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SHAKING TABLE TEST ON EARTHQUAKE RESPONSE BEHAVIOR OF 2-STORY STEEL FRAMES WITH YIELDING JOINT PANEL ZONE

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SUMMARY

This paper describes the results of shaking table test on earthquake response behavior of 2-story steel moment resisting frames, which consist of H-shaped beams and H-shaped columns with yielding panel zones. A major objective of this study is to investigate the effect of energy dissipation in panel zones on inelastic response behavior and total energy dissipation capacity of the frames. In order to achieve the objective, four specimens (H-1~H-4) were prepared for this test. These specimens were designed with three different level of panel zone strength. Specimen H-1 (none doubler plate) was designed so that all yielding would occur in the panel zones. Specimen H-2 (one 2.6mm doubler plate) was designed to promote yielding in both the panel zones and the beams. Specimen H-3 (two 6mm doubler plates) was designed so that all yielding would occur within the beams. Specimen H-4 (two 6mm doubler plates) was designed to have the reduced beam section (RBS) connection proposed in the FEMA 350. The input earthquake ground motion used in the shaking table test is a PGA of 0.8g of the JMA Kobe. The following results were obtained form the shaking table test. 1) Specimen H-2 had the largest energy dissipation capacity among four specimens. 2) Specimen H-4 had the largest beam rotation capacity among four specimens, but was not largest energy dissipation capacity.

INTRODUCTION

In seismic performance evaluation of steel moment-resisting frames, it would be necessary to grasp the capacity and demand in terms of energy dissipation and maximum inter-story drift of the frames against earthquake. In moment-resisting steel frames consist of H-shaped beam and column, panel zone strength would have an influence on the contribution of beam and column energy dissipation to the total energy dissipation of the frames, and the lateral load resistance of the frames. A major objective of this study is to investigate the effect of energy dissipation in panel zones on inelastic response behavior and total energy dissipation of the frames. In order to achieve the objective, shaking table test was conducted using 2-story steel moment-resisting frame specimens.

Four specimens (H-1~H-4) were prepared for this shaking table test. These specimens were designed with three different level of panel zone strength. Specimen H-1 was designed so that all yielding would occur

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in the panel zones. Specimen H-2 was designed to promote yielding in both the panel zones and the beams. Specimen H-3 was designed so that all yielding would occur within the beams. Specimen H-4 was designed to have the reduced beam section (RBS) connection proposed in the FEMA 350 [1]. The input earthquake ground motion used in the shaking table test is JMA Kobe. This paper describes the results of the shaking table test on earthquake response behavior of 2-story steel moment-resisting frames, which consist of H-shaped beams and columns with yielding panel zones.

METHOD OF SHAKING TABLE TEST

TEST SETUP AND SPECIMENS

The shaking table test setup is illustrated in Fig.1. The specimens are 2-story steel moment-resisting space frames consist of H-shaped beams and H-shaped columns. The weight is connected to the beams with pins at the center of beams in each story, as shown in the Fig.1. Inertia force of each story of the specimens is applied at the pin supports. The column bases of the specimens are supported by pins. The frames for collapse prevention and measurement are provided beside the specimen, as shown in Fig.1. By using of this test system, earthquake response behavior of multi-story steel frames with yielding panel zones under sever earthquake can be observed. Fig.2 illustrates a typical 2-story plane frame specimen and strain gauge position in this test. The specimen is 2-story space frame, which consist of two 2-story plane frames combined with transverse beam.

Table 1 shows a list of the four specimens (H-1~H-4) in this shaking table test. The column and beam member size was identical for four specimens. These specimens were designed so that the damage to beam-column connections would concentrate more in the 2nd floor than in the R-th floor. The specimens were designed with three different level of panel zone strength in the 2nd floor, as indicated in Table 1.

Fig.3 illustrates a typical connection detail at the 2nd floor of the specimens. Specimen H-2 through H-4 were provided with double plates, which were welded to column flanges using a full penetration welds. Specimen H-1 (none doubler plate) was designed so that all yielding would occur in the panel zones.

