested in this study are shown in Fig. 6. The crack development in the columns is shown in Fig. 9. In all tested columns, the development of cracks was generally observed as follows. (1) the occurrence of capillary cracks in the tension zone, (2) expansion of tension cracks, (3) continuing of the tension cracks in an oblique form on the sides of the column after the flexural yielding, (4) independent development of shear cracks from tension cracks, (5) expansion of the shear cracks after the peak load and decrease in strength, (6) the development of vertical adherence cracks and cover-concrete crushing and spalling in the compression zone, (7) at the advanced displacement level (after 0.8 V_{max} on descending brane) buckling in longitudinal reinforcements in the compression zone, (8) if the next step is continued, rupture in the longitudinal reinforcement of the draw zone (this step was only applied in some test columns to allow the further progress of the damage to be visible).

Significant expansion in the shear cracks of the reference columns occurred in the columns tested. In addition to this, the reduction of strength has been realized rapidly. Shear cracks occurred in the collared columns, but collars prevented these cracks expanding. The fact that the shear cracks cannot expand has led to the expansion of cracks due to the flexure. This situation has significantly prevented the brittle failure. After the peak load, expansions in the stirrups occurred and the winding effect is reduced. Deformation of the collars occurred at a further level. Therefore, it enabled the winding effect in the collars to be maintained in an effective manner. Deformed collar appearance after the test is given in Fig. 7.

4.2 Hysteretic behavior

The performance of the columns is assessed by peak load, moment capacity, reduction rate of strength and hysteretic behavior. Hysteretic behavior is so important in terms of seismic performances of structural elements. The upward slope area of the first region of the hysteretic behavior forms the linear behavior of the element. At this stage, the opening and closing capillary cracks occur. As the cycles continue, new crack developments occur depending on the brittle or ductile behavior of the element. The hysteresis loop has an upward slope in the linear region and a descending slope in the nonlinear region. Both of them reveal the strength, ductility and energy absorption capacity of the element. They also reveal the behavioral character of the element. The hysteresis loops and strength envelopes of the columns tested in the study are shown in Fig. 8. The hysteresis loops are given using the entire data set up to the end of the experiment so that behavior can be seen.

However, in the previous cycle, the envelope loops given on the same graph is marked as “ultimate point” (■) after the peak load (Vmax) before falling below the 80% level. Thus, the meaningful endpoint for evaluating the performance of the columns is limited to 0.8 Vmax. Furthermore, “yield point” (◆) and “peak load” (●) are also marked on the strength envelopes.

