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Complete collapse test of reinforced concrete columns

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Abstract. In this paper, experimental investigation into the behavior of reinforced concrete (RC) columns tested under large lateral displacement with four different types of loading arrangements is presented. Each loading arrangement has a different system for controlling the consistency of the loading condition. One of the loading arrangements used three units of link mechanism to control the parallelism of the top and bottom stub of column during testing, and the remaining employed eight hydraulic jacks for the same purpose. The loading systems condition used in this investigation were similar to the actual case in a moment-resisting frame where the tested column was displaced in a double curvature. Ten model column specimens, divided into four series were prepared. Two columns were tested monotonically until collapse, and unless failure took place at an earlier stage of loading, the remaining eight columns were tested under cyclic loading. Test results indicated that the proposed system to keep the top and bottom stubs parallel during testing performed well.

Key words: complete collapse; constant axial load; lateral load; lateral displacement; loading arrangement; parallel keeping system; reinforced concrete column.

1. Introduction

During its life time, a column, which is considered to be a very critical member in structural moment resisting frame, may be required to withstand significant overload, such as during a major earthquake. Recent earthquakes have indicated that the collapse of a column or a group of columns may lead to a partial or even total collapse of the frame, potentially leading to hundreds of deaths and huge economic loss (Dowling 1998, EERI special Report 1999, A report of the Taiwan-US Geotechnical Earthquake Engineering Reconnaissance Team 1999). Therefore, as one of the main aims of design is to provide safety to a structure, a practitioner and a researcher should have knowledge about the behavior of concrete structures under conditions of extremely large deformations.

Analytical procedures to establish the behavior of RC columns when subjected to simulated earthquake loading have been developed in recent years. However, experimental testing is still greatly necessary for assessment since some aspects of behavior of interest in seismic design cannot be determined with confidence by analytical procedures.

The need to study the behavior of RC columns when subjected to large deformation, beyond point C (see Fig. 1), is due to the fact that most of them, especially building columns, are loaded in groups where they would fail completely during a major earthquake. However, due to the

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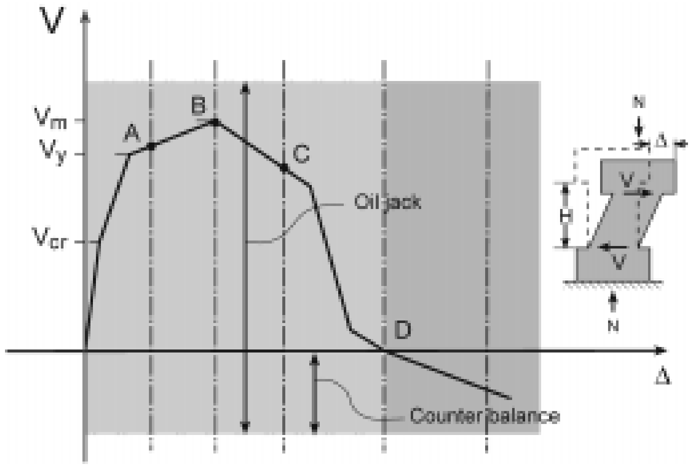


Fig. 1 Generalization of lateral load-displacement curve of RC column

difficulties in conducting such an experiment, studies and references on the complete collapse mechanism of RC columns from the initial stage of damage until total collapse are scarce (Takiguchi *et al.* 1998). Also, in most experiments on RC columns, due to stroke limitation of an actuator or a hydraulic jack acting as a lateral force, the tests had to be stopped at point C, before full information on complete collapse could be acquired.

One of the most important issues, but one that is relatively difficult to achieve in conducting double bending tests of a column, is controlling parallelism of the top and bottom stubs while they are being simultaneously subjected to both constant axial load and lateral load during testing. In the study reported herein, four loading arrangements, where the top and bottom stub rotation was controlled by different systems, are proposed to study the behavior of RC columns under large lateral displacement. The proposed loading arrangements consist of parallel keeping systems and dead weight as a constant axial load. Large stroke capacity of a hydraulic jack was used to apply the lateral load. A counter balance valve was used to control the lateral load and to prevent fast and uncontrolled increment of lateral displacement of the column within unstable stage during testing.

2. Test specimens and test procedure

2.1 Specimens design

In all, ten model columns were constructed and tested using four different loading arrangements. The specimens were divided into four series in accordance with the loading arrangement used. Columns in series 1 were tested under monotonic loading, while the rest were tested under cyclic loading. Details of the test program and material properties of the test columns are summarized in Table 1.

