Behaviour of Steel Structures in Seismic Areas, Mazzolani & Tremblay (eds) © 2000 Balkema, Rotterdam, ISBN 90 5809 130 9

Seismic performance of moment resistant steel frame with hysteretic damper

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ABSTRACT: After the Hyogoken-Nanbu earthquake (1995) in Japan, unbonded brace as a kind of hysteretic damper has been widely used in earthquake-resistant building structures. The authors of this paper have carried out a series static and dynamic loading tests for the moment resistant steel frames with or without hysteretic damper during the past year. This paper is reporting the outlines of the experiments, test methods and results. For the purpose of comparison with the experiment results, dynamic response analysis for a 3 story steel frame structure has also been carried out. It has been confirmed that hysteretic damper can help the moment resistant steel frame to absorb a majority of input energy from earthquake so that the damage to the main steel frame can be greatly relieved. Due to the contribution of hysteretic dampers to the lateral stiffness of the structural system, the main steel frame can be manufactured relative slender and to achieve good economic performance.

1 INSTRUCTIONS

The conventional moment resistant steel frames are usually designed with weak beam and strong column. It means that the two end parts of weak beam are allowed to yield during an extreme earthquake disaster. However, it has learned from the Northridge Earthquake in the USA 1994 and the Hyogoken-Nanbu Earthquake in Japan 1995, that the extraordinary large plastic strain induced in the beam end parts is the key reason to result in the collapse of conventional moment resistant steel frames at the connection between beam and column.

Some improvement methods for the conventional moment resistant steel frames, such as, using semi-rigid connection joint at beam ends; shifting the yielding part from the welds at beam ends to the inside of the beam, have been proposed. Additionally, to attach supplementary energy dissipation devices to the steel frame is one of the most effective methods to reduce the damage of moment resistant steel frames. The supplementary energy dissipation devices help the steel frame to absorb/dissipate a great amount of earthquake input energy and protect the beam end parts from large plastic yielding or collapse.

This paper presents the seismic performance of such moment resistant steel frames with unbonded brace that acted as hysteretic damper (HD) through the static cyclic loading tests and dynamic loading tests. Numerical response analysis for the test model has also been carried out for the purpose of comparison.

2 OUTLINES OF EXPERIMENTS

2.1 *Motivation of the experiments*

The bending moment distribution of a moment resistant frame is shown in figure 1 under the action of horizontal loads such as earthquake ground motion. The target of this experimental research is to see the ductility of beam end parts and the seismic performance of such frame.

Due to the limitation of the test facilities, only half span of the frame shown inside part of the circle of figure 1 is used for the test. For the convenience of loading method, the test specimen was rotated 90 degree in clockwise direction shown in right side of figure 1.

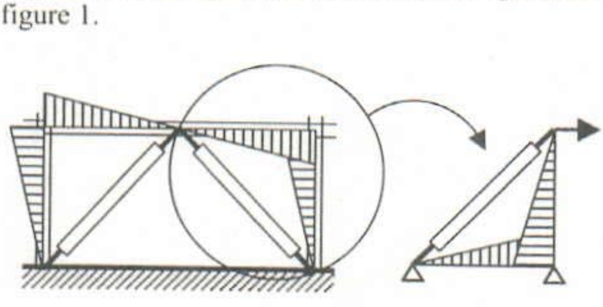


Figure 1 Part frame model

2.2 *Installation of test specimens*

Basically, two different types of specimens are used for this test series. MRF1 is a pure frame model without installation of hysteretic damper during the test shown in Photo 1. MRF2 is a frame model with hysteretic damper whose installation is shown in Photo 2.

In the photo pictures, the horizontal members are columns and the vertical members are beams. The column bottom (left side in the picture) is pin-supported on the reaction frame. The panel zone part between beam and column is supported by a pin-roller bearing so that allows horizontal displacement is allowed.

