A STUDY OF THE STRENGTH AND DEFORMATION OF PRECAST SHEAR WALL

Yoji HOSOKAWA[1], Shinji KATO[2], Koji AOTA[3] And Yasuhiro MATSUZAKI[4]

SUMMARY

We proposed that pre-cast shear wall that includes diagonal steel bars and is connected to steel beam by Hi-tensile Bolt. Their characteristic behaviors were confirmed by this experiment of 4-pieces of specimens. Little shift (about 1 to 3 mm) was confirmed at their horizontal part of connection between shear wall and beam, but there were no influence with their stiffness, maximum strength, and characteristic behaviors. There were no shift at their vertical part of connection that used shear cotter and U-shaped steel bars. We considered shear mechanism of both monolithic type and slit type and strength were able to be estimated.

INTRODUCTION

Contents of study

There will be a lot of problems about complication of joint system that of stud welding of steel beam or assuring the bond of steel bar of shear wall when we use pre-cast concrete member for steel-concrete composite structure.

We devised the way of joint system of shear wall panel for the purpose of resolving this problem and rationalization of construction. This shear wall panel has four gusset plates around each corner. And it is attached to the steel beam by high strength bolts or welding. We examined four specimens and studied the characteristic behavior of these shear walls.

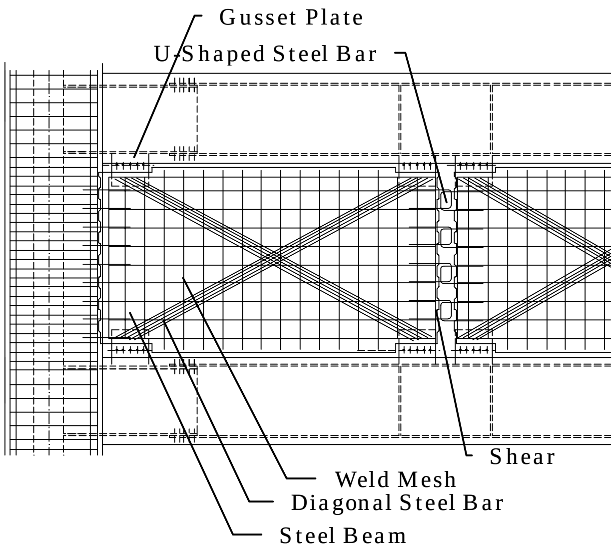


Figure 1: Composition of pre-cast shear wall

Outline of pre-cast concrete wall

We propose the construction system of pre-cast divided shear wall for steel-concrete composite structure. Horizontal joint (between wall and upper or lower beam) is only used high strength bolts without any shear cotter or steel joint bar. Vertical joint (between column

[1] *Technical Development Department Group, Maeda Corporation Email:hosokawy@jcity.maeda.co.jp*
[2] *Technical Development Department Group, Maeda Corporation*
[3] *Technical Research Institute, Maeda Corporation*
[4] *Department of Architecture, Facility of Engineering, Science University of Tokyo*

and wall or one wall and the other) are considered for two way that we use strict joint or not. They are strictly combined by using U-shaped steel bar and shear cotter when each wall need to be connected strictly for vertical part of joint. Then they are made sure of their monolithic performance. A slit which has adequate width is set between two walls when each wall aren't connected strictly for vertical part of joint. The width of slit is that walls never get in touch each other when the walls deform by the seismic force. In the wall there are steel bars diagonally and their ends are welded on steel plate. Diagonal steel bars mainly resist against seismic force as a brace. Flare welding method is adopted for joint between steel bar and steel plate. Due to the diagonal steel bar, the shear panel needs minimum vertical and horizontal steel bar.

EXPERIMENTAL SUMMARY

Test Specimen

14-story apartment building test plan that has 11.5m length of its short side had been designed for the shear wall. According to the stress on lower two stories of the test plan, 4 test specimens were designed and determined its form. Though the test plan had 3 pieces of pre-cast shear wall panels in one grid line, these specimens has 2 pieces of shear walls. These test specimens were adjusted their column sections in order to share the shear stress between column and wall equal to the test plan wall which had 3 pieces of pre-cast shear wall originally. Figure-2 shows the shape of No.1 specimen. There is H-shape beam in the column on week performance axis.

