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RETROFITTING METHOD OF EXISTING REINFORCED CONCRETE BUILDINGS USING ELASTO-PLASTIC STEEL DAMPERS

Akihiro KUNISUE[1], Norihide KOSHIKA[2], Yasushi KUROKAWA[3], Norio SUZUKI[4], Jun AGAMI[5] And Mitsuo SAKAMOTO[6]

SUMMARY

The authors have been researching and developing methods of retrofitting existing reinforced concrete buildings with elasto-plastic steel dampers. In the proposed seismic retrofitting method, dampers are installed in an existing building to increase its structural strength and at the same time to reduce its seismic response by absorbing energy. This paper reports on a structural test conducted to investigate the structural characteristics of damper-embedded frames. The test results indicate that the proposed method of retrofitting an existing building increases both its strength and its energy absorption capacity. The paper also introduces an example of an application of the proposed seismic response control retrofitting method and demonstrates the effectiveness of the retrofit through earthquake response analysis.

INTRODUCTION

Since the Hyogoken Nanbu Earthquake of 1995, vigorous efforts have been made in the area of seismic diagnosis and retrofitting of existing buildings in Japan. Conventional seismic retrofitting approaches such as adding shear walls or steel-framed bracing have many problems, such as reduction in room serviceability caused by the closing of openings and loss of building use during retrofitting work.

The authors have been researching and developing a retrofitting method using elasto-plastic dampers made of low yield steel [Suzuki, et. al, 1998]. This seismic retrofitting method increases building strength through the dampers' strength and reduces structural response through the dampers' energy-absorbing capacity. This retrofitting method, which requires fewer reinforcing elements than conventional retrofitting methods, minimizes inconvenience to occupants while maintaining a high level of seismic safety.

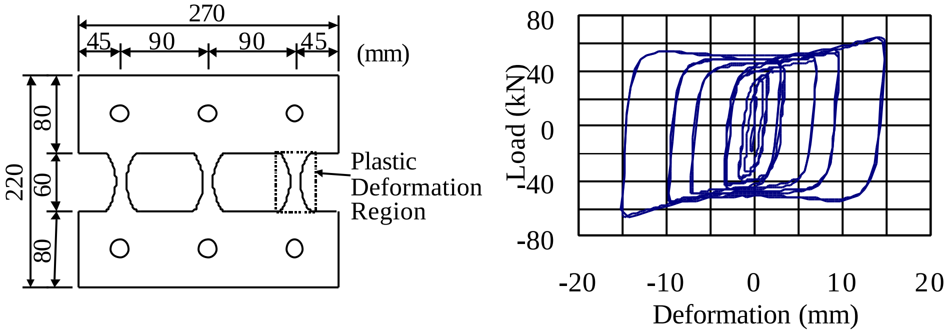
The present paper outlines the newly developed retrofitting method and reports the results of experiments carried out on a 1/2-scale one-story one-span model picked up from an existing building, with the aim of confirming the structural performance of damper-embedded frames. It also reports an application of the retrofitting method to an actual building.

OUTLINE OF RETROFITTING METHOD USING ELASTO-PLASTIC DAMPERS

The proposed retrofitting method achieves a high level of structural safety through dampers installed in an existing seismically vulnerable building more efficiently than conventional retrofitting methods. There are many ways of installing dampers in an existing building, including (1) installing steel-framed braces that incorporate dampers into an existing open frame and (2) installing damper-embedded studs into existing frame so that existing openings are maintained. The latter method can be used in cases where the building is to be strengthened internally.

[1] *Kobori Research Complex, Kajima Corporation, Tokyo, Japan Email : kunisue@krc.kajima.co.jp*
[2] *Kobori Research Complex, Kajima Corporation, Tokyo, Japan*
[3] *Kobori Research Complex, Kajima Corporation, Tokyo, Japan*
[4] *Kajima Technical Research Institute, Kajima Corporation, Tokyo, Japan*
[5] *Kajima Technical Research Institute, Kajima Corporation, Tokyo, Japan*
[6] *Kobori Research Complex Inc. , Tokyo, Japan*

Dampers used in this retrofitting method are elasto-plastic steel dampers that have honeycomb openings. The damper used in the experimental study reported in this paper is shown in Figure 1(a). The damper is designed to concentrate story drifts in its plastic deformation region and efficiently absorb earthquake energy by cyclic plastic deformations. The load-deformation relationship, as shown in Figure 1(b), plot as spindle-shaped hysteresis loops and indicate a high energy absorbing capacity.