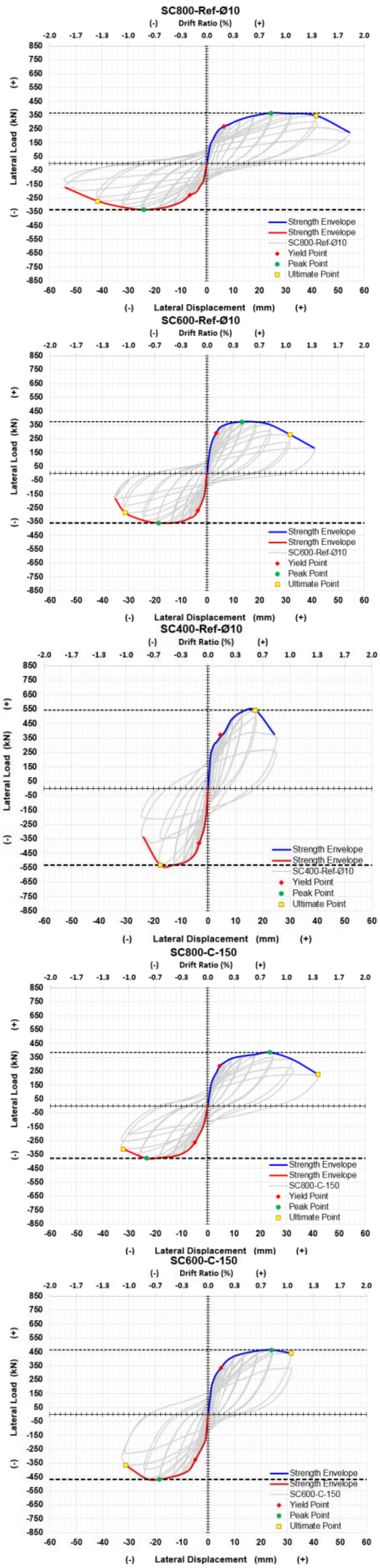

Fig. 8 Hysteresis loops and strength envelopes of all test columns

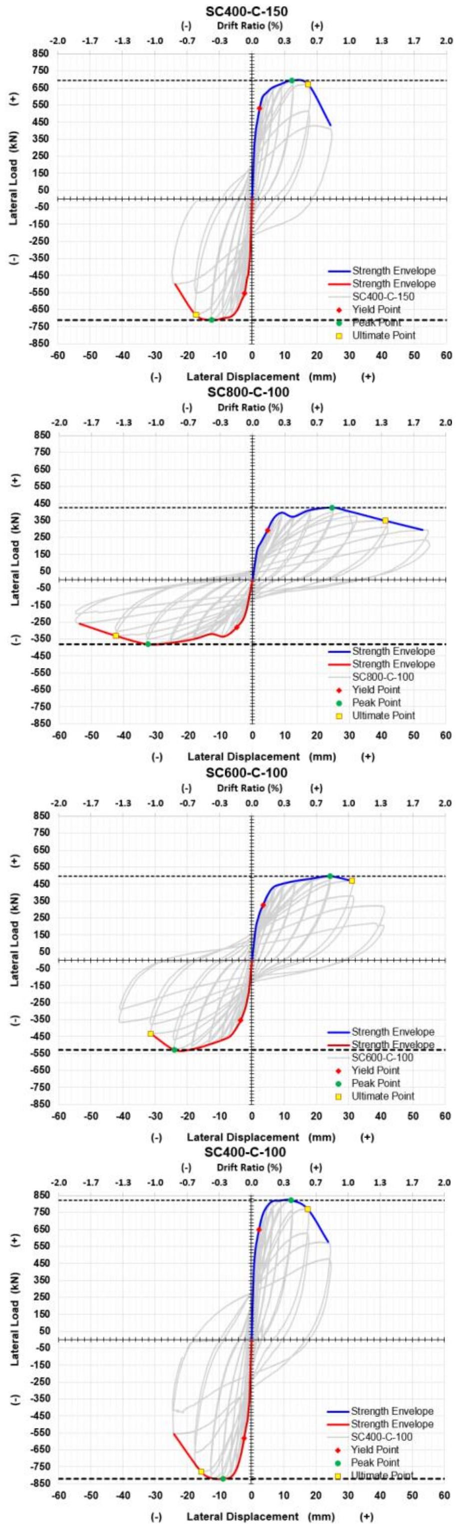

Fig. 8 Continued

The descending slope of the load after the peak load of the SC800 series columns with aspect ratio of 2 was lower, while the descending slope of the load was increased in the SC600 and SC400 columns as the aspect rate decreased. The drift ratios (%) are given on the graph as the secondary axis in Fig. 4, by taking into account the 3 m. normal floor height. The ultimate points of the SC800 series columns reached the level of 1.4%, while the SC600 and SC400

Table 1 Load/displacement relationship and peak load values of test columns

	Column	V_{max} / Δ_{peak}		V_e / Δ_e		V_u / Δ_u	
		Push	Pull	Push	Pull	Push	Pull
1	SC800-REF-Ø10	365.07 / 24.45	-334.7 / -24.16	267.60 / 6.50	-230.24 / -6.46	346.21 / 41.72	-273.17 / -41.56
2	SC800-C-150	384.97 / 23.60	-377.77 / -23.36	286.76 / 4.59	-264.53 / -4.91	209.42 / 42.01	-206.65 / -32.29
3	SC800-C-100	424.79 / 24.74	-381.27 / -32.46	290.90 / 4.62	-281.28 / -4.94	293.32 / 41.20	-331.96 / -42.39
4	SC600-REF-Ø10	372.84 / 18.05	-360.39 / -27.53	258.49 / 5.02	-271.41 / -4.53	279.06 / 41.23	-286.49 / -44.87
5	SC600-C-150	463.56 / 29.17	-467.67 / -23.06	332.32 / 5.22	-327.42 / -5.59	429.43 / 38.50	-366.64 / -41.06
6	SC600-C-100	495.80 / 30.28	-528.4 / -30.05	326.08 / 4.37	-354.97 / -5.09	468.56 / 39.05	-433.07 / -40.36
7	SC400-REF-Ø10	542.41 / 31.00	-532.17 / -22.59	395.38 / 6.67	-381.52 / -4.72	542.41 / 31.00	-532.17 / -22.59
8	SC400-C-150	694.05 / 24.71	-711.3 / -20.76	528.71 / 4.62	-555.8 / -4.09	430.70 / 34.79	-501.94 / -30.45
9	SC400-C-100	821.77 / 22.71	-820.68 / -14.72	649.20 / 4.17	-581.11 / -3.87	576.88 / 32.77	-557.62 / -27.41

(Load/Displacement*)

*V_{max} / Δ_{max}: Peak point load/disp.; V_e / Δ_e: 3/4V_{max} point load/disp.; V_u / Δ_u: Ultimate point load/disp

Fig. 9 Damage cases of the test columns after some experiments

series remained at the level of 1.05% and 0.58% respectively. In Table 1, the load/displacement data of the columns are given numerically. In the SC800 series columns, the peak load values were close to each other and the largest peak load was reached in the SC800-C-100 column. As the aspect ratio decreases, the difference between the peak loads of the reference columns and the peak loads of the externally collared columns was increased.

