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A STUDY ON THE PERFORMANCE OF A NEW TYPE OF CONNECTION IN PRECAST CONCRETE COLUMNS SUBJECTED TO SHEAR FORCES

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ABSTRACT

The structural performance of precast concrete columns was studied. It focuses on the use of the *main bar post-insertion method* in the connection 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; main bar post-insertion method; shear capacity; shear reinforcement ratio; sheath.

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 that 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 called the *main bar post-insertion method*. Here, concrete joints are situated at member ends and reinforcing bar joints at portions where design stresses are small. A simple diagram depicting the proposed system in precast concrete frame-type structures is shown in Fig. 1. This study is the third in the series of experiments (Kobayashi *et al.*, 1991, 1992) 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 or the amount of lateral reinforcement to the seismic performance of precast concrete columns.

MAIN BAR POST-INSERTION METHOD

The *main bar post-insertion method* is described as the process wherein, at the factory, the precast members are prefabricated without the presence of the main bars, and later at the construction site, the main bars are

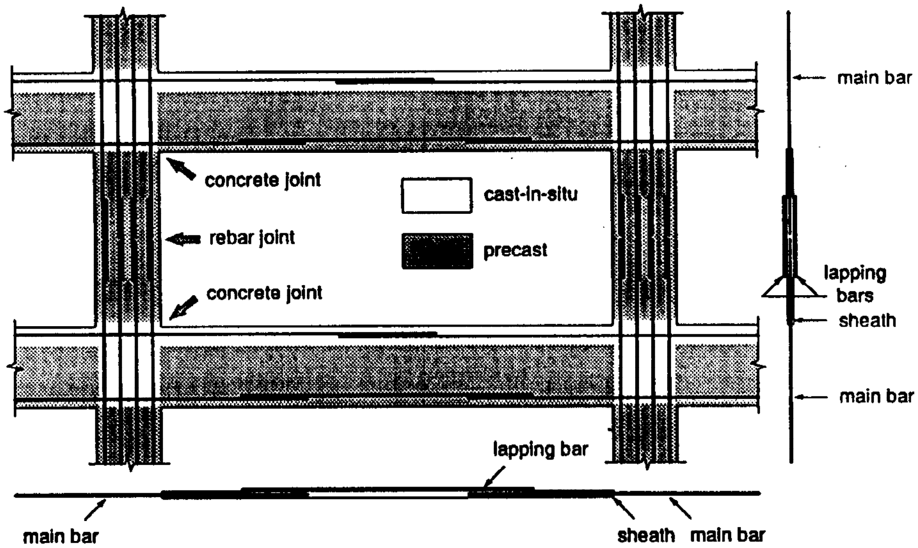


Fig. 1. Proposed Frame-Type System for Precast Concrete Structures

inserted and abutted at the middle portion of each member where the stresses due to seismic forces are small. This type of joint features the use of spiral steel sheaths which are hollow tubes positioned in place of the main bars in precast concrete members wherein such bars are to be later inserted during assembly.

The *main bar post-insertion method* applied to precast concrete columns could be described in the following step-by-step process. First, at the prefabrication site, the reinforcing bar cage is assembled. Here, the spiral steel sheaths are positioned on the location where the main bars are to be later installed. Next, two lapping bars are arranged along the sides of the middle portion of each sheath. Afterwhich, the shear reinforcement or lateral hoops are set according to the desired spacing or interval. Then, the formwork is constructed and the concrete is cast. Later, at the construction site, just before the concrete is cast on the foundation beam, steel bars which are to form the lower half of the main bars of the first-storey column are fixed on the position where the column is to stand. Next, using a crane, the precast column is carried over the protruding bars and then lowered, thereby inserting them into the sheaths through the bottom. Afterwhich, the next floor precast beams and slabs are placed in their proper positions and their main bars arranged accordingly. Then, steel bars which are to form the upper half of the main bars of the lower column and the lower half of the main bars of the upper column are inserted into the sheaths through the top. Next, high strength grout is injected into the sheath to bond together the inserted main bars with the sheath and the whole column. After the grout has hardened, an appropriate formwork for the beam-column joint is constructed and the concrete is cast. This process is simply repeated to construct the upper floors and the whole building is completed.

