コンクリート工学年次論文報告集 Vol. 17, No. 2 1995

論文 A Study on the Influence of Shear Reinforcement Ratios to the Seismic Performance of Precast Concrete Columns

Primo Allan ALCANTARA*1, Teruaki YAMAGUCHI*2 and Hiroshi IMAI*3

ABSTRACT: The structural performance of precast concrete columns was studied. It focuses on the use of the relatively new concept in the connections of frame-type structures using precast concrete members in which the concrete joints are located at member ends and rebar joints at portions where the design stresses are small. The main objective is to investigate the structural performance of precast concrete columns having ordinary to high shear reinforcement ratios.

KEYWORDS: precast concrete column, bar splice, shear capacity, shear reinforcement ratio, arch and truss model

1. INTRODUCTION

Precast concrete construction is generally characterized by the fabrication and assembly of structural members. Among its major considerations is the proper location of the concrete and steel joints in order to provide both seismic resistance and construction efficiency. Until recently, there has been no method which could satisfy both conditions at the same time. In view of this, this study aims to investigate the performance of a relatively new type of connection in precast concrete frame-type structures. Here, concrete joints are situated at member ends and reinforcing bar joints at portions where design stresses are small.

This study is the third in the series of experiments [1,2] done regarding the shear capacity of precast concrete columns. Such shear forces could be represented by the occurrence of earthquake movements. The first and the second in the series dealt with the shear performance of centrifuged precast columns and ordinary precast columns with lapping joints under shear forces, respectively. In this study, its main objective is to determine the effect of varying the shear reinforcement ratio to the seismic performance of precast concrete columns.

2. SPECIMENS AND CONSTRUCTION PROCEDURE

A total of 8 column specimens were fabricated: 5 precast concrete and 3 monolithic. These specimens were subjected to cyclic shear forces and high constant axial loads. In the actual design, ordinary strength bars are used for main bars. But in the test, high strength bars were used to make the

*1 Graduate School, University of Tsukuba, Member of JCI
*2 Technical Research Institute, Kabuki Construction Co. Ltd., Member of JCI
*3 Institute of Engineering Mechanics, University of Tsukuba, Member of JCI

specimens fail in shear. Figure 1 shows a layout of the column specimens while Table 1 depicts their column specifications. 20-D22 bars were used as main bars and were arranged around the perimeter of the column cross section. Alongside of each main bar are 2-D16 lapping bars whose total cross sectional area is slightly greater than that of the main bar. The lapping length was set at 30 times the lapping bar diameter. Thereby giving a lapped portion equal to twice the lapping length. Also, the abutting position of the main bars are located at the midheight of the column. The specimens generally vary in the amount of lateral or shear reinforcement. This is done by the variation of the spacing between the lateral hoops. Thereby producing specimens with ordinary shear reinforcement ratio of around 0.6% to a very high ratio of around 1.5%.