2.3 *Test parameters*

In order to compare the seismic performances of two different types of test specimens, the section size of MRF1 and MRF2+brace are adjusted so that they have the same strength approximately. The load displacement relationships of these test models obtained from the static structural analysis are shown in Figure 2 and 3 respectively. Due to the contribution of the unbonded brace, the section of MRF2 frame is manufactured smaller than that of MRF1 frame. Therefore, the weight of MRF2 is only about 60% of that of MRF1. The stiffness of MRF2 frame is smaller than that of MRF1 so that MRF2 has larger elastic deformation capacity than that of MRF1 and MRF2 can keep elastic behavior even under large inter story deformation. The distribution ratio of the shear forces subjected by the hysteretic damper and the main frame is about 4:6 shown in Figure 3. Two different steels SM490 (mild steel) and HT590 (high strength steel are used for the test.

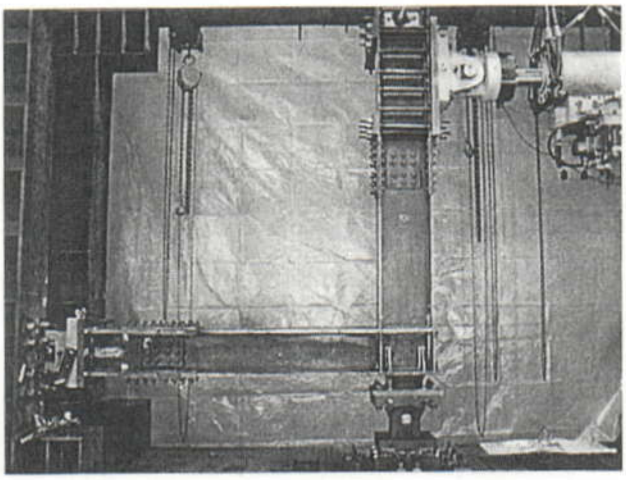


Photo 1 Specimen of pure frame (MRF1)

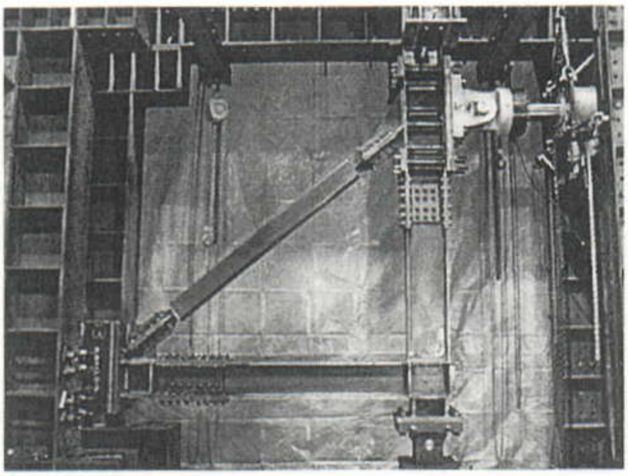


Photo 2 Specimen of frame with hysteretic damper (MRF2+brace)

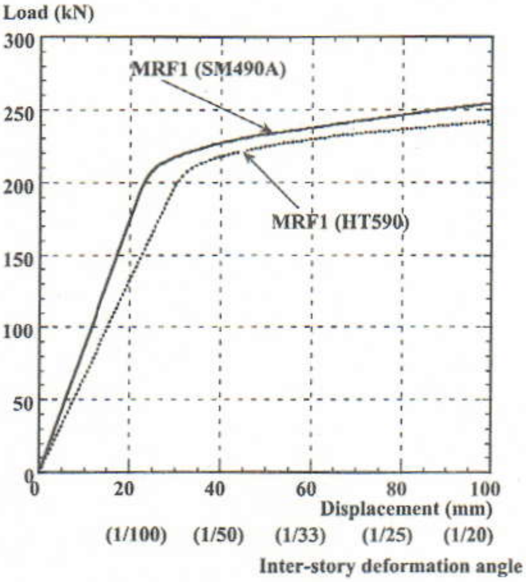


Figure 2 Load and deformation (MRF1)