SPECIMEN DETAILS

A total of 8 column specimens were fabricated: 5 precast concrete and 3 monolithic. These specimens were subjected to cyclic shear forces and to a relatively high constant axial load. The column cross-sectional dimensions were set at 45 cm by 45 cm and the column height at 135 cm. This gives a shear span ratio, M/QD, equal to 1.5. In actual design practice, ordinary strength bars are used for main bars. But for experimental purposes, high strength bars were used in order for the specimens to fail in shear. Figure 2 shows a layout of the column specimens while Table 1 depicts their column specifications. For the precast concrete specimens, 20-D22 bars were used as main bars and were inserted into the spiral steel sheaths that were arranged around the perimeter of the column cross section. High strength grout was also injected into the sheaths so as for the main bar and the sheath to act as a unit. Alongside each sheath 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. This was done to ensure adequate force transmission along the main bars. Also, the abutting positions of the main bars are located at the midheight of the column where the design stresses are minimal. On the other hand, for the monolithic specimens, a similar design to that of the precast specimens was carried out except

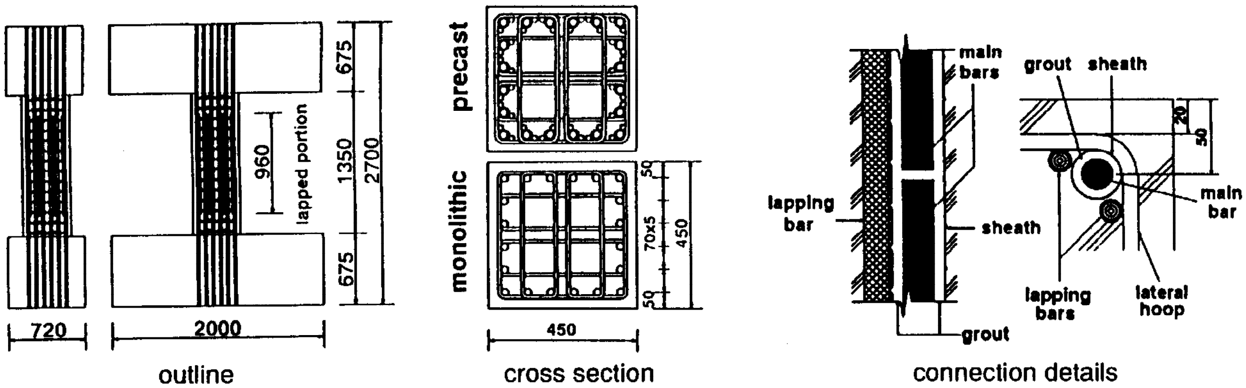


Fig. 2. Column Specimen

for the absence of the use of the spiral steel sheaths and lapping bars. Furthermore, reaction beams located on both ends of the column were fabricated and were designed in a manner wherein no apparent damage could occur during the experiment.

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%. These values were chosen in order to have a correlation of this study to the previous experiment (Kobayashi *et al.*, 1992) and to investigate the behavior of precast concrete columns having high shear reinforcement ratios. Also, monolithic specimens were tested to have a basis of comparison in the analysis of the performance of the *main bar post-insertion method.*

Table 1. Column Specifications

Specimen		Hoop Spacing	Pw	Common Details
Precast	Monolithic	mm	%	
C31P	C31M	150	0.63	b x D x h : 45 cm x 45 cm x 135 cm main bars : 20-D22 (SD 685)
C32P	-	120	0.79	lapping bars : D16 (SD 685) lateral hoops : D10 (SD 295A)
C33P	C33M	100	0.95	concrete strength : 300 kgf/cm^2 grout strength : 600 kgf/cm^2
C34P	-	80	1.19	sheath : diameter 34 mm lug height 2 mm
C35P	C35M	65	1.46	pitch 28 mm axial load : 60 kgf/cm^2

MATERIAL PROPERTIES

Except for the D22 main bars, the actual strengths of the reinforcing bars were greater than their specified values. As for the concrete, it conforms with the specified strength Fc of 300 kgf/cm^2 on the average. Lastly, the grout test results show larger values, 700-750 kgf/cm^2, than its specified strength of 600 kgf/cm^2.

