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FROM INFANCY TO MATURITY OF BUCKLING RESTRAINED BRACES RESEARCH

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SUMMARY

A brace with a stable force-deflection characteristic and that enables compressive yield strength to equal tensile yield strength was developed in around 1980 in Japan. A core steel plate is jacketed with concrete encased in a steel tube to restrain buckling, and coating materials are used between the concrete and the core plate to prevent transmission of axial forces. In the following 20 years, buckling-restrained braces have been studied in many countries and used in many buildings. The origin of buckling restrained braces is discussed from the viewpoint of rational seismic design theory.

INTRODUCTION

Japan has been importing western construction technology since the country opened to the world in 1868. Modern buildings have been constructed using reinforced concrete and structural steel technologies. Since Japan is a seismic country, in these 135 years, many of these buildings have been severely damaged each time an earthquake has occurred. As a result, seismic structural engineering has evolved. The most important lessons learnt in these 135 years are that reinforced concrete buildings that have shear walls and structural steel buildings that have many steel braces have suffered less damage than those without these seismic members.

Ductile frames with large capacity for plastic deformation have been studied since the 1960s in the USA and Japan. The concept of allowing large deformations during an earthquake has led to construction of structures without shear walls and steel braces. However, in the Northridge Earthquake in 1994 and the Hyogo-ken Nanbu Earthquake in 1995 many buildings without shear walls or braces were severely damaged, while those with these seismic members were much less damaged. As a result, studies have been carried out to incorporate them into frame design again.

There are four main reasons why shear walls and steel braces increase seismic capacity:

1. They reach maximum strength at small story drift.
2. They resist earthquake forces before frames consisting of columns and beams reach yield.
3. Because they do not support the building's gravity load, the building will not lose its stability.
4. Columns simultaneously receive X- and Y-direction seismic forces, so they are a little weak against diagonal earthquake forces as shown in Fig. 1-a. Shear walls and braces oriented in the X

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direction resist X-direction forces and those oriented in the Y direction resist Y-direction forces as shown in Fig. 1-b. As a result, they have a large capacity to resist diagonal earthquake forces.

Thus, the seismic capacity of buildings with shear walls or steel braces is much higher than that of buildings without them.

DIFFICULTIES IN STRUCTURAL DESIGN DUE TO SLENDERNESS RATIO OF BRACES

1. When the brace has a large slenderness ratio, the deformation capability will be enough for the tension force, but lateral deformation occurs easily under compression force and then the brace cannot bear the compressive load. It will show slip type hysteresis under repeated loads.
2. When the brace has a small slenderness ratios, it can also bear the compressive load and buckling will not occur. However, the brace's axial stiffness would be too high compared to that of the surrounding frame, so that there are many difficulties in selecting appropriate structural members for braces in seismic design.
3. For a frame that incorporates braces with intermediate slenderness rations, it is difficult to design for the case in which the energy absorption by plastic deformation is required, because buckling will occur under compressive forces.

As a result, it is difficult to design a frame structure containing braces. If the problem of buckling is solved, it will be possible to design the required stiffness, yield strength and deformation at yield point because it will be possible to select the yield stress and sectional area of steel members.

BUCKLING RESTRAINED BRACES AS DUCTILE SEISMIC STRUCTURAL MEMBER

However, there remain problems to be solved in applying shear walls and steel braces to seismic design of buildings. These problems are common for both shear walls and steel braces. These structural elements reach their maximum strength at small deformation. However, their strength rapidly decreases with increasing deformation, and devastating failure can occur following shear failure in the shear walls. Similarly, steel braces can buckle under compressive force.

Design of building structures for large earthquakes allows plastic deformation, and elasto-plastic dynamic analysis is used. If the building contains structural members whose strength rapidly decreases after peak strength, the following problems will arise.

If there are many seismic walls or braces in a certain story, the maximum strength of the overall structure does not equal the sum of the strengths of the individual members as shown in Fig. 2-a. This is because the seismic walls and braces do not always achieve their maximum strength at the same time when they are deformed during an earthquake. They tend to reach their maximum strengths sequentially, such that one loses strength and the load is transferred to the next, and so on. We call this behavior zipper failure.

Next, consider the dynamic behavior of multi-story buildings. If the shear walls and braces in a certain story lose their strength after reaching maximum strength, deformation due to vibration occurring thereafter is concentrated in the damaged story as shown in Fig. 3-b.

In order to solve these problems, shear walls and braces need to be designed so that they do not lose their strengths following large deformations. Shear failure of shear walls needs to be prevented by appropriate placement of shear reinforcement and confinement reinforcement, and braces need to be designed not to buckle.