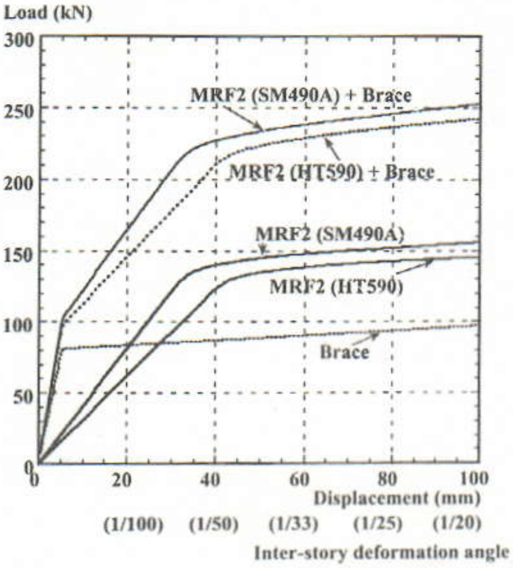


Figure 3 Load and deformation (MRF2)

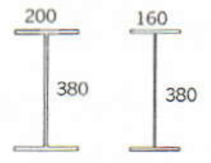
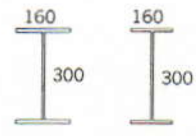
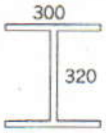
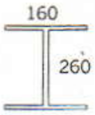
Totally, 12 specimens shown in Table 1 are used for this test series. The meanings of the symbols used in Table 1 are: SM and HT stand for the steel type SM490A and HT590 respectively; w means with and w/o means without; S_cyclic means static cyclic loading and D_cyclic means dynamic loading test. HD here means hysteretic damper or unbonded brace. End taper means that the width of end flange of beam section extends gradually just like a taper. The end taper is expected to reduce the large plastic strain concentration at the welded part at beam ends.

Table 1. List of test specimens

Specimen	Steel	HD	End taper	Loading	Number
SM-A-Sta	SM490A	w	w/o	S_cyclic	1
SM-C-Sta	SM490A	w/o	w/o	S_cyclic	1
SM-A-Dyn1	SM490A	w	w/o	D_cyclic	1
SM-A-Dyn2	SM490A	w	w/o	D_cyclic	1
SM-A-Dyn3	SM490A	w	w/o	DC_cyclic	1
SM-B-Dyn	SM490A	w	w	D_cyclic	1
SM-C-Dyn	SM490A	w/o	w/o	D_cyclic	1
SM-D-Dyn	SM490A	w/o	w	D_cyclic	1
HT-A-Dyn	HT590	w/o	w/o	D_cyclic	1
HT-B-Dyn	HT590	w	w	D_cyclic	1
HT-C-Dyn	HT590	w	w/o	D_cyclic	1
HT-D-Dyn	HT590	w/o	w	D_cyclic	1

The sectional size of the specimens used for the experiments are shown in Table 2.

Table 2. Section shape and size of the specimens

	MRF1	MRF2
Beam	 SM490A WT590	 SM490A WT590
Column	 SM490A WT590	 SM490A WT590

2.4 *Loading cycles*

The horizontal load acted on the top of the test specimen is controlled by the inter story deformation which can be transferred into inter story deformation angle through dividing the story height. The inter story deformation can be calculated from the displacement measured from the loading point.

Loading cycles for static loading test is illustrated in Figure 4. After 2 cycle loading test at the level of 1/400 inter story deformation angle, 4 cycle loading test were carried out for each level 1/200, 1/100, and 1/67 of inter story deformation angle respectively. At last, about 100 cycle loading test were continuously carried out at the level of 1/50 inter story deformation angle until the specimen collapsed.

Loading cycles for dynamic loading test is illustrated in Figure 5 which is a kind of sinusoidal wave whose frequency is 0.65Hz. At the beginning, 2 cycles dynamic loading test were carried out at the level of 1/400 inter story deformation angle. After that, 5 cycles dynamic loading test were carried out for each deformation level shown in Figure 5. For the specimen MRF2+brace, the unbonded brace was first broken. The dynamic loading test was continued for the frame only until the frame collapsed.