The cross section of columns in series 1 was 100 mm square and they had a height of 300 mm. Four deformed D-6 (diameter = 6.35 mm) and round R-3 (diameter = 3 mm) hoops spaced at 15 mm intervals were used as longitudinal and transverse reinforcement, respectively. The height and the cross section of the original columns in series 2, series 3 and series 4 were 600 mm and 120 mm

Table 1 Test program and properties of column specimens

Specification		Column									
		Series 1		Series 2			Series 3	Series 4			
		C-1	C-2	CB-1	CB-2	SCCF	SCFC	CS-1	CS-2	FSC-4L	FSC-6L
Loading test arrangement		A		B			C	D			
Axial Load		49 kN		62 kN				66kN			
Concrete properties	Water cement ratio, w/c	0.5						0.65			
	Sand cement ratio	2.0		2.5				3.75			
	Mortar strength, f'_c	54.2 MPa	59.5 MPa	43.8 MPa	49.4 MPa	50.5 MPa	48.4 MPa	33 MPa	34 MPa	33 MPa	32 MPa
Longitudinal reinforcement	Bar diameter, d_b	6.35 mm									
	Number of bar	4 pcs		12 pcs							
	Yield strain, ε_y	0.19%									
	Yield strength, f_y	373 MPa									
Lateral reinforcement	Bar diameter, d_b	3 mm		2 mm							
	spacing, s	15 mm		35 mm				50 mm			
	Yield strain, ε_y*	0.38%									
	Yield strength, f_y*	697 MPa									
Jacket section	Height of jacket, H_t	–	–	–	–	580 mm		–	–	570 mm	
	Jacket diameter, D_j	–	–	–	–	188 mm	200 mm	–	–	200 mm	
	Jacket thickness, t_j	–	–	–	–	0.2 mm	15 mm	–	–	15 mm	
	Infill mortar strength, f'_c	–	–	–	–	39.6 MPa	45.4 MPa	–	–	30 MPa	32 MPa
	Gap	–	–	–	–	10 mm		–	–	15 mm	
Jacket material properties	Number of layer	–	–	–	–	1	11	–	–	4	6
	Yield strain, ε_y	–	–	–	–	–	0.53%	–	–	0.53%	
	Yield strength, f_y	–	–	–	–	–	273 MPa*#	–	–	273 MPa*#	
	Ultimate strength, f_{ju}	–	–	–	–	2740 MPa	363 Mpa#	–	–	363 Mpa#	
	Ultimate strain, ε_{ju}	–	–	–	–	1.16%	1.50%	–	–	1.50%	

*0.2% permanent strain; #wire mesh

square and were identically reinforced with 12 deformed D-6 hoops which were distributed evenly around the perimeter of the column cross section. R-2 (diameter = 2 mm) hoops spaced at 35 mm interval were used as transverse reinforcement for columns in series 2 and series 3. R-2 was also used as transverse reinforcement at 50 mm interval for columns in series 4. Details of column specimens are shown in Fig. 2 and Fig. 3. The ratio of nominal shear strength of columns in series 1, calculated based on AIJ code approach (AIJ 1994) to shear force required to develop the theoretical nominal flexural strength, is greater than one. This ratio is approximately one for the original columns in series 2 and series 3. Meanwhile, the original columns in series 4 were designed with shear substantially less than that required to develop the flexural strength.

Ordinary portland cement and natural sand passing through JIS sieve No. 2.5 (2.38 mm) were used for the mortar. To improve workability, a superplasticizer was added at 0.05% by weight of

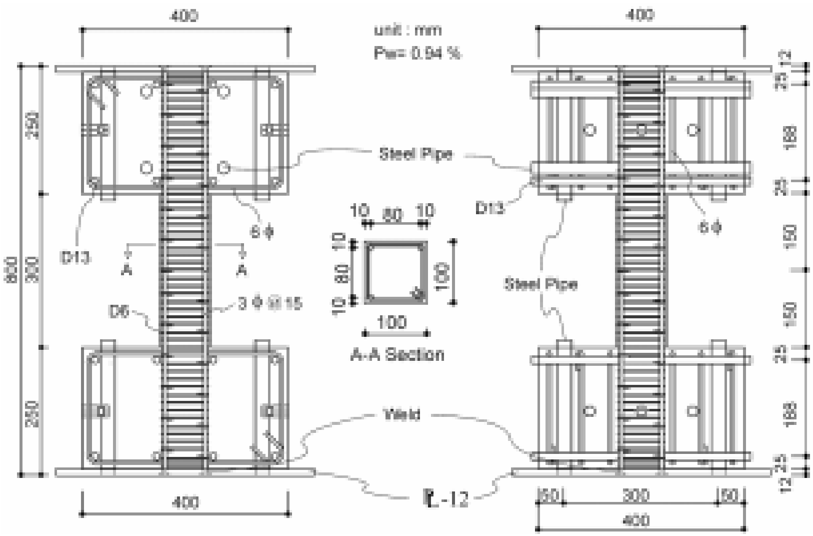


Fig. 2 Detail of column in series 1

cement. The columns were cast in a horizontal position. To minimize differences in concrete compressive strength between specimens in the same series, original columns in series 2 and series 3, and original columns in series 4, were cast with the same batch of mortars on the same day.

