EXPERIMENTAL INVESTIGATION ON ASEISMIC STRENGTHENING FOR EXISTING REINFORCED CONCRETE FRAMES

by

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SYNOPSIS

Five one-story, one-bay, one-half scale reinforced concrete frames were tested in order to obtain designing data for aseismic strengthening of the existing Morioka Station building of the Japanese National Railways (JNR). In two of them, the existing frames were strengthened with infilled shear walls, and in another, the existing columns alone were strengthened with steel plate encasing. It was confirmed that these strengthened frames all have ample earthquake resistant properties, as compared with the existing frames and monolithically cast shear walls.

INTRODUCTION

According to the plan for the construction of new Morioka Station building for the Tohoku Shinkansen (bullet train) Line of JNR now under construction, a part of the new station building will be placed on the old station building now in use. This has necessitated the aseismic strengthening in some way or other of the old one-story reinforced concrete station building with one-story basement in preparation against increasing earthquake load from the superstructure to be built upon it.

In case of the old Morioka Station building, the functionality, earthquake-proofness, and adaptability to construction and other properties of the building were examined collectively, and methods of aseismic strengthening for it were proposed as follows:

1) Infilled shear walls are additionally erected in a part of the existing frame in order to increase the aseismic strength of the entire frame.

2) All of independent columns are strengthened with steel plate encasing in order to improve the ductility of the entire frame.

This investigation was conducted with an aim at obtaining data for designing the execution of these proposed aseismic strengthening methods. Tests for confirming the earthquake resistant properties of the five frame test specimens were performed by using their one-half scale models.

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BACKGROUND

Shear Transfer Mechanism of the Frame with Infilled Shear Wall. The assumed shear transfer mechanism in the inelastic range under a severe earthquake of the frame with infilled reinforced concrete shear wall is shown in Fig. 1. The shearing force acting on the infilled walls is the total of the lateral pressure from the column top and the shearing force transferred by the mechanical anchors. Hereupon, the column top and the column bottom fall into the state of punching shear due to the lateral pressure from the wall. The punching shear becomes severer with the lessening of the transfer effects of the shearing force of the mechanical anchors. This suggests that the following points should be taken into consideration in designing infilled shear walls:

1) The joint of infilled wall to the existing frame should have sufficient strength and ductility.

2) The column top and column bottom should be reinforced sufficiently to prevent their shear failure.

3) The concrete of infilled shear walls should be thick enough and reinforced for shearing sufficiently to prevent failure by stress concentration.

In this study, specified proposals were made on these points, and their aseismic strengthening effects were confirmed experimentally.

Improved Type of Mechanical Anchor. In connecting the existing frame with infilled shear walls, mechanical anchors developed by wedging action into the existing concrete are used generally. However, in the mechanical anchor of conventional type available on the market, the skrew part of the anchored bars is located on the shear plane so that the skrew part may be ruptured by stress concentration and the joint may not have ductility. Thereupon, an improved type of mechanical anchor as shown in Fig. 2 was devised. A shearing test on the existing column-to-infilled shear wall joint using improved type mechanical anchors was conducted beforehand at the Kajima Institute of Construction Technology, and it was found that the joint using the improved type anchors have far better ductility than that using the conventional type anchors as shown in Fig. 3 [1].

Strengthening Columns with Steel Plate Encasing. As one of the aseismic strengthening systems for the existing reinforced concrete columns, the following method as shown in Fig. 4 was proposed. According to the method, after finishing materials are removed from it, the existing column is encased with relatively thin steel plates, with the seams between the plates being spot welded and the clearances between the existing column and the plates being grouted with mortar.

This reinforcement method, which was intended only for shear strengthening of columns, serves to improve the ductility of the entire frame, without changing the strength and stiffness of the existing frame considerably. The effects of shear strengthening of the existing reinforced concrete columns by this method have been confirmed through experimental investigations conducted in the past by the Kajima Institute of Construction Technology [2].