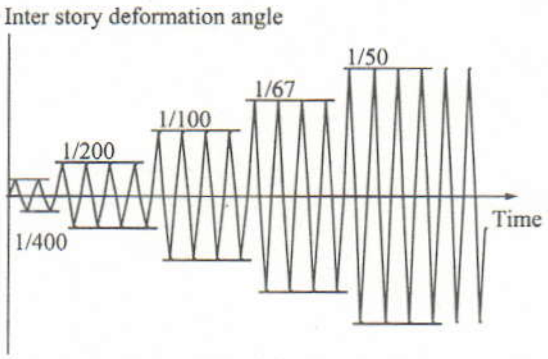


Figure 4 Static loading cycles

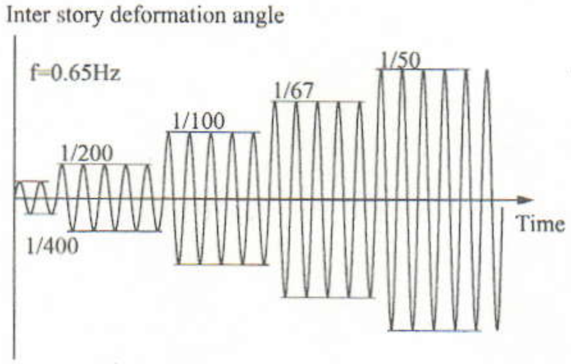


Figure 5 Dynamic loading cycles

3 ANALYSIS OF THE TEST RESULTS

3.1 *Relationship of force and deformation*

Assume the half span length of the beam to be L and the height of the column to be H as well as the horizontal displacement at top of the beam to be δ_L, then, the inter story deformation can be approximately obtained by $\delta_L * H/L$.

The relationships between the horizontal force and the inter story deformation obtained from the test results are illustrated from Figure 6 to 9 respectively for different specimens.

It is obviously understood that MRF2+brace specimen has much larger energy dissipation capacity compared with MRF1 specimen. Especially when the inter story deformation is very small like 12.5 mm (about 1/200 inter story deformation angle), MRF2+brace specimen has obvious energy dissipation capacity while the specimen of MRF1 has no any plastic loop.

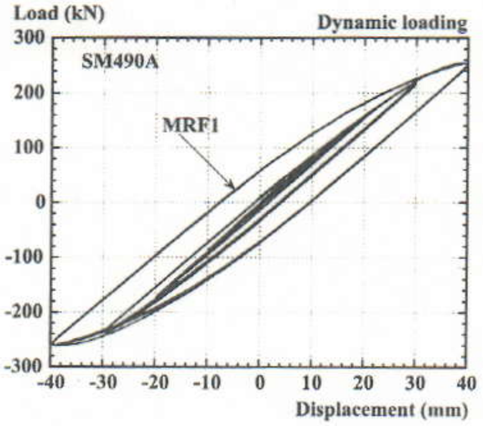


Figure 6 Load and displacement of MRF1 (SM490)

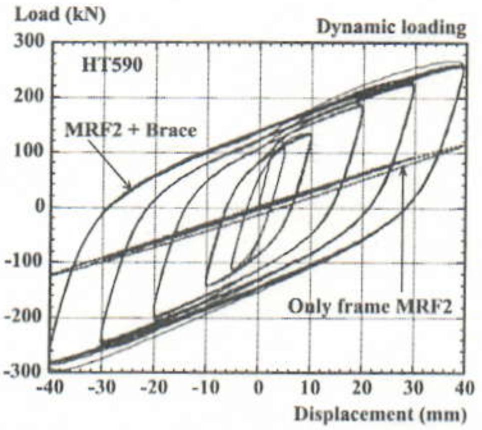


Figure 7 Load and displacement of MRF2+brace (SM490)