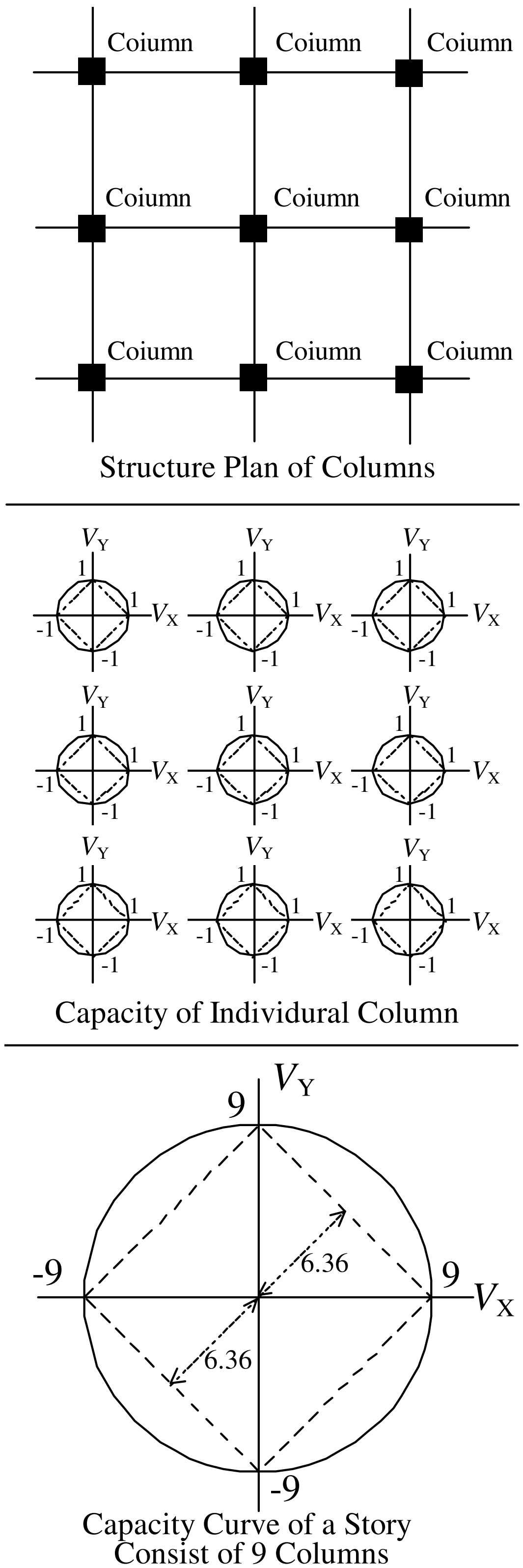

Fig. 1-a Capacity of 9 Columns Structure

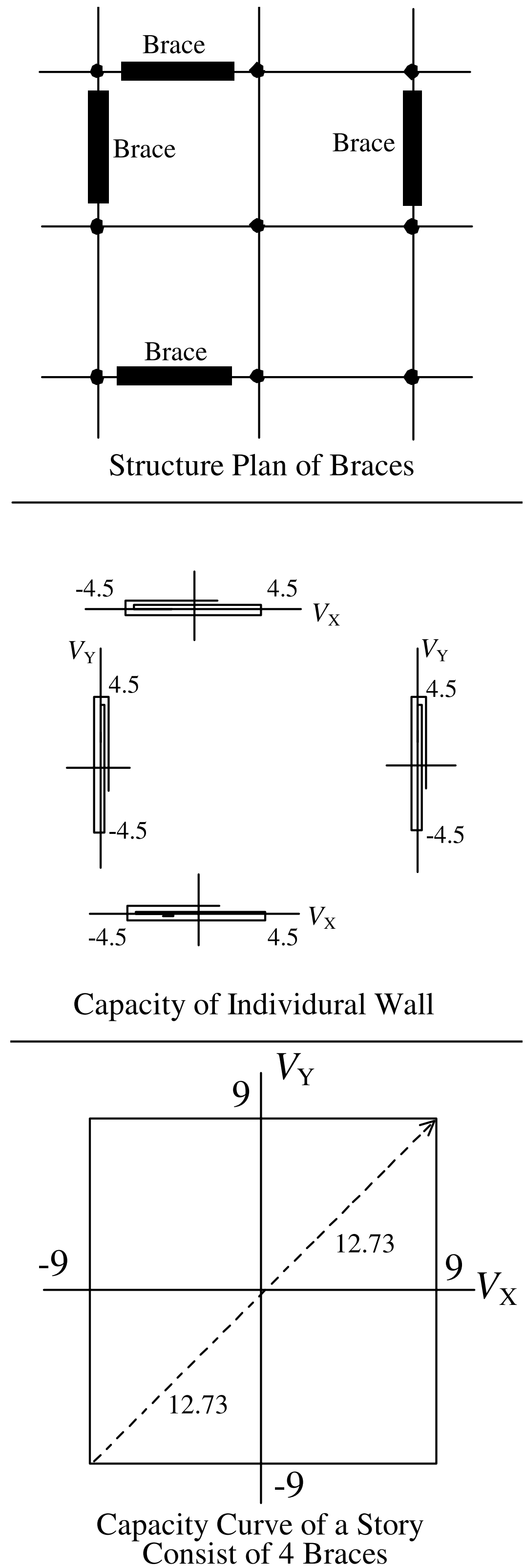

Fig. 1-b Capacity of 4 Braces Structure

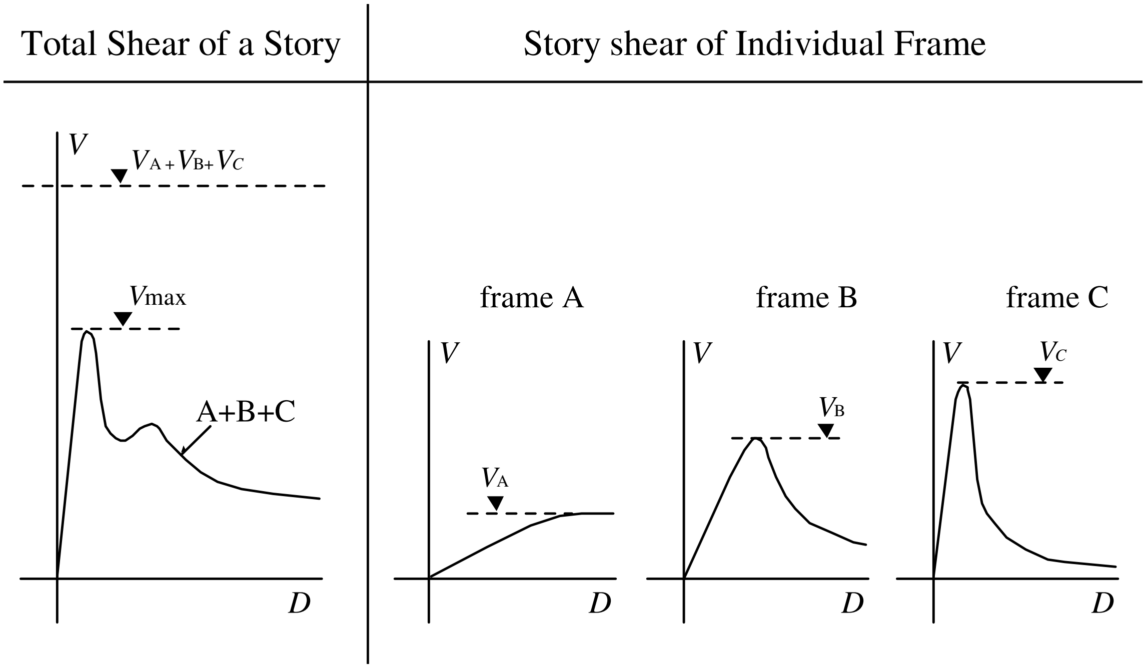

Fig. 2-a Typical Feature of Brittle Structures

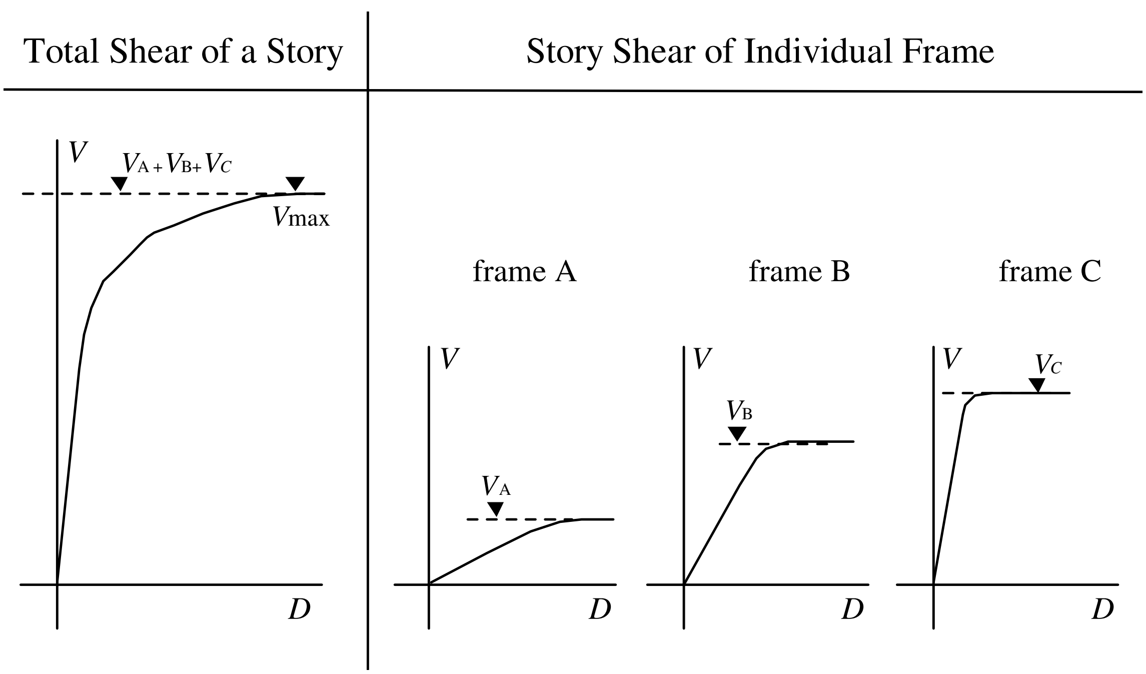

Fig. 2-b Typical Feature of Ductile Structures

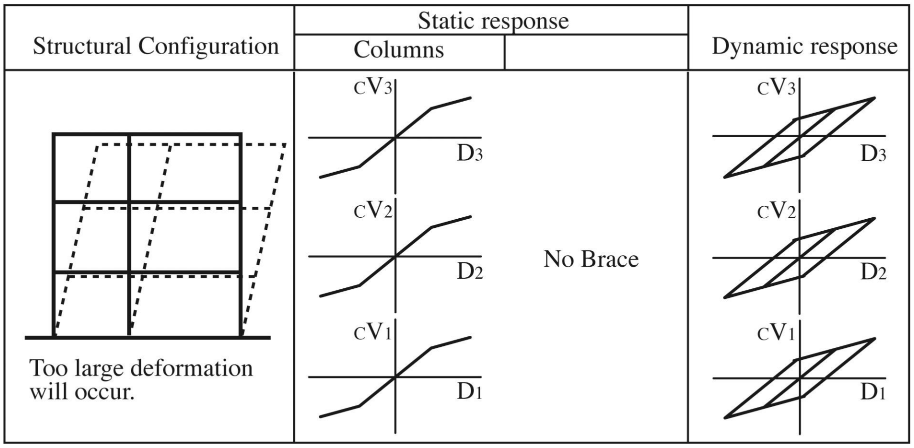

Fig. 3-a Earthquake Response of Moment Resisting Frame

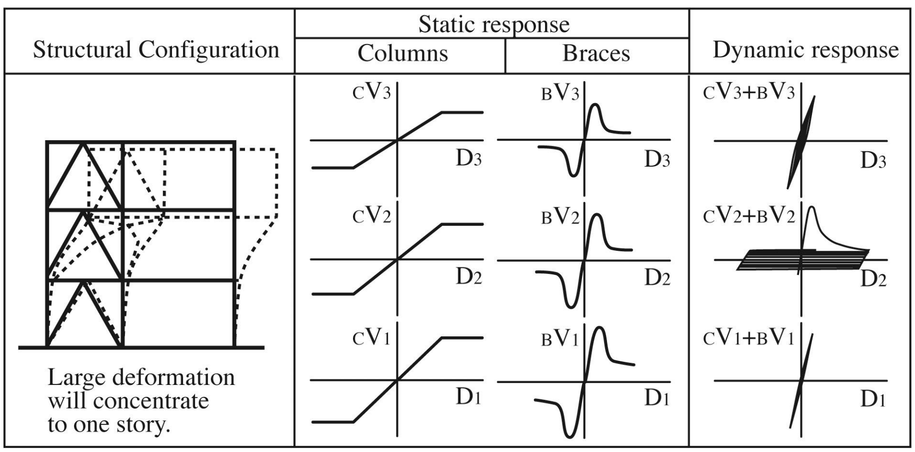