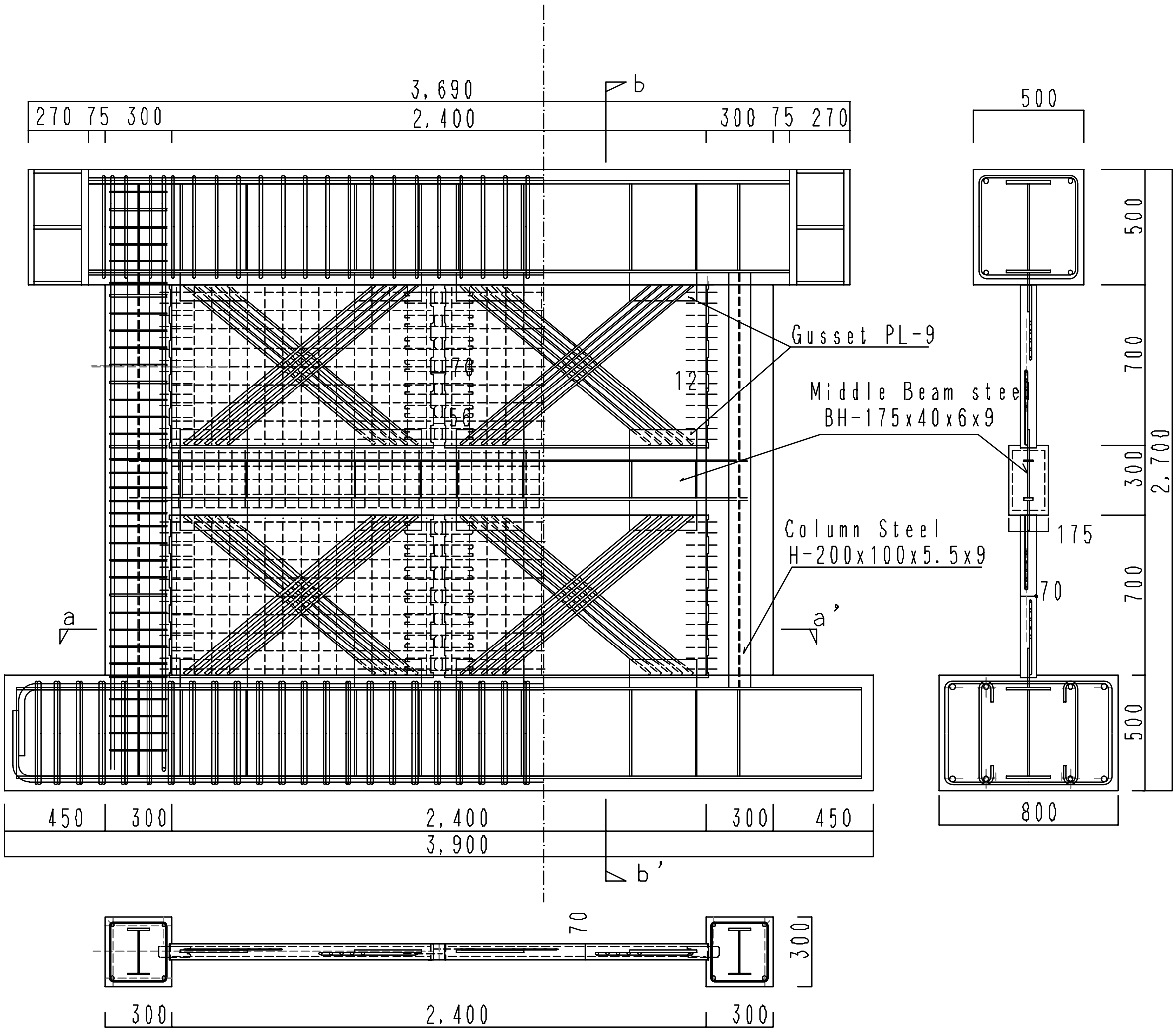
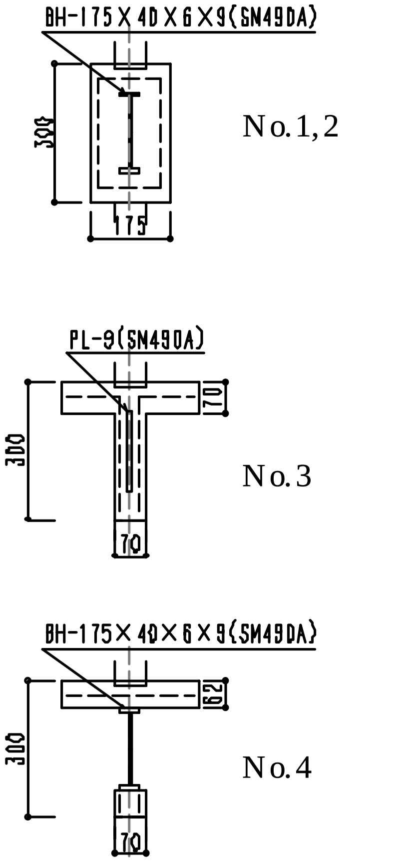


