HYBRID STEEL REINFORCED CONCRETE STRUCTURE : STRENGTHENING OF REINFORCED CONCRETE COLUMNS BY CENTRAL REINFORCING STEEL ELEMENT

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SUMMARY

In this paper, a special structure which might be called hybrid steel and reinforced concrete (HSRC) structure is recommended for aseismic structures which are lower than 6 or 7 story height and are almost without shear walls. This structure is composed of ordinary reinforced concrete (RC) beams and special steel and reinforced concrete (SRC) columns. This special SRC columns are composed of ordinary RC steel bars and a central steel H or pipe element. This central reinforcing element is used only for resisting axial force and is not expected for bending and shear resistance. Experimental study and theoretical study are reported for the reference data for HSRC structure.

INTRODUCTION

In Japan, it is a common practice to use 13mm diameter ties of 10cm or less spacing for RC columns. But in the past, we encountered some cases where RC buildings with these columns which are with a few or without shear walls were damaged completely being subjected to severe earthquakes. Photo 1 shows the one example of this case. In the Hyogoken Nambu Earthquake (Magnitude 7.2, 1995, Japan), we saw the case where a RC building of which columns were with 7.5cm spacing ties of 13mm diameter were collapsed completely.

In past destructive earthquakes, the damages of RC columns such as shown in Photo 2 have been seen. The damage is shear failure of a rather long column. Such damage of RC columns can not be reproduced in experimental studies at laboratories. It is thought that earthquake force has an impactive loading character to some extent. At the laboratory experimental tests, it is almost impossible to use actual weight for applying vertical force to columns. Furthermore the actual columns are subjected to both dynamical horizontal and vertical force during earthquakes. It is very difficult to realize these loading conditions at laboratory tests and

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also it is almost impossible to analyze theoretically the behaviors of RC columns under such loading conditions. For the time being, taking into consideration of damages of RC columns during severe earthquakes and also based on our engineering judgement, we would conclude that ordinary RC columns which are used for structures without or with a few shear walls are not dependable structural elements during severe earthquakes.

On the other hand, it has been proven that SRC structures are strong and ductile enough for earthquake force. But in the past Hyogoken Nambu Earthquake, structures with columns with a section such as shown in Fig. 1 were damaged. This SRC column section is a rather old fashioned and is composed of 4 angle steel sections and ladder type steel tie plates. The tie plates were 9mm plates and 60-70cm spacing and RC ties were 9mm diameter and 20cm spacing. This shows that even for SRC members, shear resistance reinforcing for steel parts and reinforced concrete parts is very important. Generally SRC members are rather expensive compared to RC members.

Taking into consideration of the experimental and theoretical studies so far, we would recommend special SRC column section which is composed of RC sections and a central reinforcing steel element. This central reinforcing element is expected only for resisting axial force and not expected for resisting bending moment and shear force. So rather economical special SRC column sections are expected. We carried out so far experiments for the behaviors of RC members subjected to impact loading and for the behaviors of RC column members and special SRC column members with a central reinforcing element such as reinforcing bar, H section and pipe section. Some of the experimental results and theoretical results are shown.

2. IMPACT LOADING TEST

Earthquake forces are dynamical loading which are applied to structures with time variation mainly in lateral direction. As the extreme case of dynamical loadings, impact loadings were taken up and small scaled mortar specimens were used for experimental studies.

The mortar specimens are as shown in Fig. 2. The central prestressing 12mm steel bar was used to apply axial loading to the specimen by post tensioning. This prestressing bar was not grouted. About 50 specimens were tested giving variations to axial loads (P=0, 20, 40, 80kN), model lengths (L=60, 80, 100, 130cm), tie spacings (S=0, 2, 4, 20cm) and reinforcing bars (2- 6mm, 4- 6mm). The average compressive strength of mortar at 28days was 19MPa and average tensile strength of reinforcing bars was 450MPa.

To give impact loadings to specimens, the simple loading arrangement as shown in Fig. 3 was used. The specimens, starting from rest, fallen through several heights (H=30, 40, 50, 100cm) and struck a bottom steel block. The strains were measured with SR-4 electric resistance strain gauges pasted at 7 points on specimen's surface.

Portions of the experimental results are shown in Fig. 4 through Fig. 6. From Fig. 4 which shows crack patterns for the first shock for the falling height of 40cm, it is seen that, as the spacings of ties become closer and the axial forces become larger, cracks occur more concentrically near the fixed end. The bending cracks in Fig. 4 (c) and (f) are hair cracks.

Shear cracks near the fixed end were predominant and these cracks were the main cause of the final failures of specimens. One shock for the falling height of 40cm for the specimens without main reinforcing bars and with ties of long spacings and two or three repetitions of the same shock for other specimens were enough to cause complete failures. The destruction patterns during 0.1 second interval after the instant of impact loading which are shown in Fig. 5 are sketched taking into consideration the stress time variations shown in Fig. 6.

