2 story and 2 span frame test and analysis for R/C frame with spandrel wall

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Abstract

In many cases of designing reinforced concrete (R/C) framed buildings with spandrel walls, it is common to provide slits located in the boundary line between spandrel walls and beams/columns in order to avoid uncertain effects of that on building structure. In this paper, to avoid confusion, “spandrel wall” is divided into 3 types, “wing wall” designating the subsidiary wall attached to column side, “spandrel wall” designating that attached to upper side of beam, “hanging wall” designating that attached to underside of beam. And “subsidiary wall” expresses the general term of 3 types substituting for original “spandrel wall”. Another wall used here is “mullion wall” designating vertically stood rather slender wall looked like thin column. Some reports [1,2] on great earthquake disasters clarify that adequately designed subsidiary walls could be effective to reduce response displacements of building consequently make damages decrease. Already there are a lot of research papers, test data and knowledge on members of beams and columns with subsidiary walls, but when structural designers intend to utilize subsidiary walls for structural members, they have to decide many design conditions of themselves, for instance, rigid zone length, plastic hinge location and length, M-N interaction curve of column with wing wall and so on.

Authors have carried out statically cyclic loading experiments using 2 story and 2 span R/C framed specimens [3] in order to investigate mainly whether the structural characteristics of frames are the same as the knowledges based on the results of beam and column member tests. Further, traditional design conditions for non-linear analysis of beams and columns with subsidiary walls to be adequate or not. Two of Specimens designed to assume 1st and 2nd story of middle rise buildings are supposed to have whole collapse configurations due to mainly beam yield failure computed by non-linear frame analysis rather than traditional model. Both specimens have the same R/C frame. Specimen No.1 has the spandrel wall of 100mm thick reinforced by double bar arrangement, and that of 60mm thick and single bar arrangement for No.2. Each column is subjected to lateral incremental cyclic load at column top keeping ratio of load force proportion of outer and inner column is 2:1 under constant axial forces loaded by unbonded steel bars. The R/C frame is designed to have lateral shear strength converted on base shear coefficient approximately 0.33 without subsidiary wall. Base shear coefficient means the ratio of total shear force of lowest story divided by the weight of building on top of that. The maximum lateral shear force of No.1 test results are +1,605kN, base shear coefficient 1.1 at R=+5.0/1000rad, and that of No.2 is +1,010kN, base shear coefficient 0.70 at R=+4.6/1000rad. Both of them show enough lateral shear strength to make a building to be designed in strength base, which requires minimum base shear coefficient 0.45. As effects of subsidiary wall, elastic stiffness of test results are 1.79 to 2.10 times as large as result of bare frame analysis, and ultimate strength of test results are 2.10 to 3.34 times of that of bare frame analysis.

Although the lateral shear strength of No.1 and No.2 show enough strength, the very last stage of failure configuration reveals unexpected phenomena. Major cracks initiated from the both lower corner of 2nd story’s openings extend to the base end of each column, as a result, to make flexural strength of column gradually to be weakened. Compared to non-linear analysis, test results show that initial stiffness of frames are lower than those of analysis and lateral shear strengths are also lower especially No.2 which have relatively thin subsidiary walls. We find that the traditional conditions and models of non-linear analysis when applied to R/C framed building with subsidiary walls need to be more considered.

Keywords: reinforced concrete; spandrel wall; frame experiment; frame analysis; design condition

1. Introduction

In many cases, as shown in Fig.1, R/C frame buildings in Japan have subsidiary walls so as to satisfy architectural design of space partition for dwellings, schools, office buildings and so on. Unfortunately this type of buildings tend to suffer shear fractures of columns or beams because of short clear span shown in Fig.2. From the viewpoint of structural design, subsidiary walls have been common to be detached structurally from R/C frame by slit so as to make building's behavior clear at the time of big earthquake. On the other hand, some past great earthquakes clarify that the subsidiary walls have the effect to enlarge elastic stiffness, as a result, to reduce deformation of buildings and also strengthen of both flexural moment and shear capacity. However the previously referred to advantages, they have disadvantage to make it easier to happen shear failure of beam or column due to short clear span. For long years, there have been accumulated the papers and knowledges on R/C members with subsidiary walls and consequently organized into some proposed formula on stiffness, ultimate strength, ductility and so on. But when structural designer tries to design a building utilizing subsidiary walls, there remain some challenges to be decided by designer like the length of rigid zone, location and length of hinge zone and deformation capacity after ultimate strength.

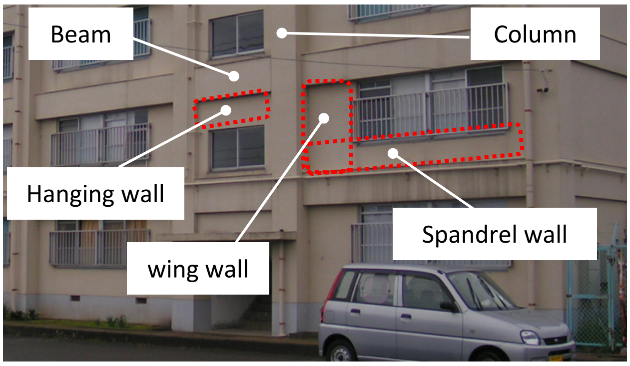


Fig. 1 – R/C frame with subsidiary walls

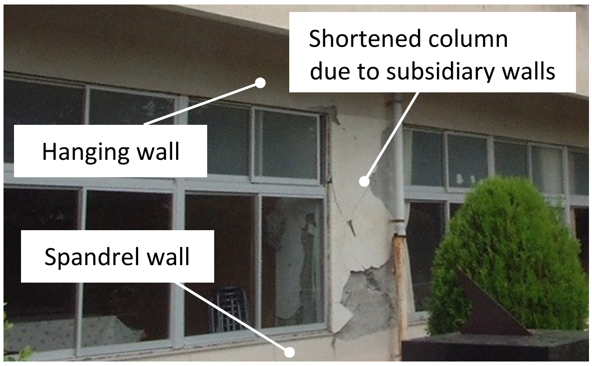


Fig. 2－Shear fracture due to short span column

2. 2 story and 2 span frame test on R/C frame with spandrel wall

2.1 Purpose of frame test

In order to investigate structural characteristics of R/C frames with subsidiary walls, we have carried out statically cyclic loading experiments using 2 story and 2 span R/C framed specimens [3]. Main purpose of frame test is to confirm the structural characteristics of frames are the same as the knowledges based on the results of beam and column member tests. Further, traditional design conditions for non-linear analysis of beams and columns with subsidiary walls, elastic stiffness, flexural strength and failure mode, are the same as we expected.