The difference between the peak loads of the (C-150)-(Ref-Ø10) and (C-100)-(Ref-Ø10) columns in the SC800 series columns was 19.90 kN/43.08 kN (push/pull) and 59.72 kN/46.57 kN respectively. This difference was 90.72 kN/107.28 kN and 122.96 kN/168.01 kN in the SC600 series and 151.64 kN/179.12 kN and 279.37 kN/288.51 kN in the SC400 series columns. In each series C-100 column has a maximum peak load. The peak load of the C-100 columns is 1.15 times higher in the SC800 series columns, 1.40 times higher in the SC600 series columns and 1.53

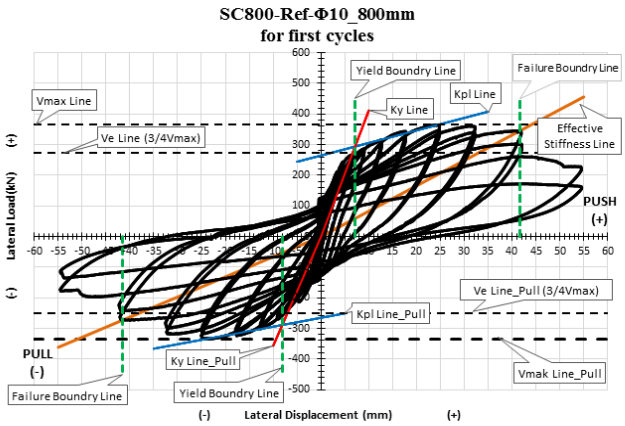

Fig. 10 Visualization of the stiffness and ductility calculation diagram

Table 2 Displacement and ductility values of short columns

Column	Loading Level (mm)	Δ_y (mm)			Δ_u (mm)			μ		
		Push	Pull	Mean	Push	Pull	Mean	Push	Pull	Mean
SC800-REF-Ø10	800	7.14	-8.10	7.62	41.72	-41.56	41.64	5.84	5.13	5.49
	600	5.67	-5.76	5.72	33.20	-31.37	32.28	5.85	5.44	5.65
	400	4.18	-4.49	4.34	24.43	-20.26	22.34	5.85	4.51	5.18
SC800-C-150	800	5.41	-6.55	5.98	42.01	-32.29	37.15	7.76	4.93	6.35
	600	4.09	-3.37	3.73	32.48	-21.20	26.84	7.93	6.28	7.11
	400	2.70	-2.20	2.45	23.82	-10.01	16.92	8.82	4.55	6.69
SC800-C-100	800	5.37	-5.68	5.53	41.20	-42.39	41.79	7.66	7.46	7.56
	600	4.17	-3.30	3.74	30.91	-30.36	30.64	7.41	9.19	8.30
	400	2.89	-2.12	2.51	20.67	-19.19	19.93	7.15	9.03	8.09
SC600-REF-Ø10	800	5.87	-5.64	5.75	41.23	-44.87	43.05	7.02	7.96	7.49
	600	4.12	-4.41	4.26	31.70	-31.24	31.47	7.70	7.09	7.39
	400	2.86	-3.06	2.96	21.13	-21.01	21.07	7.40	6.87	7.14
SC600-C-150	800	6.59	-6.99	6.79	38.50	-41.06	39.78	5.84	5.88	5.86
	600	6.10	-6.00	6.05	31.49	-31.18	31.33	5.16	5.19	5.18
	400	4.53	-5.07	4.80	22.01	-21.14	21.58	4.86	4.17	4.51
SC600-C-100	800	5.98	-6.83	6.40	39.05	-40.36	39.70	6.53	5.91	6.22
	600	4.75	-4.63	4.69	31.04	-31.38	31.21	6.53	6.78	6.66
	400	3.11	-3.43	3.27	21.34	-21.02	21.18	6.87	6.13	6.50
SC400-REF-Ø10	800	7.78	-6.10	6.94	31.00	-22.59	26.80	3.98	3.71	3.85
	600	5.97	-5.57	5.77	23.43	-20.00	21.72	3.93	3.59	3.76
	400	5.10	-4.25	4.67	17.37	-17.55	17.46	3.41	4.13	3.77
SC400-C-150	800	5.50	-4.99	5.24	34.79	-30.45	32.62	6.33	6.10	6.21
	600	4.22	-3.96	4.09	26.10	-23.96	25.03	6.18	6.04	6.11
	400	2.76	-2.79	2.78	17.31	-17.36	17.33	6.26	6.21	6.24
SC400-C-100	800	5.21	-4.90	5.06	32.77	-27.41	30.09	6.29	5.59	5.94
	600	4.03	-3.88	3.96	24.87	-21.56	23.22	6.17	5.55	5.86
	400	2.90	-2.90	2.90	17.43	-15.68	16.56	6.01	5.41	5.71

