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# Experimental studies on shear behavior of shear-wall used precast concrete

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ABSTRACT: It is important for precast concrete structures to make clear the shear behavior, especially the behavior of member-to-member joint. The objectives of this report is to make clear the effect of joint method in wall-column joint, weaving method of shear reinforcements. shear span ratio and middle beam on the deformability and destruction pattern of the shear wall used precast concrete. On this report we explain the relation between the joint method in wall-column joint and the shear behavior of specimens, the shear span ratio and the shear behavior of specimens. We also compare the shear strength formula and experimental result, then we explain that the formula makes strength of the shear wall used precast concrete larger than experimental result.

## 1 Introduction

On the reinforced precasted concrete structures, it is expected that the difference of jointmethod between members has large influence on the seismic characteristics of the structure.Especially, we cannot fully get hold of the effect of the characteristics of wall-column and wall-beam joints on the hysteresis of the shear wall that is essential seismic element.

On this study, we performed the experiments to investigate the influences of joint method between reinforced precasted concrete shear wall and surrounding frame, especially shear wall and column joint, on the seismic characteristics of the structure.

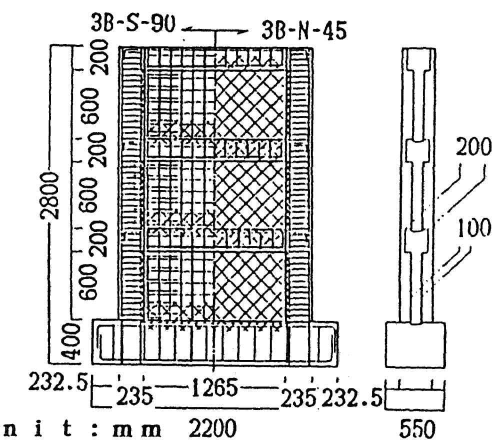


Fig.1 Details and Arrangement of Specimens

## 2 Outlines of Experiments

### 2.1 Specimens

The lists of specimens are shown in Table 1.

Table 1 List of Specimens

| Bars Arrangement and Joint Method | | | Specimen Form | | | |
|---|---|---|---|---|---|---|
| Wall Reinforcement | Wall-Beam Joint | Wall-Column Joint | 3-story with middle beam M/QL=1.53 | 2-story with middle beam M/QL=1.00 | 2-story without middle beam M/QL=1.00 | 1-story M/QL=0.47 |
| D6 @100 Double and 45 slant arrangement ps=0.64% | | Buring wall in column to 10mm | 3B-N-45 | 2B-N-45 | 2N-N-45 | Undesigned |
| | | Cotter | 3B-C-45 | 2B-C-45 | Undesigned | 1B-C-45 |
| | | Dowel bars | 3B-S-45 | Undesigned | 2N-S-45 | Undesigned |
| D6 @100 Double and normal arrangement ps=0.64% | D6 @125 single and 45 slant arrangement ps=0.57% | Dowel bars | 3B-S-90 | 2B-S-90 | Undesigned | 1B-S-90 |

Table 2 Details and Arrangement of Specimens

| Wall Thickness | Measurement | | Longitudinal Bars | | Steel Plates | | | Shear Reinforcement | |
|---|---|---|---|---|---|---|---|---|---|
| | Column | Beam | Column | Beam | Column | Beam | | Column | Beam |
| 100(mm) | 300×235 (mm) | 200×200 (mm) | 8-D13 | 4-D10 | H-200×75 ×8×16 | (center) PL-16×80 | (near column) PL-16×80+2PL-16×40 (cross section) | D6 @50 | D6 @100 |

The details and bar arrangement are shown in Fig. 1 and Table 2. Specimens are made on a scale of about one to four of the real wall of building. The thickness of specimens, however, is half of the real because there is not so small size of bar. Specimens are supposed as the shear wall in lower stories of multistory reinforced precasted concrete buildings (about 15 stories). The arrangement of wall reinforcing bar, number of stories (shear span ratio M/QL), the existence of middle beam and joint method between wall and column were chosen as parameters of specimens. Two kinds of bar arrangement in shear wall were designed (normal arrangement and inclined arrangement). There were three numbers of stories (shear span ratio, one story of shear span ratio M/QL=0.47, two stories of M/QL=1.00, and three stories of M/QL=1.53). Two types were designed on two-stories' specimens, that is, whether there is a middle beam. The joint method between wall and column is selected as three types shown in Fig. 2. The welded D6 bar mesh of 100mm grid was used in shear wall and duplicated in thickness. The wall reinforcement ratio of all specimens ($p_s$) was 0.64%. The D10 bar was used as dowel joint between wall and column.