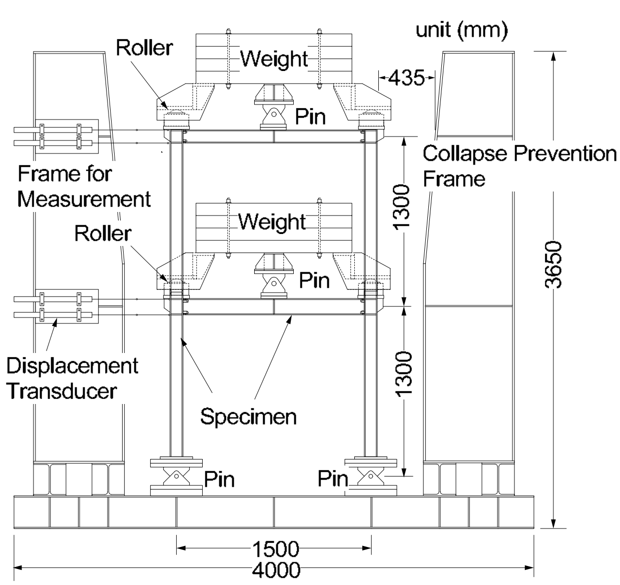

Fig.1 Test setup

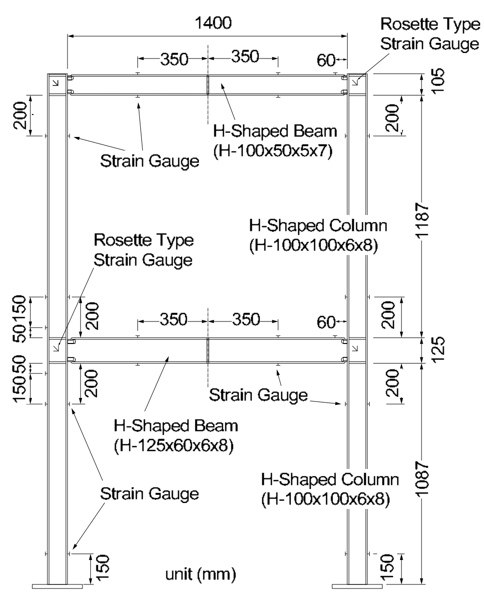

Fig.2 2-story test specimen

Table 1 Test specimens

Specimen	Column		Beam		Panel Zone Strength		Column/Beam Strength Ratio $_{cb}R_p^*$		Panel/Beam Strength Ratio $_{pb}R_p^*$		Weight		Calculated Base Shear Coefficient
	1-story	2-story	2-Floor	R-Floor	2-Floor	R-Floor	2-Floor	R-Floor	2-Floor	R-Floor	2-Floor	R-Floor	
H-1	H-100x100x6x8		H-125x60x6x8	H-100x50x5x7	Weak panel zone strength (t = 6mm: none doubler plate)	Weak panel zone strength (t = 6mm: none doubler plate)	2.69	2.04	0.61	0.74	6ton	6ton	0.43
H-2	〃		〃	〃	Balanced panel zone strength (t=8.6mm: one 2.6mm doubler plate)	Weak panel zone strength (t = 6mm: none doubler plate)	2.69	2.04	0.88	0.74	〃	〃	0.53
H-3	〃		〃	〃	Strong panel zone strength (t=18mm: two 6mm doubler plates)	Strong panel zone strength (t=18mm: two 6mm doubler plates)	2.69	2.04	1.84	2.22	〃	〃	0.64
H-4	〃		〃 (Reduced Beam Section)	〃 (Reduced Beam Section)	Strong panel zone strength (t=18mm: two 6mm doubler plates)	Strong panel zone strength (t=18mm: two 6mm doubler plates)	3.97	3.05	2.71	3.31	〃	〃	0.43

Table 2 Tension coupon data

Section	Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)	Yield Ratio (%)
H-125x60x6x8 (SS400)	330	460	71.7
H-100x50x5x7 (SS400)	379	478	79.3
H-100x100x6x8 (SS400)	383	495	77.4