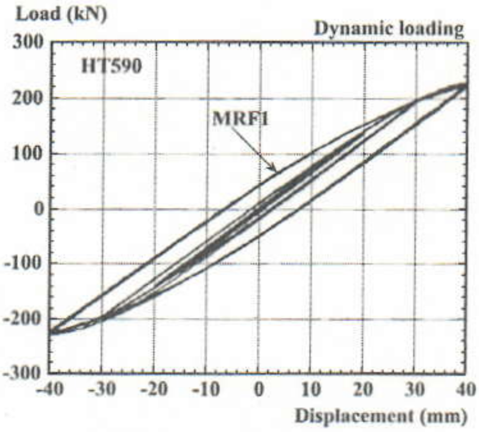


Figure 8 Load and displacement of MRF1 (HT590)

When the specimen experiences large deformation like 50mm (about 1/50 inter story deformation angle), the frame MRF1 has large plastic behavior while the main frame MRF2 keeps in almost elastic region. It can be concluded that the hysteretic damper or unbonded brace helped the main frame of the specimen to reduce plastic deformation or damage while frame MRF1 that has no hysteretic damper subjected large damage.

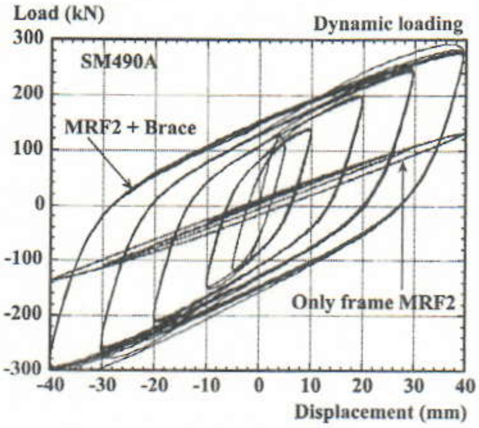


Figure 9 Load and displacement of MRF2+brace (HT590)

3.2 *Energy dissipation capacity*

Seen from the hysteresis loop shown in Figures 6 to 9, Specimen MRF2+brace model has much larger energy dissipation capacity. Integrating the load and displacement for each loop, the energy dissipated by each loop can be evaluated in quantity. Figures 10 and 11 show the energy dissipated at the second cycle for different types of specimen.

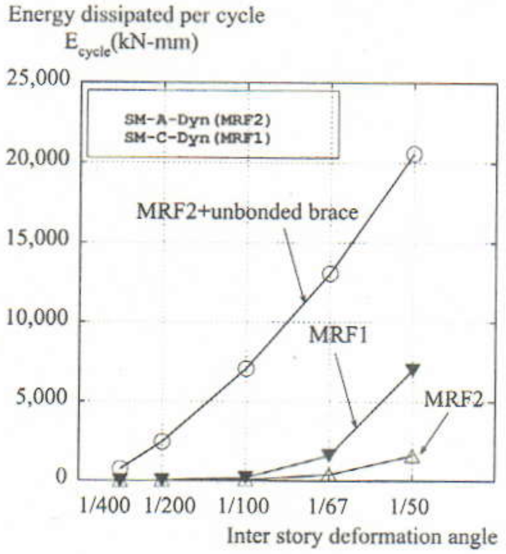


Figure 10 Energy dissipated per one cycle (SM490A)

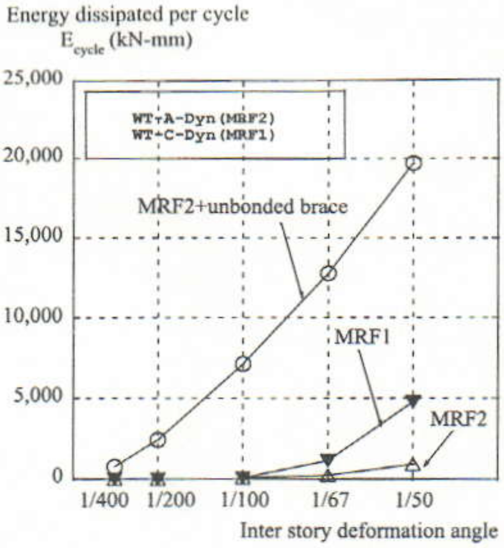


Figure 11 Energy dissipated per one cycle (HT590)