A number of 100 × 200-mm cylinders were cast for each batch of concrete to determine their compressive strength. About 4 hr after casting, the specimens were covered with damp burlap to prevent moisture loss. The specimens were stripped of the molds 7 days after casting and then air-cured in the laboratory before testing. Similar treatment was employed after infill mortar for strengthened specimens was cast through four steel pipe sleeves made in the bottom stub or was cast under pressure through holes made in the steel molds. The strengths of materials used for the test columns are shown in Table 1.

2.2 Strengthening procedure

The diameter of the strengthening jacket was chosen so that the axis of the transverse tensile force developed in the jacket is identical. The same batch of mortar was used as the infill mortar and was cast on the same day in an unloaded condition. Gaps, see Table 1, were provided between ends of the jackets and the adjacent column stubs to avoid additional strength or stiffness from the strengthening jacket. To investigate the properties of the strengthening materials, tensile tests were conducted on three identical specimens for each material, and the results are summarized in Table 1.

2.2.1 Ferrocement jacket

A woven wire mesh of 2.5-mm square opening and 0.45-mm wire diameter was used. The required widths and lengths of wire mesh were cut and properly wrapped around the entire column. At several places, the first and the second layer of the wire mesh were tied together with the same diameter of steel wire. This process was repeated when the third, and the fourth layers were wrapped around the column. One hundred millimeters overlapping of wire mesh was provided in the lateral direction. Bonding 5 mm square, and of 3-mm thick steel plates at several places

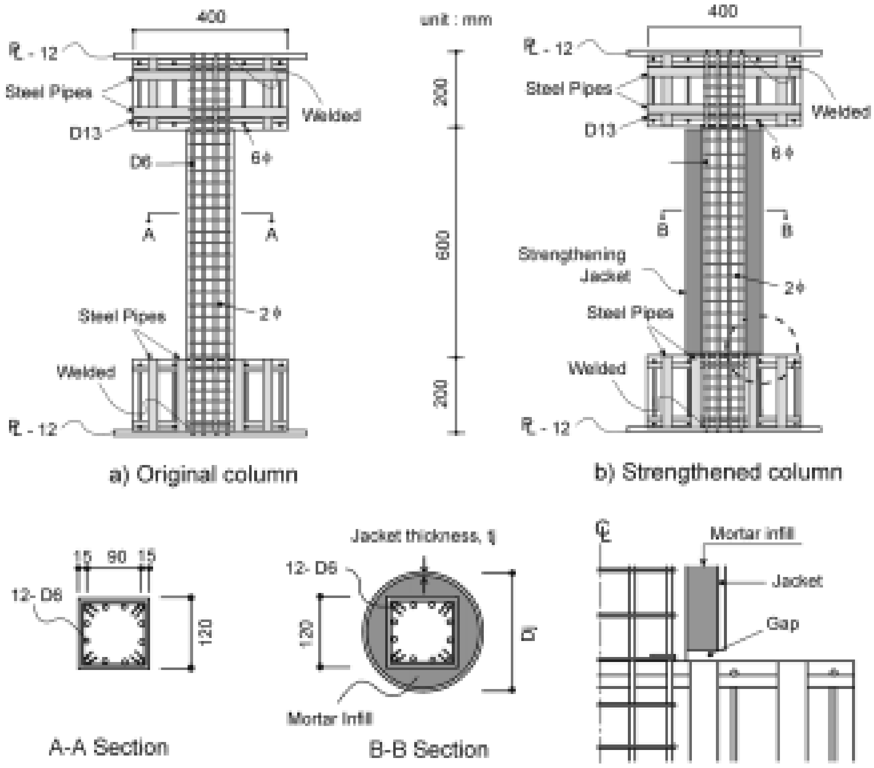


Fig. 3 Detail of columns in series 2-4

provided a clear cover of 3-mm on outer face of jacket.

Infill mortar of strengthened columns in series 2 (column SCCF) and series 3 was made with a water-cement ratio of 0.55 and cement sand ratio of 1:2.5. A smaller maximum size of natural sand passing through JIS sieve designed No. 1.2 (1.19 mm) was used. In order to improve workability, a superplasticizer was added at 0.05% by weight of cement. Note that, even though special care was applied when fresh infill mortar with a slump of 180 mm was cast in the vertical position, and properly vibrated by two units of hand vibrators, some defects were observed on the surface of the jacket of column SCFC. Therefore, repair work by epoxy resin was executed. Meanwhile, cement slurry mixed in proportions in accordance with the manufacturers recommendation was used as infill mortar and for a ferrocement jacket of strengthened columns in series 4 (FSC-4L and FSC-6L). It was injected under pressure through a hole provided in the steel mold. A number of 50×100-mm cylinders were cast to determine their compressive strength. The test result on the compressive strength of the slurry paste is shown in Table 1.