2.2 Design of frame specimen

The outline and reinforcing bar arrangement of the specimens are shown in Fig.3. Left half is No.1 and right half is No.2. The assumed building is a 6-story dwelling house. The size of specimens is scaled by 1/2 and targeted to the lowest 2 stories, made of 2 span frame of RC with subsidiary walls. Above 2nd story, just half height of 3rd story is built and reinforced bar arrangements are rather strong enough to avoid 3rd story columns and wing walls to be fractured because of directly attached to actuators. The distances between the adjacent columns are 3,500mm, and heights between each beam (height of story) are 1,500mm. There are 2 specimens named No.1 and No.2 with variable of wall thickness and reinforced bar arrangement.

Fig.4 and Fig.5 show the section of column with wing wall and beam with spandrel and hanging wall. The common points of both specimens are section size 400 mm $\times$ 400 mm of column with longitudinal bar arrangement of 16-D16 and hoop of 2-D10 (D: diameter of deformed steel bar) @50, (@100:outer column), and section size of beam 250mm$\times$350mm with longitudinal bar arrangement of each upper and lower of 3-

D16, (4-D16:attached to outer column), and stirrup of 4-D10@100mm, as an exception, in center of each span stirrup are arranged @50mm against short clear span. The R/C frame is designed to have base shear coefficient approximately 0.33 without subsidiary wall. Base shear coefficient means the ratio of total shear force of lowest story divided by the weight of building on top of that. Subsidiary wall of No.1 has thickness of 100mm, is arranged of 2-D6@50. Subsidiary wall of No.2 has thickness of 60mm, is arranged of 1-D6@100. Diameters of reinforcement for opening, arranged the nearest to opening, are D10.