The surrounding frame (beam, columns and stub) was made by steel reinforced concrete structure. In column and stub, we used H-shaped steel and the direction of weak axis of it agrees with the direction of loading. In beam, we used steel plate in center part of beam and crossed steel near the beam-column joint. The mechanical properties of materials are shown in Fig. 3 and 4.

At first, the wall and beam reinforcement were arranged. Secondly concrete was casted in wall and beam as one body. Thirdly, the concrete panel, which is united beam and column in a body, was hanged and column and base reinforcements were arranged surrounding it. At last concrete is casted in base and columns.

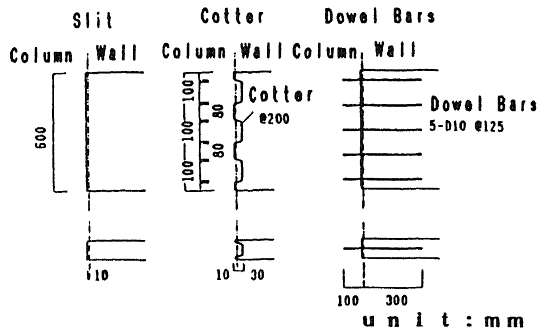

Fig. 2 Details of Wall-Column Joint

Table 3 Mechanical Properties of Concrete

| Concrete | $c\sigma c$ (N/mm²) | $c\sigma t$ (N/mm²) | $cE$ (kN/mm²) |
|---|---|---|---|
| for PC Wall & PC Beam | 34.9 | 3.3 | 26 |
| for Column | 26.6 | 2.8 | 23 |
| for Stub | 32.8 | 2.9 | 22 |

$c\sigma c$:Compressive Strength
$c\sigma t$:Tensile Strength
$cE$ :Young modulus (1/3 secant)

Table 4 Mechanical Properties of Steel

| Bars and Steel Plate | | $s\sigma y$ (N/mm²) | $s\sigma t$ (N/mm²) | $sE$ (kN/mm²) |
|---|---|---|---|---|
| Bars | D6 | 411 | 570 | $2.0\times10^2$ |
| | D10(SD30) | 385 | 550 | $1.9\times10^2$ |
| | D10(SD35) | 370 | 521 | $1.9\times10^2$ |
| | D13 | 370 | 547 | $1.8\times10^2$ |
| | D6 grid | 369 | 518 | $2.0\times10^2$ |
| | D10 grid | 358 | 510 | $1.9\times10^2$ |
| Steel | 6mm | 436 | 559 | $2.1\times10^2$ |
| | 9mm | 400 | 545 | $2.0\times10^2$ |
| | 12mm | 392 | 526 | $2.1\times10^2$ |
| | 16mm | 375 | 532 | $2.2\times10^2$ |

$s\sigma y$:Yield Strength
$s\sigma t$:Maximum Tentiie Strength
$sE$ :Young Modulus

## 2.2 Method of loading

The loading apparatus are shown in Fig. 3. The specimens were subjected to cyclic horizontal (shear) load by two oil jacks.The loading was controlled by horizontal displacement (displacement angle) at beam-column joint. Loading path are shown in Fig.4. Column were subjected to axial load of 784kN.

## 3 Test Result

### 3.1 Relation between shear force and displacement and failure condition

The relations between shear force (Q) and horizontal displacement ( $\delta$ ) at the beam-column joint are shown in Fig.5 and 6. The envelope of the relation between shear force (Q) and horizontal displacement angle (R) of all specimens are shown in Fig.7,8,9,10. (1) When R is $5\times10^{-4}$ rad., the slanting shear crack arose on the wall. (2) When R is $1\times10^{-2}$ rad., the horizontal load (shear force) reached to the maximum. (3) After R is $1.5\times10^{-2}$ rad., the specimens reached ultimate condition and failed.