2.2.2 Carbon fiber sheet

Prior to the application of the epoxy coating to the bare column (column SCCF), the concrete surface was cleaned of dust. The carbon fiber sheet, available in a 300 mm wide roll, was then wrapped directly on the fresh epoxy. A hundred millimeters overlapping was provided in the lateral direction with no overlapping for the vertical direction. Any air trapped underneath the carbon fiber sheet was forced out by a hand operated pressure roller. About 30 minutes later, the second epoxy coating was applied on the surface of the carbon fiber sheet.

2.3 Test setup

Four different loading arrangements in controlling the parallelism of the top and bottom stubs during testing, referred to as loading arrangements A, B, C, and D, were used in this investigation. The top and bottom stubs of the column specimens were postoensioned to a number of steel plates (see Table 1) acting as a constant axial load, and to a reaction floor beam, respectively. The loading systems conditions used in this investigation were similar to the actual case in a moment-resisting frame where the tested column was displaced in a double curvature with the point of inflection occurring at the column mid height. Rotation of top stub, if any, can be taken into account when determining the real displacement by adding to or subtracting from the measured lateral displacement, Δ, a quantity of θH, where θ is the measured rotation of the top stub and H is the height of column.

As shown in Fig. 4, a parallel keeping system which consists of a three unit link mechanism was used in loading arrangement A to assure that the top and bottom stubs were consistently parallel. A similar system has been successfully used in the past by Takiguchi *et al.* (1979), and the maximum rotation of the top stub was only 2.0×10^{-3} rad.

Instead of the three unit link mechanism, eight units of hydraulic jack were used in loading arrangements B, C, and D. As it is illustrated in Figs. 5, 6, and 7, the parallelism of the top and bottom stubs of columns were tested by using loading arrangement B and D while in testing was relied upon self adjustment of the hydraulic jacks. However, as indicated in Fig. 6b, in the loading arrangement C, a pump was used to adjust the lack of parallelism during testing.

Although loading arrangements B and D, basically, are similar in controlling the rotation of the top stub, there are some differences in arranging the hydraulic jacks for the parallel keeping system. In loading arrangement B, hydraulic jacks were separated into two series (see Fig. 6a) and post tensioned to the dead load and were placed on top of it. Meanwhile, in loading arrangement D, the distance between hydraulic jacks in the loading direction was four time loading arrangement B. The hydraulic jacks in loading arrangement D were connected to each other to formed one unit