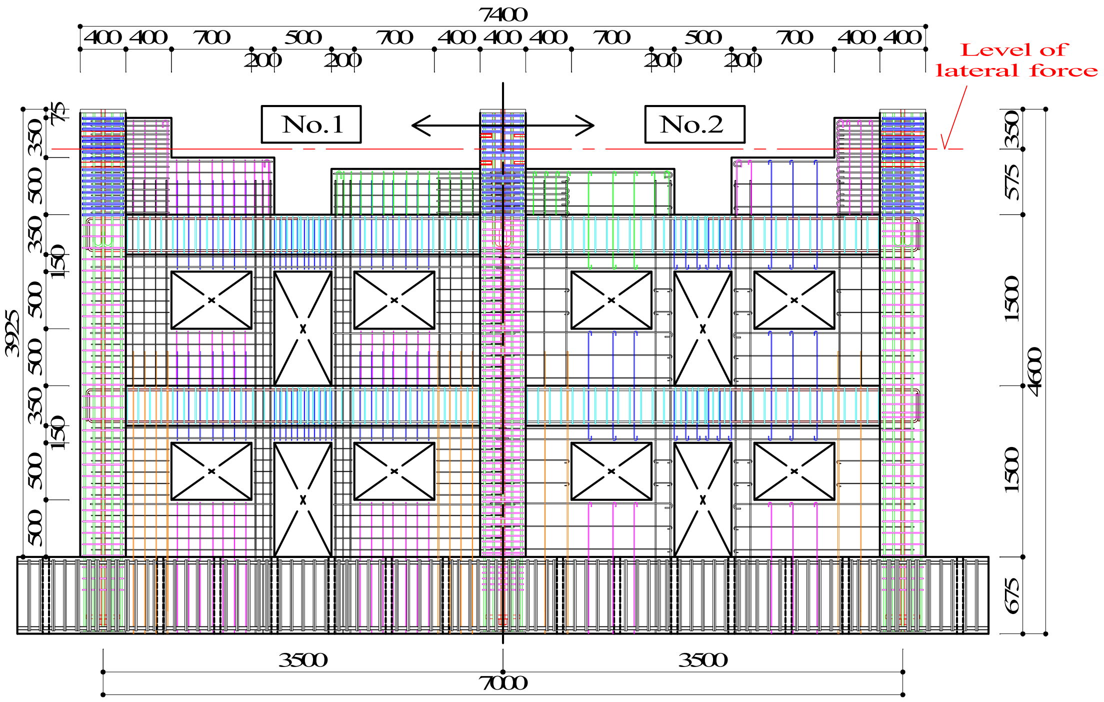


Fig. 3 – Outline and bar arangement of specimen

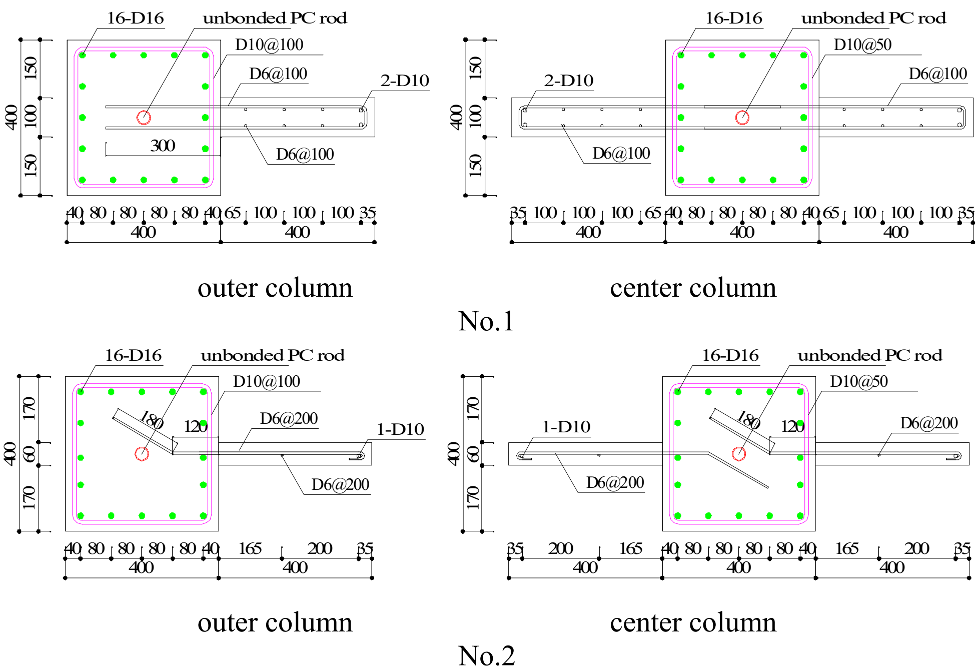
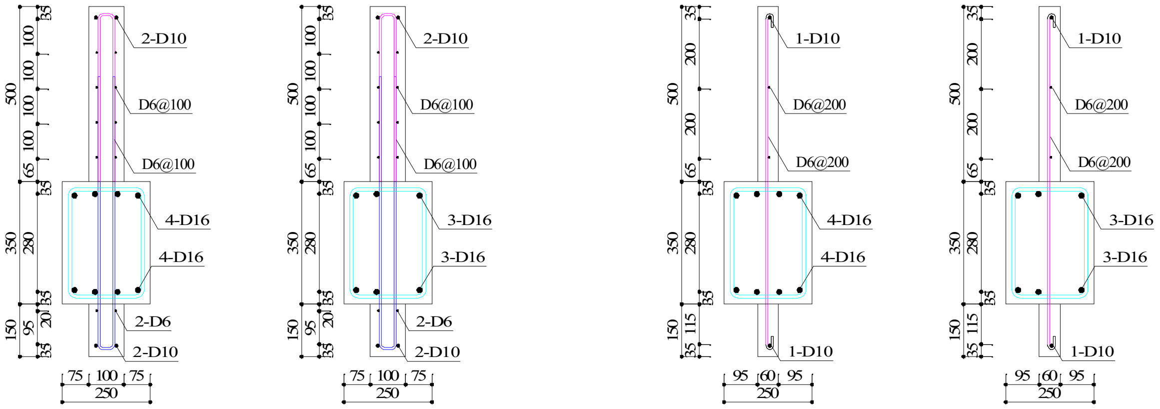


Fig. 4 – Section of column with wing wall



outer-beam inner-beam outer-beam inner-beam

No.1 No.2

Fig. 5 – Section of beam with spandrell and hanging wall

The mechanical properties of steel bars are shown in Table 1, and compressive strengths of concrete are in Table 2.

Table 1 – Mechanical properties of steel bar

	Diameter	Yield stress
Column bar, beam bar	16mm	380 N/mm^2
Hoop, stirrup, wall bar (opening reinforcement)	10mm	382 N/mm^2
Wall bar	6mm	454 N/mm^2

Table 2 – Compressive strength of concrete

	No.1	No.2
C, W(3F)	46.6	50.6
C(2F),W(2F),B(3F)	38.1	33.6
C(1F),W(1F),B(2F)	37.2	36.5
Foudation	62.8	60.0

C: column, B: beam, W: wall Unit: N/mm^2

2.2 Test setup and loading cycle

Test setup is illustrated in Fig.6. Constant axial forces are loaded by unbonded prestressed steel bars embedded along the center of column sections from the bottom of foudation to top of column. Axial forces are respectively 0.075 $b \times D \times \sigma_B$ for outer column, 0.15 $b \times D \times \sigma_B$ for center column, here "b" and "D" are width and depth of column and "σ_B" is compressive strength of concrete as shown in Table 2.

Cyclic lateral shear forces are loaded statically at the center height of 3rd columns from actuators respectively keeping the shear force ratio of outer column against center column equal to 1:2. The distance between the center of actuator and upper surface of foudation is 3,575mm.

The value of lateral shear force is measured with loadcell incorporated to each actuator. The horizontal displacement of beam-column joint is measured with contact type electric displacement sensor. All displacement sensors are fixed to steel frame isolated from specimen. Q_{total} is defied total loads of all actuators. And $\delta 1^{st}$, $\delta 2^{nd}$, R_{total} are defined as Eq. (1), Eq. (2), Eq. (3).

$$\delta 1^{st} = (D4+D5+D6)/3 \tag{1}$$

$$\delta 2^{nd} = (D1+D2+D3)/3 - \delta 1^{st} \tag{2}$$

$$R_{total} = (D1+D2+D3)/3/2{,}825 \tag{3}$$

Loading cycle schedule is shown in Fig.7. R_{total}=±1/3200 , ±1/1600, ±1/800 are 1 cycle for each, R_{total}=±1/400, ±1/200, ±1/100, ±1/50, ±1/33, ±1/25 are 2 cycles for each. As for No.1, loading cycle of R_{total}=±1/33, ±1/25 are saved, because shear force fall down rather rapidly.