times higher in the SC400 series columns than the reference columns. The shear reinforcement spacing of the externally collared columns are less than the reference columns. Considering this, it is seen that the collar is very effective in

Table 3 Stiffness values obtained by load-displacement relation at 800 mm, 600 mm and 400 mm height

Test Column	Cycle Definition	800 mm Level Initial Stiffness *K_y* (*kN/mm*)	800 mm Level Effective Stiffness *K_{eff}* (*kN/mm*)	800 mm Level K_y / K_{eff}	600 mm Level Initial Stiffness *K_y* (*kN/mm*)	600 mm Level Effective Stiffness *K_{eff}* (*kN/mm*)	600 mm Level Ky / Ks	400 mm Level Initial Stiffness *K_y* (*kN/mm*)	400 mm Level Effective Stiffness *K_{eff}* (*kN/mm*)	400 mm Level K_y/K_{eff}
SC800-REF-Ø10	Push	41.19	8.30	4.96	51.43	10.43	4.93	68.79	14.17	4.85
	Pull	35.62	6.57	5.42	49.95	8.71	5.74	64.74	13.49	4.80
SC800-C-150	Push	62.53	5.41	11.56	82.62	7.00	11.81	125.06	9.54	13.11
	Pull	53.92	6.40	8.43	104.67	9.75	10.74	160.68	20.65	7.78
SC800-C-100	Push	63.02	8.43	7.47	81.19	11.24	7.22	117.09	16.81	6.96
	Pull	56.99	7.83	7.28	98.91	10.93	9.05	153.34	17.30	8.86
SC600-REF-Ø10	Push	58.17	6.77	8.59	82.77	8.80	9.40	119.57	13.21	9.05
	Pull	59.96	6.38	9.39	77.39	9.17	8.44	111.47	13.64	8.17
SC600-C-150	Push	63.63	11.43	5.57	68.71	13.97	4.92	92.41	19.99	4.62
	Pull	58.55	8.93	6.56	68.16	11.76	5.80	80.82	17.34	4.66
SC600-C-100	Push	74.59	12.00	6.22	93.59	15.10	6.20	142.83	21.95	6.51
	Pull	69.68	10.73	6.49	101.79	13.80	7.38	137.06	20.60	6.65
SC400-REF-Ø10	Push	59.30	17.50	3.39	76.04	23.15	3.28	88.83	31.23	2.84
	Pull	80.87	23.55	3.43	90.13	26.61	3.39	119.34	30.32	3.94
SC400-C-150	Push	114.47	19.32	5.92	148.79	25.76	5.78	226.99	38.83	5.85
	Pull	135.75	22.39	6.06	170.85	28.46	6.00	242.04	39.28	6.16
SC400-C-100	Push	155.67	23.53	6.62	201.20	31.01	6.49	279.51	44.24	6.32
	Pull	150.03	28.47	5.27	189.19	36.19	5.23	252.70	49.75	5.08

increasing the shear strength. Some sample illustrations for the post-test damage of the columns are given in Fig. 9.

4.2 Stiffness and ductility

Stiffness is an important parameter in terms of the behavior of the columns under the influence of the lateral load. A structural element must have sufficient stiffness in terms of the design of earthquake-resistant structures. In this section, the secant and effective stiffness of the test columns are compared. Secant stiffness is the slope of the K_y line passing 75% of the peak load, and the effective stiffness is the slope of the line combining the starting point and the ultimate point. Effective stiffness is defined as the slope of the line combining the starting point with the design point. In the scope of the study, the performance of the test columns was investigated. For this reason, the effective stiffness was calculated by taking into account the ultimate point (Δ_u). Furthermore, the design point can be examined at the ultimate point where the load falls to 80% because the response spectrum of the test columns does not intersect the spectrum curve in the Acceleration Displacement Response Spectrum (ADRS) diagram for the displacement-based design. The displacement at this ultimate point is used in the evaluation of ductility (μ=Δ_u/Δ_{mak}). K_{pl} line is defined to obtain the yield displacement (Δ_y). K_{pl} line is the line which combines the coordinates of the peak point of the circle (V_{mak}, Δ_{max}) and the coordinates of the peak point of the cycle before the two cycles (V_{mak-2}, Δ_{mak-2}). One of the graphical representations used to calculate the secant and effective stiffness and ductility values of the columns is given as an image in Fig. 10. Ductility and stiffness values of the columns are given numerically in Tables 2 and 3 respectively.