Figure 2: Specimen Detail

Table-1 lists specification of specimens. Table-2 lists specification of steel materials. No.2 specimen has 1.43 times as many diagonal steel bars as No.1 in order to look into the effect of steel bar quantity. No.3 specimen has middle beam that has equal width to wall and involves steel flat plate that has equal area to that of No.1. Then No.3 has a flat shape as if it had no middle beam from outside view. No.4 has middle beam which is consisted only H-shaped steel beam, and has 20 mm width slit between two walls and 10 mm width slit between column and wall without connecting them in order to look into the effect of stiffness, strength and failure mode. Then from No.1 to No.3 could be called monolithic type, No.4 is slit type.
Figure-3 shows the sections of middle beam. Figure-4 shows the detail of vertical connection between column and wall and between two walls. No.3 and No.4 have slabs whose width is equal to column so as not to buckle of the beams. All the specimens were made following to the real construction way. Shear wall panel and steel frame were made separately, and shear wall panel were included inside the steel frame and connected by welding. Then concrete was poured to form columns, beams, slabs, and connections.



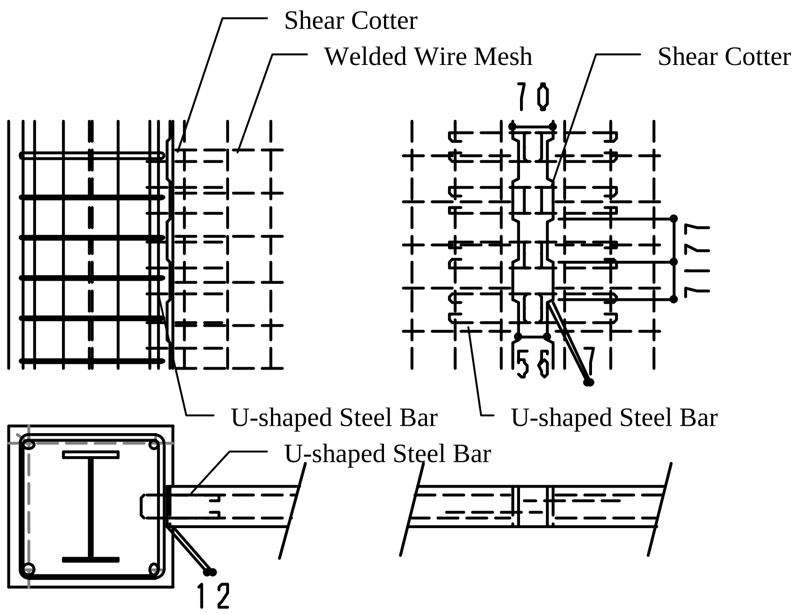


Figure 4: Detail of Connection

Figure 3: Detail of Middle Beam