(a) Dimension **(b) Load-deformation relationship**

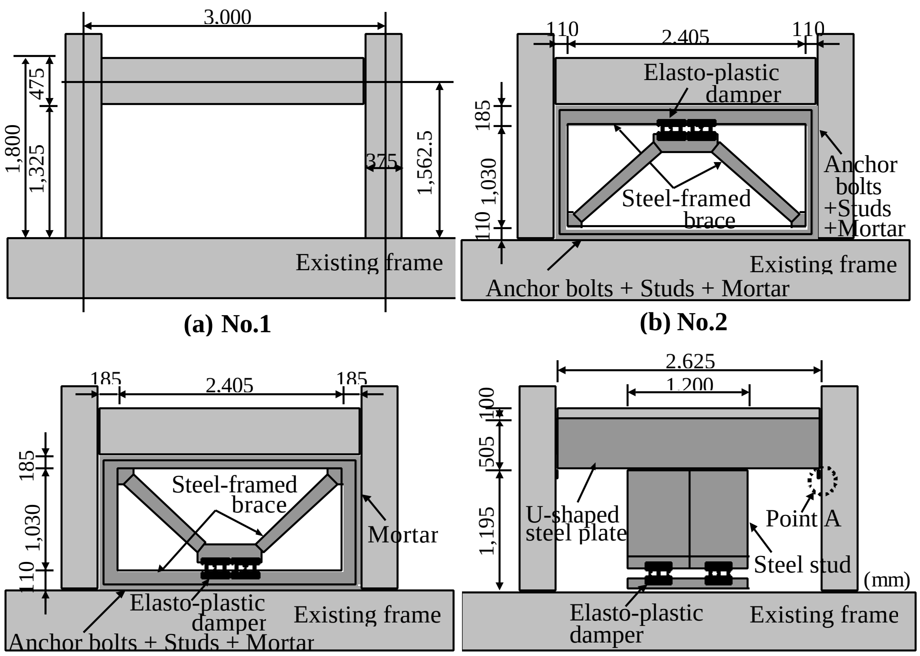
Fig.1: Elasto-plastic damper

3. EXPERIMENTAL STUDY

The objective of the structural experiment was to investigate the structural characteristics of damper-embedded frames and, by comparing them with existing frames, to evaluate the effectiveness of the proposed seismic retrofitting method.

3.1 Test Specimens

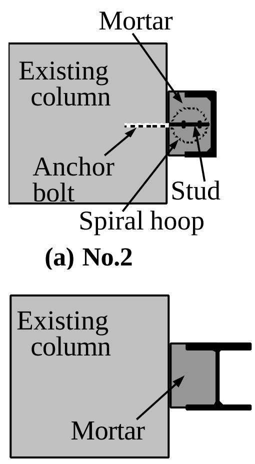
The test specimens, along with their dimensions, are shown in Figure 2. Table 1 shows the cross sections of the members, Table 2 shows the compressive strength of the concrete and mortar used in each specimen, and Table 3 shows the mechanical properties of the reinforcing bars and steel. The existing frame portion of each specimen is a 1/2-scale model of the first-floor frame extracted from the six-story existing building constructed in the 1960s. The specimens were strengthened with four sets of dampers shown in Figure 1(a). The strength of the dampers is roughly one-half that of the frame. Each test specimen is described below.



(a) No.1 **(b) No.2**

(c) No.3 **(d) No.4**

Fig.2: Test specimens



(a) No.2

(b) No.3

Fig.3: Detail of connection

Specimen No. 1: Existing open frame
This specimen was used to investigate the basic characteristics of the originally designed frame. This frame was designed to form plastic hinges at its beam ends and column bottom ends.

Specimen No. 2: Damper-embedded frame system
This specimen had an additional steel-framed brace, as well as the existing frame, with elasto-plastic dampers embedded between the frame and the brace top. The existing frame-steel frame connections were provided with anchors, studs, and spiral hoops and were filled with mortar to achieve structural integrity (see Figure 3(a)).

Specimen No. 3: Damper-embedded frame system with simplified connections
This specimen was basically the same as Test Specimen No. 2. However, it had simpler exiting frame-steel frame connections, which complicated the construction work. As shown in Figure 3(b), the spiral hoops along the four sides were omitted by making the steel frame flanges function as constraints. The anchors and studs near the columns were also omitted as a rational connection method commensurate with damper strength.