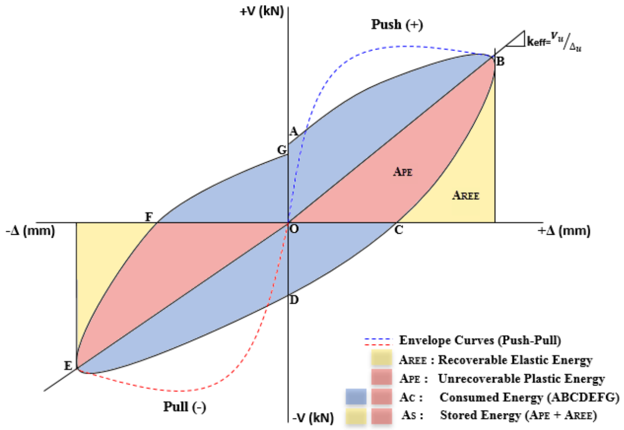

Fig. 11 Definitions of energy consumption zones

Table 2 shows the Δ_y, Δ_u and μ values obtained at each level of the columns. The reference column SC600 series showed more ductile behavior than the collared columns in the column. However, ductility of the collared column is higher in SC800 and especially in SC400 series columns. The aspect ratio of 1 causes a considerable decrease in the ductility of the reference column. The comparison of ductility is based on the results obtained from the displacement in the loading levels.

Although the collared columns cannot be effective at the desired level due to increasing the ductility, they are considerably effective in obtaining sufficient stiffness due to the increase in strength. The secant and effective stiffness values obtained from the data which is gathered from each level (400-600-800 mm) are also given in Table 3. These values are considered important to examine the effect of the aspect ratio on the stiffness values of the measured heights. In addition to this, the assessment should be based on the values obtained from each loading level for each column series. Therefore, the stiffness values which were obtained from the load-displacement relation carried out at the 800 mm level for the SC800 series, 600 mm level for the SC600 series and 400 mm level for the SC400 series. They are highlighted in the table. As the aspect ratio decreases, the stiffness increases. The mean initial stiffness (push-pull) values of the reference columns increased by 2.08 and 2.71 times for SC600/SC800 and SC400/SC800 respectively, while the effective stiffness ratios increased by 1.21 and 4.14 times. These increase rates were 1.18-4.03 and 2.18-6.61 for the C-150 columns and 1.63-4.43 and 1.78-5.78 times for the C-100 columns. Descending in aspect ratio (α_s) is very effective in increasing the effective stiffness. The stiffness value is the C-100 column, which is the highest of every series. The initial stiffness ratio of SC800-C-100/SC800-Ref-Ø10 was 1.56 in SC800 series columns with α_s=2, while this ratio increased up to 1.22 when α_s=1.5 to 2.56 when α_s=1. The increase in the effectiveness of

Table 4 Energy data calculated for test columns

Test Column	Et (kNmm)*	Ed (kNmm)	Recoverable Elastic Energy (kNmm)	Unrecoverable Plastic Energy (kNmm)
SC800-REF-Ø10	98477.42	94193.79	42829.16	51364.63
SC800-C-150	81209.61	76898.58	38184.73	69233.69
SC800-C-100	96673.85	109897.56	57502.67	52394.89
SC600-REF-Ø10	100676.47	103084.94	42328.45	60756.50
SC600-C-150	113298.30	111567.95	50486.38	61081.57
SC600-C-100	122210.84	127804.80	63617.05	64187.74
SC400-REF-Ø10	92152.24	85099.49	49134.77	28148.45
SC400-C-150	128970.44	135070.93	64976.62	70094.31
SC400-C-100	148012.66	151032.20	72043.12	78989.08

*kNmm=Nm

shear dominant behavior of externally collar provides a significant contribution to the stiffness.

4.3 Energy dissipation

The energy consumption capacity is an important parameter in assessing the performance of columns that are very important structural system components in reinforced concrete frames and/or mixed systems. The energy consumption of the columns depends on axial load level, yield displacement, number of cycles, support conditions, cross-sectional details, material properties and reached peak load level. The identification of energy consumption zones is given visually in Fig. 11.

Energy consumption values of test columns are given in Table 4. Due to the bending-dominated behavior in SC800 series column, no significant difference is observed in energy consumption data between the SC800 series column.