TEST METHOD

Each of the column specimens was set under the loading apparatus shown in Fig. 3. These specimens were subjected to varying shear forces that 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). Each column specimen was set using oil jacks found on both sides of the upper and lower beams. Here, the shear force was applied through the horizontal actuator while the axial load was provided by the two vertical actuators shown.

The application of shear forces was controlled by the following loading history. For the proper simulation of seismic behavior, each specimen was made to drift once at a drift angle R equal to $\pm 1/800$, then twice at R of $\pm 1/400$, $\pm 1/200$, $\pm 1/100$ and $\pm 1/50$ and again once at R equal to $\pm 1/25$. Here, the drift angle is defined as the ratio of the relative displacement between the upper and lower beams to the column height. To follow the prescribed loading history, 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 also shown in Fig. 3. Asides from these, strain gauges were strategically positioned all over the reinforcing bars of the specimen.

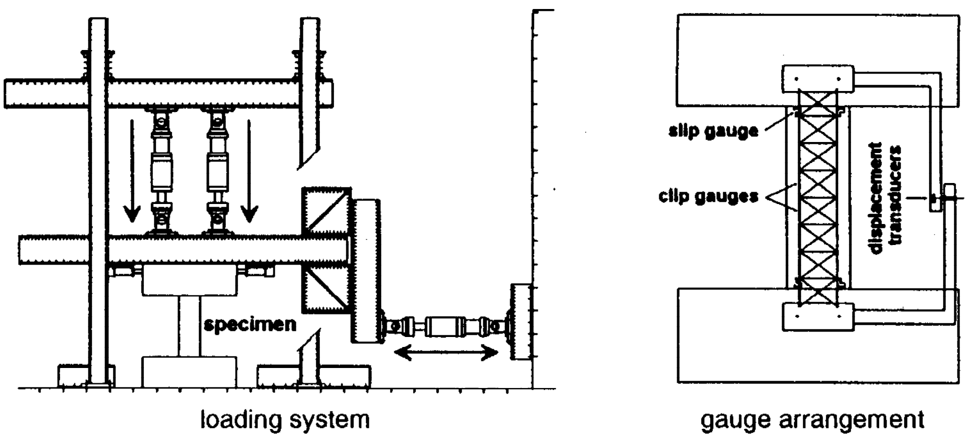


Fig. 3. Experimental Procedure

EXPERIMENTAL RESULTS AND ANALYSIS

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 fewer 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, spalling of the concrete cover and appearance of more fine cracks could be seen on precast concrete columns compared to monolithic columns while wider cracks were present in monolithic than in precast. For all specimens, they all failed in shear. However, bond splitting cracks could be seen on the sides of the columns.