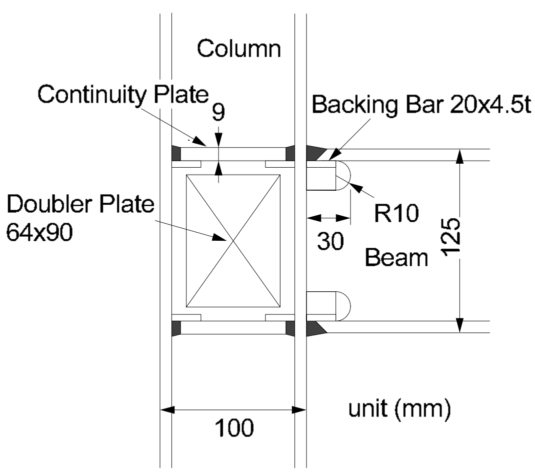

Fig.3 Typical connection detail

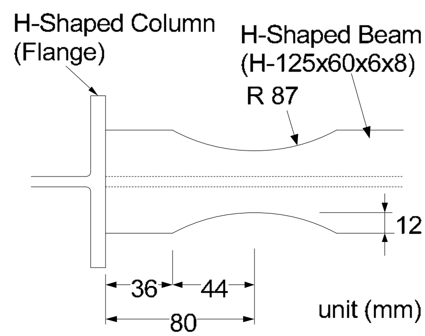

Fig.4 Reduced beam section connection detail

Specimen H-2 (one 2.6mm doubler plate) was designed to promote yielding in both the panel zones and the beams. Specimen H-3 (two 6mm doubler plates) was designed so that all yielding would occur within the beams. Specimen H-4 (two 6mm doubler plates) was designed to have the reduced beam section (RBS) connection as shown in Fig.4. The depth of the RBS cut was chosen to provide a 67 percent flange reduction at the minimum section of the RBS. The choice of the RBS dimension was based on the FEMA 350 [1].

The beam and panel zone energy dissipation would strongly depend on the panel/beam strength ratio (${}_{pb}R_p^*$) and the base share coefficient at the failure mechanism of the specimens. These values of each specimen are listed in Table 1. The column/beam strength ratio (${}_{cb}R_p^*$), the panel/beam strength ratio (${}_{pb}R_p^*$) and the base share coefficient of each specimen in Table 1 are calculated by using of tension coupon data listed in Table 2. The column/beam strength ratio (${}_{cb}R_p^*$) and the panel/beam strength ratio (${}_{pb}R_p^*$) are defined as follows.

$$ {}_{cb}R_p^* = \Sigma_c M_p^* \;/\; \Sigma_b M_p^* \tag{1}$$

$$ {}_{pb}R_p^* = {}_{p}M_p^* \;/\; \Sigma_b M_p^* \tag{2}$$

where,

$$ \Sigma_c M_p^* = \frac{1}{1-\dfrac{d_b}{H_U}}\,{}_{c}M_{pU} + \frac{1}{1-\dfrac{d_b}{H_L}}\,{}_{c}M_{pL} \tag{3}$$

$$ \Sigma_b M_p^* = \frac{1}{1-\dfrac{d_c}{L_L}}\,{}_{b}M_{pL} + \frac{1}{1-\dfrac{d_c}{L_R}}\,{}_{b}M_{pR} \tag{4}$$

$$ {}_{p}M_p^* = \frac{1}{1-\left(\dfrac{d_c}{L} + \dfrac{d_b}{H}\right)}\,{}_{p}M_p \tag{5}$$

Where, ${}_{c}M_{pU}$, ${}_{c}M_{pL}$ are full plastic moment of columns above and below a panel zone respectively, ${}_{b}M_{pL}$, ${}_{b}M_{pR}$ are full plastic moment of beams to the left and right of a panel zone respectively, ${}_{p}M_p$ is full plastic moment of a panel zone, H_U, H_L are story height above and below a panel zone respectively, H is average value of story height above and below a panel zone, L_L, L_R are span lengths of the left and right of a panel zone respectively, L is average span length of the left and right of a panel zone, d_b is depth of a beam, d_c is depth of a column.

INSTRUMENTATION AND TESTING

Instrumentation of the test setup included accelerometers on the shaking table and each floor of the specimens, displacement transducers to measure the inter-story drift and the rotation of the beam at the connection as shown in Fig.1, displacement transducers to measure the panel zone rotation, strain gauges to measure the beam and column flange strains, and strain gauge rosettes to measure the panel zone strains as shown in Fig.2.

The input earthquake ground motion used in the test is the south-north component of earthquake ground motion recorded at the Kobe Marine Observatory (JMA Kobe) during the 1995 Hyogo-ken Nanbu Earthquake. The peak ground acceleration (PGA) of JMA Kobe was scaled to be about 0.8g. The shaking table test used the earthquake motion of JMA Kobe was repeated until either the specimen collapsed or fracture of any member occurred. The collapse of a specimen in the test means that the specimen touches the collapse prevention frames shown in Fig.1.