Specimen No. 4: Damper-embedded studded frame system
This specimen had a damper-embedded steel stud installed in the middle of the plane frame. It was designed to directly transfer additional stresses from the damper to the columns through the U-shaped steel plate bonded to the beam surface.

Table 1: Member sections

Member	Column	Beam	
		End	Mid.
Section	375 × 375 (mm)	475 × 225	475 × 225
Bar arrangement	20-13ϕ	Top : 6-D13	Top : 3-D13
	Hoop :End 4ϕ@50	Bot. : 4-D13	Bot. : 3-D13
	:Mid 4ϕ@75	St : 4ϕ@75	St : 4ϕ@75

Table 2: Concrete strength

Concrete	Frame				Mortar		
	No.1	No.2	No.3	No.4	No.2	No.3	No.4
Compressive(MPa)	20.7	21.6	20.8	21.7	52.7	59.6	76.4
Split Tensile(MPa)	2.33	2.26	1.93	1.86	---	---	---

Table 3: Material properties

Reinf. Bar & Steel	Size		Yield Strength (MPa)	Tensile Strength (MPa)
Column Long. Reinf.	13ϕ	No.1,2	333	458
		No.3,4	336	473
Beam Long. Reinf.	D13	No.1,2	343	494
		No.3,4	354	513
Hoop & Stirrup	4ϕ	No.1,2	209	317
		No.3,4	222	320
Steel Frame	H-150 x 150 x 7 x 10		392	---
	C-150 x 75 x 9 x 12.5		392	---
Brace	H-150 x 150 x 7 x 10		392	---
Damper	PL-9	No.2	305	432
		No.3,4	314	447

3.2 Test Method

The method of applying loads to the test specimen is shown in Figure 4. In the test, a static 2 MN actuator and a tension rod were used to apply cyclic horizontal loads to the uppermost part of the test specimen from one direction. While the test specimen was loaded horizontally, a force of 461 kN was applied to each column so that they were equal to the permanent axial loads acting on the building.

The measurement items were horizontal load, story deflection, damper deformation, and axial strains in the main reinforcement at the column and beam ends and in the steel frames.