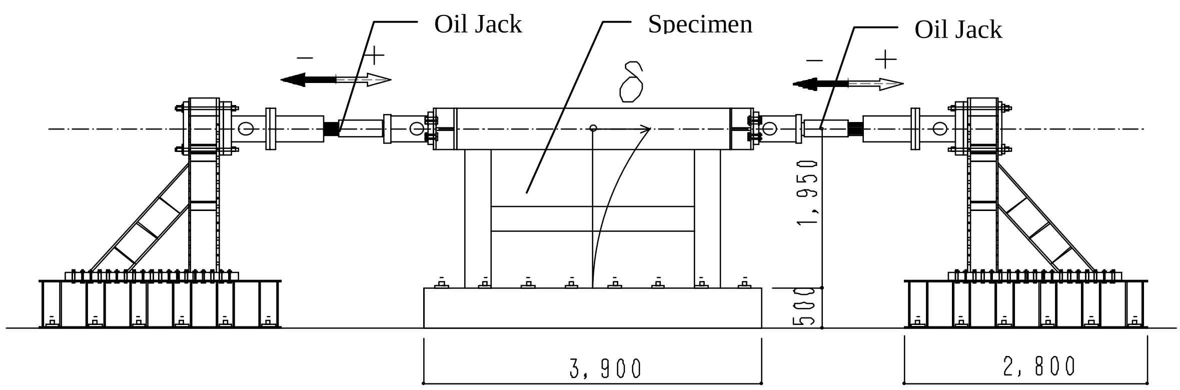


Figure 5: Loading Apparatus

Loading and Measurement

Figure-5 shows loading way. Figure-6 shows loading schedule. The loading was negative-positive alternative loading by two hydraulic jacks settled horizontally on upper loading beam. Two jacks were kept the equal loading each other. Loading was measured by load cell between jack and specimen. Horizontal deformation of each story, shear deformation of shear wall panel, and gap of connection were measured. Strain of steel bar of column, steel of column, steel of middle beam, gusset plate and diagonal steel bar were measured by strain gauge.

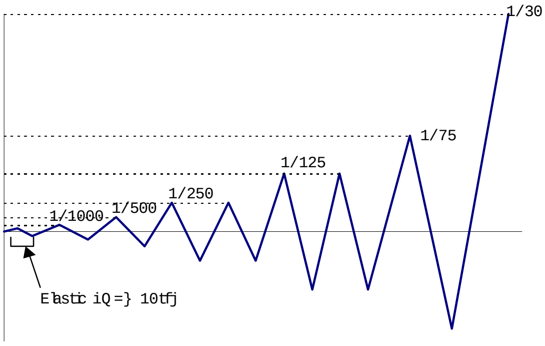


Figure 6: Loading Schedule

Table 2 Mechanical Properties of Material

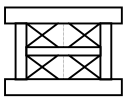
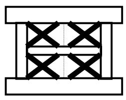
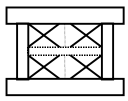
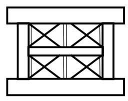
			Yield Stress σ_y	Tensile Strength σ_u	Young's Modulus E
Middle Beam	Web	PL6	372	560	1.985E+07
	Flange	PL9	371	540	1.766E+07
Column	Steel Plate		312	457	1.872E+07
	Steel Bar	D16	410	579	1.912E+07
	Hoop	D6	301	491	1.667E+07
Diagonal Steel Bar		D10	390	549	1.863E+07
		D13	368	533	1.902E+07
Connecting Steel Bar		2.6ϕ	508	629	1.236E+07
		3.2ϕ	622	691	1.579E+07

TEST RESULT

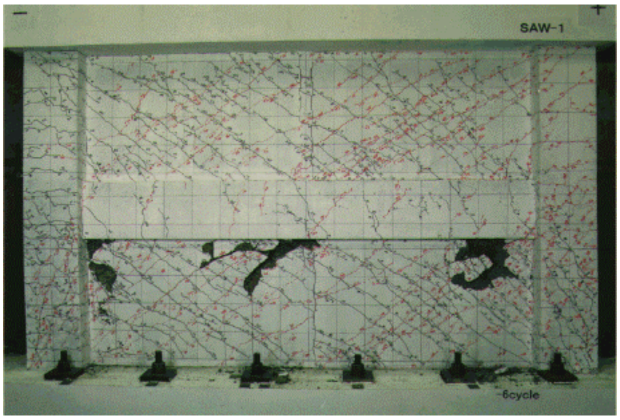
Failure process and load-deformation relationship