However, the energy data of the collared columns differs from the reference column by the reduction of the aspect ratio. The energy of the collared columns is well above the reference column especially in the SC400 series columns (α_s=1). The energy consumption of the Ref column in the SC800 series (E_t) is higher than the collared columns, while the energy consumption of the collared columns in the SC600 and SC400 series is higher than the Ref column. The recoverable elastic energy and unrecoverable plastic energy values of the collared columns are higher than the reference column in each series. This difference increases prominently as the aspect ratio decreases.

4.4 Shear strength envelope

Many researchers (Jin *et al.* 2017, Cai *et al.* 2015, Park *et al.* 2012, Shin *et al.* 2013, Pujol *et al*. 2016) are studying the evaluation of the shear strength of reinforced concrete columns. Most of these studies are mainly based on the testing of columns. The contribution of the shear strength envelope, depending on some parameters, to the shear strength of the concrete is expressed by evaluating the obtained results. These shear strength envelopes are formed in the reinforced concrete column. The most important design parameters considered for assessing the contribution to the shear strength of concrete are; displacement ductility (μ), axial load level (P), longitudinal reinforcement ratio (ρ_l), size effect and effective shear area (A_{sh}).

In most of the models, the effective shear area is expressed as 80% of the gross cross-sectional area (A_g) of the column. In some models, it is expressed as the product of the section width (b_w) and effective depth (d). ($A_{sh}=0{,}80A_g$ *or* $A_{sh}=b_w d$)

Priestley *et al.* (1994) evaluated the shear strengths of reinforced concrete columns in part of concrete contribution, the contribution of the shear reinforcement and axial load. They have defined the decrease in the shear strength of concrete according to increasing ductility with the following Eq. (1).

$$V_c = \gamma\sqrt{f_c^{'}}(0.8A_g) \tag{1}$$

Here γ is 0.29 if the displacement ductility (μ) is less than 2, 0.10 if it is greater than 4. The γ factor is linearly decreasing when the ductility is between 2 and 4.

Xiao and Martirossyan (1998) proposed that the contribution of concrete to shear decreases more dramatically when they investigate the seismic performances of high strength concrete columns. Accordingly, the γ factor is defined by the following equations.

$$\begin{aligned}
&\gamma = 0.29 && for\ \ \mu \le 2\\
&\gamma = 0.29 - 0.12(\mu-2) && for\ \ 2 \le \mu \le 4\\
&\gamma = 0.05 - 0.025(\mu-4) && for\ \ 4 \le \mu \le 6\\
&\gamma = 0 && for\ \ 6 \le \mu
\end{aligned} \tag{2}$$

Recently, Howser *et al.* (2010) have conducted a numerical parameter study. They evaluated the change of ductility depending on the longitudinal reinforcement ratio and concrete strength (Eq. (3)). Therefore, the determination of the γ factor and the contribution to the shear strength of the concrete has been revised by taking into consideration both the strength and the longitudinal reinforcement of the section.

$$\begin{aligned}
&\gamma = 0.29 && for\ \ \mu \le 2\\
&\gamma = 0.29 - 0.12(\mu-2) && for\ \ 2 \le \mu \le r\\
&\gamma = 0.53 - 0.095r - 0.025\mu && for\ \ r \le \mu \le q\\
&\gamma = 0.53 - 0.095r - 0.025q && for\ \ q \le \mu\\
\\
&r = 35\rho_t - 0.011f_c^{'} + 3.8 &&\\
&q = -144\rho_t - 0.03f_c^{'} + 4.3 && for\ \ r \le q\\
&q = r && for\ \ q < r
\end{aligned} \tag{3}$$

The strength envelopes of the test columns were reduced to the shear force-ductility axis in Fig. 12. In these graphs, the shear strength envelopes and the strength envelopes of the test column are compared. Therefore, depending on the ductility, an important visual is generated in the shear effect