Observing these experimental results, it is presumed that when concrete columns are subjected to impact loadings shear failures near the fixed ends will likely to develop even for rather long columns and concrete columns will be of lower ductilities.

3. STATIC LOADING TEST

Here the experimental results for eight HSRC column specimens among about 60 specimens are shown. The variables which are considered to affect the behavior of RC columns subjected to axial load and cyclic shear load are the magnitude of axial load, tie ratio and main bar ratio. As a central reinforcing element, a reinforcing bar, a steel H section, a steel pipe without inside concrete and a steel pipe filled with inside concrete are considered.

For all the column specimens, the cross section is 20cmx20cm, the length is 100cm, the ratio of shear span to depth is 2.5. The configurations of the specimens are shown in Fig. 7. The variables which are considered in this study are as follows;

(a) Reinforcing element

MB: main bar (*Pt*=1.44%(4-D19), *Pc*=0%)
CB: central bar (*Pt*=1.00%(4-D16), *Pc*=0.97%(1-D22))
CP: central pipe (*Pt*=1.00%(4-D16), *Pc*=1.08%(steel pipe φ 42.7x3.5))
CH: central H section (*Pt*=1.00%(4-D16), *Pc*=2.20%(built H-50x50x5x7))

(b) Tie ratio

Pw=1.28%(2-9 φ @50) and *Pw*=0.56%(2-9 φ @110)

(c) Axial load ratio ($\sigma = N/(Fc \cdot Ac + Fs \cdot As)$)

0.4 and 0.2

where, "*Pt*" is tensile steel bar ratio, "*Pc*" is central reinforcing element ratio, "*Pw*" is tie bar ratio, "*Fc*" is compressive strength of concrete, "*Ac*" is cross sectional area of concrete, "*Fs*" is yield strength of steel and "*As*" is sectional area of steel.

Table 1 shows the properties of specimens. Specimen names are given in the order of [reinforcing element]-[tie ratio]-[axial load ratio]. The loading setup and the loading process are shown in Fig.8 and Fig. 9, respectively.

Table 2 shows experimental results. Figure 10 shows shear load-deflection relationships and Fig. 11 shows cracks patterns. In Figs. 10 and 11, the results for CB types are not shown.

From the experimental results, it is observed that a central reinforcing element is effective for keeping the ductility of the member. For lower axial force ($\sigma = 0.2$), MB type shows rather brittle behavior while CP type shows rather ductile behavior. As seen from crack patterns, the former type shows the tendency of bond slip failure and latter shows the tendency of concentrated shear and bending failure near the end portions of the member.

For higher axial force ($\sigma = 0.4$), the tie reinforcing effect becomes more important. For higher tie reinforcing (Pw=1.28%), the members show higher ductility and concentric crack occurrence near the end portions is prevented. In this case, the reinforcing effects of a central reinforcing element become higher.

Figure 12 shows the shear loading and axial displacement relationship. The ordinate of Fig. 12 shows the axial displacement increase after 1/100 deflection angle loading. From these figures, it is seen that the members with a central steel pipe and a steel H section show rather high ductility. Also from Fig. 12, it is clear that as a central reinforcing element, a steel pipe should be filled with inside concrete.

4. ANALYTICAL STUDY

The finite element method is applied for analytical study. One example of the analytical model is shown in Fig. 13. Concrete and the central reinforcing element are represented by constant strain triangular elements. Tensile steel and ties are represented by one-dimensional element. To express the stress and bond slip relationship between tensile steels and concrete, bond link element is used. To represent the behavior of cracking in concrete, the discrete cracking model modeled by two orthogonal springs between the cracks formed by separation of previously common element edges is used. Crack position is decided from experimental results.

Two examples for shear load-deflection relationships for analytical results and experimental result are shown in Fig. 14. The cracks which occurred near the end portions do not progress to the middle portion due to high axial load.

The distributions of principal stress at the near maximum shear load indicate that the model with central reinforcing element resists compression in wider area at the end than those in the model with ordinary reinforcing, because the principal shear directions of the model with central reinforcing element are more near to the vertical direction than those in the ordinary model. The high ductility of RC columns with central reinforcing element is expected

5. HSRC STRUCTURE

Taking into considerations of the experimental and theoretical results and basing upon the observations of the damages of RC structures during severe earthquakes, we recommend a structure which might be called Hybrid SRC(HSRC) structures for structures of low and middle height and with a few or without shear walls which might be subjected to very severe earthquakes.

It would be concluded that ordinary RC beams are generally earthquake resistant members because they are not subjected to large axial force. Also beam members are stiffened by slab members which are cast monolithically with beam members in most cases.