TEST PROGRAMS

Object. In this experiment, lateral loading tests were conducted on one-half scale models of the frames of the existing Morioka Station building, which were aseismically strengthened with infilled shear walls and steel plate encasing of columns, respectively. Through these tests, the experiment was aimed at obtaining the structural properties of the strengthened frames from the elastic state to their failure, thereby obtaining data for designing the execution of the strengthening methods.

Test Specimens. For the experiment, five test specimens summarized in Table 1 were used. Specimen No. 1 is the frame, extracted as a portal frame, of the first-story section of the existing Morioka Station building, with its details given in Fig. 5. Specimens No. 2 and No. 3 are the frames made by reinforcing the existing frames (Specimen No. 1) with 15cm-thick infilled shear walls. The details of these infilled shear walls are shown in Fig. 6. Specimen No. 4 is a test specimen of monolithically cast shear wall for comparison use. Specimen No. 5 is columns of the existing frame, reinforced with steel plate encasing (thickness of the plate: 3.2mm), with the beam being stiffened to make the column a failure type one.

The strengthening designs for these test specimens are based on the "Design Recommendation for Repair and Strengthening of Reinforced Concrete Building" proposed in 1978 in Japan [3].

The specified compression strength of the concrete used was $Fc = 210kg/cm^2$, and according to the results of the compression test of concrete cylinders at the time of the experiment, the averaged compression strength of the concrete in the existing frame part was $c\sigma_B = 273kg/cm^2$ and that of infilled shear wall concrete was $c\sigma_B = 230kg/cm^2$. According to the results of tension tests on the steel bars used, the yield strength was $s\sigma y = 3.57t/cm^2$ in the longitudinal reinforcement of columns and beams (13ϕ), and $s\sigma y = 3.66t/cm^2$ in shear reinforcement of walls (DIO).

Loading. The tests were conducted by using the apparatus shown in Fig. 7. In the tests, the base sections of the specimens were fixed to the large-size loading frame, and vertical load $N = 0.15Fc \cdot B \cdot D$ (B and D are column width and column height) was brought by hydraulic jacks to act on both the column tops first, and under such a condition the lateral force was brought by the hydraulic jacks to repeatedly act on the central position of the beam to the left and right alternately. The loading procedure for each of the test specimens is given in Table 2.

Data Acquisition. On all of the test specimens, story displacements and rotational displacements at the beam-to-column joints were measured with dial gauges, strains of longitudinal reinforcements at the ends of columns and beams were measured with wire strain gauges, and the crack incidence was measured with the eye. For the shear wall type specimens, longitudinal displacements of columns, slips and separation displacements between columns and beams on one hand and walls on the other, and strains of wall reinforcements were measured additionally.

TEST RESULTS AND CONSIDERATIONS

The results of the tests are tabulated in Table 3. The comparisons of post-test cracking patterns and load-deflection curves between the infilled wall specimen (No. 3) and the monolithic wall specimen (No. 4) are given in Fig. 8. Fig. 9 shows the comparison of envelopes of load-deflection curves between the specimens. Fig. 10 shows rotational behavior of the beam-to-column joints.

The Frames with Infilled Shear Walls. The frames reinforced with infilled walls (Specimens No. 2 and No. 3) reached maximum strength at the story deflection angle R ($R = \delta/h$ where, δ is story deflection, h is story height) of about 1/200 rad. With the strength falling slowly with the subsequent increase of deflection, the frame showed a ample earthquake resistant property of bending yield type due to overturning moment.

The initial stiffness of the infilled shear wall frame represented some 64 percent of the monolithically cast shear wall frame (Specimen No. 4) and about 19 times as high as that of the existing frame (Specimen No. 1). And the maximum strength of the infilled wall frame was some 74 percent of the monolithic wall frame and about 5.6 times as high as that of the existing frame.

In the infilled shear wall frame, the averaged shearing stress was $\tau \doteqdot 10 kg/cm^2$, and the slips ana separations between the frame and the walls were observed. It became known from the rotational deflection quantity at the beam-to-column joints that the columns and beams increase their deformation properties as frames.