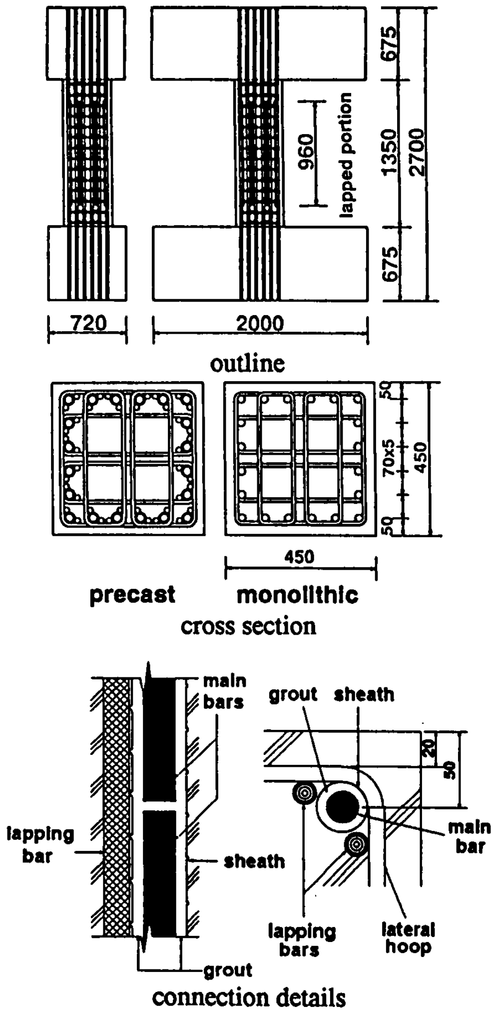

Fig. 1 Column Specimen

The precast concrete column specimen is cast in the following manner. First, the bar cages composed of spiral steel sheaths, lapping bars and closed welded lateral hoops are assembled. It is then placed inside the column formwork and the concrete is cast in a horizontal manner. Once the concrete has set, the formwork is removed. Next, the column is positioned in a manner in which the main bars protruding from the previously made lower beam coincide with the sheaths inside the column. This is then lowered, thereby inserting the lower main bars into the sheaths through its bottom. The outside portion of the column base is then sealed with high strength mortar. After the mortar has completely set, the upper main bars are then inserted into the sheaths through its top. Then, the partially precast upper beam is positioned accordingly. Afterwhich, the unoccupied space between the spiral sheaths and the main bars is filled at the same time with high strength grout from the bottom. Finally, once the grout has completely set, the beam-column joint is cast using ordinary concrete.

Table 1 Column Specifications

Specimen Precast	Specimen Monolithic	Hoop Spacing mm	Pw %	Axial Load kgf/cm^2
C31P	C31M	150	0.63	60
C32P	-	120	0.79	"
C33P	C33M	100	0.95	"
C34P	-	80	1.19	"
C35P	C35M	65	1.46	"

Common b x D x h : 45 cm x 45 cm x 135 cm
main bars : 20-D22 (SD 685)
lapping bars : D16 (SD 685)
lateral hoops : D10 (SD 295A)
concrete strength : 300 kgf/cm^2
grout strength : 600 kgf/cm^2
sheath : diameter 34 mm
lug height 2 mm
pitch 28 mm

3. MATERIAL PROPERTIES

Table 2 shows the test results of the rebars, concrete and the grout used. As for the concrete, it conforms with the specified strength Fc of 300 kgf/cm^2 on the average. As for the grout, test results show a very much larger value than its specified strength.

4. TEST METHOD

The column specimens were subjected to varying shear forces which were applied continuously in a cyclic manner producing anti-symmetric bending moment distribution while being acted upon by a constant axial load (60 kgf/cm^2 = 0.2 Fc). The application of shear forces were controlled by the following loading history. Each column specimen was made to drift once at a drift angle R equal to 1/800, then twice at R of 1/400, 1/200, 1/100 and 1/50 and again once at R equal to 1/25. Figure 2 shows the loading setup. The column specimen is set using oil jacks found on both sides of the upper and lower beams. Here, the shear force is applied through the horizontal actuator.

Relative displacements between the upper and lower beams were measured using displacement transducers located on both sides of the specimen. Along the height of the specimen, clip gauges were systematically arranged to measure local deformation. Also, the slip between the column and the beam were measured. These gauges are shown in Fig. 3.

Table 2 Material Test Results

(a) Reinforcing Bars (kgf/cm^2)

Size	Grade	Young's Modulus	Yield Stress	Tensile Stress	Elongation %	Remarks
D22	SD 685	1.86x10^6	6,870	8,920	13.5	Main
D16	SD 685	2.02x10^6	7,500	9,720	12.0	Lapping
D10	SD 295A	1.87x10^6	3,720	5,310	28.0	Hoop

(b) Concrete (kgf/cm^2)

Specimen	Compression	Split
C31P	301	27.7
C32P	301	28.9
C33P	299	32.3
C34P	302	33.9
C35P	286	29.8
C31M	296	24.9
C33M	294	22.6
C35M	305	24.0