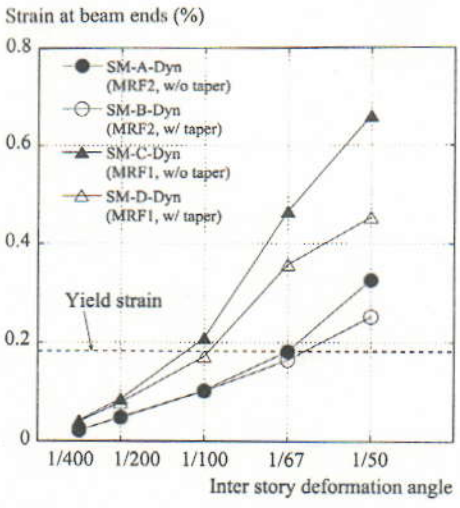


Figure 12 Local plastic strain at beam end part (SM490)

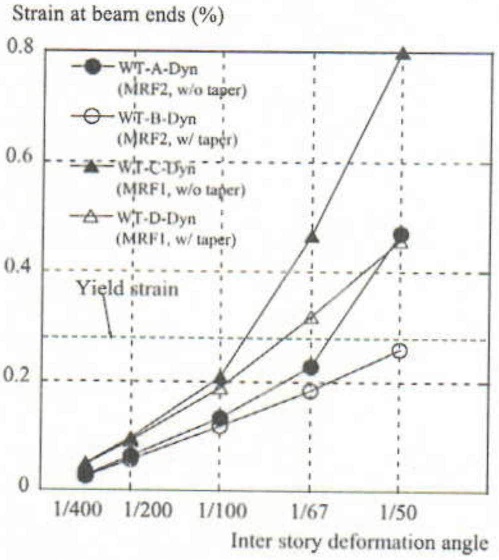


Figure 13 Local plastic strain at beam end part (HT590)

The curve denoted as MRF2 in the figures indicates the MRF2 frame only without hysteretic damper. Both the specimen manufactured of mild steel SM490A and the specimen manufactured of high strength steel have the same trend.

Under the little deformation whose inter story deformation angle is smaller than 1/100, both MRF1 frame and MRF2 frame has almost no energy dissipatd. However, under large deformation whose inter story deformation angle is about 1/50, MRF1 frame has much energy dissipated than MRF2. It means that MRF1 frame subjects much plastic deformation or damage than MRF2. MRF2 is much safer than MRF1. However, the specimen of MRF2+brace has much larger energy dissipation capacity than single MRF1 frame even under small deformation like 1/200. Due to the contribution of unbonded brace or hysteretic damer, MRF2 can keep in safe condition even under extreme large deformation.

3.3 *Local plastic strain at beam end part*

The trends of plastic strain at the center of beam end part increased with the increase of inter story deformation angle are shown in Figures 12 and 13 for the mild steel specimen and high strength steel specimen respectively. The plastic strain of MRF1 frame is about double larger than those of MRF2 frame. MRF1 frame begin to yield at the beam end part when the inter story deformation angle is 1/100 while MRF2 has not begun to yield even the inter story deformation angle is as large as 1/67. Taper shape flange at beam end can reduce the strain concentration at the end welded part. For the specimen MRF2+brace using high strength steel HT590 and with taper shaped flange at beam end, the largest strain at beam end is still kept in elastic even under large deformation with 1/50 inter story deformation angle.