Fig. 3-b Earthquake Response of Moment Resisting Frame with Ordinary Braces

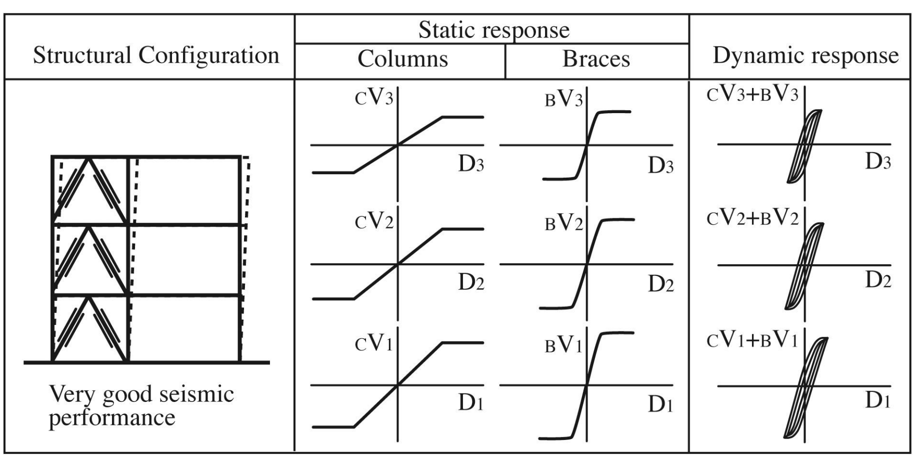

Fig. 3-c Earthquake Response of Moment Resisting Frame with Buckling Restrained Braces

This has led to the development of buckling restrained braces. These braces have the following characteristics. After the core steel member reaches yield, the brace can adapt to large deformation without decreasing its strength. It maintains its strength during repeated loading and gradually increases its strength after plastic deformation occurs.