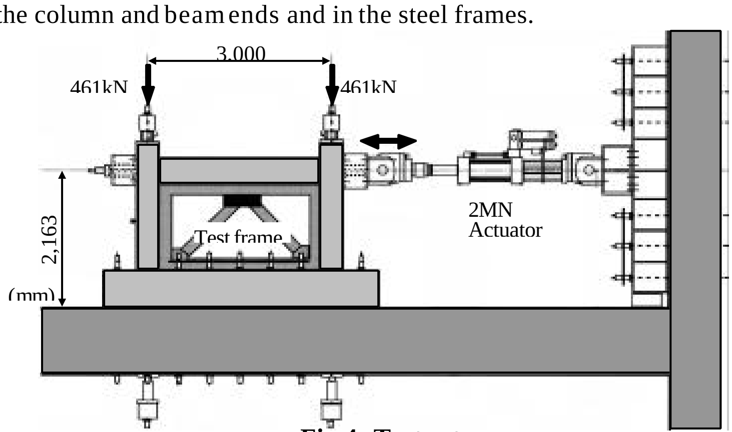


Fig.4: Test setup

3.3 Test Results

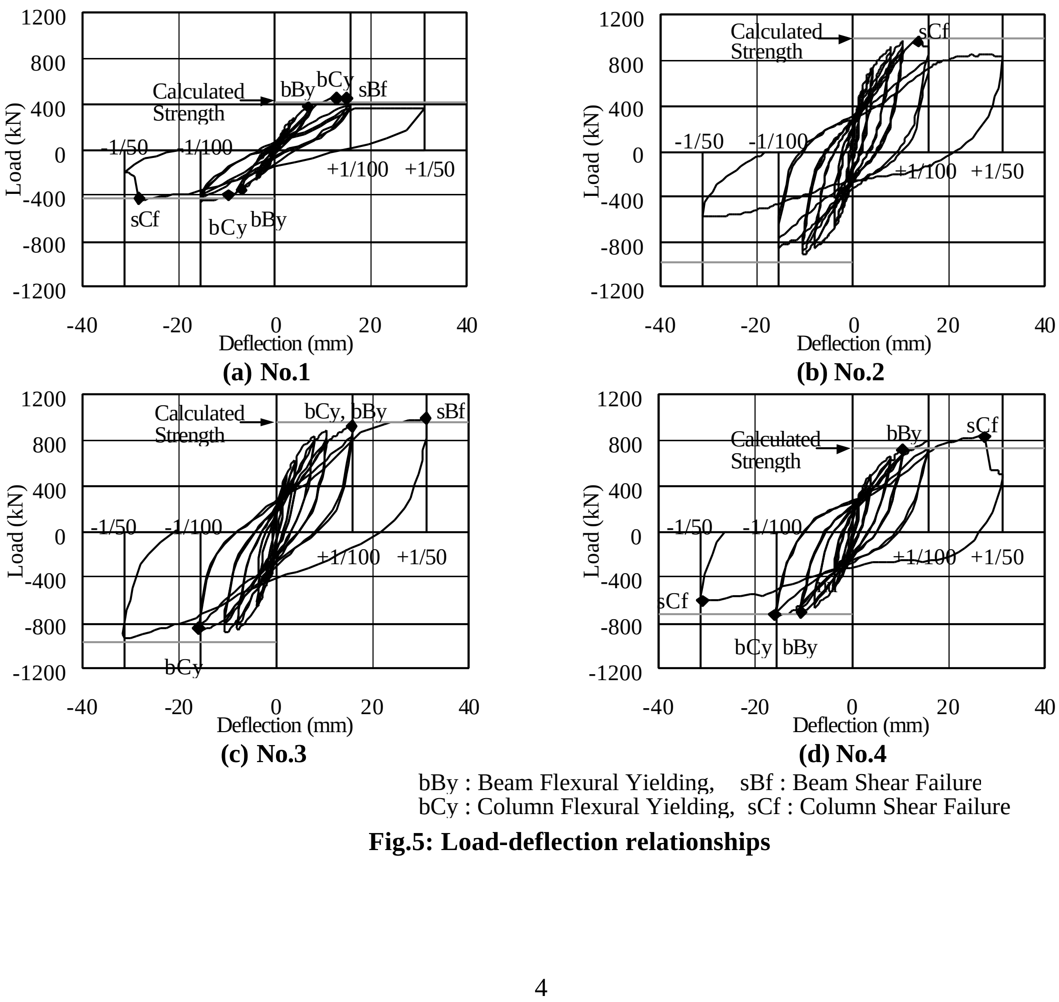
Table 4 summarizes the maximum load and the failure mode for each test specimen. Figure 5 shows the horizontal load-deflection relationships; Figure 6, cracks at a story drift angle of 1/100; and Figure 7, equivalent viscous damping factors for typical drift angles. The results obtained for each test specimen are described below.

No. 1: Flexural yielding occurred first at the beam ends and column bottom ends, and shear failure occurred at a column bottom end when the story drift angle was about 1/50. This is because the shear reinforcement was inadequate and the bending ultimate strength was too close to the shear ultimate strength. The hysteresis loop shows a typical degrading tri-linear type characteristic of a reinforced concrete building frame.

No. 2: Shear failure occurred before flexural yielding at the column bottom ends when the story drift angle was 1/100, but the strength did not decrease substantially after the shear failure. The maximum strength showed values greater than the sums of the strengths of the dampers and Test Specimen No. 1. This is thought to be due to increases in the maximum strength of the members under the influence of the steel frame. The hysteresis loops are spindle-shaped and indicate a large energy absorbing capacity. The equivalent viscous damping factor was about two times that of No. 1.