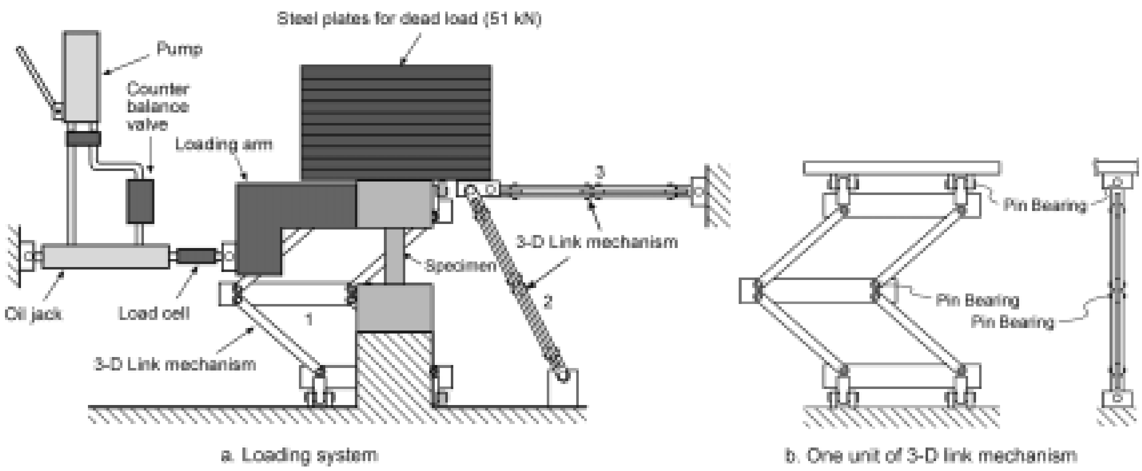


Fig. 4 Loading arrangement A

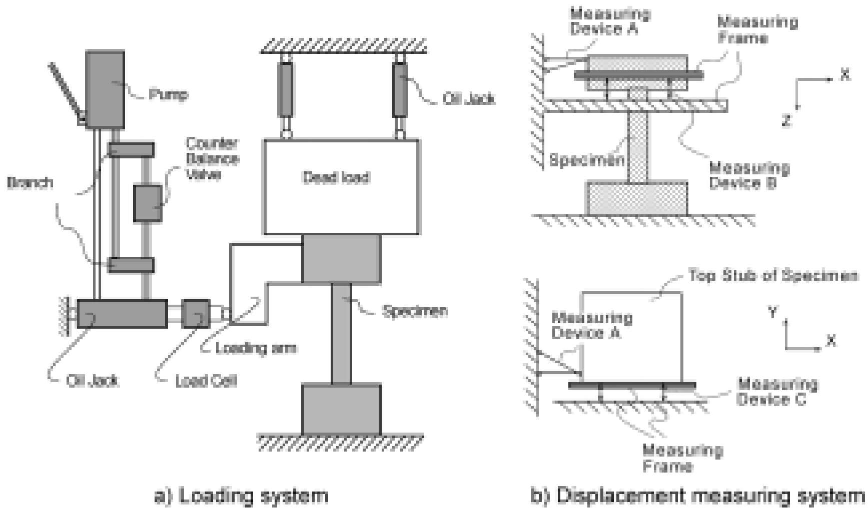


Fig. 5 Loading arrangements B and C

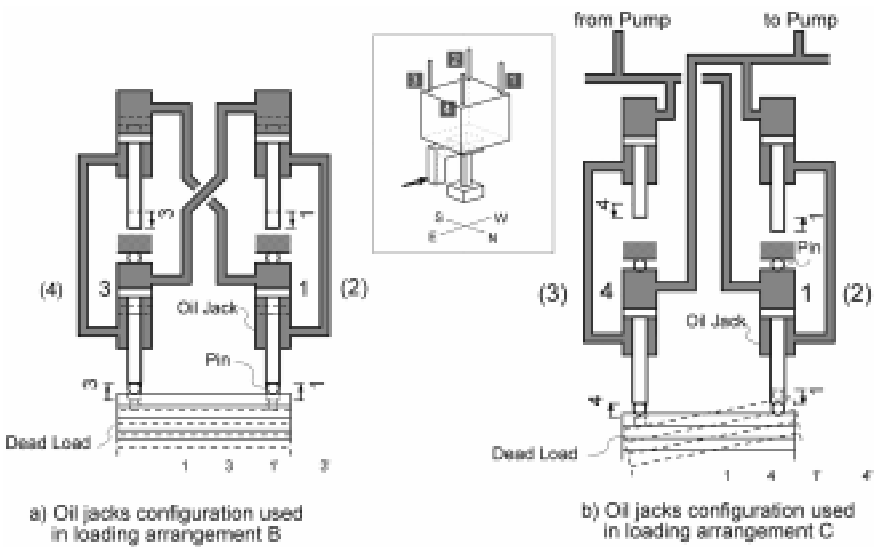


Fig. 6 Detail of oil/hydraulic jacks configuration

configuration and were placed below the dead load (see Fig. 7).

The lateral load was applied by a 500 mm stroke hydraulic jack, with a capacity of 200 kN, connected to the L-shape loading arm at the mid-height of the column. A calibrated load cell was used to monitor and record the applied lateral load. The lateral load sequence was controlled by lateral force or lateral displacement increment.

Specimens in series 1 were tested under a monotonic lateral load while simultaneously being subjected to a 51 kN constant axial load. Meanwhile, unless failure occurred at an earlier stage of loading, specimens in series 2, 3, and series 4 were tested under constant axial load and cyclic lateral forces. Two, and three full cycles of lateral load were loaded to specimens in series 2 and

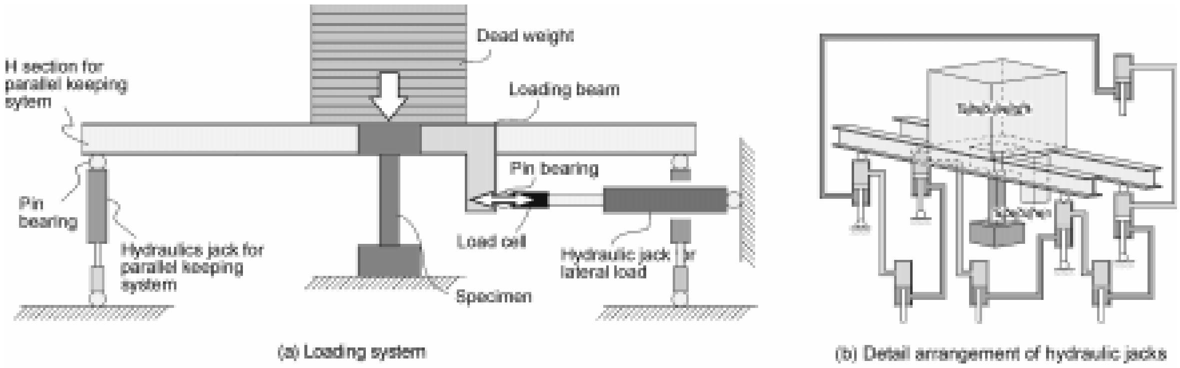


Fig. 7 Loading arrangement D

series 3, and specimens in series 4, respectively, before loading monotonically to failure.