As a result, the overall strength of buckling restrained braces that are placed in a certain story is the sum of their individual strengths in the story as shown in Fig. 2-b. After a certain story becomes plastic, the strength of the story gradually increases. Thus, earthquake energy is dispersed to the other stories as shown in Fig. 3-c.

TEST SPECIMENS AND EXPERIMENT

As shown in Fig. 4, all the core members used as specimens are 19 mm x 90 mm plates. The length of the specimens is 3290mm as shown in Fig. 5. Fig. 6 shows the detail at both ends of the specimens. The core members were coated with concrete encased in a steel tube. The steel SS41 of the Japanese Industrial Standard was used as the material for the core members and the yield stress by the material test was 288 MPa. The steel STK50 of the JIS was used as the material for the outer steel tubes and the yield stress by the material test was 370 MPa. The cross-shaped core member is exposed at the ends of a specimen and a cross-shaped buckling prevention steel member is embedded in the concrete encased in a steel tube at both end. Furthermore, in view of the effect of Poisson's ratio vinyl/mastic tape was used in the thickness direction of both sides between the core member and the concrete and 3mm thick foaming polystyrol was also used in the width direction on both sides between the core member and the concrete.

The experiment was conducted on a total of five specimens with the ratio of Euler load, P_E of the steel tube to the yield load P_y, working on the core member. P_E/P_y between 0.55 and 3.82 by carrying the sectional dimensions of the steel tube with the size of the core brace kept constant. Calculated strength values of each specimen are shown in left part of Table 1.

In the experiment, horizontal forces were applied to the frame using an 1100kN actuator as shown in Fig. 7 and Photo. 1. The braces were tested by repeated load on both directions at 8 cycles until story displacement angles become 1/400 to 1/50.