RESULTS OF SHAKING TABLE TEST

NATURAL PERIOD AND VISCOUS DAMPING

Table 3 shows natural period and viscous damping of four specimens obtained from the free vibration test. The natural period of these specimens ranged from 0.48 to 0.50 seconds, and the natural period of specimen H-3 was slightly shorter than that of the other specimens. It is assumed that a slight difference of the natural periods in these specimens would not affect to total energy input of these specimens. The viscous damping of the specimens was in a range from 0.7% to 1.2%.

TOTAL ENERGY INPUT

Fig.5 illustrates the energy spectrum [2] of the input ground motion (JMA Kobe), which was measured on the shaking table during testing of each specimen. The symbols △,○,□ in the figure are the total energy input (E) to the specimen, which is expressed by an equivalent velocity (V_E) defined as follows.

Table 3 Natural period and viscous damping

Specimen	Natural Period (sec)	V iscous Damping
H-1	0.50	0.8 %
H-2	0.50	1.2 %
H-3	0.48	0.7 %
H-4	0.50	1.1 %

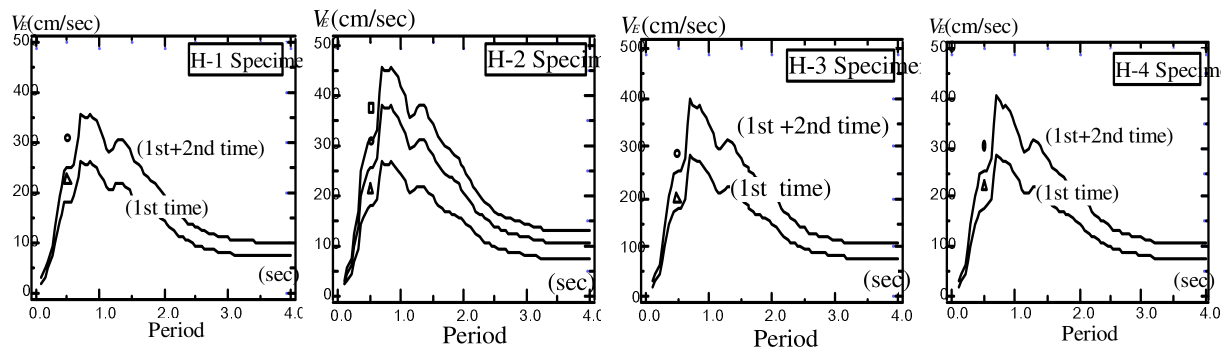

Fig.5 Total input energy spectrum

$$V_E = \sqrt{\frac{2E}{M}} \tag{6}$$

$$E = -\sum_{i=1}^{2} m_i \int_0^{t_0} \ddot{Z}_0 \dot{y}_i d_t \tag{7}$$

Where, M is total mass of the specimen, m_i is the mass of story i, t_0 is the duration of the input earthquake ground motion, $\ddot{Z}_0$ is the acceleration of the input earthquake ground motion measured on the shaking table, y_i is story drift of story i.

In this shaking table test, the shaking table test using the JMA Kobe was repeated until either the specimen collapsed or fracture of any member of the specimen occurred. The bracket in Fig.5 indicates the frequency of the shaking table test for each specimen. The frequency of the shaking table test was twice for specimens H-1, H-3 and H-4, and three times for specimen H-2. This figure shows that the energy spectrum of the input ground motion corresponds closely to the equivalent velocity of the total energy input to the specimen. The equivalent velocity of the total energy input of specimen H-2 was approximately 1.5 times that of the other three specimens. The energy spectrum shapes for each specimen are almost identical, so it is assumed that the specimens were all tested by almost identical input earthquake ground motion.

INELASTIC RESPONSE BEHAVIOR OF THE SPECIMENS

Specimen H-1 was designed so that virtually all yielding occurred within the panel zones. In the first time shaking test of the specimen, shear deformation in the panel zones occurred without the development of instability. During the 2nd time shaking test of the specimen, the beam fracture occurred and resulted in collusion of the specimen with the collapse prevention frame. Specimen H-2 was designed to promote yielding in both panel zones and beams. In the 2nd time shaking table test of the specimen, the shear deformation in the panel zones and the buckling of beam flange occurred without the development of instability. During the 3rd time shaking test, the fracture of beam occurred and resulted in collusion of the specimen with the collapse prevention frame. Specimen H-3 was designed so that virtually all yielding would occur within the beams. In the first time shaking test of the specimen, the yielding of beam occurred, but no local buckling was observed. During the 2nd time shaking test of the specimen, the fracture of the beam flange and crack of the beam web occurred. Specimen H-4 was designed to have the reduced beam section (RBS) connection with strong panel zones. During the 2nd time shaking test of the specimen, the collusion of the specimen with the collapse prevention frame occurred. Large local buckling of the beam web occurred, but the fracture of beam was not observed.