Four unit wire transducers of 1000 mm measuring capacity and four unit LVDTs of 100 mm strokes attached to measuring frames fixed to both side of the top stubs, were used to measure the displacement of the specimens in series 1 during testing. The same measuring system, as illustrated in Fig. 5b, was used to measure the displacement of columns in series 2, series 3, and series 4. The displacements in vertical, horizontal and diagonal directions were measured and recorded by a displacement measuring system consisting of 3 units wire transducers of 1000 mm measuring capacity, and 3 and 2 unit LVDTs of 100 mm and 25 mm strokes, respectively. The measuring system used in this experiment was able to take into account any rotation of the top stub that may occur during testing when the real lateral displacement was determined.

3. Test results and discussions

3.1 General observation

In general, every loading arrangement performed very well in keeping the top stubs of the columns parallel during testing. Fig. 8 shows photographs of specimens tested with different loading arrangements during testing. From this figure it can be seen that the parallel keeping systems employed in all loading arrangements controlled the top stub consistently parallel to the bottom stub even after an extremely large lateral displacement was applied. It is observed, however, that every loading arrangements employed in this investigation had both advantages and disadvantages as well. The installation of the link-mechanism used in loading arrangement A required more time, and more people, and also extra safety precaution was required. Meanwhile, even though installation of parallel keeping systems using oil jacks in the loading arrangements B and D was simple, it has the disadvantage, that when rotation of the top stub occurs during testing, nothing can be done to adjust the unwanted rotation and this rotation may keep increasing as the lateral load is increased. This problem can be solved by attaching a pump to the jacking system, as was proposed in loading arrangement C. Nevertheless, as it was noticed during testing, the testing by using hydraulic jacks in the loading arrangement C was time consuming.

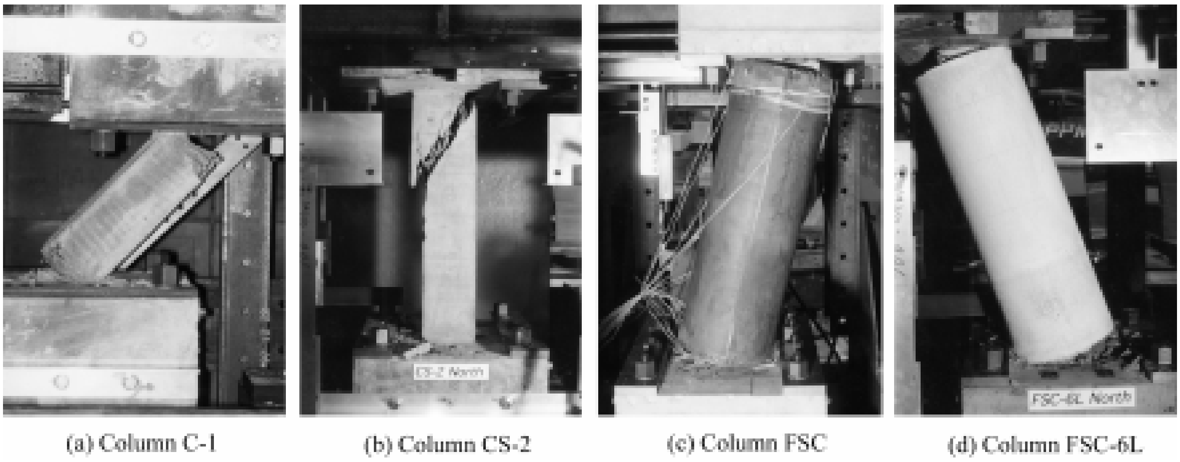


Fig. 8 Columns under testing with different types of loading arrangement

3.2 Performance of parallel keeping systems

Typical rotation of top stub and lateral displacement relationships of specimens during testing are shown in Fig. 9. From this figure it can be seen that both loading arrangements A and D perform well in controlling parallelism of the top and bottom stubs of the columns. Fig. 9a shows that loading arrangement D performed extremely well, where the rotations of the top stub of column CS-2 was very small although the specimens exhibited brittle shear failure and became too unstable to carry horizontal and axial loads due to the penetration to the column by more cracks and transverse reinforcements fracture, as shown in Fig. 8b. However, the rotation of top stub of column CB-2, tested using loading arrangement B after shear failure took place, was significant. This indicates that widening the distance between hydraulic jacks in the loading direction resulted in better performance of loading arrangement D. Fig. 9b shows the top stub rotations and lateral displacement relations of columns C-2 and FSC-6L. Although the rotation of the top stub of column FSC-6L kept increasing during testing, the maximum point was almost the same as was recorded in column C-2 (see Fig. 9b) which was tested using loading arrangement A.