The comparisons between maximum shear force of experimental result and that of calculations are shown in Table 5. The initial stiffness of each specimens are also shown in it. In Table 5 we used the so-called "Hirosawa's formula" and "Arakawa's formula" as calculation. On specimens of two-story with middle beam and three-story, the shear force of experimental result were larger than that of calculation. But on specimens of one-story and two-story without middle beam, the shear force calculated by "Hirosawa's formula" ($Q_{ws1}$) were larger than that of experimental result. The shear force calculated by "Arakawa's formula" ($Q_{ws2}$) were considerably larger than that of experimental result except specimens of two-story

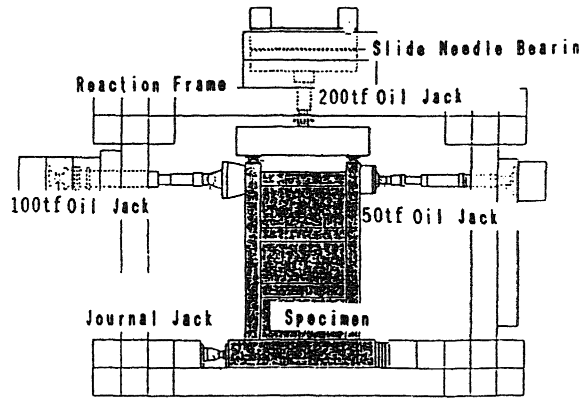


Fig. 3 Loading Apparatus

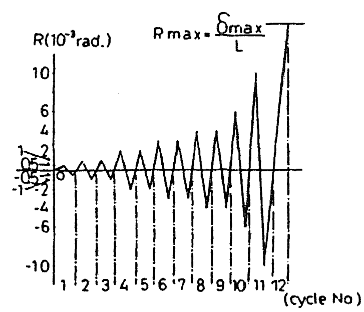


Fig. 4 Loading Path

Table 5 Test Result and Calculated Value etc.

| Specimens | Gf(kN/rad.) | Qmax(kN) | δQmax(mm) | U.D.P. | Qws1(kN) | Qmax/Qws1 | Qws2(kN) | Qmax/Qws2 |
|---|---|---|---|---|---|---|---|---|
| 1B-S-90 | $11.8\times10^5$ | $13.8\times10^2$ | 4.06 | SL | $14.7\times10^2$ | 0.94 | $9.2\times10^2$ | 1.49 |
| 1B-C-45 | $31.4\times10^5$ | $14.5\times10^2$ | 2.15 | PP·PS | | 0.98 | | 1.57 |
| 2N-S-45 | $6.9\times10^5$ | $10.0\times10^2$ | 11.98 | PP·PS | $11.5\times10^2$ | 0.87 | $9.2\times10^2$ | 1.08 |
| 2N-N-45 | $5.9\times10^5$ | $10.0\times10^2$ | 16.05 | PP | | 0.87 | | 1.08 |
| 2B-S-90 | $5.9\times10^5$ | $11.9\times10^2$ | 11.47 | PS·SL | | 1.03 | | 1.29 |
| 2B-C-45 | $20.6\times10^5$ | $13.7\times10^2$ | 14.52 | PP·PS | | 1.20 | | 1.49 |
| 2B-N-45 | $6.9\times10^5$ | $12.3\times10^2$ | 14.93 | PP·PS | | 1.07 | | 1.34 |
| 3B-C-45 | $20.6\times10^5$ | $11.8\times10^2$ | 19.19 | PC | $10.0\times10^2$ | 1.19 | $7.2\times10^2$ | 1.64 |
| 3B-S-45 | $7.8\times10^5$ | $10.2\times10^2$ | 15.65 | PP·PS | | 1.02 | | 1.41 |
| 3B-S-90 | $7.8\times10^5$ | $11.9\times10^2$ | 23.26 | SL | | 1.19 | | 1.64 |
| 3B-N-45 | $3.9\times10^5$ | $11.3\times10^2$ | 27.85 | PP | | 1.13 | | 1.56 |