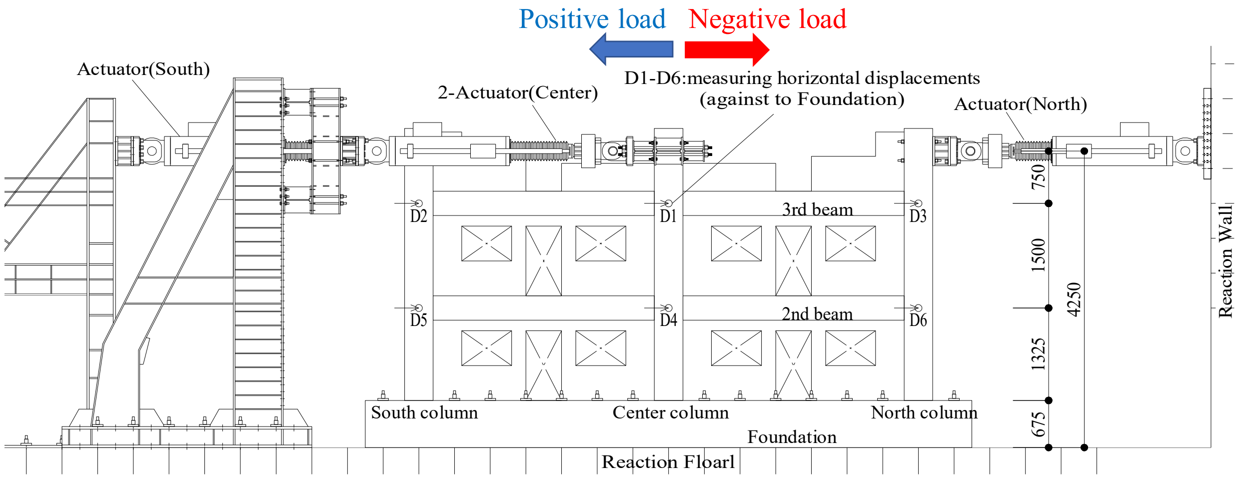


Fig. 6 – Setup of loading system

Table 3 – Constant axial force

Specimen	No.1		No.2	
Column	Outer (south, north)	Inner (Center)	Outer (south, north)	Inner (Center)
Axial force【kN】	377	745	375	741

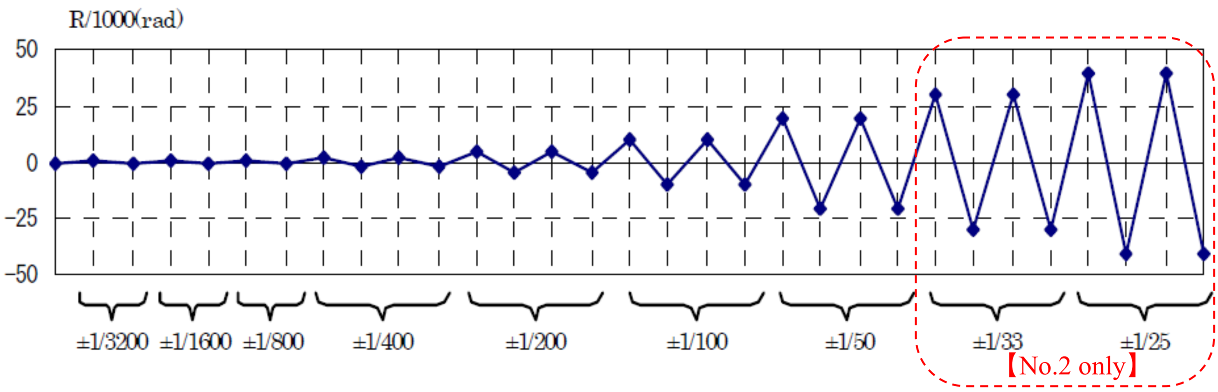


Fig. 7 – Loading cycle schedule

2.3. Test result

The crack and damage condition of No.1 at R_{total}= -1/200, -1/50 are shown in Fig.8, and that of No.2 at R_{total}= -1/200, -1/25 are shown in Fig.9. In cycle of R_{total}= ±1/200 in left-hand illustrations, both specimens show ultimate strength. Right-hand pictures are conditions of very last cycle of each specimen.

Damage process of No.1 is that flexural diagonal cracks appear in wing, spandrel and hanging walls of 1st and 2nd story at the corner of openings in the cycle of R_{total}=1/3200~1/1600. Flexural cracks of beams appear in the cycle of R_{total}=1/1600~1/800. Diagonal shear cracks appear in wing walls, mullion walls and spandrel walls in the cycle of R_{total}=1/400. Flexural cracks in columns of 1st and 2nd story appear in the cycle of R_{total}=1/200 and shows ultimate shear strength in both positive and negative directions. After ultimate strength, as shown red lined box in right-hand picture, shear cracks and compressive fractures of concrete of wing walls in the 2nd story progress and shear force rapidly drop down. The failure mode is partly 2nd story failure yielded in beams of 3rd and column base of 2nd story.

Damage process of No.2 is that flexural vertical cracks in wing, spandrel and hanging walls of 1st and 2nd story at the corner of openings and flexural cracks of beams appear in the cycle of R_{total}=1/3200~1/1600. Diagonal shear cracks appear in wing walls, mullion walls and spandrel walls in the cycle of R_{total}=1/800. Flexural cracks in columns of 1st and 2nd story appear in the cycle of R_{total}=1/400. Ultimate shear strength in both positive and negative directions are shown in the cycle of R_{total}=1/200. After ultimate strength, shear cracks and compressive fractures of concrete of wing walls and mullion walls in the 1st and 2nd story progress and shear force moderately drop down. The failure mode is whole frame failure yielded in beams of 2nd and 3rd and column base of 1st story.

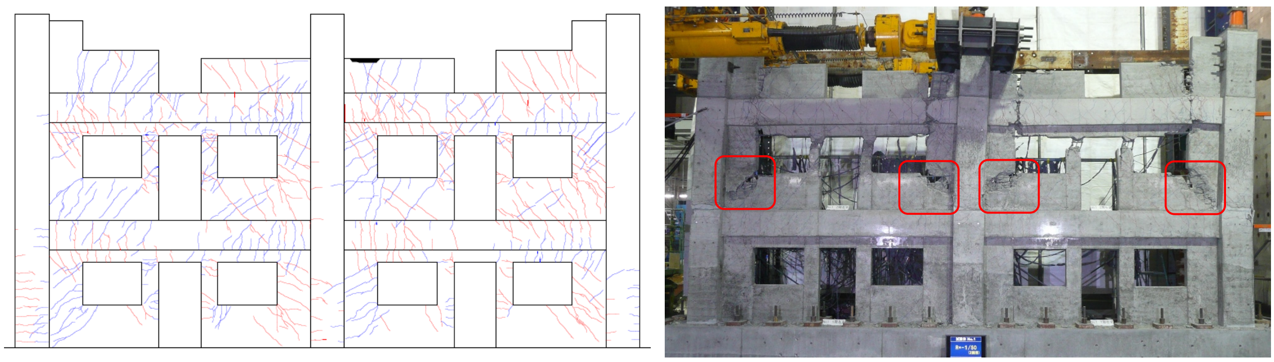


Fig. 8 – Damage condition of No.1 specimen(left: R_{total} =-1/200rad, right: R_{total} =-1/50rad)