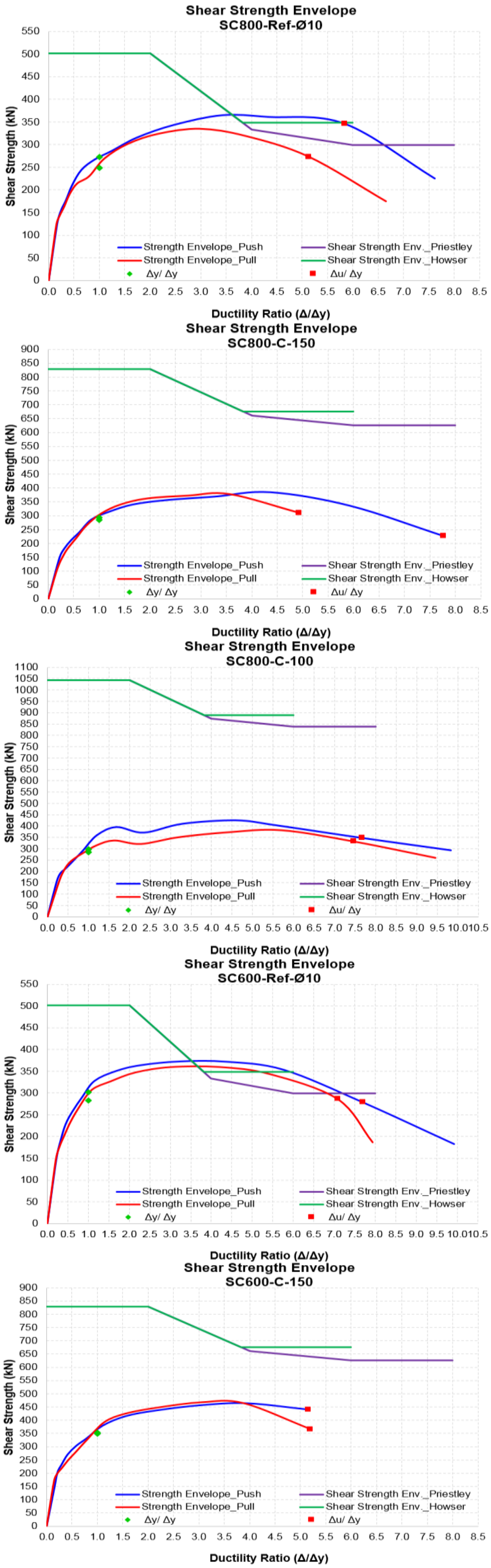

Fig. 12 Shear strength envelope and failure behavior

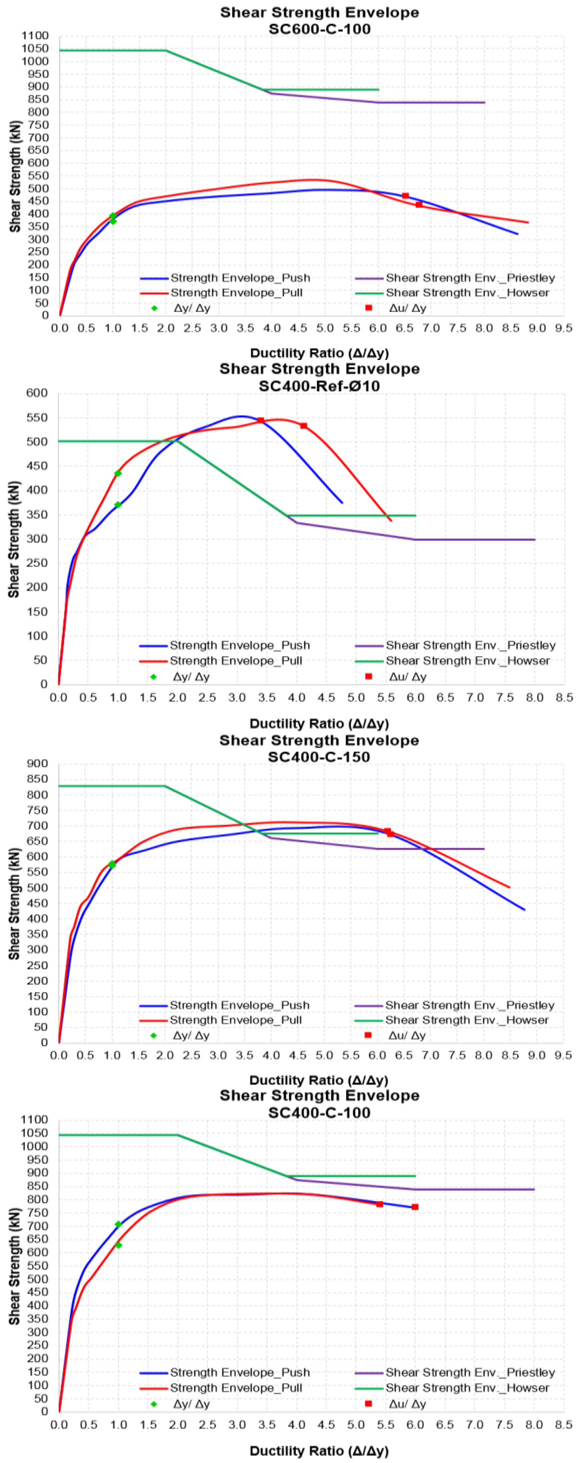

Fig. 12 Continued

of the behavior of the column. It is seen that the strength envelopes of the reference columns intersect the shear strength envelopes at each α_s ratio. This indicates that the reference columns are directed to shear failure in each series. Reference columns are directed to shear failure after μ=3.5 when α_s=2 and α_s=1.5, and after μ=2 when α_s=1. When the C-150 column α_s = 1, intersect the shear strength envelope at μ=3.5 level. The C-100 column did not intersect the shear strength envelope and reached its capacity due to damage at the column-foundation junction.