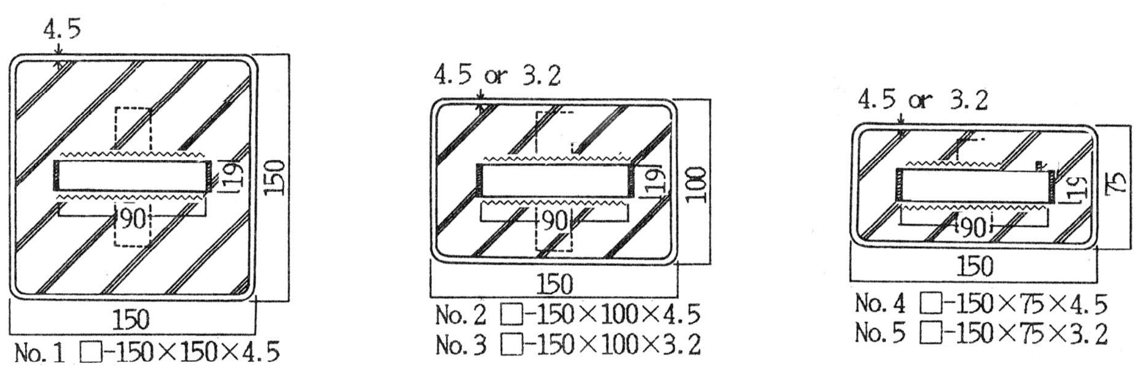

Fig. 4 Cross sections of 5 specimens

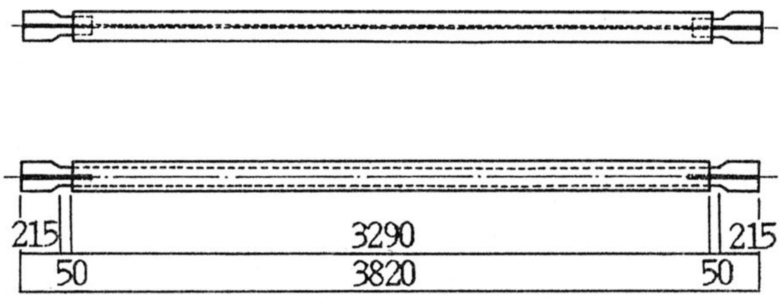

Fig. 5 Typical Configuration of the Specimen

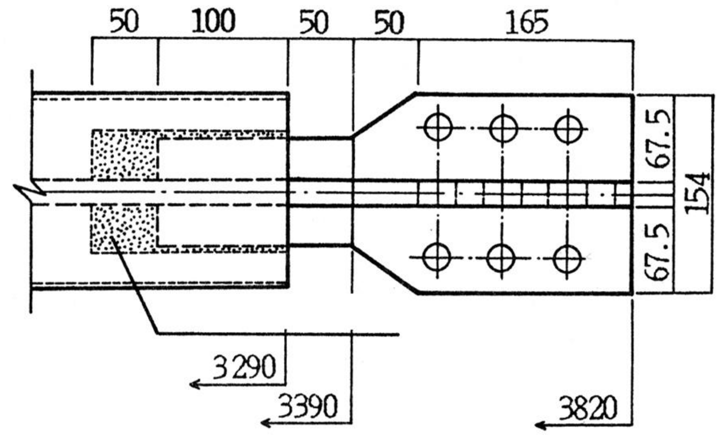

Fig. 6 Right End Detail of the Specimen

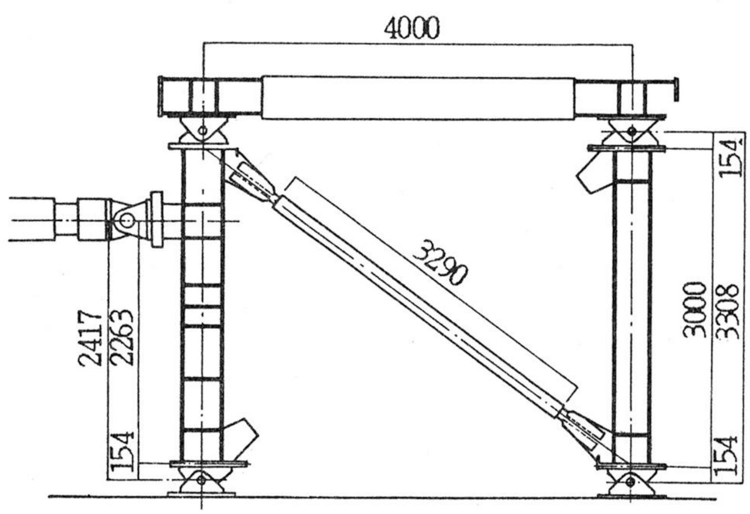

Fig. 7 Loading apparatus

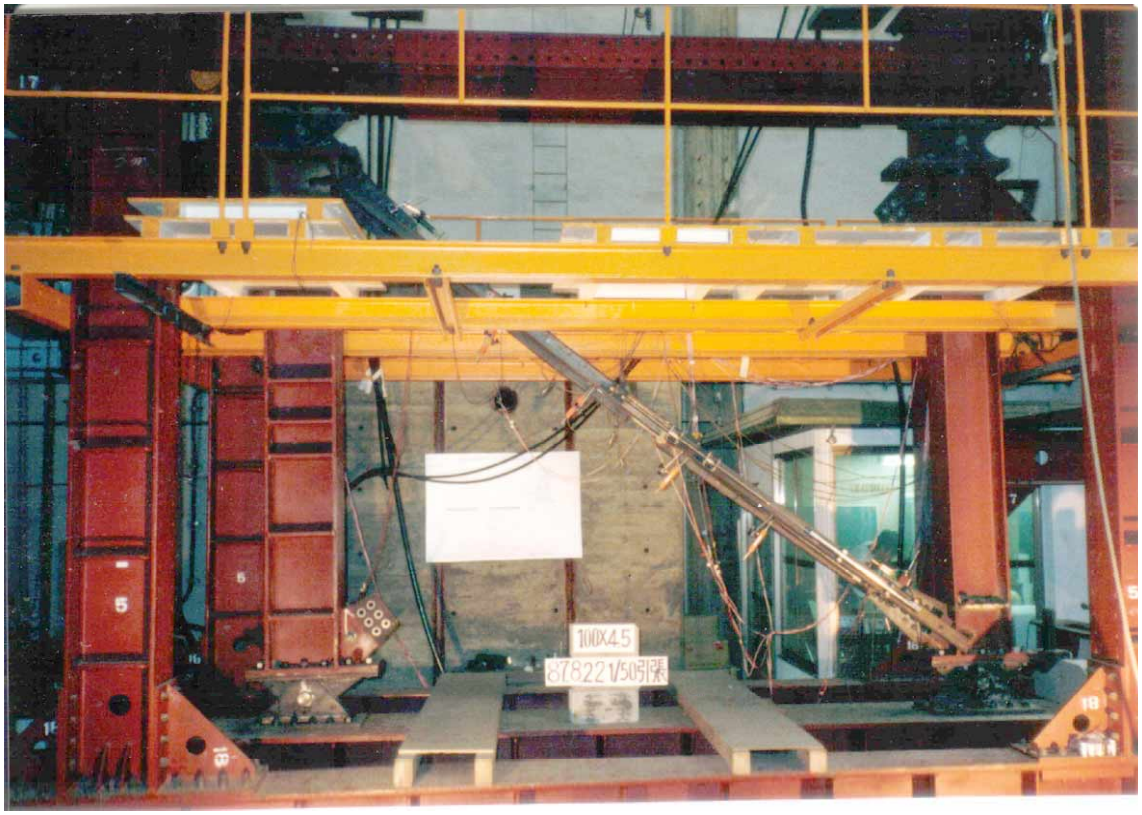

Photo 1 The Specimen and Loading apparatus

TEST RESULTS

Results of the experiment are shown in Table 1. For the specimens in which the buckling strength exceeded the yield stress of the core member (No.1, No.2 and No.3 specimens), buckling did not occur even under compression loading and, as shown in Figs 8, 9 and 10, a lot of energy was absorbed. Thus, the hysteresis characteristic was stable in these specimens. The story displacement angle at yield point was about 1/500. A stable hysteresis characteristic was observed even at the final story displacement angle of 1/50. It can be said, therefore, that these three specimens have hysteresis characteristics that coordinate sufficiently with the deformation of ordinary moment resisting frames of 1/200 to 1/50.
Because the initial stiffness corresponds closely to the stiffness of the core member only and because the axial force transmitted to the concrete encased in the steel tube was about 5% of the total even under compression loading, it might be thought that the bond and friction between the core member and the concrete encased in the steel tube could be eliminated. It may be said that the effect of the coating materials (vinyl/mastic tape plus foaming polystyrol) used in the experiment was ascertained.
In specimens in which the buckling yield strength of the concrete encased in the steel tube was lower that that of the core member (No.4 and No.5 specimens), buckling occurred before the yield of the core member during compression and the yield strength decreased abruptly as shown in Figs. 8 to 9. The condition of the specimens after the experiment is shown in Photo 2. The specimens shown are No.1 to No.5 from left to right.

	B×D×t (mm)	I_k (cm^4)	P_E (ton)	A (cm^2)	Py (ton)	P_E / Py	Pt (ton)	Pt / Py	Pc (ton)	Pc / Py	Pcr (ton)	Pcr / Py	Pcr / P_E
No. 1	150x150x4.5	896	171.0	16.84	48.50	3.53	48.6	1.00	51.5	1.06	—	—	—
No. 2	150x100x4.5	352	67.4	16.84	48.50	1.39	48.3	1.00	51.8	1.07	—	—	—
No. 3	150x100x3.2	262	50.2	16.88	48.61	1.03	47.6	0.98	49.3	1.01	—	—	—
No. 4	150x75x4.5	183	35.0	16.84	48.50	0.72	48.3	1.00	—	—	46.5	0.96	1.33
No. 5	150x75x3.2	137	26.2	16.62	47.87	0.55	47.9	1.00	—	—	43.1	0.90	1.65