Details of the experimental results are presented in Table 2. The measured rotations of columns tested with loading arrangements A and D show fairly similar results except on the maximum rotation recorded during testing. The highest rotation of top stub recorded in this experimental investigation was when column SCCF was tested using loading arrangement B.

Test results on specimens loaded to large lateral displacement indicated that the counter balance valve attached to the loading systems responded very well. It prevented columns C-1 and C-2 in series 1 from sudden collapse within the unstable stage where the lateral displacement would increase uncontrolled due to P-Δ effect only. Also, it was observed that the counter balance valve controlled the increment of lateral displacement of columns C-1 and C-2 gradually, even after a drift ratio of more than 65%. Here, the drift ratio is defined as the lateral displacement divided by column height. In this investigation, Columns C-1 and C-2 were tested to their total collapse, whereas other columns were tested to their failure, but were terminated when the continuation of the testing was considered to be unsafe.

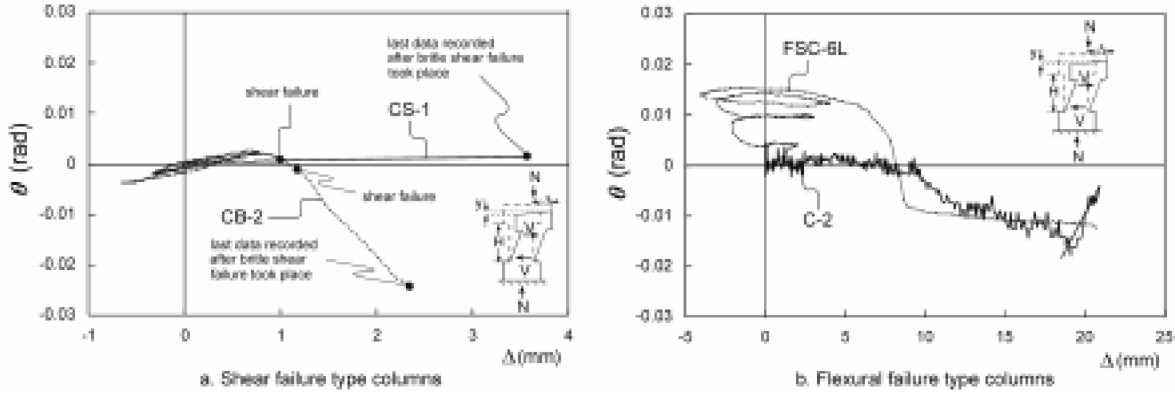


Fig. 9 Typical top stub rotation and displacement curves

Table 2 Summary of test results

Column unit	Loading types	V_f^a (kN)	V_{exp}^b (kN)	Δ_{max}^c (mm)	Δ_u^d (mm)	V_{exp}/V_f	Top stub rotation		Drift ratio	
							at c (%)	at d (%)	at c (%)	at d (%)
C-1	A	28.0	28.6	7.5	40.0	1.02	0.001	0.002	2.1	14.3
C-2		28.2	29.0	15.2	51.8	1.03	0.001	0.001	5.1	17.3
CB-1	B	35.4	28.9	8.3	8.3	0.82	–	–	1.7	1.7
CB-2		35.9	28.3	11.5	11.5	0.79	0.001	0.001	2.3	2.3
SCCF		36.0	30.4	17.0	40.2	0.84	0.010	0.043	2.8	6.7
SCFC	C	35.8	36.5	42.7	135.9	1.02	0.004	0.001	7.1	22.7
CS-1	D	32.1	27.1	10.6	10.6	0.84	0.001	0.001	1.8	1.8
CS-2		32.3	23.4	7.2	7.2	0.72	0.001	0.001	1.2	1.2
FSC-4L		32.1	31.8	18.2	88.6	0.99	0.002	0.010	3.0	14.8
FSC-6L		31.9	32.3	18.4	110.0	1.01	0.005	0.010	3.1	18.3

Δ=lateral displacement.
[a]Shear at theoretical flexural strength.
[b]Maximum measured shear strength.
[c]At maximum shear force.
[d]At failure or test stopped.

3.3 Shear failure type columns

The original columns in series 2 and series 4 fail in shear, as expected, at a drift ratio less than 2.5%. These columns failed by disintegration of core concrete resulting from a lack of concrete confinement and yielding of transverse reinforcement. Both columns CB-1 and CB-2 could not develop their flexural strength and failed in push direction of loading. The maximum measured moments are about 80% of the theoretical strength calculated including the P-Δ effect. Column CB-1 exhibited brittle shear failure before two full cycle of lateral load were completed at a drift ratio of about 1.7%. Similarly, even though a limited ductile response was achieved by column CB-2, it failed in sudden shear failure at a drift ratio of about 2.3%.

As expected, both the CS-1 and CS-2 specimens exhibited brittle shear failure and failed at drift ratios of about 1.8% and 1.2%, respectively, before the target displacement of the first cycle in the push direction was achieved. The ratios of maximum measured shear strength to nominal shear strength, V_n, calculated based on AIJ code approach (AIJ 1994) were 1.04 and 0.89 for column CS-1 and column CS-2, respectively.