4 DYNAMIC RESPONSE ANLAYSIS

4.1 *Structural models for response analysis*

In the static and dynamic cyclic loading test mentioned above, both the specimen MRF1 frame and MRF2+brace are subjected the same amplitude displacement. Since MRF2+brace model has much larger energy dissipation capacity, MRF2+brace model should has smaller displacement response than that of MRF1 frame. In order to confirm the test results, a few cases of dynamic response analysis have been carried out using 2 three story steel frames. One simulates MRF1 frame without any energy dissipation members. Another simulates MRF2+brace model with 2 unbonded braces in each story. The span length is 10 meter and story height is 6 meter. These 2 structural analysis models are shown in Figure 14. The member sizes are adjusted based on the results of static structural analysis under the action of A_i distributed external lateral force which are specified by the Japanese Building Construction Law. In the response analysis, 2% structure natural damping has been considered.

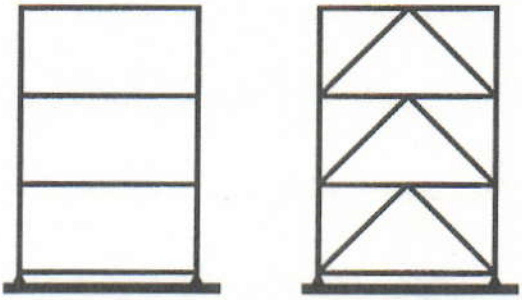


Figure 14 Structural model for dynamic response analysis

The relationship between the horizontal force and the inter story deformation at each story obtained from the static structural analysis are illustrated in Figure 15.

The solid curves in Figure 15 denote the results of MRF1 frame model while the dotted curves denote the results of MRF2+brace model. Due to the contribution of unbonded brace, the initial elastic lateral stiffness of MRF2+brace model is larger than that of MRF1 frame. After the unbonded brace yields, the lateral stiffness of MRF2+brace become smaller than that of MRF1.

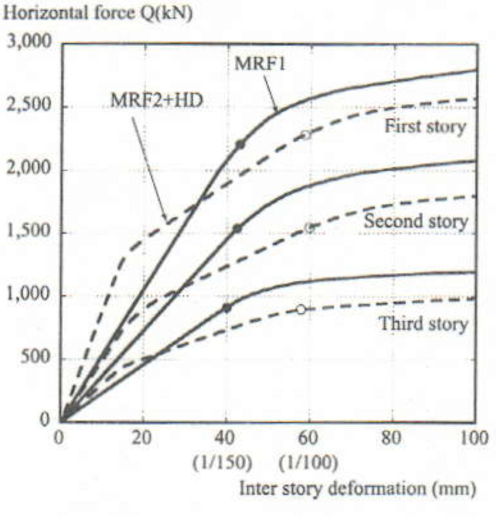


Figure 15 Relationship between force and inter story deformation for the analytical structural models

4.2 *Maximum response of inter story deformation*

The ground motion used for the time history response analysis is a kind of artificial ground motion that was generated to fit for a reasonable design spectrum using the phase filtered from the real earthquake ground motion EL Centro NS component. The strength of input ground motion was adjusted according to the level of induced maximum inter story deformation angle for MRF1 model. The distribution of the maximum inter story deformation of MRF1 and MRF2+brace models are illustrated in Figure 16 where the maximum inter story deformation angle is just equal to 1/100 for MRF1 frame. It is understood that the maximum inter story deformation angle induced in MRF2+brace model is only about 40% to 60% of that of MRF1 frame.

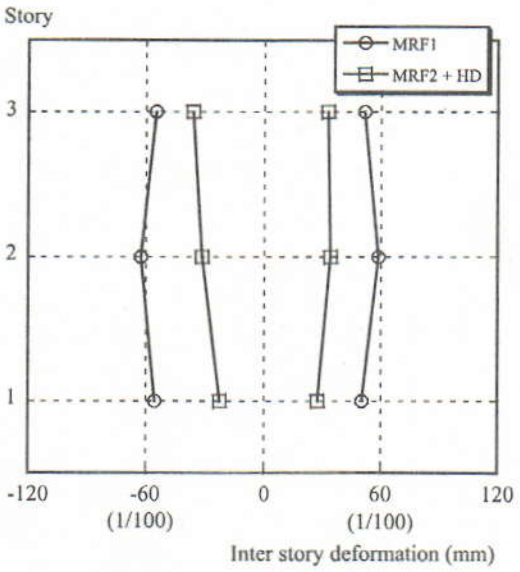


Figure 16. Maximum inter story deformation.