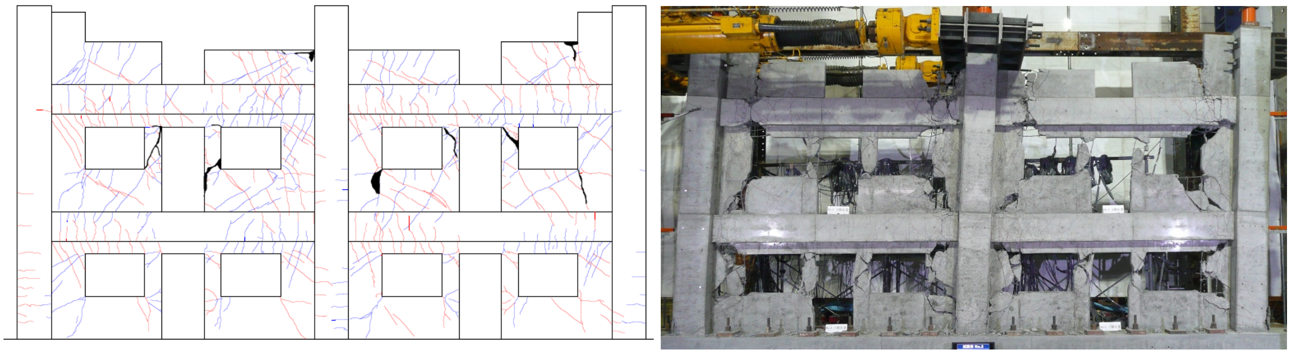


Fig. 9 – Damage condition of No.2 specimen(left: R_{total} =-1/200rad, right: R_{total} =-1/25rad)

Relationship between Q_{total} and R_{total} is shown in Fig.10, black solid-line express test and blue dotted-line express analysis of bare frame without subsidiary walls. The ultimate lateral shear force of No.1 test results are +1,605kN, base shear coefficient 1.1 at R_{total} =+5.0/1000rad, and that of No.2 is +1,010kN, base shear coefficient 0.70 at R_{total} =+4.6/1000rad. Both of them show enough lateral shear strength to make a building to be designed in strength base frame. However both specimens show shear force drop after ultimate shear strength, hystelysis do not demonstrate pinched loop or reverse S shape loop those are unfavorable in energy consumption.

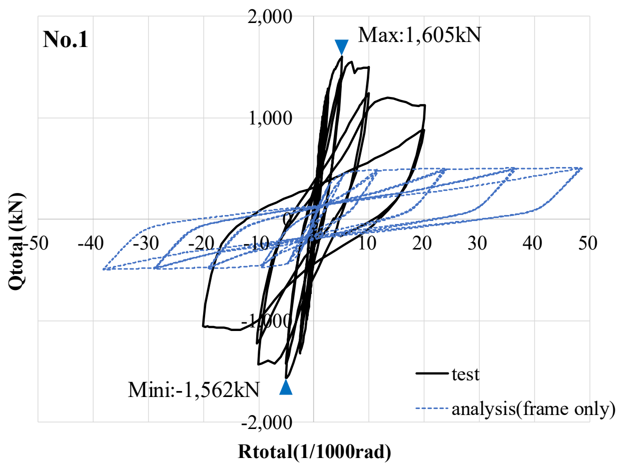


Fig. 10 – Relation of Q_{total} and R_{total}

As for elastic stiffness, test results are 1.79 to 2.10 times as large as result of bare frame analysis, and as for ultimate strength, 2.10 to 3.34 times shown in Table 4.

Table 4 – Comparison of elastic stiffness and ultimate shear strength with bare frame analysis

	Analysis(*A*)	Test(*T*)			
	Bare frame	No.1	*T*／*A*	No.2	*T*／*A*
Elastic stiffness 【kN/mm】	168	353	2.10	301	1.79
ultimate shear strength 【kN】	480	1605	3.34	1010	2.10

A course of ratio of $\delta 1^{st}$ story and $\delta 2^{nd}$ story are shown in Fig.11. As for No.1, the ratio of the 1st story starts from 40% at *R*=1/3200, almost linearly dopping down to 8% at *R*=1/50. This verifies fracture mode is partly 2nd story failure. As for No.2, the ratio of the 1st story starts from 30% at *R*=1/3200, and exceeds 25% in *R*=1/100rad, and ends 38% at *R*=1/25. This also verifies fracture mode is wholle frame failure.

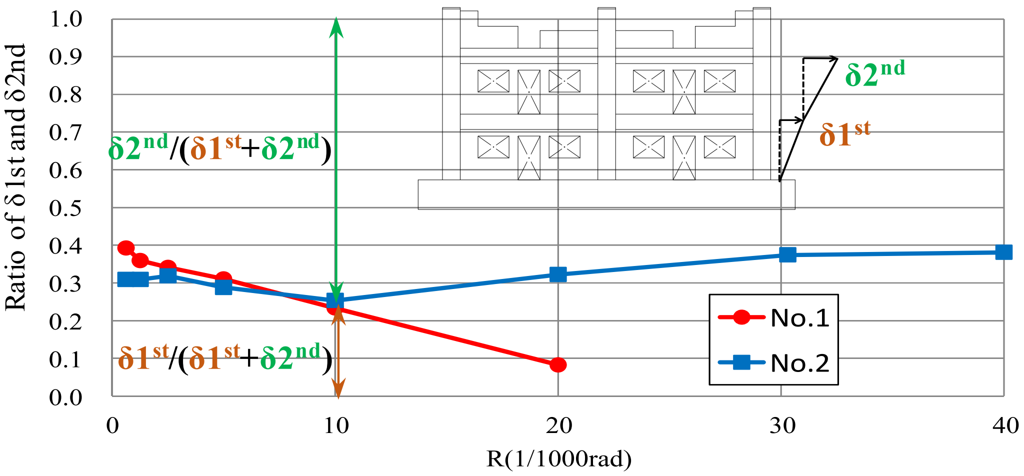


Fig. 11 – Ratio of *δ1* and *δ2*

3. Non-linear analysis for frame test

3.1. Frame model

In order to clarify peculiarity of R/C frame with subsidiary walls, we compare test results to results of non-linear frame analysis modeled by ordinal process.