Fig.6 shows plots of story shear force coefficient versus inter-story drift angle of 1st story of each specimen obtained from the shaking table test. The story shear force coefficient (Q_1/W_T) is a value obtained by dividing the story shear force of 1st story (Q_1) by total mass of the specimen (W_T). Fig.7 shows plots of story shear force coefficient versus inter-story drift angle of 2nd story of each specimen. The story shear force coefficient (Q_2/W_2) is a value obtained by dividing the story shear force of 2nd story (Q_2) by mass of 2nd story of the specimen (W_2). These story shear forces (Q_1, Q_2) are obtained by deducting the strength reduction caused by the P-delta effect from the shear force of column calculated based on the values obtained by the strain gauges attached to the elastic part of the columns. Comparing the story shear coefficient vs. inter-story drift angle relationships of specimen H-1 to H-3 in Fig.6 shows that the maximum inter-story drift angle of H-1 specimen during the first time shaking test was approximately 0.1radian, which was the largest of the three specimens. The maximum inter-story drift angle of specimens H-2 and H-3 during the first time shaking test was approximately 0.08 and 0.07radian respectively. The thickness of the doubler plate in panel zones caused differences in the maximum inter-story drift angle of the specimens. It would be necessary to account the strength of pane zones by the doubler plates in order to appropriately evaluate the maximum inter-story drift of the frames. The maximum inter-story drift angle of specimen H-4 having the RBS connections during the first time shaking test was approximately 0.11radian, which was the largest of all the specimens. In the 2nd story shown in Fig.7, the maximum inter-story drift angle was smaller than that in the 1st story.

Fig.8 shows plots of panel moment vs. shear deformation angle of panel zone in the 2nd floor of each specimen obtained from the shaking table test. The panel moment is obtained by subtracting the moment caused by the column shear forces from the beam moment at the column face. The beam moment and column shear force are calculated from the strain gauges attached to the beams and columns as shown in Fig.2. The panel shear deformation is measured by the displacement transducers in the panel zone. In specimen H-1, large plastic deformation in the panel zone occurred, and the maximum deformation angle of the panel zone during the 1st time shaking test approximately reached to 0.06radian, as shown in Fig.8. On the other hand, in specimens H-3 and H-4 having strong panel zones, there was almost no plastic deformation of the panel zones. In the case of specimen H-2, the maximum deformation angle of the panel zone during the 1st time shaking test was approximately 0.02radian. The results shown in Fig.8 indicate that the maximum plastic deformation of the panel zones would depend on the thickness of doubler plate for reinforcement.

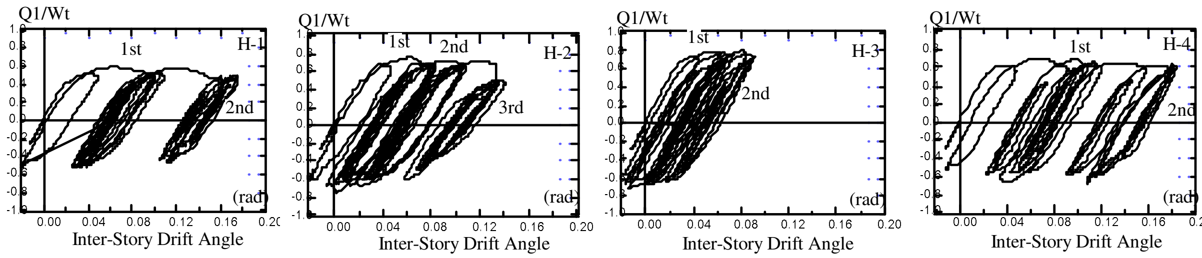

Fig.6 Story shear force coefficient vs. inter-story drift angle (1st story)