Gf:Initial Stiffness  Qmax:Maximum Shear Force  δQmax:Displacemant at Qmax
U.D.P.:Ultimate Destruction Pattern
PP:Comressive Failure of Wall Concrete near Compressive Loading Point
PC:Compressive Failure of Wall Concrete near Bottom of Compressive Column
PS:Shear Failure of Top Beam  SL:Slip Failure of Wall
Qws1:Shear Force Calculated by HIROSAWA's Formula
Qws1:Shear Force Calculated by ARAKAWA's Formula

$$Q_{ws1}=\{0.068\cdot p_{te}^{0.23}\cdot(F_c+180)/\sqrt{M/QL+0.12}+2.7\cdot\sqrt{P_w\cdot {}_w\sigma_y}+0.1\cdot\sigma_0\}\cdot b_e\cdot j$$
$$Q_{ws2}=\{0.053\cdot p_{te}^{0.23}\cdot(F_c+180)/(M/QL+0.12)+2.7\cdot\sqrt{P_w\cdot {}_w\sigma_y}+0.1\cdot\sigma_0\}\cdot b_e\cdot j$$

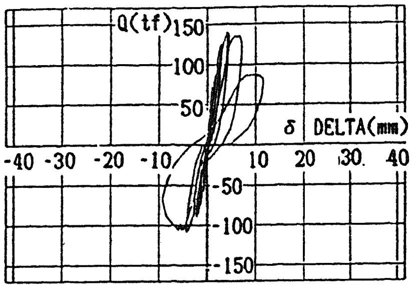


Fig.5 Q-δ Curve (1B-S-90)

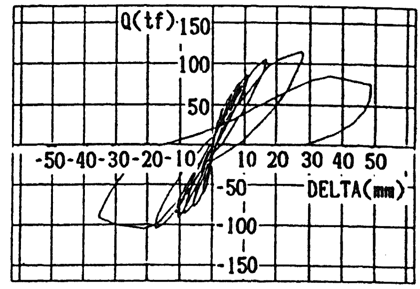


Fig.6 Q-δ Curve (3B-N-45)

without middle beam.

On the three specimens which have normal arrangement of wall reinforcement (specimen 1B-S-90,2B-S-90,3B-S-90), slip failure was occurred with suddenly concrete fall and large noise near wall-(top)beam joint at the maximum shear force or at the peak of loading cycle immediately after the maximum shear force. That failure was attached with suddenly force-falling and reached the ultimate destruction. The area of hysteresis loop of them were small.

On the other eight specimens which have the slant arrangement of wall reinforcement, such slip failure did not occurred and the area of hysteresis loop of them were larger than that of three specimens which have normal arrangement.The eight specimens reached to the ultimate destruction with compressive failure of wall concrete near the part of the compressive side of loadings or near the bottom of compressive column. That compressive failure was local and silent.

On the three specimens 2N-S-45,2B-C-45 and 3B-S-45, the buckling of wall reinforcement with wall concrete failure was occurred.

The failure mode of shear wall panel at the ultimate destruction was not determined by the wall-column joint method but by the arrangement method of wall reinforcement.

The typical crack patterns at the ultimate destruction is shown in Fig.11 and 12.

### 3.2 Influence of wall-column joint

To investigate the influence of only joint method, we compare the Q-R curve of each specimens. The comparisons between 2N-S-45 and 2N-N-45 is shown in Fig.8, 2B-C-45 and 2B-N-45 is shown in Fig.9, and 3B-C-45, 3B-S-45 and 3B-N-45 is shown in Fig.10.

As shown in Fig.9 and 10, the specimens with cotter (2B-C-45) and (3B-C-45) have the largest initial stiffness and maximum shear force. 3B-S-45 with dowel has larger initial stiffness than that of 3B-N-45 with slit, but it has smaller maximum shear force than that of 3B-N-45.