It was found that there are no great difference in restoring characteristics between two infilled shear walls, while some disparity exists between them in cracking patterns, and the columns not reinforced with steel plate encasing are conspicuous in rupture of the concrete in their bottoms ultimately.

The Frame with Columns Reinforced with Steel Plate Encasing. The frame with columns reinforced with steel plate encasing reached maximum strength at $R \doteqdot 1/100$ rad, and the strength showed a tendency to rise against the subsequent increasing deflection. Even at $R = 1/20$ rad, the frame demonstrated ample earthquake resistant property without any serious damages sustained.

Failure Mechanism of Specimens. The ultimate failure mechanism of Specimen No. 1 was due to bending yield of the both ends of beams and column bottoms, and that of Specimen No.5 was due to bending yield of column tops and bottoms. They were in conformity with the failure mechanism estimated at the time of designing.

On the other hand, in the wall-type Specimens (Nos. 2, 3 and 4), the ultimate failure mechanism was bending yield due to overturning moment at the base, and was different from shearing failure mechanism of earthquake resisting walls estimated at the time of designing. This is judged to have been caused by the fact that the shearing capacity of the shear walls was higher than the value estimated by the design estimation formula. The established design guidelines can be used safely in designing, but requires caution in estimating a failure mode.

CONCLUSION

Through this experimental investigation, useful data for desiging the execution of aseismic strengthening methods for the existing Morioka Station building. Main conclusions reached through this experiment are as follows:

(1) The existing frame aseismically strengthened with infilled shear wall demonstrates ample earthquake resistant property of bending yield type, and its maximum strength is about 5.6 times as high as the existing frame unreinforced.

(2) However, in comparison with the monolithically cast shear wall, its maximum capacity represents only some 74 percent and its initial stiffness only about 64 percent. This poses an important question for the method of connecting the existing frame with infilled wall. The mechanical anchors of improved type used in this experiment serves to improve ductility of the joints.

(3) In the existing frame with its columns reinforced with steel plate encasing, the column tops and column bottoms fell into bending yield, and it was confirmed that it maintains sufficient ductility up to the story deflection angle of $R = 1/20$ rad.

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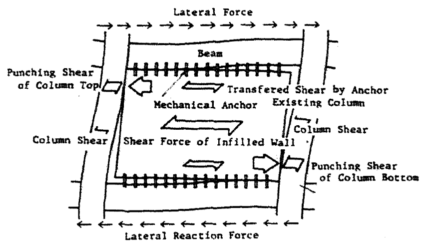

Fig.1 Shear Transfer Mechanism of the Frame with Infilled Shear Wall

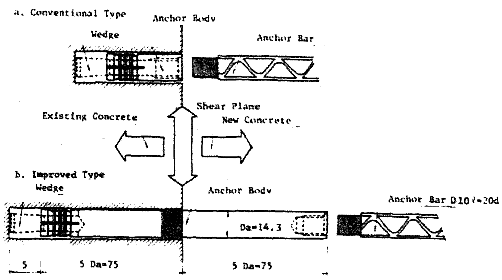

Fig.2 Newly Devised Mechanical Anchor

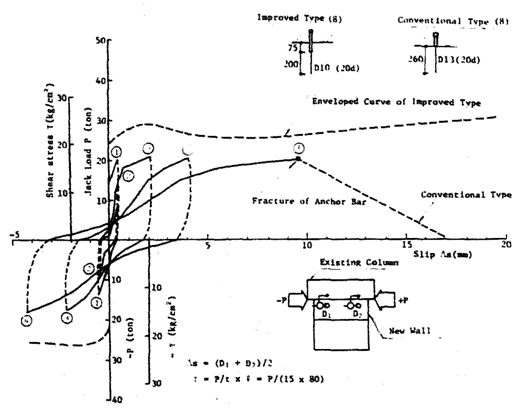

Fig.3 Comparison of Shear Transfer Effect between Conventional and Improved Anchor(1)