Frame model is indicated in Fig.12. To take *M-N* interaction into consideration, Multi-spring(MS) model is employed. In Fig.13, sections of columns and beams with subsidiary walls are devided into concrete and steel elements. Stress and strain curves of concrete and steel are defined as shown in Fig.14. Every MS spring is located at critical section and the length of hinge zone is *D*/2, here *D* denotes depth of column or beam. And the intermediate between both hinge zones is elastic line. Fig.14 shows also the length of rigid zone. The boundary line of rigid zone and critical section located precisely at the end of opening. Shear behavior is modeled by elastic sigle axis spring considering only elastic stiffness of shear.

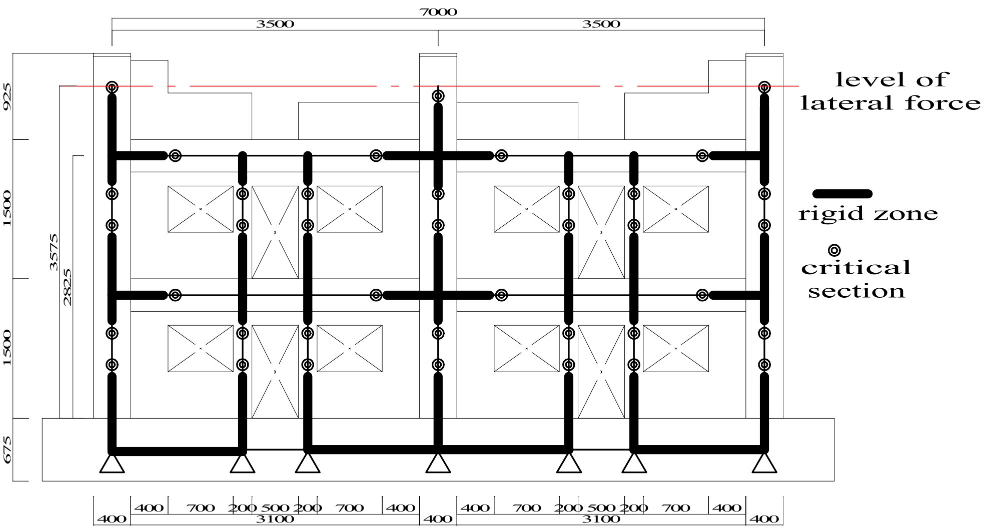


Fig. 12 – Frame model

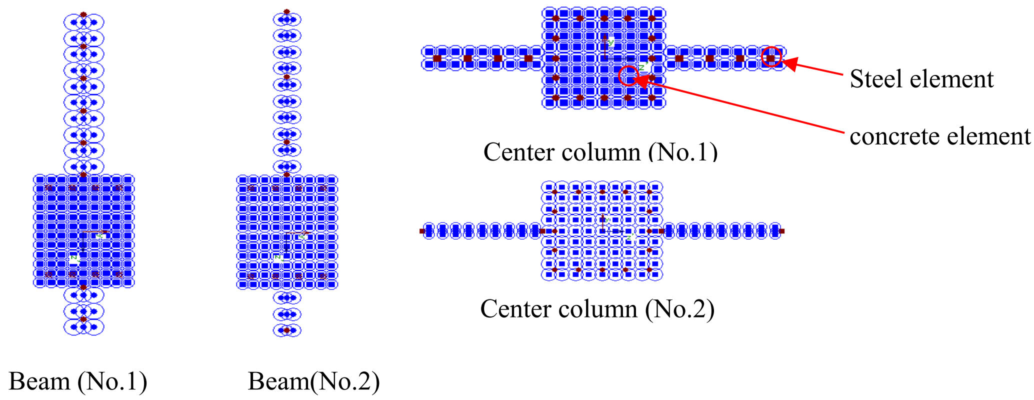
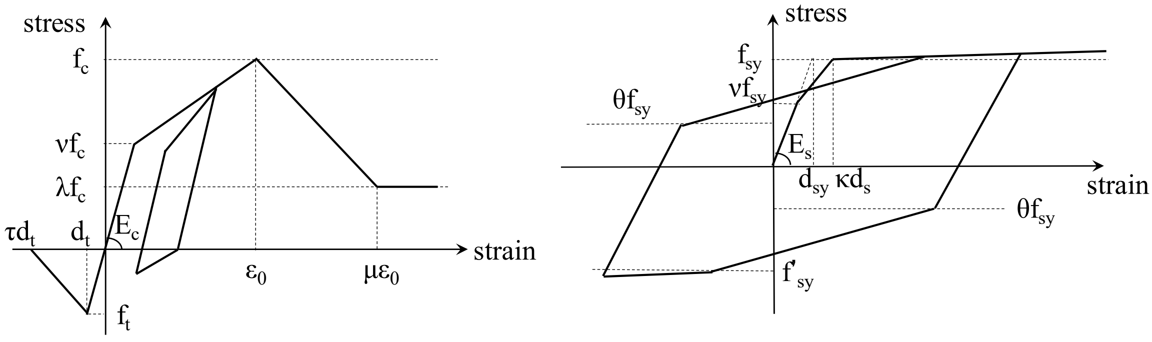