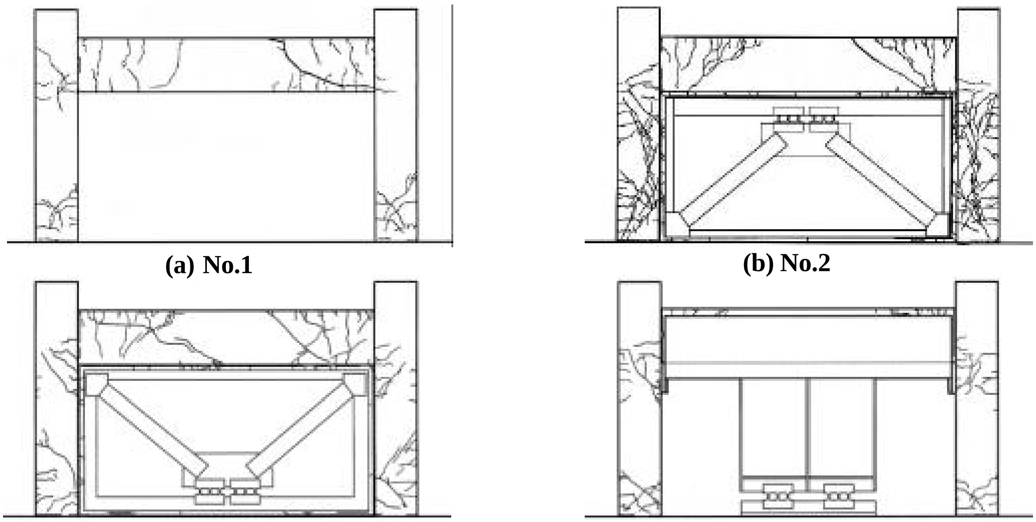
No. 3: Flexural yielding occurred first at the beam ends and column bottom ends, and shear failure of the beam occurred when the story drift angle was 1/50. The hysteresis loops are spindle-shaped and indicate a large energy absorbing capacity. This specimen showed values of maximum strength and equivalent viscous damping factor comparable to those for No. 2. Furthermore, the rate of deterioration after the maximum strength was lower than for No. 2. This indicates that the simplification of connections did not have any noticeable adverse effect.

No. 4: Flexural yielding occurred first at the beam ends and column bottom ends, and, when the story drift angle was a little smaller than 1/50, shear failure of the columns began at the top where beam steel plate fixed to the column (see Figure 2(d), Point A). The maximum strength showed values greater than the sums of the strengths of the dampers and Test Specimen No. 1. Although strength fell sharply as a result of the column shear failures, the hysteresis loops show a typical spindle-shaped mode at story drift angles of 1/100 or less. The equivalent viscous damping factor took values similar to those indicated for the frame system, indicating satisfactory structural characteristics.



bBy : Beam Flexural Yielding, sBf : Beam Shear Failure
bCy : Column Flexural Yielding, sCf : Column Shear Failure

Fig.5: Load-deflection relationships



(a) No.1 (b) No.2 (c) No.3 (d) No.4

Fig.6: Crack distributions (Story drift angle: 1/100 rad)

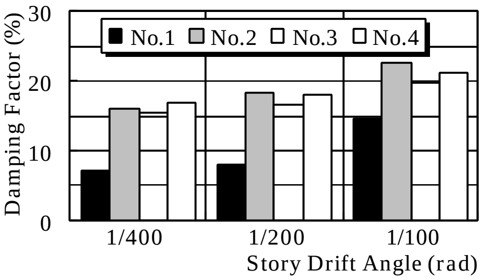


Fig.7: Equivalent viscous damping factors

Table 4: Maximum load and failure mode

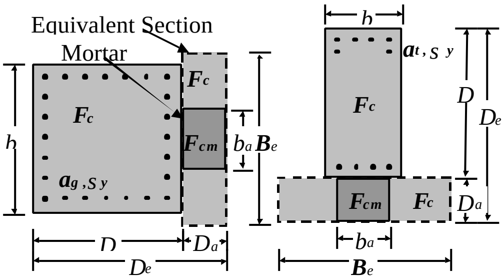
Specimen	Max. Strength (kN)	Failure Mode
No.1	453	Shear failure at column bottom end after flexural yielding of beam and columns end
No.2	973	Shear failure at column bottom end
No.3	987	Shear failure at beam end after flexural yielding of beam and columns end
No.4	1010	Shear failure at column bottom end after flexural yielding of beam and columns end

3.4 Evaluation of Strength of Test Specimens

This section discusses the strength of each test specimen at flexural yielding. Simplified formulas for the ultimate bending strength of the members [AIJ, 1990] are shown in Table 5. The symbols used in the formulas are shown in Figure 8. The formulas used for mortar-filled members (columns and beams of No. 2 and No. 3, beam of No. 4) are as follows. Where the mortar is in compression, Equation 1 is used, assuming an equivalent cross section of the existing member-mortar composite. Where the mortar is in tension, Equation 2 or Equation 3 is used, assuming that only the cross section of the existing member is effective. The loads carried by the columns are calculated from the equilibrium of forces in the mechanism of bending failure due to flexural yielding at column bottom ends and beam ends, by using the M-N relationship calculated from both the simplified formula and a cross section analysis using a fiber model. The loads carried by the dampers in Test Specimens No. 2, No. 3, and No. 4 are calculated from the results of a test in which the damper displacements obtained by experiment are applied to the