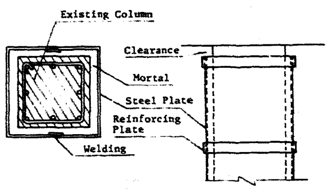

Fig.4 Aseismic Reinforcing System of Existing Columns

Table 1 Test Specimens

	Out line	Designed Capacities	
1	P → 2,150; 3,000	P → 5.62 9.32 6.42 5.30 10.44 9.55 12.16 14.12 15.70 O: Yield Hinge	P = 21.7ton Column τ_c=6.8kg/cm²
2	P → 2,150; 3,000	P → 10.2 80.6 11.3 Shear failure of Infilled Wall	P = 102ton Wall τ_w=22.8kg/cm²
3	P → 2,150; 3,000	P → 10.2 80.6 11.3 Shear failure of Infilled Wall	P = 102 ton Wall τ_w=22.8kg/cm²
4	P → 2,150; 3,000	P → 151 Shear failure of Shear Wall	P = 151ton Wall τ_w=33.6kg/cm²
5	P → 2,150; 3,000	P → 16.5 13.2 16.9 13.5 13.2 16.5 O: Yield Hinge	P = 30.4 ton Column τ_c=7.9kg/cm²

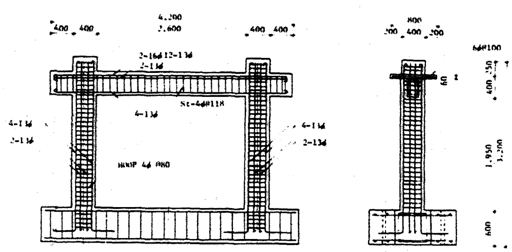

Fig.5 Details of Specimen No.1

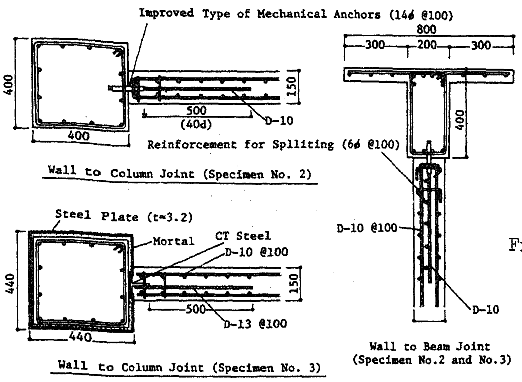

Fig.6 Details of Infilled Wall Specimens

Fig.7 Test Set-up

Table 2 Loading Procedure

No.	No.1,5	No.2,3	No.4	No.	No.1,5	No.2,3	No.4
1	R=1/400	$\bar{\tau}=20kg/cm^2$	$\bar{\tau}=20kg/cm^2$	11	R=1/100 4	R=1/100 3	R=1/100 2
2	R=1/200 1	R=1/400	$\bar{\tau}=30kg/cm^2$	12	" 5	" 4	" 3
3	" 2	R=1/200 1	R=1/400	13	R=1/200	" 5	" 4
4	" 3	" 2	R=1/200 1	14	R=1/50 1	R=1/200	" 5
5	" 4	" 3	" 2	15	" 2	R=1/50	R=1/200
6	" 5	" 4	" 3	16	" 3	R=1/30*	R=1/50
7	R=1/400	" 5	" 4	17	" 4		
8	R=1/100 1	R=1/400	" 5	18	" 5		
9	" 2	R=1/100 1	R=1/400	19	R=1/25*		
10	" 3	" 2	R=1/100 1				

* Only Positive Loading

$R = \delta/h$, δ: Story Deflection
h: Story height

Table 3 Test Results

	No.1	No.2	No.3	No.4	No.5
Initial Stiffness (t/min)	6.4	122	127	195	12.1
Flexure Cracking of Column (t)	14	72	90	99	18
Flexure Cracking of Beam (t)	9	72	108	-	31
Shear Cracking of Wall (t)	-	98	90	108	-
Slip of Wall-to-Beam (t)	-	63	45	-	-
Slip of Wall-to-Column (t)	-	72	45	-	-
Slip of Wall-to-Base (t)	-	117	99	189	-
Yield of Column Reinforcement (t)	29.1	160	168	189	35.4
Maximum Strength (t)	29.1	160	168	222	36.1