Fig. 13 – MS model for beam and column with spandrel wall



ν=0.5, λ=0.2, μ=30, τ=3 ν=0.8, κ=1.2, θ=0, β=0.001

(a) Concrete (b)Steel bar

Fig. 14 – MS model for materials

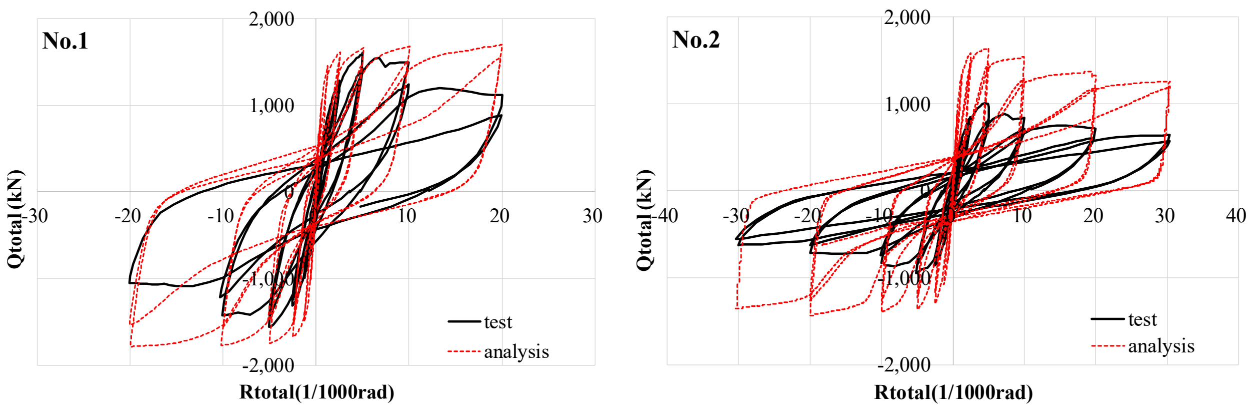


Fig. 15 – Comparison of test and analysis of Q_{total}-R_{total} relation

Fig.15 shows comparison of test and analysis of Q_{total}-R_{total} relation. Black solid-line is test result and red dotted-line is alalysis. Table 5 shows elastic stiffness and ultimate shear strength of test and analysis. Elastic stiffness is defined as the tangent of both positive and negative peak points of first cyclye. Tests results are 0.438 and 0.546 of that of analysis. The reason why elastic stiffness of analysis are larger than that of analysis might be that analysis models have clear rigid zone identical with subsidiary walls, on the other subsidiary walls of specimens might as well are affected by exsisting stresses and a lot of cracks are observed in subsidiary walls. The length of rigid zone should be revised.

The ultimate shear strengths of test results are 0.874, 0.944 of that of analysis for No.1. They relatively coinside with each other. As for No.2 ratio of test results/analysis are 0.616, 0.661. In Fig.9 mullion walls in 2nd story are already shear fractured and may lost shear strength, and also wing walls that constitute rigid zone for beams might loosen their binding effects.

Table 5 – Comparison of elastic stiffness and ultimate shear strength

	No.1			No.2		
	Test(T)	Analysis(A)	T/A	Test(T)	Analysis(A)	T/A
Elastic stiffness 【kN/mm】	353	805	0.438	301	551	0.546
Ultimate shear strength(+) 【kN】	1,605	1,700	0.944	1,010	1,640	0.616
Ultimate shear strength(-) 【kN】	-1,562	-1,787	0.874	-945	-1,430	0.661

4. Comparison with moment distribution assumed from the reinforcing bars strain of test and that of analysis

4.1. Assumption of moment of test (M-test)

Moment distribution and each column's exsisting shear force of test is not determined because frame specimen is statically indeterminate. This study utilises measured strain of steel bars to assume moment distribution.

Process of assumption of moment distribution, illustrated in Fig.16, is as follows. (1) Strains for constant axial force are compted under Bernoulli-Navier theory instead of test strain. Because loading axial force uniformly into all section area is quite difficult. (2) Strain of longitudinal bar is equal to concrete strain of the same location. (3) Tensil strength of concrete is neglected. (4) Strain of steel bar which is not measured is interpolated by the nearest measured strain of adjacent bars of both sides. (5) Compressive stress-strain curve of concrete is modeled with 4 linear lines shown in Fig.17. 1st line represents initial stiffness ended at $0.8\sigma_B$ for No.1 and $0.6\sigma_B$ for No.2. 2nd line ends at compressive strength and strain of material tests. So the area of material test and model until compressive strength that expresses the energy consumption become nearly equal to each other. 3rd line ends at $0.5\sigma_B$ and strain value 0.38%, 4th line keeps $0.5\sigma_B$. (7) M-test is calculated around the center of beam or column.