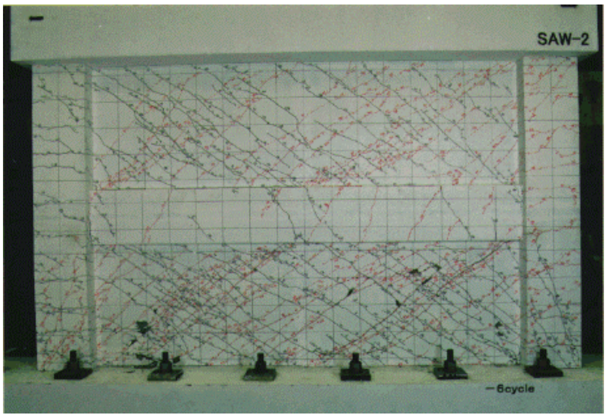
Figure-5 shows load-deformation relationship. On the whole at each specimens shear crack appeared on shear

Table 1 Specimens

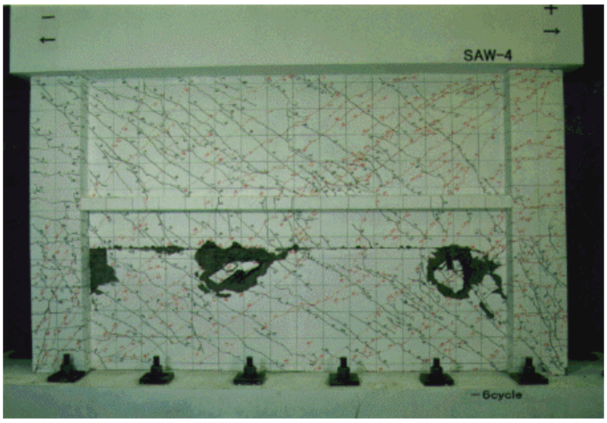
		Test Plan Design	Monolithic Type Specimens			Slit Type Specimen
			No. 1	No. 2	No. 3	No. 4
Spec.		14 Story Apertment	 Basic	 Diagonal steel bar	 Middle Beam	 Slit
Side Column	BxD	1000x1200	300x300			
	Steel	H-700x300x14x28	H-200x100x5.5x8(SS400)			
	Steel Bar	20-D29(SD345)	4-D16 (SD345)			
	Hoop	□-D13@100 (SD295A)	□-D6@75 (SD295A)			
	Concrete	36N/mm^2	37.2N/mm^2	41.4N/mm^2	42.5N/mm^2	32.6N/mm^2
Shear Panel	tw x lw x hw	200x10600x2155	70x2400x700			
	Wall	Mesh 6ϕ200W (pw=0.155%)	2.6ϕ@80W (pw=0.189%)			
	Concrete	36N/mm^2	48.2N/mm^2	50.9N/mm^2	48.7N/mm^2	39.5N/mm^2
	Diagonal Bar	6-D25x6 (SD345)	5-D10x4 (SD345)	4-D13x4 (SD345)	5-D10x4 (SD345)	
Connection	shear cotter W x H x D	200x200x20	70x71x7			-
	Connecting Column-Wall Wall-Wall	28-D6 14-D10	28-2.6ϕ 14-3.2ϕ			-
Middle Beam	B x D	950x500	300x175		300x70	300x175
	Steel Beam	H-700x200x9(13)x16	BH-175x40x6x9(SM490Ａ)		175x9(SM490Ａ)	175x40x6x9(〃)