(c) Grout (kgf/cm^2)

specified strength	600
7 days	589
28 days	681
48 days (first day of expt)	728
77 days (last day of expt)	748

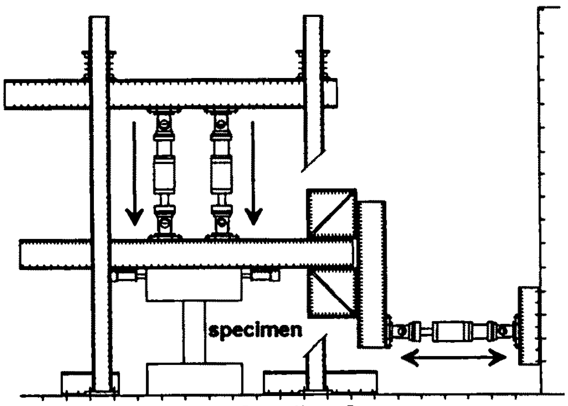

Fig. 2 Loading System

5. TEST RESULTS AND DISCUSSIONS

5.1 CRACK PATTERNS

For all the specimens, initial cracking due to flexure took place when R was 1/800. Progressive cracking on the initially damaged portion continued at R=1/400 as well as the appearance of initial shear cracks except for C35P and C35M which have the greatest number of lateral hoops. At R=1/200, shear cracks continue to propagate, extending over the height of the column, and this continued until R=1/100 where the shear cracks became rampant. However, as seen from Fig. 4, there are less cracks present for columns with a higher shear reinforcement ratio Pw. Diagonal cracks running from corner to corner can already be seen at R=1/50, however, they become less pronounced as Pw increases. Also at this stage, the widening of the shear cracks occurred except for columns with high shear reinforcement ratio wherein this took place at R=1/25. At R=1/25, chipping-off of the concrete cover and appearance of more fine cracks could be seen on precast concrete columns compared to monolithic columns. However, wider cracks were present in monolithic than in precast. For all specimens, they all failed in shear or bond.

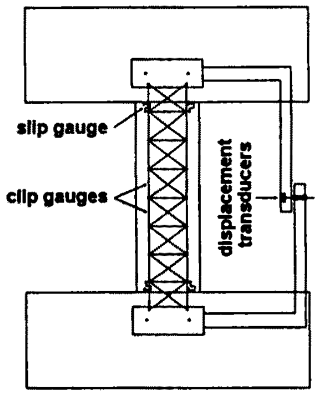

Fig. 3 Gauge Arrangement

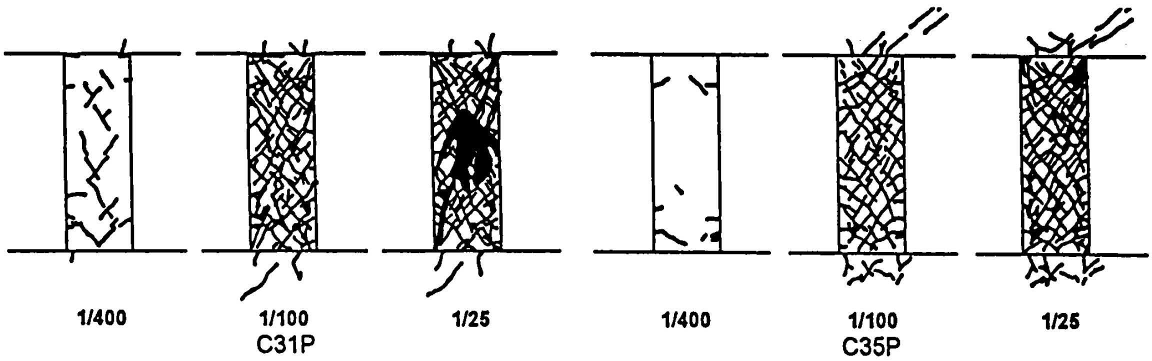

Fig. 4 Crack Patterns

5.2 LOAD-DISPLACEMENT RELATIONSHIPS

Figure 5 shows curves depicting the relationship between the applied shear force Q and the horizontal displacement or drift angle R. It can be observed that there is little difference in the overall shape of the curves which shows the similarity of the behavior of the specimens under loading. The only difference lies on the maximum shear load. Also, these diagrams generally feature a shear-type of failure. However, bond splitting cracks could be seen on the sides of the column.