Table 5: Strength formulas

Unified mortar	Column, Beam	$M_u = \alpha\, a_g \sigma_y g_1 D_e + 0.5 N D_e \left(1 - \dfrac{N}{B_e D_e F_c}\right)$	Eq. 1
Existing member	Column	$M_u = 0.5\, a_g \sigma_y g_1 D + 0.5 N D \left(1 - \dfrac{N}{b D F_c}\right)$	Eq. 2
	Beam	$M_u = 0.8\, a_t \sigma_y D + 0.5 N D \left(1 - \dfrac{N}{b D F_c}\right)$	Eq. 3



$B_e = b_a F_{cm} / F_c$

g_1 : Reinforcement interval divided by depth of member

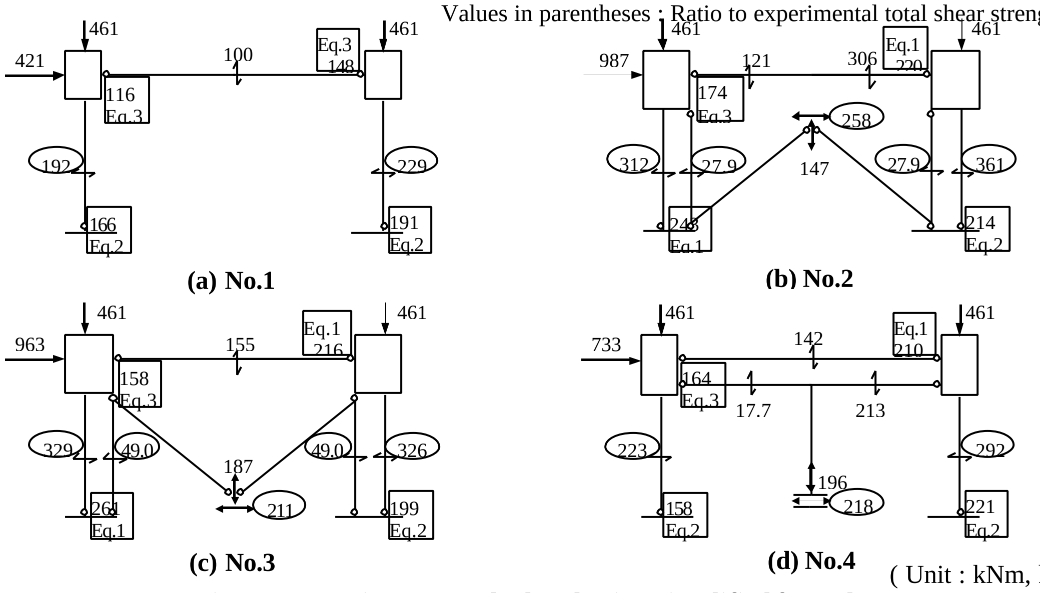
Fig.8: Symbols used in formulas

dampers separately. The loads carried by the steel frames in Test Specimens No. 2 and No. 3 are calculated from axial strains in the steel frame. The strength of each test specimen at flexural yielding is calculated as follows: (1) for No. 1, by summing the loads carried by the right and left columns, (2) for No. 2 and No. 3, by summing the loads carried by the right and left columns and the loads carried by the dampers and the steel frame, and (3) for No. 4, by summing the loads carried by the right and left columns and the loads carried by the dampers. The calculation results are shown in Table 6 and Figure 9. The coefficient α in Equation 1 expresses the influence of shifting the neutral axis due to mortar filling. Calculations assuming α=0.75 gave values close to those obtained through the cross section analysis using the fiber model. This indicates that the strength of all test specimens can be evaluated by the calculation method described above.