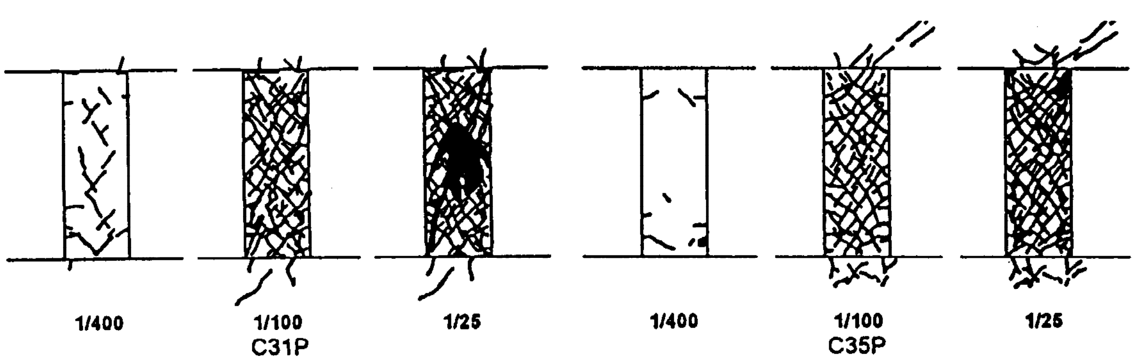


Fig. 4. Crack Patterns

Load-Displacement Relationships

Figure 5 shows curves depicting the relationship between the applied shear force Q and the drift angle R as well as their envelope curves. From the set of graphs on the left, it can be observed that there is little difference in the overall shape of the curves between the precast and monolithic specimens, which shows the similarity of the behavior of the specimens under loading. The only difference lies on the maximum shear load attained. Except for C31M, the specimens attained their maximum shear load at R equal to 1/50. Also, these diagrams generally feature a shear or bond failure type. Furthermore, the graphs on the right show a comparative analysis on the effect of the shear reinforcement ratio as well as the casting method based on the envelope curves of the shear force - drift angle relationships. It can be observed from the upper graphs that the shear strength capacity of the specimen increases as the shear reinforcement ratio Pw is increased from ordinary to high for both the precast concrete and monolithic types. This proves that column having a large amount of shear reinforcement is more ductile and stronger in resisting seismic shear forces. Also, it can be seen from lower graphs that the shear capacity of the precast concrete specimens is generally higher than that of their monolithic counterparts having the same amount of lateral reinforcement.

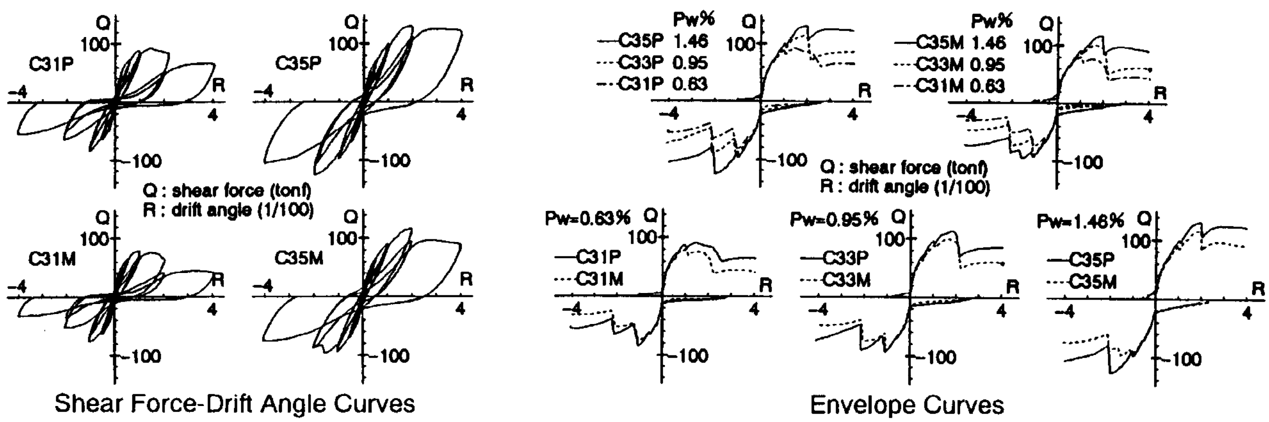


Fig. 5. Load-Displacement Relationships

Partial Deformation

In order to calculate the deformations due to shear and flexure as shown in Fig. 6, it is necessary to first determine the shear distortion and curvature values at specific locations along the height of the column. Shear distortion and curvature of the column could be calculated using the clip gauges found on Fig. 3. The diagonal clip gauges were used in determining the shear distortion while the vertical ones give the curvature for each subdivision. With regards to shear distortions, they tend to become large at the middle portion of