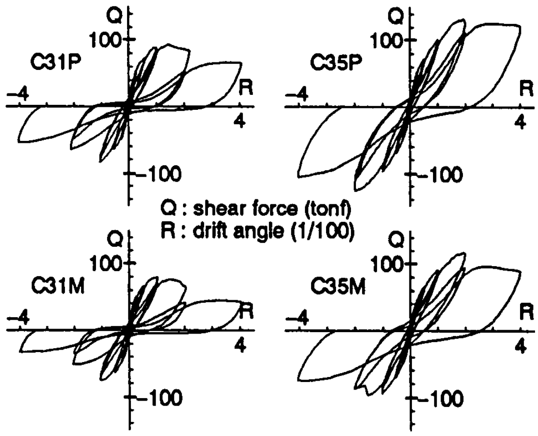

Fig. 5 Shear Force-Drift Angle Curves

Figure 6 shows the envelope curves of the shear force - drift angle relationships. It can be observed that the shear capacity of the specimen increases as the shear reinforcement ratio Pw is increased from ordinary to high. Also, it can be seen that the shear capacity of the precast concrete specimens are generally higher than that of the monolithic ones.

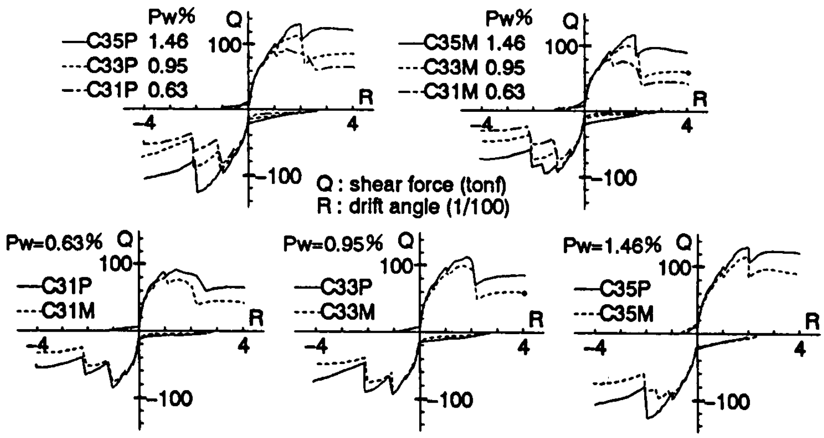

Fig. 6 Shear Force-Drift Angle Envelope Curves

5.3 PARTIAL DEFORMATION

A typical distribution of shear distortions and curvatures are shown in Fig. 7. Shear distortions tend to become large at the middle portion of the columns while the curvatures are seen to be similar to the shape of the assumed bending moment distribution. Also, there is no significant difference between precast and monolithic specimens with regards to their shear distortions and curvatures.

Calculations using such shear distortions and curvatures give partial deformations due to shear and bending, respectively. These deformations would add up to the total deformation on the column which is very close to the actual relative displacement between the upper and lower beams at each cycle. Such deformation components are shown in Fig. 8. From the diagrams, it can be seen that at small drift angles, most of the deformation is due to the bending component because of the presence of initial flexural cracks. And as the drift angle increases, the bending component decreases and the shear component gradually increases due to the occurrence of shear cracks. Also, the percentage of shear deformation is greater for specimens with lesser shear reinforcement and the curves of partial deformations of precast and monolithic specimens of identical reinforcement seem to coincide as the shear reinforcement ratio increases.