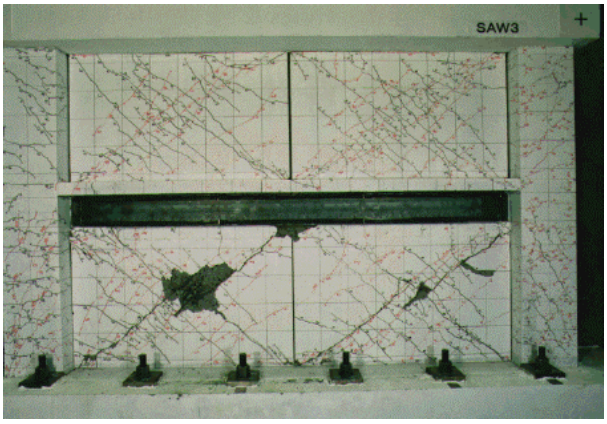
panel when the load was around 200 kN. Just after that their stiffness descended and kept constant stiffness until diagonal steel bar began to yield. After yielding of diagonal steel bar, column steel bar yielded. But the load went up still more. When deformation angle was from 4/1000rad. to 8/1000rad. each specimen showed that the collapse of lower shear panel appeared and the strength started to decrease. The strength of No.1 and No.3 started to decrease while the loading cycle of 7.3/1000rad.
No.1, No.2, and No.3 showed that shear cracks appeared at the end and center of middle beam under 1/1000rad. Especially No.3 had the tendency that its shear crack spread from wall to whole beam. The web plate of middle steel beam of No.4 specimen revealed shear yielding ,the others didn't yield. There were few shear cracks penetrated through the vertical connection, but there were no shift or separation. There was little shift at the horizontal connection, 1mm upper side of middle beam of No.1, 1mm under side of that of No.2, and 3mm under side of that of No.3. A collapse occurred at the corner of the panel of No.4, but shift of horizontal connection didn't appear. Generally after maximum strength, there was little failure on upper side of the shear panel, failure and deformation concentrated on lower side. All of them kept constant load after decreasing their strength. Photo-1 shows the state just after maximum strength of each specimen.





No.1 **No.2**





No.3 **No.4**

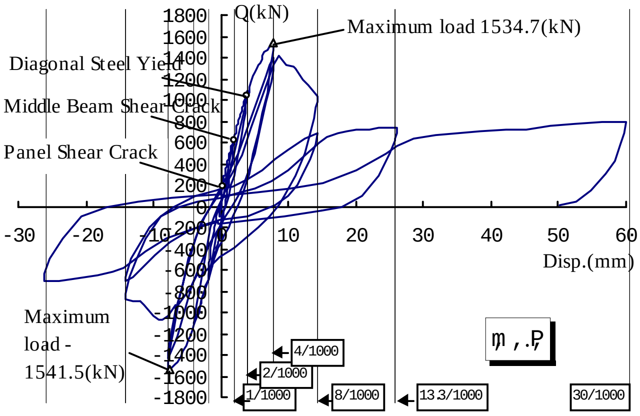
Photo-1 Failure of Each Specimems (R=7.3/1000 rad.)

Stiffness of Initial and After Cracking

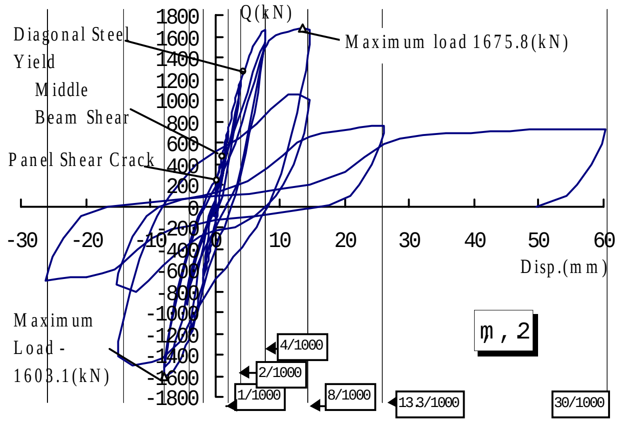
Fig.8 and Table-3 show initial stiffness by experiment and by calculation that was lead by the elastic theory considered deformation of bending and shear. Neither steel bar nor steel beam was counted in calculation. And the calculation is considered following three types of stiffness, whole flexural, whole shear, and flexural one of each stories. Initial stiffness of No.4 is 0.69 times as large as No.1. Initial stiffness of each specimen are smaller than calculation. After cracking Stiffness of No.1 and No.3 are approximately equal to each other. No.2 is around 1.17 times larger than No.1.