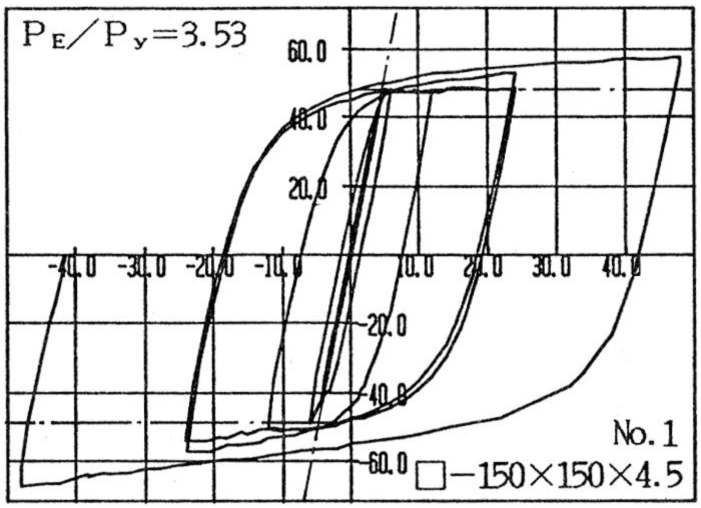

Fig. 8 Test result of specimen No.1

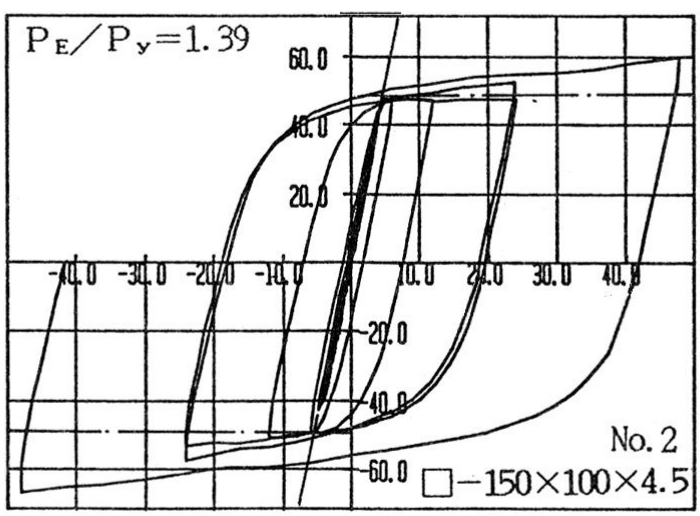

Fig. 9 Test result of specimen No.2

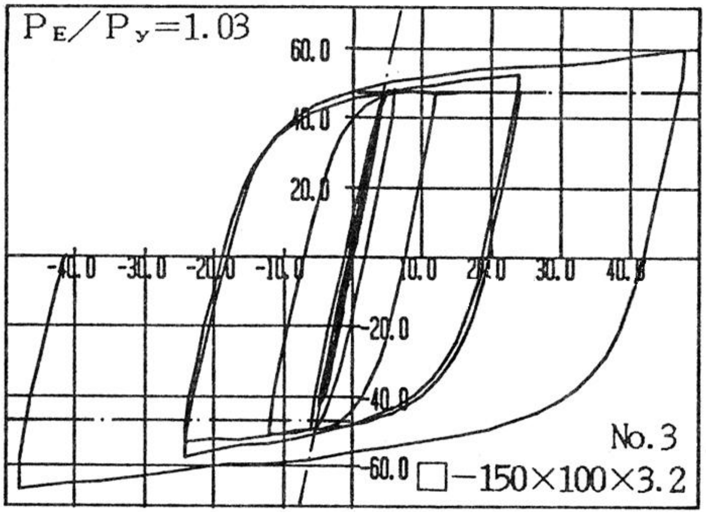

Fig. 10 Test result of specimen No.3

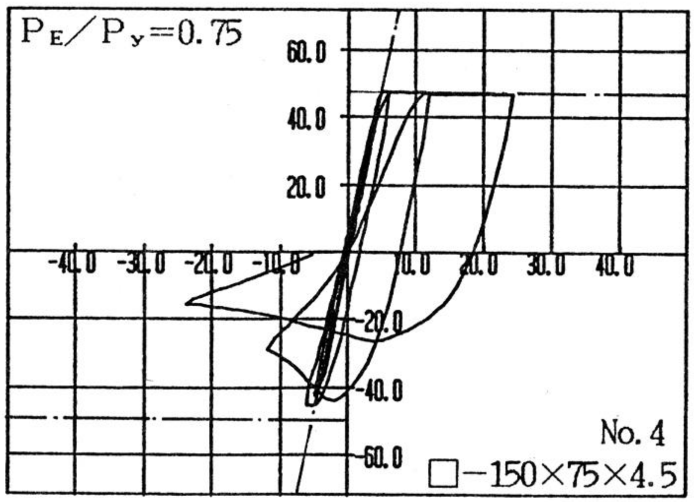

Fig. 11 Test result of specimen No.4

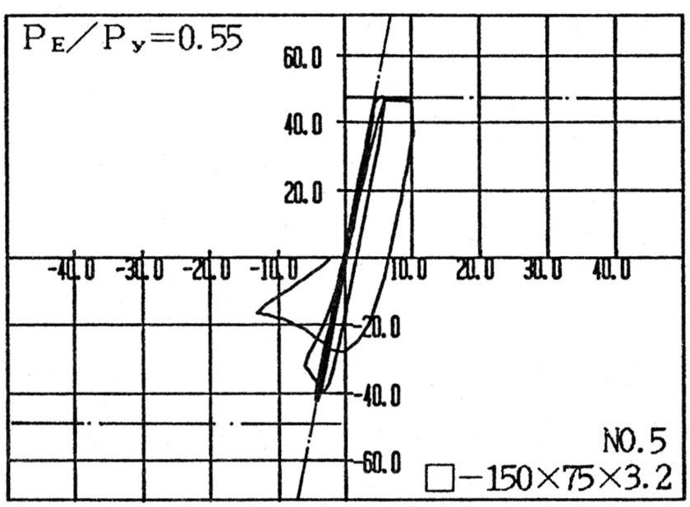

Fig. 12 Test result of specimen No.5

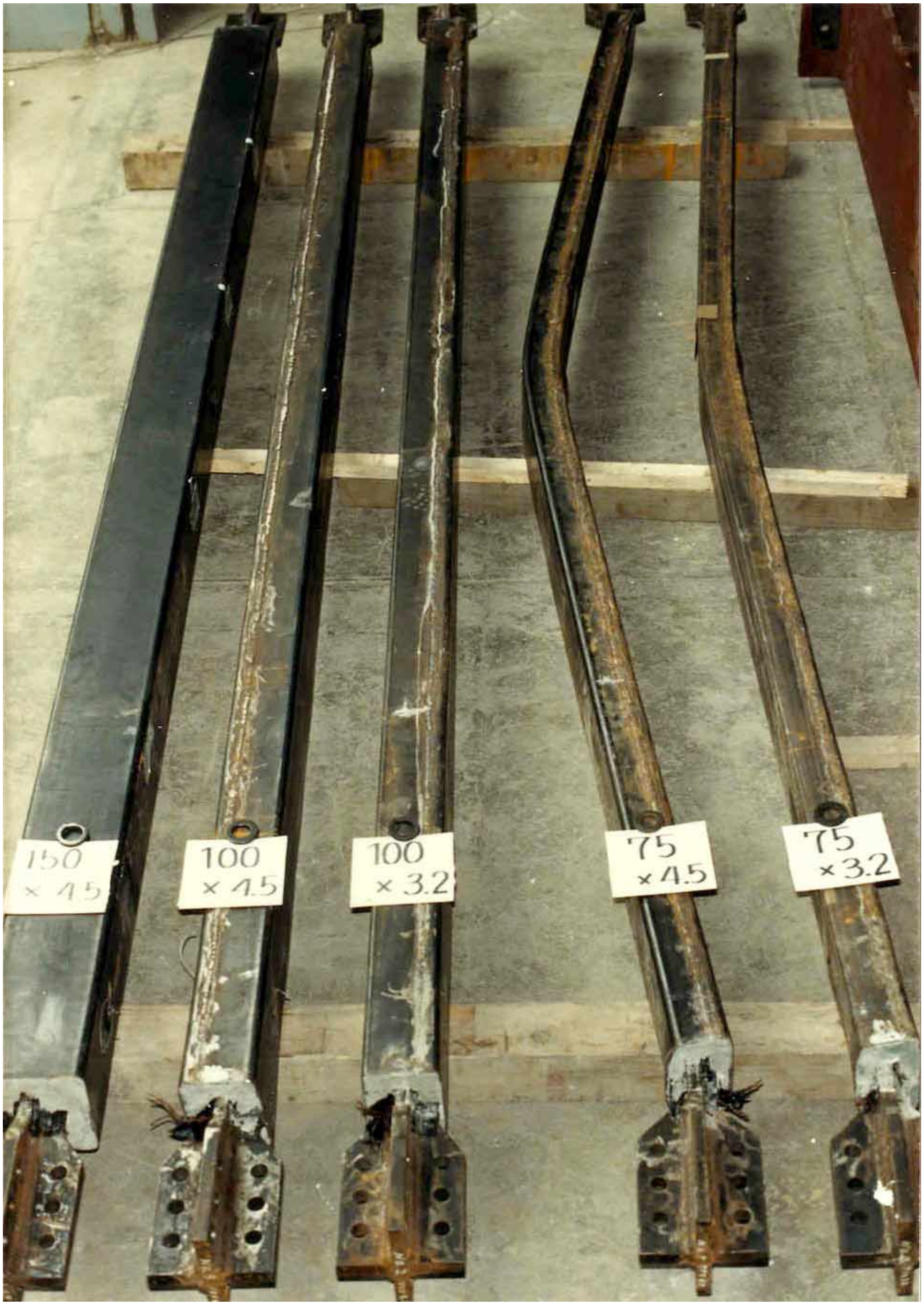

Photo 2 The Specimens after the test

NECESSARY EULER BUCKLING LOAD OF TUBE TO YIELD FORCE OF CORE STEEL

No axial force is acting on the outer steel tube because there are unbonded materials around the core steel plate, and elastic flexural stiffness of the tube is maintained. Therefore, if the Euler buckling load of the outer tube is kept even slightly larger than the yield load of the core steel plate, even if only the core brace reaches yield, buckling does not occur. Thus, a stable force/displacement relationship is obtained. However, in reality, buckling strength may decrease due to the effects of initial deflection, etc.

The following experiments were performed as examples. The core steel plate was 19mm x 90mm. Deflection due to the self-weight was 19 mm at the center for the both-end-pinned condition. This corresponds to 1/174 for a member length of 3290 mm. Because actual manufacturing is carried out by standing the member upright, an initial deflection as large as 1/174 would not occur. However, the effects of initial deflection on buckling of the brace need to be discussed.