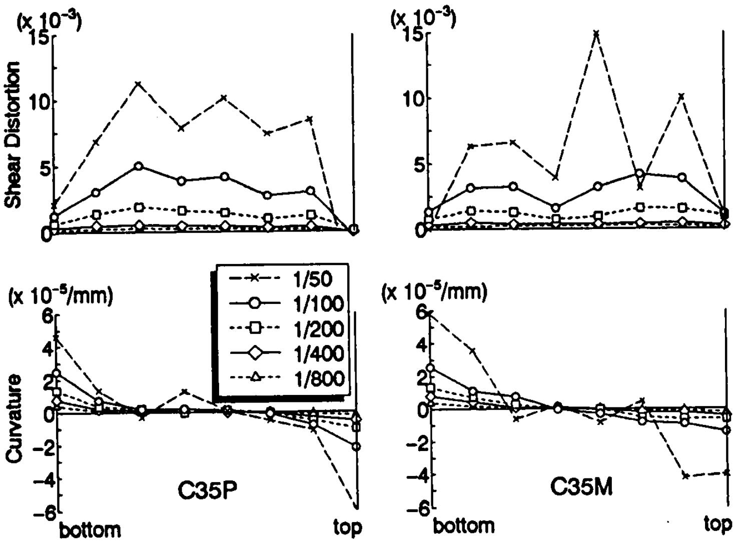

Fig. 7 Shear Distortion and Curvature Distribution

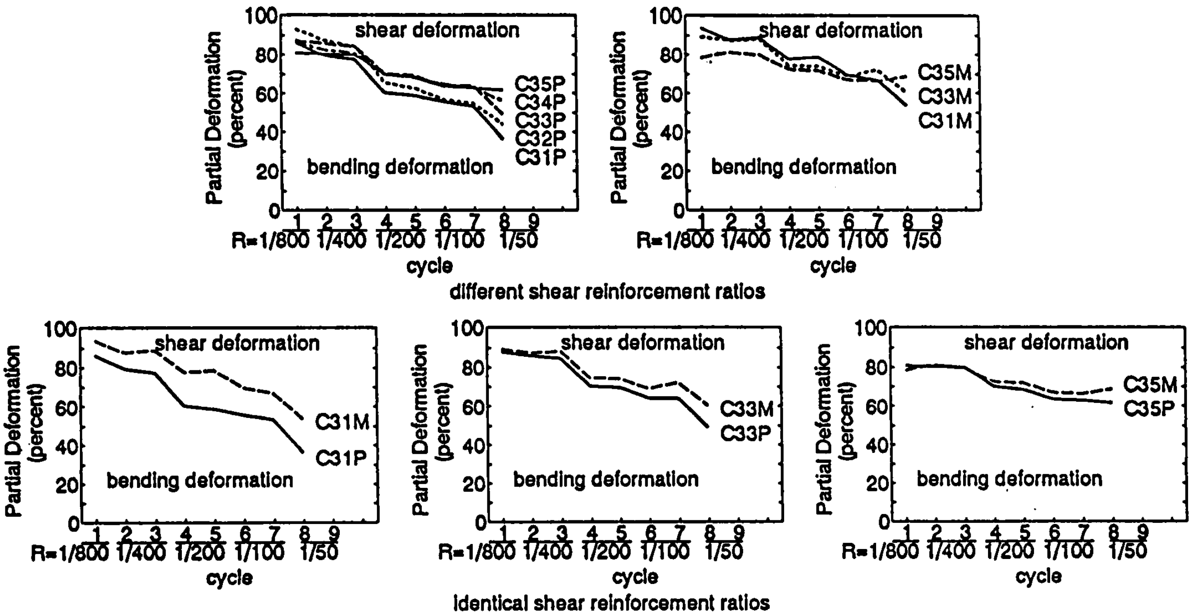

Fig. 8 Partial Deformation

5.4 ULTIMATE SHEAR STRENGTH

The shear capacity of columns could be calculated by the strength equation given in the Architectural Institute of Japan's Structural Design Guidelines [3] shown below.

$$Q_{su} = bj_t p_w \sigma_{wy} \cot\phi + \tan\theta(1-\beta)bD\nu\sigma_B/2 \quad (1)$$

This equation is actually based on the Arch and Truss Model Theory in structural mechanics wherein the left expression is the one contributed by the truss mechanism while the right expression by the arch mechanism.