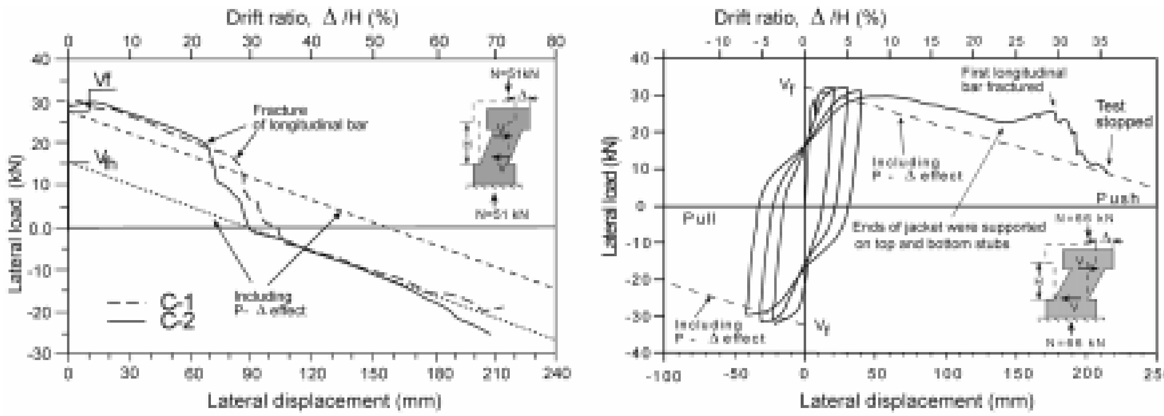
3.4 Flexural failure type columns

The brittle shear failure that occurred in the original columns in series 2 and series 4 was completely prevented by circular ferrocement jackets and the carbon fiber sheet. Note that, due to safety reasons, application of a lateral load on columns SCCF and SCFC that were strengthened with one layer of carbon fiber sheet and eleven layers of wire mesh, respectively, was stopped before full information on ductility could be acquired.

Stable responses were observed until failure during testing of columns FSC-4L and FSC-6L. Column FSC-4L failed due to a fractured jacket followed by a rupturing of transverse reinforcement, and the test was stopped at a drift ratio of 30.1%. Meanwhile, the application of lateral load to column FSC-6L was stopped at drift ratio of 35.6%, when seven out of eight longitudinal bars in both the top and bottom tensile zone fractured.

Typical lateral load-versus-lateral displacement relationships of the tested columns are shown in Fig. 10. Also shown in these figures are the corresponding theoretical lateral loads, V_f, calculated based on an additional theorem where the stress-strain of both steel reinforcement and concrete is assumed to be rigid and in perfectly plastic relation. In this theorem, the yield strength f_y of steel reinforcement and the cylinder compressive strength f_c' of concrete is used and the tensile strength f_t of concrete is assumed as zero in determining the interaction diagram of the column section. Detailed explanations of this theorem are available elsewhere (Suzuki *et al.* 1985, Takiguchi *et al*. 1994). Note that, the shear load V_{fh} in Fig. 10a was calculated by assuming that the longitudinal reinforcements were provided at two corners of the square section only.

Both columns C-1 and C-2 failed in ductile flexural mode. A shear load about 15% higher than the calculated flexural strength was achieved at different drift ratios. First longitudinal reinforcement



a. Columns C-1 and C-2 b. Column FSC-6L

Fig. 10 Typical lateral load-displacement relationships was fractured at drift ratios of about 28.4% and 23.6% for columns C-1 and C-1, respectively, followed by extremely rapid degradation of strength. Similarly, although gaps were provided in between ends of the jacket and adjacent stubs, first longitudinal reinforcement of column FSC-6L was fractured at a drift ratio of about 29.5%.

4. Conclusions

A total of 10 columns tested using 4 different loading arrangements have been presented in this paper. From the test results discussed herein, the following conclusions may be drawn:

1. It was found that in general the loading arrangements performed very well. Depending on their designed strength, every columns tested by the same loading arrangement failed in the same manner.
2. The proposed loading arrangements C and D were able to maintain the parallelism of the top and bottom stubs of columns even whan extremely large lateral displacement was applied.
3. By using loading arrangement A, imposing lateral load on the columns up to their complete collapse while simultaneously keeping the top stub parallel during testing is possible.
4. By attaching a counter balance valve to the loading systems, the lateral displacement of tested columns could be controlled to displaced gradually even within the unstable stage where otherwise the displacement would increase uncontrollably due to P-Δ effect only.
5. In both monotonic and cyclic loading of flexural failure type columns tested in this investigation, the first longitudinal reinforcement fracture was observed at drift ratios around 23.6-29.5%.

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