Table 6: Comparison between experimental strength and calculated strength

Specimen		No.1	No.2	No.3	No.4
Story drift angle at flexural yielding (rad)		1/114	1/112	1/100	1/100
Experimental (kN)	Total Shear	453 (1.00)	967 (1.00)	925 (1.00)	787 (1.00)
	Damper	---	258	211	218
	Steel frame	---	55.8	98.0	---
Calculated (kN)	Simplified formulas	421 (0.93)	987 (1.02)	963 (1.04)	733 (0.93)
	Analysis using fiber model	426 (0.94)	989 (1.02)	968 (1.05)	724 (0.92)

Values in parentheses : Ratio to experimental total shear strength



(Unit : kNm, kN)

Fig.9: Stress Diagram (Calculated using simplified formulas)

4. APPLICATION EXAMPLE

From the seismic retrofit techniques considered above, the method used for Test Specimen No. 2 was applied to an actual building (4-story local government office building completed in 1962)[Kunisue, et. al, 1998]. The following sections introduce this example.

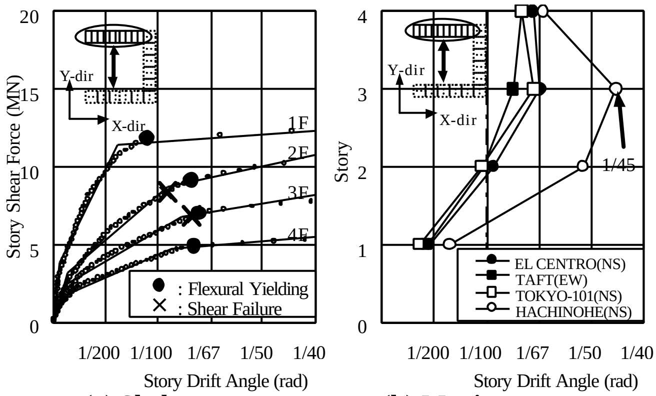
4.1 Overview of Building

The building consists of three 16.5 m x 58.5 m rectangular blocks and one 16.5 m x 16.5 m square block arranged in a "U" shape. The blocks are separated by expansion joints and are structurally independent of each other. The building is a reinforced-concrete rigid-frame structure whose columns are mostly arranged on a 6 m x 12 m grid.

As a first step, the earthquake resistance of the building was evaluated on the basis of a dynamic analysis. Four earthquake ground motions (El Centro 1940 NS, Taft 1952 EW, Hachinohe 1968 NS, Tokyo-101 1956 NS) normalized to the maximum acceleration of 400 cm/s^2 were adopted as input motions. The analysis revealed that shear failure of second- and third-story columns might occur before flexural yielding of beam ends, resulting in a fall of the upper stories because of inadequate strength and ductility (see Figure 10).

Table 7: Input earthquake motion

Earthquake	Time (sec)	Maximum Acceleration (cm/s^2)
EL CENTRO 1940 NS	30	400
TAFT 1952 EW	30	400
HACHINOHE 1968 NS	30	400
TOKYO-101 1956 NS	11	400



(a) Skeleton curve **(b) Maximum response**

Fig.10: Seismic responses (Existing building)

4.2 Seismic Retrofit Design

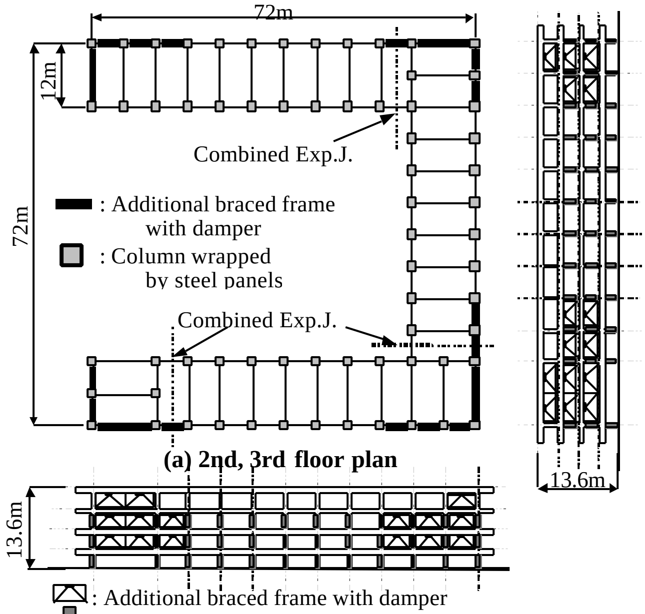
To improve the horizontal strength and ductility, which had proven to be inadequate, the following retrofit plan was drawn up:

(1) The ductility of all second- and third-floor columns will be increased by steel-plate wrapping. The first-floor columns have already been so treated.
(2) To achieve a well-balanced arrangement of seismic response control framing and a high level of constructibility by concentrating them at the four corners of the outer frame on the balcony side, the expansion joints are turned into rigid connections so as to attain structural integrity.
(3) A damper-embedded steel frame will be installed on the stories that are not strong enough.
(4) The foundation and piling under the seismic retrofit zones will be strengthened to prevent floating of the foundation due to overturning moment during earthquakes and resist compressive forces.