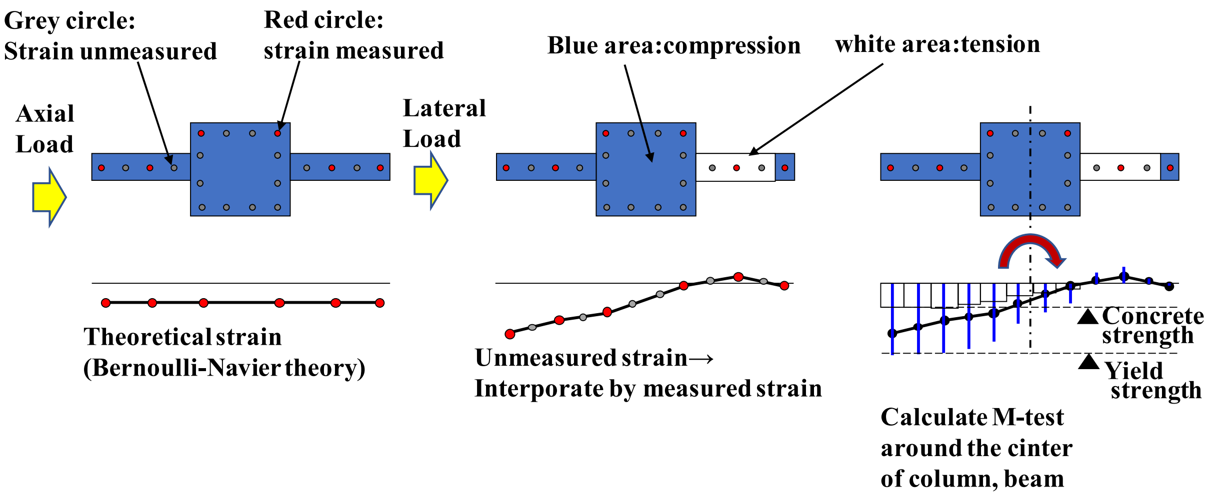


Fig. 16 – Process to asume flexural moment of section

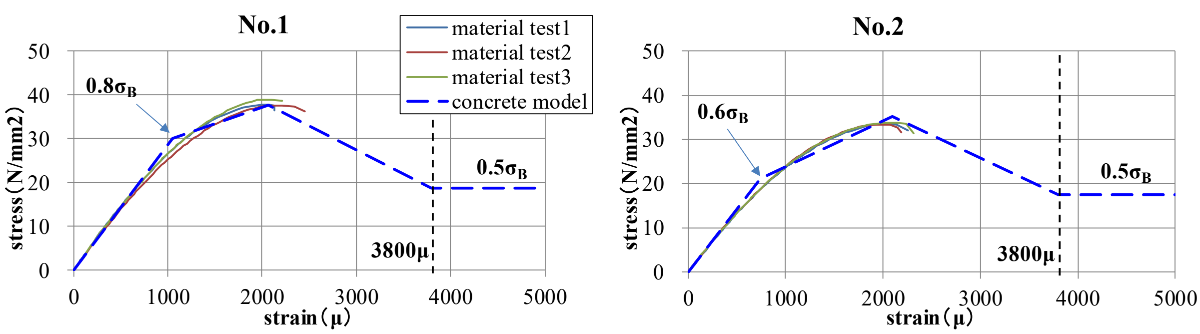


Fig. 17 – Concrete model employed in assuming *M* distribution

Table 6 – Comparison of shear force of each column at R_{total}=+1/200

Specimen	No.1			No.2		
$\sum Q$ (story force)	M-Test (Q_T) 【kN】	Actuators (Q_A) 【kN】	$\frac{Q_T}{Q_A}$	M-Test (Q_T) 【kN】	Actuators (Q_A) 【kN】	$\frac{Q_T}{Q_A}$
2F	1,536	1,423	1.08	1,072	1,013	1.08
1F	1,332	1,423	0.94	1,102	1,013	1.09

In Table 6, each story forces (Q_T) calculated from M-test is compared to total shear force (Q_A) of 3 actuators. Q_T is calculated of each column at top and bottom of opening devided by height of opening 500mm. The ratios of Q_T / Q_A are 0.94~1.09, show good correspondances.

Fig.18 compares *M* distribution of M-test and analysis. As for No.1, assumed from M-test, inflection point of north column in 2nd story and center column and north column of 1st story are lower than that of analysis. And *M* distribution of beam in 2nd story matches relatively good.

As for No.2, assumed from M-test, inflection point of center column in 2nd story is higher than that of anaysis. And *M* distribution of beam in 2nd story matches relatively good.

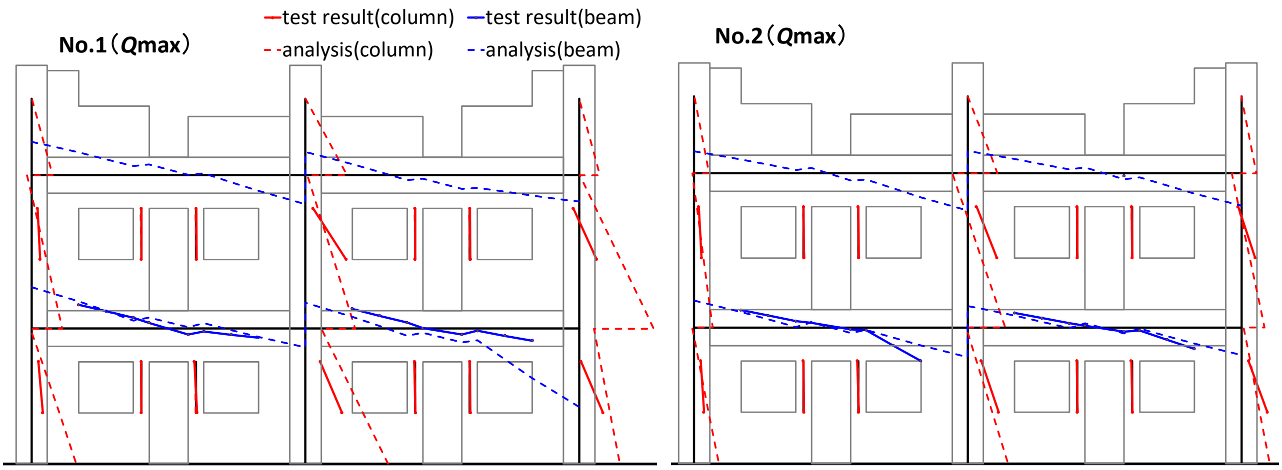


Fig. 18 – Comparison of *M* distribution of M-test and analysis at R_{total}=+1/200

5. Conclusions

Authors have carried out statically cyclic loading experimental study on R/C frame with subsidiary walls, and compare to the result of non-linear analysis based on the ordinal conditions.

・The failure mode of 2 specimens differ. No.1 with thick and well reinforced subsidiary walls fails as partly 2nd story failure. No.2 with thin and poorly reinforced subsidiary walls fails as whole frame failure.

・The maximum lateral shear force of No.1 is +1,605kN, base shear coefficient 1.62 at *R*=+5.0/1000rad, and that of No.2 is +1,010kN, base shear coefficient 0.73 at *R*=+4.6/1000rad. Both of them show enough lateral shear strength to make a building to be designed in strength base.

・As for elastic stiffness, test results are 1.79 to 2.10 times as large as result of bare frame analysis, and as for ultimate strength, 2.10 to 3.34 times.

・Compared to analysis, elastic stiffness of test results are 0.438 and 0.546. Thus analysis models have clear rigid zone identical with subsidiary walls should be revised.

・The ultimate shear strengths of test results are 0.874, 0.944 of that of analysis for No.1. They relatively coincide with each other. For No.2, ratio of test results/analysis are 0.616, 0.661. Mullion walls in 2nd story are already shear fractured.

・This study utilizes measured strain of steel bars to assume moment distribution. The story shear force assumed from strain of steel bars show good correspondences with total shear forces of 4 actuators.

・Compared M distribution of M-test with analysis, as for No.1 inflection point of north column in 2nd story and center column and north column of 1st story, which are assumed from M-test, are lower than that of analysis. As for No.2 inflection point of center column in 2nd story assumed from M-test is higher than that of analysis. Moment distribution of beam in 2nd story of both No.1 and No.2 match relatively good.

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