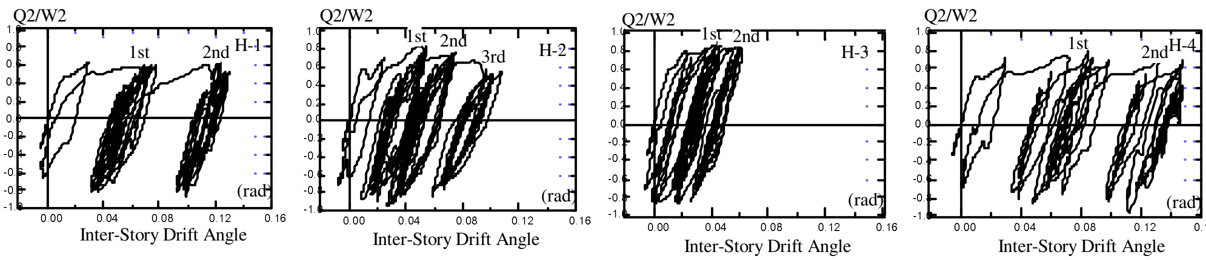

Fig.7 Story shear force coefficient vs. inter-story drift angle (2nd story)

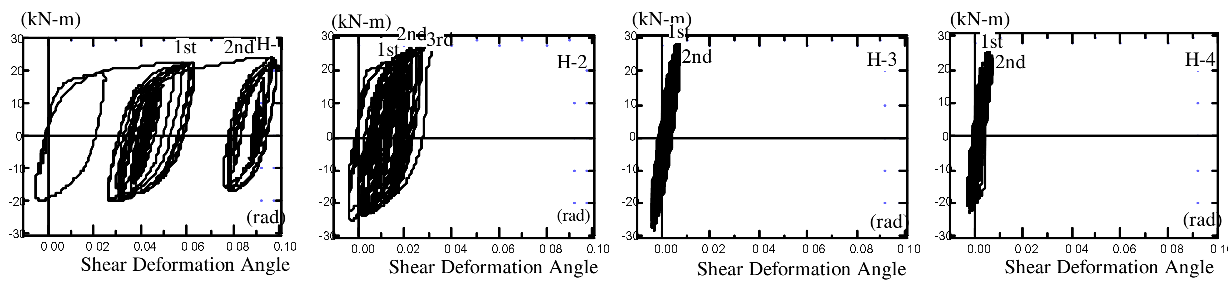

Fig.8 Moment vs. shear deformation angle of panel zone (2nd floor)

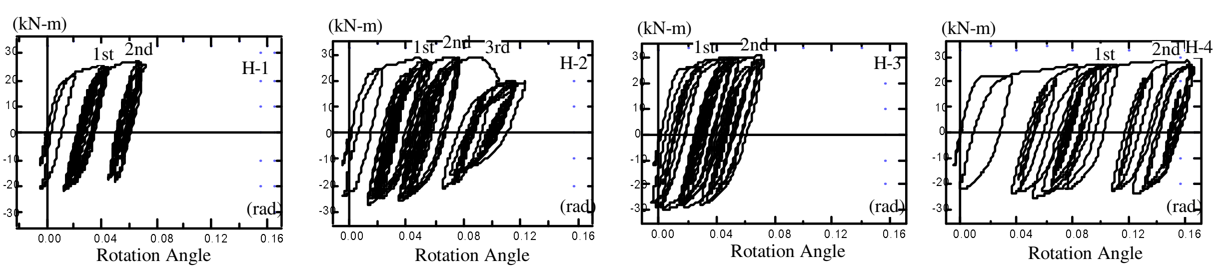

Fig.9 Moment vs. rotation angle of beam (2nd floor)

Fig.9 shows plots of beam moment at the column face vs. rotation angle of beam in the 2nd floor of each specimen obtained from the shaking table test. The beam rotation angle is obtained by dividing the displacement differential of the displacement transducers at the top and bottom flanges of the beam by the length of the beam depth. The calculated full plastic moment of the beam is 25.68 kN-m, and in specimen H-3 having strong panel zones, the calculated value is almost identical to the test value as shown in Fig.9. In the both specimens H-2 and H-3, the maximum rotation angle of the beam during the 1st time shaking test approximately reached to 0.05radian. Even in the H-1 specimen having weak panel zones, the maximum rotation angle of the beam during the 1st time shaking test was approximately 0.04radian. The maximum rotation capacity of the beam until the fracture occurred in specimens H-1 and H-3 was approximately 0.07 radian, and it was approximately 0.08 radian in specimen H-2 as shown in Fig.9. In specimen H-4, the maximum rotation angle of the beam during the 1st time shaking test reached approximately more than 0.1radian. But in only this specimen, the fracture of any beam connection in the specimen did not occurred even when the specimen impacted the collapse prevention frame. The maximum rotation angle of the beam at this time was more than 0.16radian, and it was revealed that the RBS beam connection used in the specimen have more resistant to fracture and have larger plastic deformation capacity than normal beam connection.