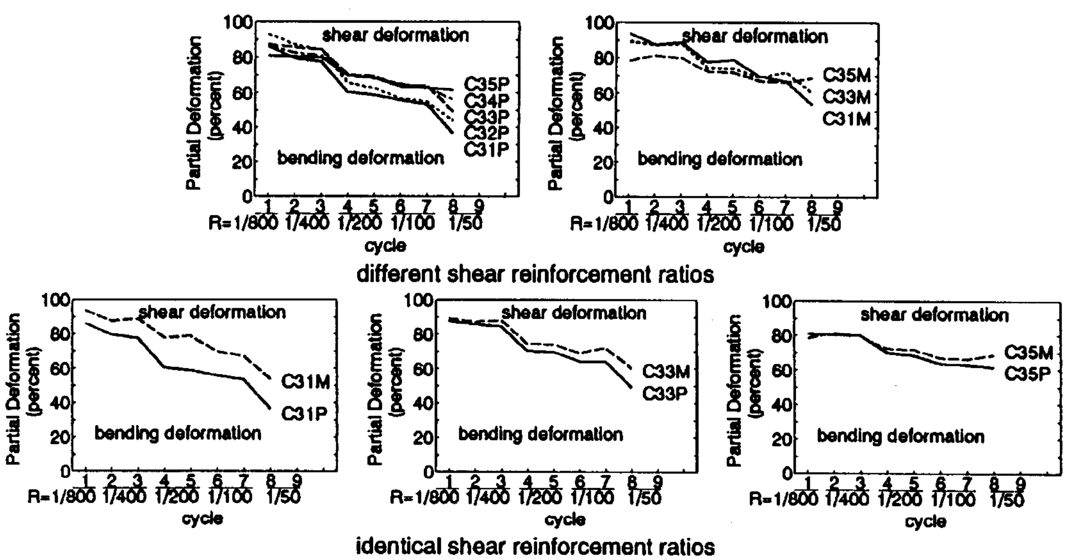


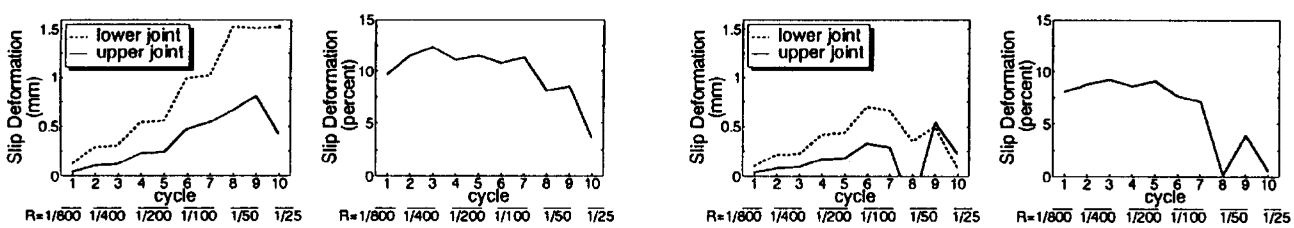
Fig. 6. Partial Deformation Curves

the columns and their peak values tend to decrease as the shear reinforcement ratio increases. On the other hand, 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 flexure, 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 for each step. From the diagrams, it can be seen that at small drift angles, most of the deformation is due to the flexural component because of the presence of initial bending cracks. And as the drift angle increases, the bending component decreases and the shear component gradually increases due to the occurrence of shear cracks. Lastly, it could be observed that the percentage of shear deformation is greater for the precast concrete specimens as compared to their monolithic counterparts having the same Pw. However, these curves seem to coincide as the shear reinforcement ratio increases.

Slip Deformation

In order to determine the performance of the concrete joints, slip gauges were installed at the column ends, as shown in Fig. 3. Two slip gauges were arranged at the upper joint wherein ordinary concrete was used in the casting of the beam-column joint while another two were put at the lower joint wherein a high strength mortar seal was used. The data from which was also used in the comparison with the monolithic specimens.

Actual slip deformation of the columns at the upper and lower portions as well as its percentage in the total deformation at each cycle for specimens C35P and C35M are shown in Fig. 7. As for the slip deformation values recorded, none of the precast concrete specimens exceeded 1.5 mm. while none of the monolithic specimens exceeded half that value. Thus it could be considered that the monolithic specimens are more resistant in slip compared to their precast concrete counterparts. This is naturally expected since in the precast concrete specimens, the column and beam members are cast individually and are just connected at a later time, while in the monolithic specimens, they are cast as a whole. However, considering the magnitude of the slip deformation, these values are reasonably negligible compared to the total deformation given to the columns. This could be seen on the right hand graphs wherein the percentage of slip deformation is only 10 percent of the total deformation on the column. Therefore, the performance of the beam-column joints is said to be satisfactory.