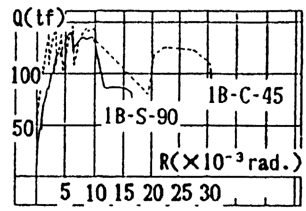


Fig.7 Q-R Curve (1-story Specimens)

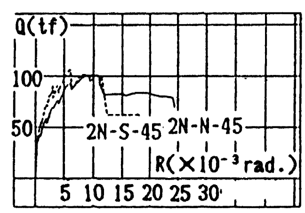


Fig.8 Q-R Curve (2-story Specimens without Middle Beam)

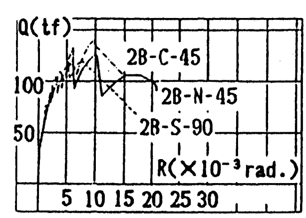


Fig.9 Q-R Curve (2-story Specimens with Middle Beam)

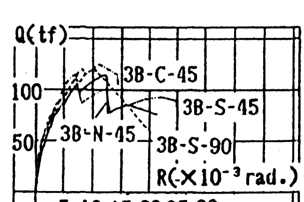


Fig.10 Q-R Curve (3-story Specimens)

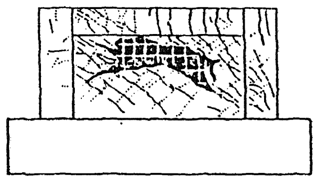

Fig. 11 Ultimate Crack Pattern (1B-S-90)

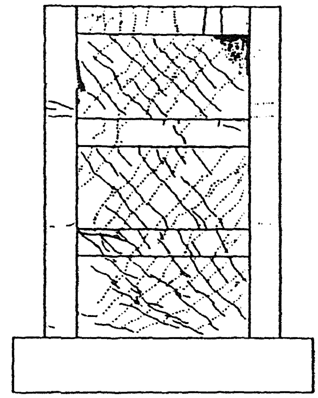

Fig. 12 Ultimate Crack Pattern (3B-N-45)

On the Q-R curve after maximum shear force, the specimens with cotter (2B-C-45 and 3B-C-45) reached the ultimate destruction when R became $15\times10^{-3}$ rad. and occurred large force-falling. But other specimens reached to the ultimate destruction when R became $20\text{-}25\times10^{-3}$ rad..

As shown in Table 5, the specimens with cotter have large initial stiffness and maximum shear force, but have smaller area surrounded by Q- $\delta$ curve and larger force-falling than the other specimens with the other two joint method. The specimens with other two method have nearly same initial stiffness and have large area surrounded by Q- $\delta$ curve. It seemed that the cotter has the largest shear stiffness of wall-column joint method, that the slit has the smallest one and that the dowel bar has the stiffness between the cotter and slit. That order of the stiffness of joint is equal to the order of initial stiffness and maximum shear force gotten by experiment result. So we can say that the shear stiffness of wall-column joint method has the large influence on the initial stiffness and maximum shear force of shear wall and that the shear wall with joint which has small shear stiffness has small shear fall after maximum shear force.

3.3 Relationship between wall-column joint and.shear span ratio

The influence of the relationships between wall-column joint method and shear span ratio on the Q-R curve of each specimens are investigated. The Q-R relations of specimens with dowel bars (1B-S-90, 2B-S-90 and 3B-S-90) are shown in Fig. 7,9,10. Those of specimens with cotter (1B-C-45, 2B-C-45 and 3B-C-45) are shown in Fig. 7,9,10). Those of specimens with slit (2B-N-45 and 3B-N-45) are shown in Fig.9,10). On the influences of cotter and slit, the order of magnitude of initial stiffness, maximum shear force and force-holding after maximum shear force are as follows; 1B-C-45>2B-C-45>3B-C-45 and 2B-N-45>3B-N-45. On the that of dowel, the order of magnitude of initial stiffness is as follows; 1B-S-90=2B-S-90>3B-S-90 and that of maximum shear force is as follows; 1B-S-90>2B-S-90=3B-S-90. We can say that the growth of shear span ratio causes the decreasing of the initial stiffness, maximum shear force or force-holding after maximum shear force or force-holding after maximum shear force and causes the fall of hysteresis of shear wall.

3.4 Relationship between wall-column joint and middle beam

To investigate the influence of the relationship between wall-column joint method and middle beam, we compare the Q-R curve of the specimens 2N-N-45 (specimens with slit and without middle beam, as shown in Fig.8) with that of the specimen 2B-N-45 (with slit and middle .pa beam, as shown in Fig.9).

The specimens 2B-N-45 have larger initial stiffness and maximum shear force than those of 2N-N-45. The force-holding ability after maximum shear force of 2N-N-45 is slight higher than that of 2B-N-45, but that of 2B-N-45 is relatively good. The middle beam improve the hysteresis of shear wall.

