

EXPERIMENTAL STUDY ON STRUCTURAL PERFORMANCE OF WALL-TYPE PRECAST CONCRETE PANEL HOUSES

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SUMMARY

The purpose of this study is to acquire fundamental data to construct analytical models of members applicable to nonlinear static analysis for wall-type precast concrete panel houses of two stories. The precast panels are thin walls, floors and beams with reinforcing ribs. The house is built to fasten each panel by joint steel bars. The wall panels have 2700mm-high, 897mm-wide, 120mm-thick ribs and a 46mm-thick wall. Two kinds of experiments were carried out. One was a test for the wall panels and the other was for the frame. A total of four wall specimens and one frame specimen were prepared. Each wall specimen was fabricated by a single wall panel, while the frame specimen consists of two wall panels, floor panels and beam panels. The variables investigated were the level of axial forces and the shear span ratio for the wall panels test and the disposition of wall panels. The specimens in both experiments were loaded with lateral cyclic shear forces and an axial compression.

This paper shows that the flexural strength of wall panels is decided by the yield of their anchor bars, the flexural strengths are represented by the existing equations of reinforced concrete walls and the deformation capacity of wall panels is influenced seriously by the rotation distortion of wall panels. It also shows that the strength and deformation capacity of the frame are affected by the disposition of wall panels.

INTRODUCTION

"The method of construction made from Medium size concrete panels with ribs" used for a concrete prefabricated house of 2 stories is a wall type constitution which assembles the standardized precast reinforced-concrete panels with ribs by bolt junction, and is a method of construction which enables shortening of the necessary construction time and cutting the house frame cost.

The authors acquired experimental work data about each detail in the horizontal load experiment of the concrete wall panels with ribs and the frame assembled concrete wall panels, floor panels, and a sagging wall and orthogonal wall panels in order to grasp the structural performance of the structural elements of

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this method in mind constructing analytical models of members applicable to nonlinear static analysis required for structural calculation. While describing the scheme of experiments, this paper shows the examination result of the structural performance of wall panels and a frame.

ANTISYMMETRICAL LOADING TEST OF THE WALL PANELS

Specimens

x120

4

The list of specimens is shown in Table 1, and the details of the specimens are shown in Fig.1. The four wall panels of the first floor of a 2-story house. The size of a specimen is the wall thickness x length= 120mmx 897mm and the shell thickness = 46mm (par wall thickness is 57.5mm). Main reinforcements of the specimen are 2-D13 (SD295A), shell section reinforcements were the $2.9\phi@60$ mesh and anchor bolts were D19 (SD295A). Then their specified concrete strength was 30N/mm² and their specified mortal strength filled in the joint was $60N/mm^2$. The variables considered here were axial load (constant, fluctuation) and shear span ratio (a/D=3.42, 1.74). In addition, a/D=1.74 was the assumed stress condition of the external corner wall, and a/D=3.42 was the assumed stress condition of the wall which an inside grid line and an outside grid line crossed at right-angles. The high of a/D=3.42 was 2700mm, and the high of a/D=1.74 was 1200mm. The mechanical properties of concrete and steel are shown in Table 2 and Table 3.

Measurement(mm) Longitudinal Anchor Shell Shear Span Ratio Length No main rein-Axial Load Shell Bolts Reinforcement (a/D) ×High forcements Thickness ×Thickness 1 Constant 897x 2700 3.42 x120 2 Fluctuation 46 2-D19 2-D13 2.90@60 3 Constant 897x 1200 1.74



 Table 1 Details and arrangement of specimens

Table 2 Mechanical properties of concrete

Fluctuation

No.	Compressive strength (N/mm ²)		
	Concrete	Mortar	
1	46.47	82.17	
2	50.25	85.21	
3	50.72	78.60	
4	46.47	83.10	

Table 3 Mechanical properties of steel

Part		Anchor bolt	Wall's main re- inforcement
Specification		D19 (SD295A)	D13 (SD295A)
Viald point	Stress (N/mm ²)	395	367
i iela politi	Strain (µ)	1890	2198
Tensile str	ength (N/mm ²)	578	508
Elongation ratio (%)		24.0	23.2
Young's modulus $\times 10^{5}$ (N/mm ²)		2.16	1.85