As shown in Figure 13, let the initial deflection at the center be e. Then, the deformation curve $y(x)$ is obtained as a sine wave, as shown by:

$$y(x) = e\sin(\pi x/l) \quad \text{----- (1)}$$

With an axial load, this deflection increases. Total deformation curve $v(x)$ is obtained from:

$$v(x) = v_0 \sin(\pi x/l) \quad \text{-----(2)}$$

Bending moment distribution $M_e(x)$ in this member can be calculated from the axial load P multiplied by the total deformation $v(x)$ as shown by:

$$M_e(x) = Pv(x) = Pv_0 \sin(\pi x/l) \quad \text{-----(3)}$$

Bending moment $M_i(x)$ in this member due to deformation is expressed by:

$$\begin{aligned} M_i(x) &= -EI\phi(x) \\ &= -EI\frac{d^2}{dx^2}(v_0 - e)\sin(\pi x/l) \\ &= \frac{\pi^2 EI}{l^2}(v_0 - e)\sin(\pi x/l) \end{aligned}$$

Since $M_e(x)$ must be equal to $M_i(x)$ under the equilibrium condition, the relationship shown in Equation (4) is introduced.

$$Pv_0 \sin(\pi x/l) = \frac{\pi^2 EI}{l^2}(v_0 - e)\sin(\pi x/l) \quad \text{-----(4)}$$

Using $P_E = \dfrac{\pi^2 EI}{l^2}$, Equation (4) yields:

$$Pv_0 = P_E(v_0 - e) \quad \text{-----(5)}$$

Central deflection v_0 is obtained from:

$$v_0 = \frac{P_E e}{P - P_E} \quad \text{-----(6)}$$

Bending moment M_0 at the center is expressed by:

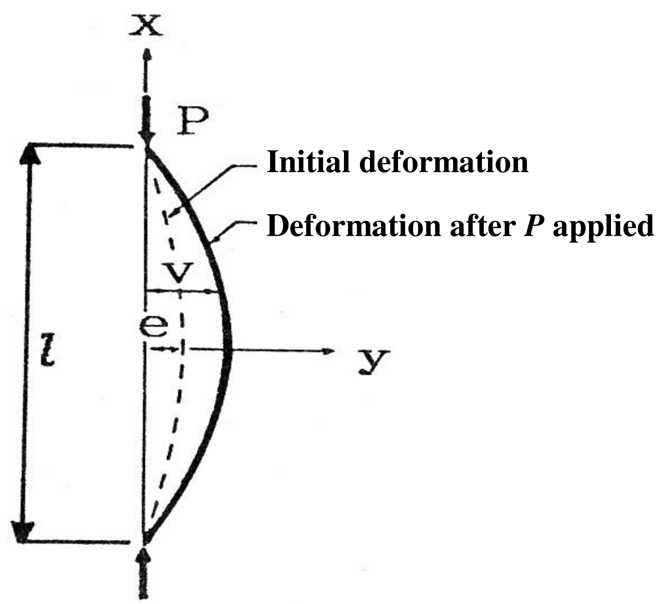

Fig. 13 Initial Deformation and Final Deformation after *P* applied

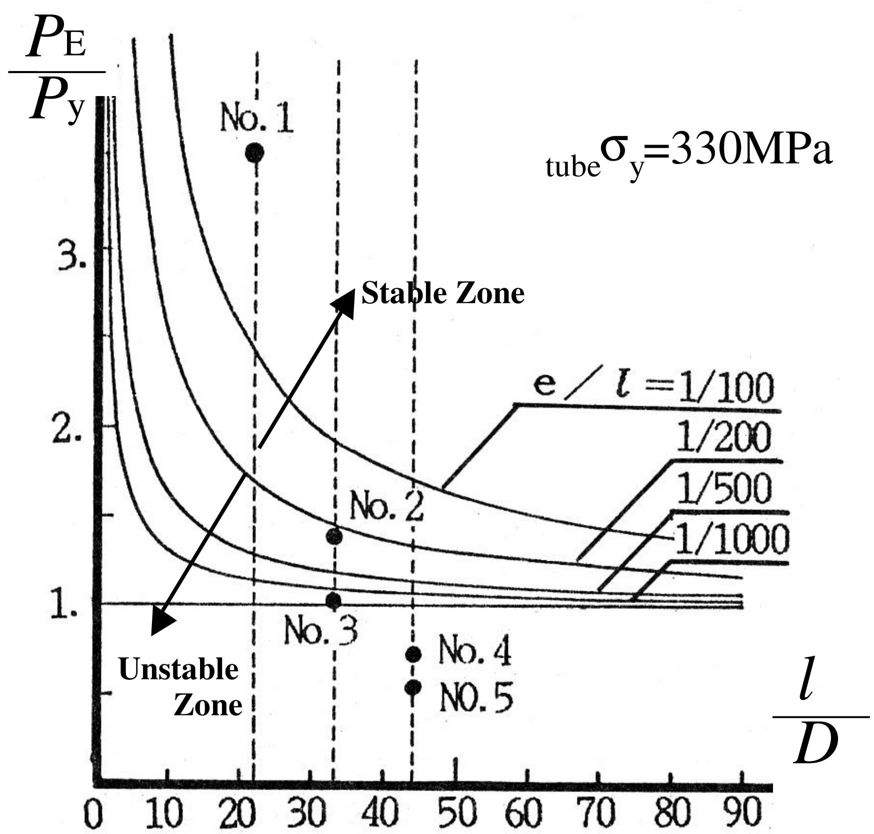

Fig. 14 Stability Chart for Buckling Restrained Braces

$$M_0 = Pv_0 = \frac{PP_E e}{P - P_E} \quad \text{-----(7)}$$

If this bending moment in the steel tube does not exceed the yield bending moment ${}_{tube}M_{yeild}$, buckling does not occur at this member.

$${}_{tube}M_{yeild} = {}_{tube}\sigma_y Z = {}_{tube}\sigma_y \frac{I}{D/2} \quad \text{-----(8)}$$

${}_{tube}M_{yeild} > M_0$, thus:

$${}_{tube}\sigma_y \frac{2I}{D} > \frac{PP_E e}{(P - P_E)}$$

$${}_{tube}\sigma_y \frac{2I}{D} > \frac{Pe}{(P - P_E)} \frac{\pi^2 EI}{l^2} \quad \text{-----(9)}$$

Rearranging equation (9) yields:

$$\frac{P_E}{P} > 1 + \left(\frac{\pi^2}{2} \frac{E}{{}_{tube}\sigma_y} \frac{e}{l}\right) / \left(\frac{l}{D}\right) \quad \text{-----(10)}$$

Because P does not exceed P_y in the unbonded braces, it is necessary to satisfy:

$$\frac{P_E}{P_y} > 1 + \left(\frac{\pi^2}{2} \frac{E}{{}_{tube}\sigma_y} \frac{e}{l}\right) / \left(\frac{l}{D}\right) \quad \text{-----(11)}$$

Figure 14 shows the range given in Equation (11) with P_E/P_y on the Y-axis and l/D on the X-axis. The five lines shown here are based on ${}_{tube}\sigma_y = 330MPa$, and are for initial deflections of 1/100, 1/200, 1/500, 1/1000 and zero. For these initial deflections, the upper right part of each line shows the region where buckling does not occur. The test pieces used in these experiments are plotted on this chart. No. 1 is in the perfect stable region. Nos. 2 and 3 are situated generally on the lines of initial deflections of 1/200 and zero, respectively. Nos. 4 and 5 are in the region where buckling occurs even if there is no initial deflection. Buckling actually occurred in these cases in the experiments. The reason why buckling did not occur in No. 3 is considered to be that the internal concrete had a hybrid effect for the flexural stiffness and the boundary condition was not a perfect pin.

From Figure 18, when an initial deflection of up to 1/200 is allowed, a P_E/P_y of about 1.5 is sufficient for $l/D = 32.9$, and this corresponds to Nos. 2 and 3 in the current experiments.

CONCLUSIONS

The following were the findings of this study. If the braces described in this paper were actually incorporated into a frame and the member ends were subjected to a bending moment, overall buckling would not occur providing the Euler buckling load of the outer tube was larger than the yield strength of the core steel. Furthermore, a stable force/displacement relationship would be obtained even after the core steel plate reached yield.

Actual design would require coverage with an outer steel tube that had an Euler buckling load about 1.5 times larger than the yield force of the core steel plate. Buckling was avoided in the experiments even in No. 3 with P_E/P_y of 1.03, because the concrete inside the steel tube contributed to its flexural stiffness. It is thus possible to decrease the cross-sectional area and thickness of the steel tube by taking into account the effect of the inner concrete. However, this was ignored when considering the safety margin.

The buckling restrained braces enabled us to determine the initial stiffness and yield strength of the brace member from the characteristics of the core steel plate as well as the compressive and tensile characteristics by isolating the buckling problem. It is also possible to utilize this for hysteretic damping in order to demonstrate stable behavior after the core steel plate reaches yield.

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