Table 3 shows the ultimate shear strength calculations for both the A and B methods. For the precast concrete specimens, it shows that the experimental results are higher than the theoretical values for both methods. However, for the monolithic specimens, the experimental results show a slightly lower value for the A method. Therefore, it could be considered that the ultimate strength methods proposed by the AIJ both proved to be conservative representations of the actual shear strength especially for the precast concrete type.

Lastly, Fig. 9 shows the behavior of the shear strength Q_{su} against the shear reinforcement ratio Pw. From the graph, it could be seen that there is a rough agreement between the test results and the calculated values except that the actual capacity for precast concrete columns is greater.

Table 3 Ultimate Shear Strength

(tonf)

Specimen	Calculated USD A	Calculated USD B	Experimental	Expt./Calc. USD A	Expt./Calc. USD B
C31P	81.7	78.5	92.7	1.13	1.18
C32P	95.4	85.8	105.0	1.10	1.22
C33P	106.5	93.2	113.8	1.07	1.22
C34P	114.9	104.1	126.9	1.10	1.22
C35P	122.0	116.8	131.5	1.08	1.13
C31M	81.7	78.5	80.8	0.99	1.03
C33M	106.5	93.2	100.5	0.94	1.08
C35M	122.0	116.8	117.2	0.96	1.00

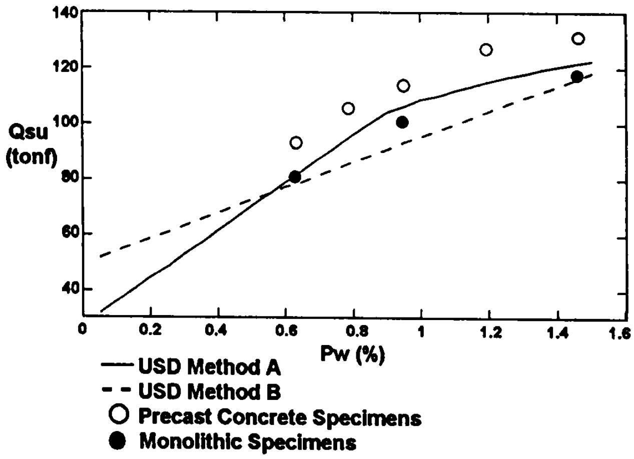

Fig. 9 Ultimate Shear Strengths

6. CONCLUSIONS

The following could be concluded from the experiment and the analysis made:

a) The performance of precast concrete columns with lapping joints is generally similar, or even better, than that of monolithic columns with identical concrete strength and reinforcement.
b) The Arch and Truss Model Theory can be used in calculating shear strengths of precast concrete columns having ordinary as well as high shear reinforcement ratios.
c) Also, the shear capacity of precast concrete columns is higher than that of monolithic columns.

REFERENCES

1. Kobayashi, T., Yamaguchi, T., and Imai, H., "Shear Performance of Centrifuged Precast Columns with Lapping Joints," Trans. of the Japan Concrete Institute, Vol. 13, 1991, pp.589-596.
2. Kobayashi, T. and et al., "An Experimental Study on the Seismic Performance of Precast Concrete Columns under Shear Forces," Trans. of the Japan Concrete Institute, Vol. 14, 1992, pp.409-416.
3. Architectural Institute of Japan, "AIJ Structural Design Guidelines for Reinforced Concrete Buildings," 1994, pp.77-123.