Fig.1 Specimen

Loading system

A loading apparatus is shown in Fig.2, and the loading path is shown in Fig.3. The specimen was installed on the precast lower stub fixed by the PC bar in the reaction frame, and was joined with the anchor bolts. In addition, the circumference's clearance of the anchor bolts was filled up with expanded mortar. The upper precast stub was joined to the specimen and the horizontal load apparatus using vertical bolts.

The specimens were subjected to repeat loading with the 500kN actuator attached horizontally. The height of the loading point was set to 3077mm and 1567mm. Loading of the constant axial load of 29kN (axial stress was $0.56N/mm^2$) or the fluctuation axial load (N=29+1.2 x Q kN, Q: horizontal force) of 0-150kN (axial stress was 0-2.90N/mm²) was carried out, which assumed a long time load acting on the first floor wall of the 2 stories with a 350kN hydraulic jack.

A measuring apparatus is shown in Fig.4. Horizontal and vertical displacement of the wall panel's top, the open and slip displacement of the wall panel's leg, and elongation displacement of wall side faces were measured by displacement meters. Strains of main reinforcements, mesh lines and anchor bolts were measured by wire strain gages. In addition, the horizontal force was measured by the 500kN load cell, and the axial load was measured by the pressure intensifier.

Test results and discussions

Failure pattern

The ultimate destructive situation of each test specimen is shown in Fig.5.





Fig.4 Measuring apparatus



Fig.5 Ultimate destructive situation

In regard to the side splitting failure, the decline of horizontal force caused by exfoliation of the concrete covering the wall's main-reinforcement on the tension side in No 1 and 2 of a/D=3.42 and No3 of a/D=1.74 in constant axial load was shown. Also the bending compression destruction of the longitudinal rib was shown in No4 of fluctuation axial load and a/D=1.74 was shown.

Deformation

The load (Q)-displacement (δ) curves obtained from the experiment are shown in Figs. 6-.9. In allspecimens, hysteresis showed the slipped type. When seen in detail, from Figs.6 and 7, in No 1 and 2 of a/D=3.42, first, the transverse tension crack was generated in the cycle of (rotation angle of member) R=1 / 400. Second, the bond crack of the anchor bolts arose in the cycle of R=1/400-1/200. Third, the anchor bolts yielded in the cycle of R=1/100. Fourth, the horizontal force did not decline until the cycle of R=1/50. Afterward, the wall's main reinforcement yielded in the cycle of R=1/30, and the horizontal force declined gradually. In addition, it was shown that No2 of the fluctuation axial load, which the degree of axial stress is high at the positive load and is low at the negative load as compared with No1, had high horizontal force at positive load, and low horizontal force at negative load. Thus, it is thought that the axial load affects the horizontal force. From Fig.8, in No3 (a/D=1.74) of the constant axial load, first, the bond crack of an anchor bolt arose in the cycle of R=1/400. Second, anchor bolts yielded in the cycle of R=1/100. Third, the horizontal force did not decline until the cycle of R=1/50. Then the wall's main reinforcement yielded in the cycle of R=1/30, the horizontal force declined by exfoliation of the concrete covering the wall's main-reinforcement on the tension side in the cycle of R=1/20.



From Fig.9, in No4 (a/D=1.74) of fluctuation axial load, first, the transverse tension crack and the bond crack of anchor bolts arose in the cycle of R=1/400. Second, the anchor bolts yielded in the cycle of

R=1/100 at the positive load and R=1/67 at the negative load. Third, horizontal force did not decline until the cycle of R=1/50. Then, the wall's longitudinal rib on the compression side carried out the bending compression destruction in the cycle of R=1/30, and the horizontal load declined suddenly.