5. Conclusions

Based on a comprehensive experimental study of the behavior of nine collared RC columns under combined constant axial loading and cyclic loading, suggested collared rehabilitation is shown to be effective for application to deficient short RC columns. The main observations and conclusions of this study are summarized as follows:

- The crack widths and characters differ depending on the aspect ratio. In the SC800 series columns with $\alpha_s = 2$, joint damages were apparently effective in behavior. The effect of bending is more determinative than the effect of the shear effect in the behavior of these columns. While the shear dominant behavior increases in SC600 series columns, the behavior of Ref. and C-150 columns in the SC400 series columns has been completely under the shear effect. The SC800 and SC600 series were observed to be buckled in longitudinal columns after 10th step (20-21th cycles). In the intermediate area of the collars, spalling of the cover concrete and section reduction were observed. Moreover, buckling of the longitudinal reinforcement was found in collar intermediate regions. Collars have been severely deformed by preventing the longitudinal reinforcement from being buckled. The decrease of the collar spacing (150 mm to 100 mm) makes it difficult to buckle the longitudinal reinforcement.
- Due to the failure of the SC400-Ref-Ø10 columns, the development of the classic X crack has happened. Shear crack damage that occurs in the collared columns is not at the level that will cause failure. During the experiment, the slippage did not occur. Even if crushing and spalling of the cover concrete occurred between the collars, deformation was prevented in the zones where collars were present.
- As the aspect ratio decreases, the difference between the peak loads of the reference columns and the peak loads of the externally collared columns increased. The difference between the peak loads of the (C-150)-(Ref-Ø10) and (C-100)-(Ref-Ø10) columns in the SC800 series columns was 19.90 kN/43.08 kN (push/pull) and 59.72 kN/46.57 kN respectively. This difference was 90.72 kN/107.28 kN and 122.96 kN/168.01 kN in the SC600 series and 151.64 kN/179.12 kN and 279.37 kN/288.51 kN in the SC400 series columns. In each series, C-100 column has a maximum peak load. The peak load of the C-100 columns is 1.15 times higher in the SC800 series columns, 1.40 times higher in the SC600 series columns and 1.53 times higher in the SC400 series columns than the reference columns. The peak loads of the C-100 columns are higher than that of the C-150 columns, so the reduction of the collar spacing (150 mm to 100 mm) increases the shear strength of the column. The C-100 / C-150 peak load ratios in the SC800, SC600, and SC400 series columns were 1.06, 1.27 and 1.17 respectively.
- As the aspect ratio decreases, the stiffness increases. The mean initial stiffness (push-pull) values of the reference columns increased by 2.08 and 2.71 times for SC600 / SC800 and SC400 / SC800 respectively, while the effective stiffness ratios increased by 1.21 and 4.14 times. These increase rates were 1.18-4.03 and 2.18-6.61 for the C-150 columns and 1.63-4.43 and 1.78-5.78 times for the C-100 columns. Descending in aspect ratio (αs) is very effective in increasing the effective stiffness. The stiffness value is the C-100 column, which is the highest of every series. The initial stiffness ratio of SC800-C-100 / SC800-Ref-Ø10 was 1.56 in SC800 series columns with α_s=2, while this ratio increased up to 1.22 when α_s=1.5 to 2.56 when α_s=1. The increase in the effectiveness of shear dominant behavior of externally collar provides the significant contribution to the stiffness.
- From the shear strength envelope graphs, it is clear that the behavior of the reference columns is the shear dominant behavior. Apart from the SC400-C-150 column, collared columns did not intersect the shear strength envelope.
- Due to the bending-dominated behavior in SC800 series column, no significant difference is observed in energy consumption data between the SC800 series column. However, the energy data of the collared columns differs from the reference column by the reduction of the aspect ratio. The energy of the collared columns is well above the reference column especially in the SC400 series columns (α_s=1). The recoverable elastic energy and unrecoverable plastic energy values of the collared columns are higher than the reference column in each series. This difference increases prominently as the aspect ratio decreases.

Experimental studies and results have shown that collared steel is very effective in increasing the performance of columns with low aspect ratio. The increase in the stiffness, strength and energy consumption capacities of columns with collared steel is regarded as a positive contribution to the literature. In addition to the material, spacing, and geometry of the collar, further studies can be done by the diversification of the cross section dimensions of the column. This situation shows that the study is open to new developments.

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