4.3 *Time history of energy dissipated*

Energy dissipated by the structural system was calculated in each time step. The time history of energies for MRF1 frame and MRF2+brace model are illustrated in Figure 17 and 18 respectively.

Seen from Figure 17, about 67% of total input energy has to be dissipated by the main frame MRF1. It means that MRF1 has to subject so much plastic deformation or damage. The rest 33% of total input energy was dissipated by the 2% natural structure damping.

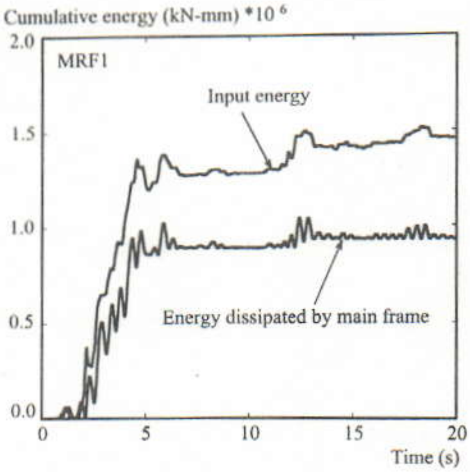


Figure 17 Time history of energy dissipated by MRF1 frame

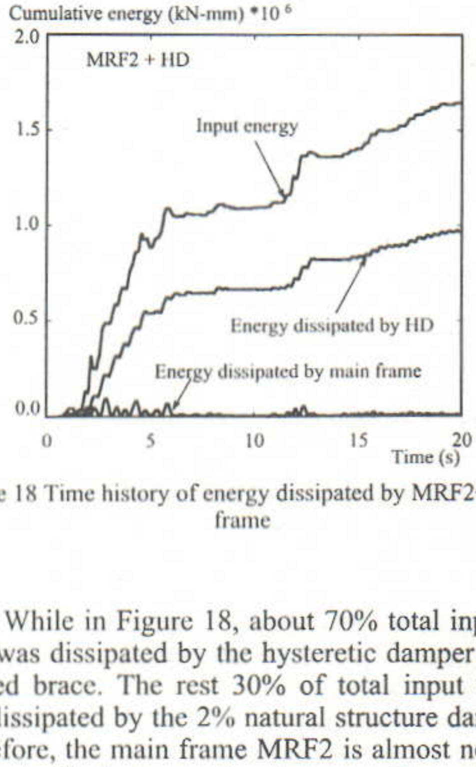


Figure 18 Time history of energy dissipated by MRF2+brace frame

While in Figure 18, about 70% total input energy was dissipated by the hysteretic damper or unbonded brace. The rest 30% of total input energy was dissipated by the 2% natural structure damping. Therefore, the main frame MRF2 is almost not necessary to dissipate any input energy. It means that the MRF2 frame can be protected from the major damage.

5 CONCLUSIONS

The following conclusions can be obtained from the experimental studies of this paper:

1) A pure steel frame without any energy dissipation members should absorb the majority of input energy and subject large damage itself. While the frame with supplemental hysteretic damper does not need to absorb the input energy by itself so that the main frame can be kept in elastic.

2) Due to the contribution of unbonded brace to the lateral stiffness of the whole structural system, the main frame can be manufactured relative slender with small section compared with that of regular pure steel frame. The main frame with supplemental hysteretic damper will keep in elastic behavior even it subject large inter story deformation angle. It can also reach good economic effect because it can be made lightly.

3) If the flange at beam end part is made in taper shape, it will relief the strain produced in the welds and reduced the damage to the weld parts during large earthquakes.

4) Although high strength steel has large elastic deformation capacity compared with mild strength steel, if the supplemental hysteretic damper is used in the steel frame, even mild steel can make the steel frame to have large elastic deformation capacity. Therefore, mild steel frame will have approximately the same seismic performance with that made of high strength steel.

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