Flexural strength

Comparison between the experimental result at the time the anchor-bolt yielded and the estimated result are shown in Table 4.The bending-moment (M)-axial load (N) correlation by the bending analysis assumed flat-surface holding and the existing formula (1)[1] are shown in Fig.10.

No	Direction	Flexural strength		
		Experiment	Calculation	eMy/cMy
		eMy(kN⋅m)	cMy(kN⋅m)	
1	+	94.09	94.77	0.99
2	+	109.45	109.71	1.00
	-	84.74	83.89	1.01
3	+	82.93	94.19	0.88
4	+	139.29	132.22	1.05
	-	98.74	94.95	1.04







(1)

In all test specimens, the experimental result and the estimate showed a good relationship. In the range of the axial load of this experiment, the bending strength of the anchor bolt yield can be evaluated in general by the formula (1).

 $cMy = a_t \cdot \sigma_y \cdot j + 0.5 \cdot N \cdot j$

 a_t : Sectional area of an anchor bolt (mm²)

 σ_y : Yield point strength of an anchor bolt (N/mm²)

j: Lever arm 0.74(m)

N : Constant or fluctuation axial load (N)

The stress -elongation of anchor bolt

The stress –elongation curve of the anchor bolt is shown in Fig.11. It is shown slipped type hysteresis like the Q- δ curve. The slipped type hysteresis of Q- δ curve is considered that this behavior of the anchor bolt has influenced as one factor. The behavior of the anchor bolt observed in this experiment is shown in Fig.12. The anchor bolt is jointed with a nut at the wall. In the condition that the anchor bolt yielded and



Fig.11 Stress-elongation curve of anchor bolt

Fig.12 Behavior of anchor bolt

extended, after the wall rotated and the nut touched at the wall leg rib, the anchor bolt began (Q- δ curve began to recover from a slip condition) to be effective as a tension member, and, after the bond had deteriorated, the anchor bolt slipped out at the time of the removing load.

Moreover, it was shown that the tension stiffness of the anchor bolt lowered once in the cycle of R=1/200 from Fig.11. This was in agreement with the cycle which the bond crack of the anchor bolt (shown in Fig.5) arose. It is thought that looseness arose due to bond deterioration on the anchorage section of the anchor bolt into the wall. And in Q- δ curve of Fig.6-Fig.9, it has appeared as stiffness depression in the cycle of R=1/400-1/200.

Deformation component rate

A deformation component rate is shown in Fig.14. Using the measurements of displacement meters shown in Fig.4, each deformation is calculated from the geometric condition shown in Fig.17. It is shown that the rotation distortion by the elongation of the anchor bolt forms about 70% of all deformations from Fig.16 and that the rate becomes large as the rotation angle of the member becomes large. Moreover, it is shown that a horizontal slip deformation is comparatively small [about 3% of all deformations].



Fig.13 Geometric condition of deformation

Fig.14 Deformation component rate

ANTISYMMETRICAL LOADING TEST OF THE FREAM

Specimen

A frame specimen detail and a bar arrangement of a wall panel are shown in Fig.15 and Table 6. The frame specimen with orthogonal wall panels was set up. It was simulated the first floor of a 2-story house. The wall used for the experiment were concrete panels with ribs, and they had height of 2,700mm, rib thickness of 120mm, shell thickness of 46mm, and wall length of 897mm. Moreover, the floor was made into a shape which combines three 3810mm-length floor panels as shown in Fig.15 a long arm of floors was put on the plane wall panels. In addition, the floor panels had thickness of 150mm, shell thickness of 46mm.

The longitudinal rib's main reinforcements of the wall panel were 2-D13 (SD295A), and shell reinforcements were $2.9\phi@60$ mesh, and the rib main reinforcements of the floor panel were 4-D13 (SD295A), and shell reinforcements of the floor panel were $2.9\phi@100$ mesh. Moreover, foundation and wall panels were jointed by 2-D19 (SD295A) anchor bolts of mortar fitting combined use, a wall panel and a floor panel were jointed by 2-16 ϕ (SS400) bolts of mortar fitting combined use, and a wall panel and a wall panel were jointed by $3-13\phi$ (SS400) bolts, and a floor panel and a floor panel were jointed by 13ϕ (SS400) bolts and shear cotter of mortar fitting. In addition, specified concrete and mortar strength were $30N/mm^2$.