C35P C35M

Fig. 7. Slip Deformation

Strain Distribution

Typical samples of the strain distribution on both the main bar and the lapping bars at a corner section are shown in Fig. 8. By simply combining both diagrams, a representation of the actual strain distribution on a continuous longitudinal bar could be derived. As seen from the right-hand side of the figure, such strain distribution is very much similar to the actual strain distribution of their monolithic counterparts as well as to the assumed bending moment distribution on the column. Also, it could be inferred that there is adequate force transfer from the main to the lapping bars of the proposed connection system used.

With regards to the shear reinforcement, strain gauges were attached on bars along and perpendicular to the shear loading. Considering the strain of the bars positioned along the loading direction, it was observed that the lateral hoops situated at the middle portion tend to yield first due to the presence of the shear cracks. Moreover, the lateral hoops for specimens with lesser amount of shear reinforcement yielded at R=1/100 while those with a higher amount yielded at R=1/50 to 1/25. On the other hand, for bars positioned perpendicular to the loading direction, there is not much yielding of the lateral hoops in general.

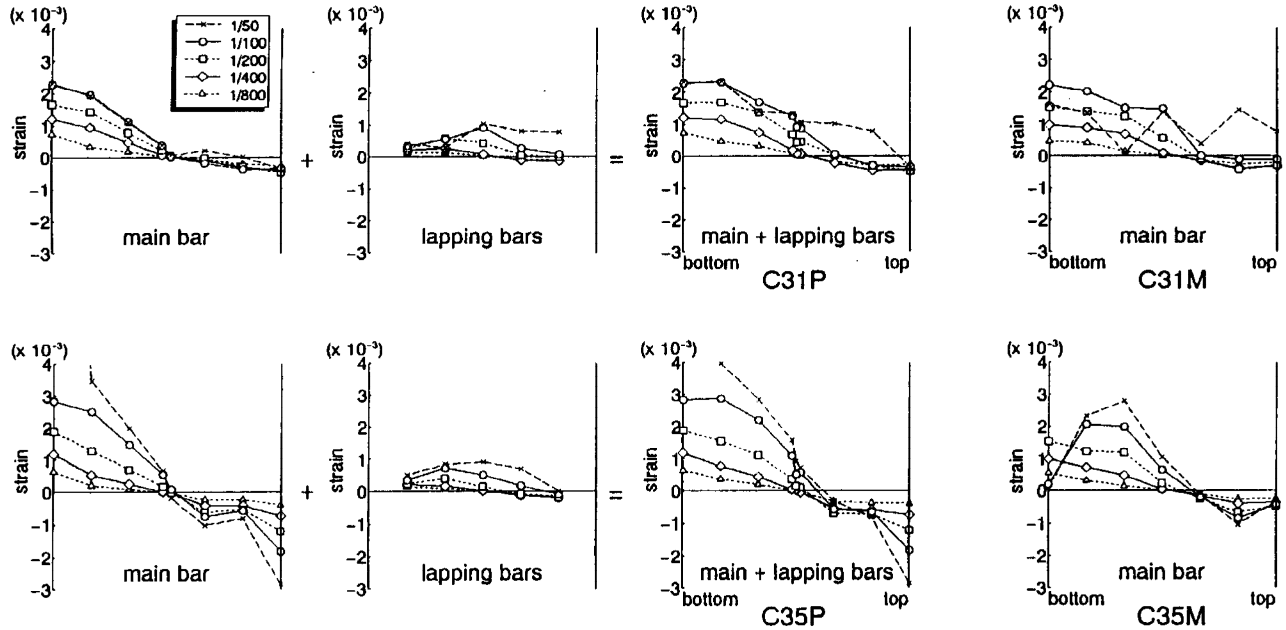


Fig. 8. Strain Distribution

Ultimate Shear Strength

The shear capacity of columns could be calculated by the strength equation given in the Architectural Institute of Japan's Ultimate Strength Design Guidelines (1994), hereafter, USD, which is actually based on the Arch and Truss Model Theory in structural mechanics. Table 2 shows the ultimate shear strength calculations (shear and bond splitting failures) for both the A and B methods as well as the flexural capacities and experimental results for the specimens. Here, the bond splitting capacities were calculated using equations proposed by Fujii-Morita (1982) and Kaku (1992) wherein the bond strength is obtained using the sheath diameter as the main bar diameter for the precast concrete specimens. Based on the observed crack patterns and time of yielding of the lateral hoops during the experiment, it could be said that all the specimens failed in shear first followed by bond splitting. However, according to the USD Method A, which is a more accurate representation based on the angle of the cracks observed, the specimens should have failed in bond splitting rather than in shear. Thus, the calculated values for bond splitting capacity could be considered as conservative representations of the actual bond strength since the experimental results show values closer to the calculated shear capacities rather than the bond splitting capacities.