HYSTERESIS DISSIPATION ENERGY OF BEAM AND PANEL ZONE

Fig.10 shows the hysteresis dissipation energy of the beam and panel in the 2nd floor of each specimen. The frequency of the shaking table test was twice for specimens H-1, H-3 and H-4, and three times for specimen H-2. The total hysteresis dissipation energy of specimen H-2 was approximately 1.6 times that of the other three specimens, and it was little difference between the three specimens other than specimen H-2. Fig.10 shows that as intended by the design of the specimens, in specimen H-1 (weak panel zone), the hysteresis energy was primarily dissipated in the panel zones, in specimen H-2 (balanced panel zone), the hysteresis energy was dissipated by both beams and panel zones, in specimens H-3 and H-4 (strong panel zone), the hysteresis energy was primarily dissipated in the beams.

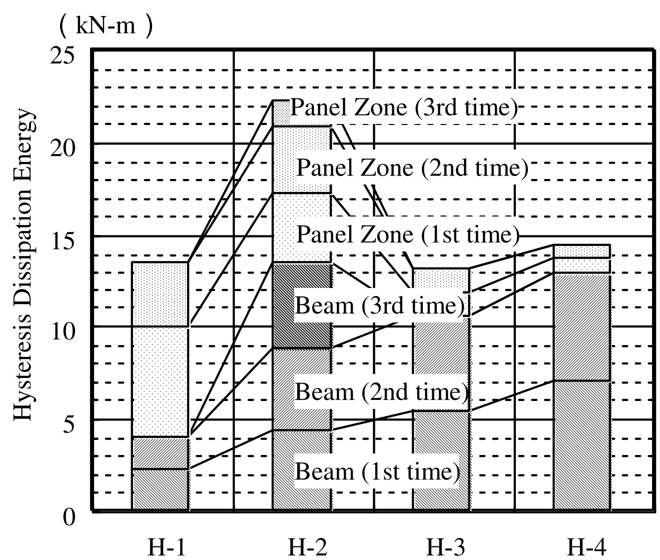

Fig.10 Hysteresis dissipation energy of beam and panel zone

The results shown in Fig.10 indicate that the thickness of doubler plate would cause big differences in the energy dissipation capacity until the frames fail. The frame designed to promote yielding in both panel zones and beams would develop the largest energy dissipation capacity. In specimen H-4 having the RBS connections, the plastic deformation capacity of the beam connections was extremely large. But the energy dissipation capacity until failure was almost the same in case of both specimens H-1 and H-3 as shown in Fig.10. Because the maximum inter-story drift of specimen H-4 against the earthquake ground motion became the largest of all the specimens.

SUMMARY AND CONCLUSIONS

This paper described the results of the shaking table test of 2-story steel frames. A major objective of this test is to investigate the effect of energy dissipation in panel zones on inelastic response behavior and total

energy dissipation capacity of the frames. Based on the results of the shaking table test reported herein the following conclusions are noted:
(1) The thickness of the doubler plate in panel zones caused differences in the maximum inter-story drift angle of the specimens. It would be necessary to account the strength increase of panel zones by the doubler plates in order to appropriately evaluate the maximum inter-story drift of the frames.
(2) Regarding the total energy input of the specimens, the equivalent velocity of the total energy input of specimen H-2 (balanced panel zone strength) was approximately 1.5 times that of the other three specimens. Comparison of the hysteresis dissipation energy in the 2nd floor of the specimens shows that the dissipation energy of the H-2 specimen was 1.6 times as much as the other specimens.
(3) In specimen H-4 having the RBS connections, the plastic deformation capacity of the beam connections was extremely large. But the energy dissipation capacity until failure was almost the same in case of both specimens H-1 and H-3. Because the maximum inter-story drift of specimen H-4 against the earthquake ground motion became the largest of all the specimens.

ACKNOWLEDGMENTS

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REFERENCES

1. FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, The Federal Emergency Management Agency, Washington, DC., 2000
2. Akiyama H. ”Earthquake-Resistant Limit-State Design for Buildings” University of Tokyo Press, 1985