Loading system

The scheme of a loading apparatus is shown in Figs.16. The experiment was carried out by loading the repeat horizontal force (height of the loading point is 2,775mm) with the 500kN actuator horizontally attached to an abutment test wall, after it was loaded the constant axial load of 29kN (orthogonal walls are 14.5kN) which were equivalent to long time loading of a first floor wall with the 500kN hydraulic jack installed on the floor panel through the PC bar which anchored to the lower stub along the central heart of each wall.



Fig.16 Load apparatus

Fig.17 Displacement

A measuring method is shown in Fig.17. The horizontal and vertical displacement of the wall panel's top and the floor panel, and the aperture and slip displacement of the wall panel's leg, and the open displace-

ment of a sagging wall panel -a floor panel and a wall panel, and the slip displacement of a wall panel -acolumn panel were measured by displacement meters. Moreover, strains of main reinforcements, mesh lines and joint bolts were measured by wire strain gages.

Test results and discussions

Failure pattern

The crack pattern (rotation angle of member R=1/50) of the specimen is shown in Fig.18. The horizontal load depression was rising gradually by compression failure of the compression side longitudinal ribs of the plane walls, and the specimen showed the type of rupture near the wall panel specimen No4.



Fig.18 Crack Pattern

specimen shows slipped type hysteresis like the result of ally when a diagonal crack arose to the floor rib on the orthogonal wall in the cycle of R=+1/200. Fourth, anchor bolts yielded at load 158.32kN in the cycle of R=+1/100 and load -153.14kN in the cycle of R=-1/67. In the cycle

of R=1/67 and 1/50, the horizontal load did not decline. And the longitudinal-rib leg concrete on the compression side of the plane wall deteriorated in the cycle of R=1/33, after the load fell to about 90% of the maximum load, the horizontal load declined further in the cycle of R=1/20.

Although the failure type showed the same aspect of compression destruction on the compression side longitudinal rib as compared with the wall panel specimen No4 loaded the fluctuation axial load, it was shown that the suddenly horizontal force depression (R=1/40) seen in the wall panel specimen No4 did not arise in the frame specimen. So it is thought that the deformation performance improves by forming frame.

Inflection Point

Stress distribution of the plane wall's main reinforcements obtained from the main reinforcement strains of the plane walls is shown in Fig.20. From Fig.20, tensile stress has occurred in the main reinforcement on the right hand side of both the wall's leg and the left hand side of both wall's head at the positive load, and it is possible that the inflection point arises in the plane walls in the frame. Moreover, also in the crack pattern of Fig.18, in the wall upside, the transverse tension cracks are shown.



2W

2W6

Q(+)

1W3

1W2

W4

Wall

2W3 1W6

2W2 1W5

Wall



Flexural strength

The relationship between the lift up displacement of the plane wall's top and total tensile force of the vertical joint bolts of the orthogonal wall's top is shown in Fig.22. It is shown that the tensile force arisen in the vertical joint bolt of the orthogonal wall's top becomes larger as the plane wall rotates from Fig.22. The tension force arises in the vertical joint bolts of the orthogonal wall's top as floors are lifted up by rotation of the plane wall when the orthogonal walls are placed by the tension side of the plane wall's leg. And the tension force restrains the lift up of floors. It is shown that the orthogonal wall's effect[2] shown in Fig.21 that the pressing axial load of floors reposed on the plane wall applies to the frame also. Therefore it is thought that the orthogonal wall's effect can be similarly evaluated in the frame. In addition, also in the crack pattern of Fig.18, the punching-like shear crack occurred to the floor rib on the orthogonal wall. Thus the floor is pressing down the rotation of the plane wall.



Comparison between the experimental result at the time of anchor-bolt yield and the estimate calculated in consideration of the bending moment of the sagging wall and the floor and the orthogonal wall's effect is shown in Table 7.