Strength

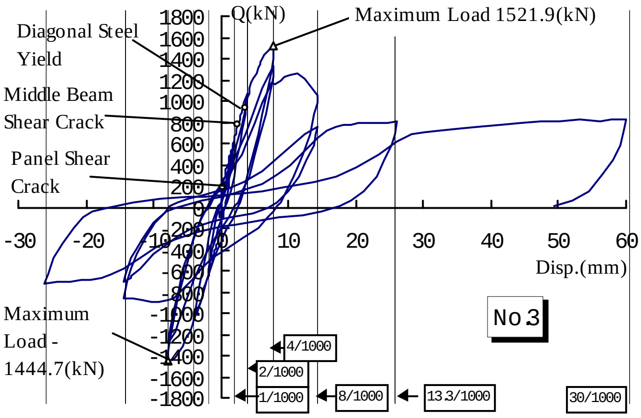
Table-3 shows experimental and calculation shear strength. Strength of No.1 and No.3 descended up to R=1/8000rad., due to failure of shear panel. The strength of No.2 is 141.2kN larger than that of No.1. This value is approximately equal to the difference of calculated strength between No.1 and No.2. The strength of No.1 is mostly as large as that of No.3. This fact indicates that the difference of middle beam has no influence to the maximum strength.



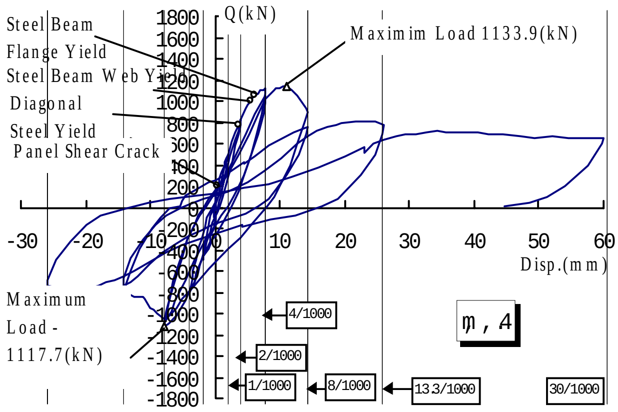
No.1



No.2



No.3



No.4

Figure 7 Relationship of Shearing Force(Q) and Deformation

(1) Monolithic Type

The strength of monolithic type (No.1,2,3) is estimated according to the reference 1) that is based on the truss structure theory. Figure-9 shows the concept of shear wall mechanism. Shear reinforcing wire mesh bar is counted for truss mechanism, but steel of middle beam is not. Arch strut is assumed that it reached from loading point to opposite end of basement penetrating middle beam. Diagonal steel bars are considered to be available for counting as a shear reinforcing factor according to reference 2), because it was confirmed all the diagonal steel bars both tensile and compressed yielded. As a result all the calculating values are larger than those of experiment.

Table 3: Strength of Experiment and Calculation

No.	Initial Stiffness (kN/mm)		Shear Strength (kN)	
	Exp.	Cal.	Exp.	Cal.
1	886.9	954.5 (0.93)	1534.7	1716.0 *1 (0.89)
2	818.3	954.5 (0.86)	1675.8	1938.4 *1 (0.86)
3	826.1	954.5 (0.87)	1521.9	1783.6 *1 (0.85)
4	670.3	808.5 (0.83)	1133.9	1130.9 *2 (1.00)

( ) reveal [Experiment]/[calculation]
*1 is according to Equation (1), *2 is (8)

Equations of shear strength of monolithic type (from reference 1)