3.5 Local displacement between wall and column

Fig.13 show the relative displacement of wall against column. They show the slip and open of joint on two-story wall of specimens 3B-C-45 and 3B-N-45. The behavior of specimen 3B-C-45 with cotter is nearly same as that of specimen 3B-S-45 with dowel bars. On specimen 3B-N-45 with slit, negative slip displacement about 2mm occurred when R was $-15\times10^{-3}$ rad.. At the same time the wall concrete near pressure loading point came to compressive failure. It seemed that at that time the wall concrete or the column concrete touching the measuring point came to compressive failure.

On 3B-C-45 the maximum open displacement was about 1mm. That was small displacement. We can say that the cotter and the dowel bars are effective to restrain from occurring the open displacement between wall and column.

On the other hand, the slip displacement was larger than open displacement. On the ultimate destruction of 3B-C-45 slip displacement suddenly occurred. As shown in Fig.14 the side incline of cotter of 3B-C-45 was about 14 degree, but the wall moving incline calculated from the wall displacement in ultimate destruction was about 83 degree against horizontal direction. It seemed that on 3B-C-45 the cotter failed by compression at ultimate destruction because the slip displacement was about 1mm.

### 3.6 Shear destruction of the top beam

On all specimens except three specimens 2N-N-45, 3B-C-45 and 3B-N-45, shear destruction occurred on an end of the top beam at the ultimate destruction. That shear destruction occurred according to the following order, (1) at first the inclined crack grew according to the loading, (2) that crack crossed the top beam, (3) at last the shear destruction occurred with the breakaway of beam concrete.

The incline of the crack crossing the top beam was about 30 degree in eight specimens. On specimens 1B-C-45 and 2N-S-45 the destruction was very hard and we confirmed the buckling of the longitudinal bars in the top beam. It seemed that the shear destruction of top beam was caused by the punching shear force from the wall. We may do the effective reinforcement on the top beam.

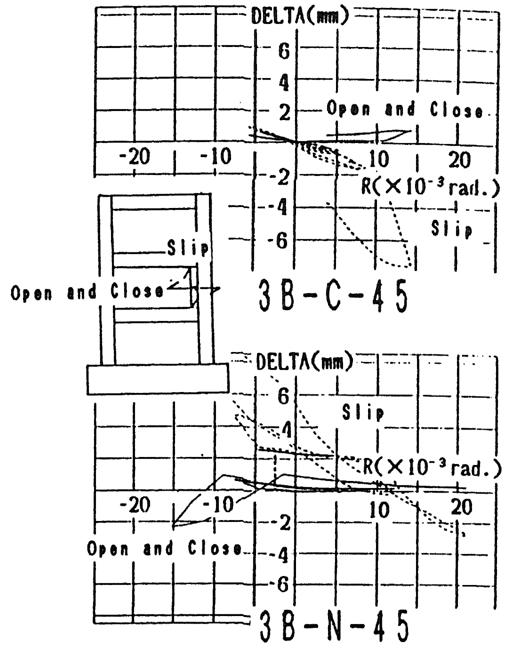


Fig. 13 Local Deformation of Wall near Wall-Column Joint

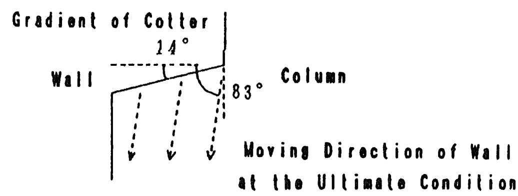


Fig. 14 Gradient of Cotter

## 4 Conclusion

We obtain the following result,

1. The whole hysteresis of the shear wall is effected by the wall-column joint method. The wall-column joint method which have large shear stiffness brings large initial stiffness and maximum shear force, but brings large force-falling after the maximum shear force.
2. The whole hysteresis of the shear wall is effected by the shear span ratio regardless of the wall-column joint method.
3. The whole hysteresis of the shear wall is effected by the middle beam regardless of the wall-column joint method.
4. The local wall displacement is effected by the wall-column joint method.

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## Referecne

Guide for Design and Prefabrication of Precast Reinforced Concrete Structures, AIJ, October 1986. Japan