In addition, the flexural strength estimate of the plane wall at the time of anchor-bolt yield is calculated by formula (2) ²⁾, as shown in Fig.24.The bending moment of the sagging wall and the floor calculated by flexural strength formula(3) of a beam which gave as σ_b the tensile stress(Fig.23) calculated from the strain of sagging wall-wall joint bolts and a floor's main reinforcement (I thought that all the cross sections of the main reinforcements on tension side were effective to bending because the bending crack of the floor on sagging wall-wall junction had arisen over the floor-panel and the next floor panel.) at the time of anchor bolt yield (rotation angle of member 1/100).

 $cMy = a_t \cdot \sigma_v \cdot j + 0.5 \cdot (N + Ns) \cdot j + 0.9 \cdot (N_t + N_{Tt}) \cdot D$ (2)

- a_t : Anchor bolt cross-sectional area (mm²)
- σ_v : The yield point of an anchor bolt (N/mm²)
- j: Lever arm 0.74(m), N: Axial load of a plane wall (N)
- Nt : Axial load of a orthogonal wall (N)
- N_{Tt} : Pressing axial load by a floor (N)
- Ns: Shear force due to the bending moment of the
- sagging wall and the floors (N)
- D : The plane wall length (m)

Fable 7	Flexural	strength
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Direc-	Flexural strength		
tion	Experiment eQy(kN)	Calculation cQy(kN)	eQy/cQy
+	158.32	143.34	1.10
_	153.14	135.32	1.13

Note) N_t and N_{Tt} take into consideration about the plane wall to which orthogonal wall exists in the leg tension side of a plane wall.

 $cM_{by} = 0.9 \cdot at_b \cdot \sigma_{yb} \cdot d$ (3)

Floor main reinforcement

at b: A floor's main reinforcements or junction bolts cross-sectional area (mm²)





In addition, as N_s , N_{Tt} and N_c of a formula (2), the shear arisen due to the bending moment of the sagging wall and the floor, the tensile force arisen in the vertical joint bolts of the orthogonal wall's head, and the shear arisen due to the bending of the compound beam of a sagging wall and a floor were given. From Table 7, the experimental result and the calculation show a good relationship. It is thought that the flexural strength of the frame can estimate in the scope of this experiment by taking account of the bending moment of the sagging wall and the floors and the orthogonal wall's effect.

Equivalent viscous damping factor

The relationship between the equivalent viscous damping factor and the rotation angle which were obtained from the hysteresis curve is shown in Fig.25. In addition, the hysteresis curve is adopted the twocycle of each rotation angle. Moreover, the experimental results of the wall panel are also shown all over Fig.25 for comparison. It is shown that the equivalent viscous damping factor of the frame is 11 - 16% from Fig.25, and improves about 3 to 4% from the wall panels. In addition, it is thought that about 11% is adequately as an equivalent viscous damping factor for the structural calculation of the frame because the equivalent viscous damping factor in the rotation angle over R=1/100 shows the steady value which is about 11%.



Fig.25 Equivalent viscous damping factor

CONCLUSIONS

The following results were acquired from these horizontal load experiments of the wall panels with ribs, and the frame.

- 1) The wall panels with ribs and the frame have the deformation capacity about 1/50 rotation angle. Moreover, the horizontal strength depression in rotation angle over R=1/50 becomes gradually by forming the frame.
- 2) The rate of the rotation deformation in the wall specimens occupies about 70% of all deformations, and that the horizontal slip deformation is comparatively as small as about 3%.
- 3) Inflection point arises in the plane walls by forming the frame.
- 4) It is thought that the orthogonal wall's effect can also be evaluated in the frame.
- 5) The bending strength of the wall at the time of an anchor bolt yield that is calculated by Eq.(1) shows a good relationship. Furthermore, it is thought that the horizontal strength of the frame can also be estimated by taking into consideration the orthogonal wall effect and the bending resistance of a sagging wall and floors.
- 6) The equivalent viscous damping factor of the frame is about 11%.

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