$$V_{ucal} = V_t + V_a + V_x \quad (1)$$

$$V_t = t_w \ell_{wb} p_s \sigma_{sy} \cot\phi \quad (2)$$

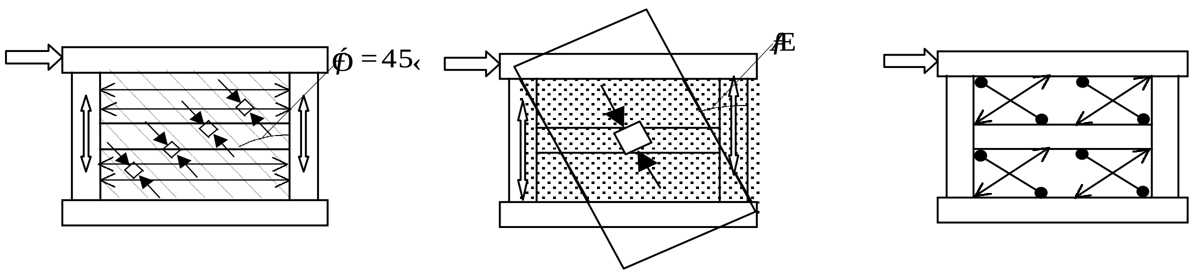
$$V_a = \tan\theta \; (1-\beta) \; t_w \ell_{wa} \nu \sigma_B / 2 \quad (3)$$

$$V_x = A_x \sigma_{xy} \sin\theta_x \quad (4)$$

$$\tan\theta = \left[\sqrt{(h_w/\ell_{wa})^2 + 1} - h_w/\ell_{wa} \right] \quad (5)$$

$$\beta = \left(1 + \cot^2\phi \right) \; p_{sy} \sigma_{sy} / (\nu \sigma_B) \quad (6)$$

$$\nu = \nu_0 = 0.7 - \sigma_B/200 \quad (7)$$



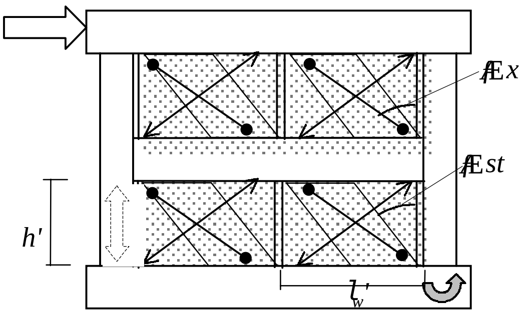
Truss Mechanism Arch Mechanism Diagonal Steel Bar Mechanism

Figure 9: Shear Mechanism of monolithic type

(2) Slit Type

Columns and walls of slit type wall behave separately because shear force never go through between column and wall or two walls due to the existence of slit. Figure-10 shows concept of shear wall mechanism. At the experiment steel bars inside the compressed column yielded at maximum load by bending force, but steel beam inside the compressed column didn't yield. Then we assumed that the shear strength sheared by the compressed column regarded as the one of flexural yielding on its base.

On the other hand there were a lot of cracks on tensile column, all the steel bars and steel were tensile condition. Then tensile column didn't count into shear strength. Diagonal steel bars are considered to be available for counting like the monolithic type. In the shear strength expression, calculating value was equal to that of experiment if width of strut were assumed 0.6lw.



Equations of shear strength of Slit Type

$$V_{ucal} = \sum {}_wV_{su} + V_x + {}_cV_b \qquad (8)$$

$${}_wV_{su} = t_w \cdot 0.6 \cdot \ell'_w \sin 2\theta_{st} \nu\, \sigma_B / 2 \qquad (9)$$

$$\sin 2\theta_{st} = 4 \cdot \ell'_w h' \Big/ \left(4 \cdot h'^2 + \ell'^2_w\right) \qquad (10)$$

Figure 10: Shear Mechanism of Slit Type

CONCLUSIONS

This paper mainly says that test result of pre-cast divided shear wall and estimation of its strength. Conclusions are as follows.

1) The vertical connection of monolithic type of specimen showed that there were no shift or separation even at the final state.
2) According to the estimation of strength that is assumed one shear resist mechanism, the calculating strength of monolithic type were from 0.85 to 0.89 times as large as the numerical value of experiment.
3) The calculating strength of slit type that is assumed its width of strut to be 0.6lw was approximately equal to the numerical value of experiment.

REFERENCES

1) Architectural Institute of Japan (1990), Design Guidelines for Earthquake Resistant Reinforced Concrete Building Based On Ultimate Strength Concept
2) Japan Federation of Architect Office Association (1990), Design and Construction of Diagonally Reinforced Concrete
3) Osanai.y and the others (1995), Experimental Study on Seismic Capacity of Precast Concrete Wall Isolated form Boundary Column
4) Aota.k and the others (1998), Study on Precast Divided Shear Wall (Part1), (Part2)