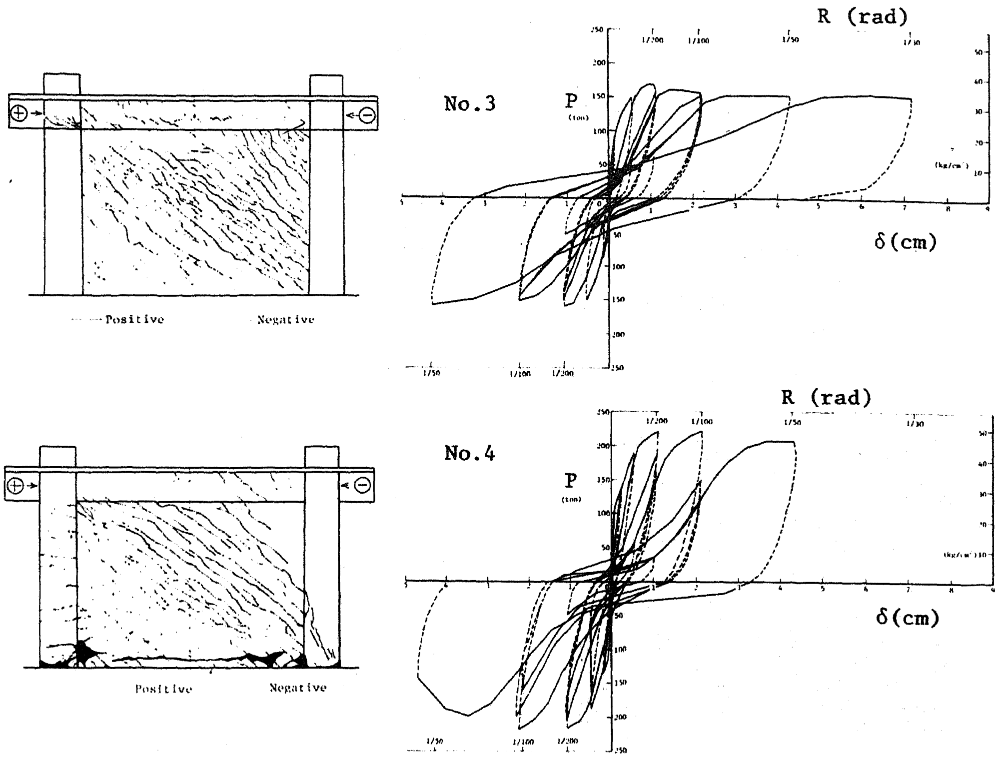

Fig.8 Comparison of Cracking Patterns and Restoring Characteristic between Infilled and Monolithic Wall

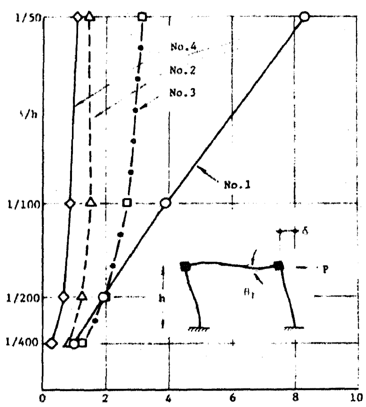

Fig.10 Rotational Behavior of Beam-to-Column Joints

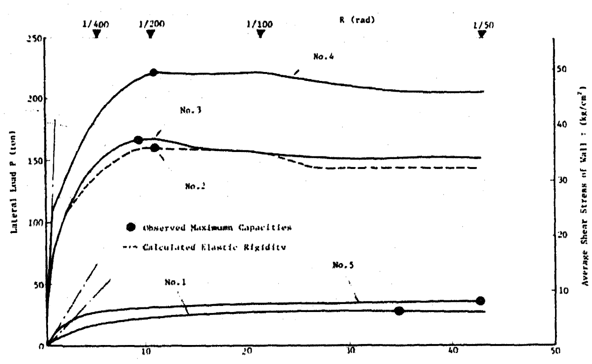

Fig.9 Envelopes of Load-Deflection Curves