Figure 11 shows the arrangement of the damper-embedded frames.

4.3 Analytical Model

The building's expansion joints had been replaced with rigid connections and had been strengthened by casting damper-embedded steel frames and wrapping columns with steel plates. It was modeled at the member level, and a static incremental load elasto-plasticity analysis was carried out. On the basis of the analytical results, a three-



(a) 2nd, 3rd floor plan

(b) Framing elevation

Fig.11: Locations of reinforcing frame

dimensional vibration analysis model was constructed by replacing each frame with an equivalent shear element and a damper-embedded retrofit frame with an equivalent spring element. Degrading tri-linear model hysteresis characteristics were adopted for the equivalent shear elements, and no-degrading tri-linear model hysteresis characteristics were adopted for the equivalent spring elements. An internal viscous damping of 3%, which is proportional to the initial stiffness of the existing framing, was also given.

4.4 Analytical Results

Figure 12 shows the seismic responses in terms of maximum story drift angle, Figure 13 shows the energy distribution at each level where the El Centro 1940 NS motion was inputted, and Figure 14 illustrates the plasticization of the members at maximum response. These results indicate the following: (1) the story drift angle is smaller than about 1/100, (2) although the columns and beams have plasticized, they have not yet formed a collapse mechanism as a frame, and (3) before the retrofit, building damage is concentrated on the third floor, while after the retrofit, frame damage is reduced to a nearly constant, very small level as the dampers absorb about 60% of the input earthquake energy.

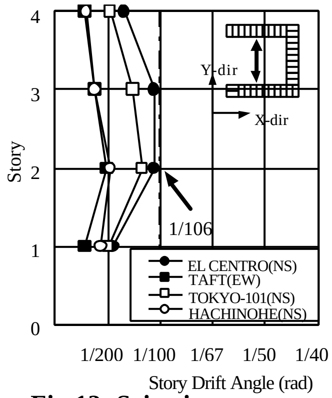
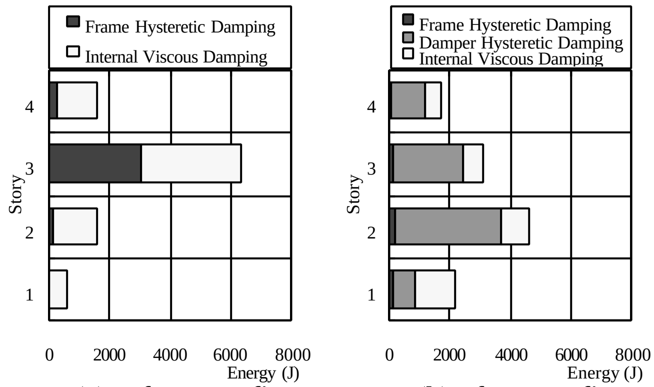


Fig.12: Seismic responses (After retrofit)



(a) Before retrofit **(b) After retrofit**

Fig.13: Distribution of absorbed energy

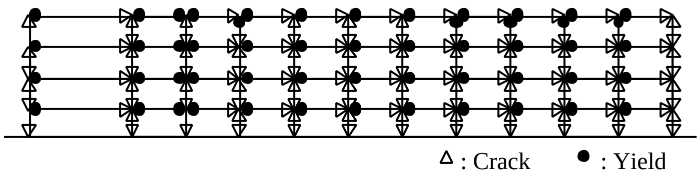


Fig.14: Plasticization of members at maximum response

5. CONCLUSION

This paper has reported on a structural experiment conducted to verify the effectiveness of a seismic retrofitting method for reinforced concrete buildings that uses elasto-plastic steel dampers. The experimental results indicate that the strength and energy absorption capacity of an existing building can be increased by retrofitting the building using this method. Procedures for evaluating the strength of retrofitting frames were presented, and good agreement between the calculated values and the test results was confirmed. Finally, an example of a building retrofitted by the method has been introduced. It was confirmed that retrofitting method is effective to reduce the seismic response and increase the earthquake resistance of the building with a relatively small number of retrofitting frames.

6. REFERENCES

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