Table 2. Ultimate Strength

(tonf)

Specimen	Calculated Values							Test Results
	Shear Capacity		Bond Capacity (Fujii-Morita)		Bond Capacity (Kaku)		Flexural Capacity	
	USD A	USD B	USD A	USD B	USD A	USD B		
C31P	81.7	78.5	66.0	90.9	69.5	94.6	135.5	92.7
C32P	95.4	85.8	68.6	93.8	74.6	100.2	135.3	105.0
C33P	106.5	93.2	71.3	96.6	79.7	105.6	135.3	113.8
C34P	114.9	104.1	75.3	100.9	87.4	113.8	135.3	126.9
C35P	122.0	116.8	79.9	105.8	96.2	123.2	135.3	131.5
C31M	81.7	78.5	65.0	89.8	71.0	96.3	135.3	80.8
C33M	106.5	93.2	70.3	95.5	81.8	107.8	135.3	100.5
C35M	122.0	116.8	78.9	104.7	98.7	126.0	135.3	117.2

Lastly, Fig. 9 shows the behavior of the shear strength Q_{su} against the shear reinforcement ratio Pw. The figure also includes results from the previous experiment (Kobayashi *et al.*, 1992) which has specimens with a low 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 is greater. However, the actual capacity shown by their monolithic counterparts is a little bit lower than the calculated values obtained from USD Method A. Such a discrepancy between the actual capacities of precast and monolithic specimens with identical reinforcement could be qualitatively explained by the presence of both the sheath and the lapping bars which provide better core concrete confinement.

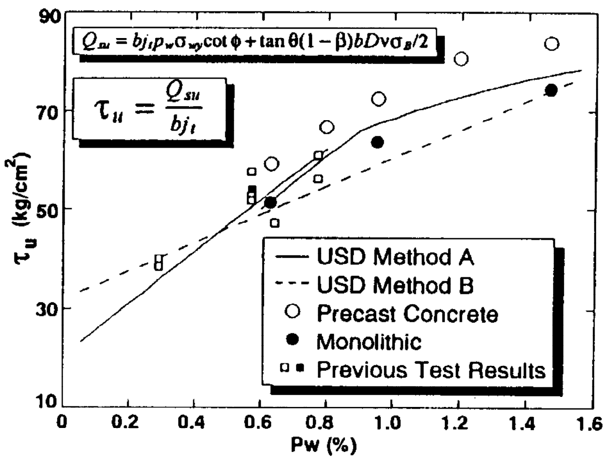


Fig. 9. Ultimate Shear Strength

CONCLUSIONS

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

a) The performance of precast concrete columns using the *main bar post-insertion method* is generally similar, or even better, than that of monolithic columns with identical concrete strength and reinforcement.

b) The main bar joints and the concrete joints of the proposed connection system showed satisfactory performance based on the reinforcing bar strain distributions and slip deformation values recorded.

c) 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.

d) The calculated bond splitting strengths using Fujii-Morita and Kaku equations could be considered conservative representations of the actual bond strength for columns of high Pw.

e) The shear capacity of precast concrete columns is higher than that of monolithic columns.

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