As a central reinforcing element for special SRC column members, central H section, central pipe section and central pipe section filled with inside concrete are considered. For central pipe section, it will be necessary to fill the inside with concrete. From the practical point of view, it would be rather difficult work at fields, so here we recommend steel H section for a central reinforcing element.

For the determination of the central steel H section, only permanent axial load and additional axial load due to earthquake force should be considered and in this case it is not necessary to consider buckling effect because the central H section is covered by concrete. Bending and shear resistances are expected for the portion of RC section. Of course, the central H section would add some bending and shear resistance. For the column-beam panel zone, the H section would be cut and in this case H sections may be connected by four bolts as shown in Fig. 15. By this connection, main bars of beams would be easily positioned as designed as RC members.

6. CONCLUSION

Within the scope of the above study and also being based on our engineering judgement from the past experiences, following conclusions would be obtained. It is expected that reinforced concrete columns with central reinforcing element become ductile enough for severe earthquake force. As a central reinforcing element, to apply a concrete filled steel pipe or a steel H section is advisable.

The minimum sectional area of a central steel section which is demanded for securing a safe aseismic column member would be the amount which can resist the axial force due to permanent load plus earthquake force.

The HSRC construction for middle and lower height buildings with a few or without shear walls which is recommended in this report would aseismic against severe earthquakes.

As a practical design, the idea of HSRC construction was applied to the structural design of an actual building. This building is for a research center at my university. This building is situated just behind the Naito Memorial Laboratory of Waseda University. This building is almost free from aseismic walls and all the reinforced concrete columns are strengthened by a central steel H section. The plan and the section of this building are as shown in Fig. 16 and in this figure the section of the column is also shown.

7. REFERENCE

Tanaka,Y., Yashiro, H., Ro, Y. and Sato, N.(1992), "Study on the formation of plastic hinges and the failure of reinforced concrete columns", 10 WCEE Proceedings, Madrid, 2977-2982.
Tanaka,Y., Ro, Y. Nakagawa. O. and Kawahara, T.(1995), "Strengthening of reinforced concrete columns by central reinforcing element", The International Conference on Earthquake Engineering, Jordan.
Tanaka,Y., Ro, Y. Nakagawa. O. and Kawahara, T.(1996), "Study on reinforced concrete columns with central reinforcing element", 11 WCEE Proceedings, Acapulco, Ref. No. 00744.

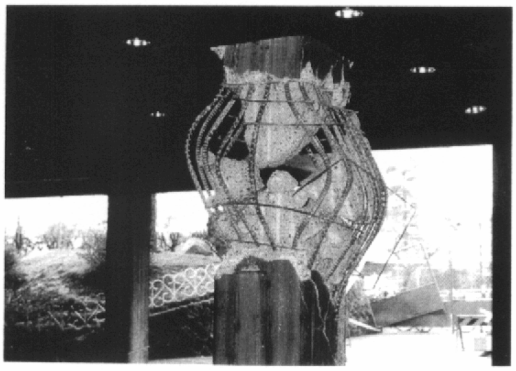


Photo 1 Damage of RC column

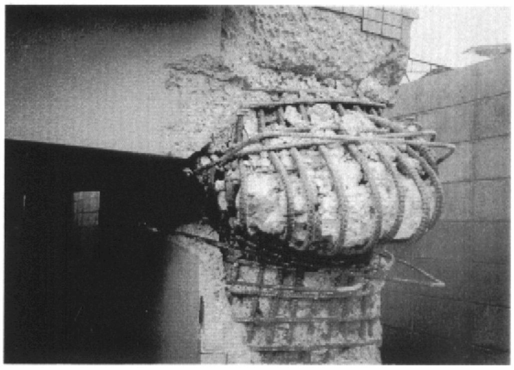


Photo 2 Damage of RC column

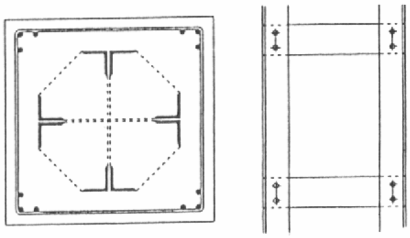


Fig. 1 SRC column section and tie plate arrangement

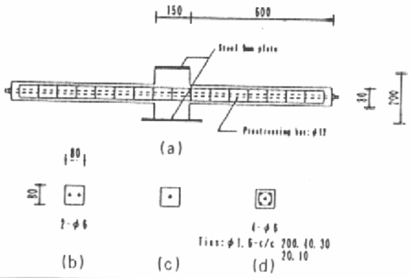


Fig. 2 Mortar specimen (unit:mm)

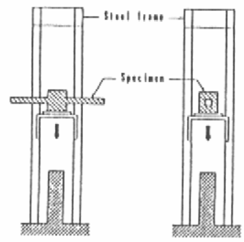


Fig. 3 Impact loading arrangement

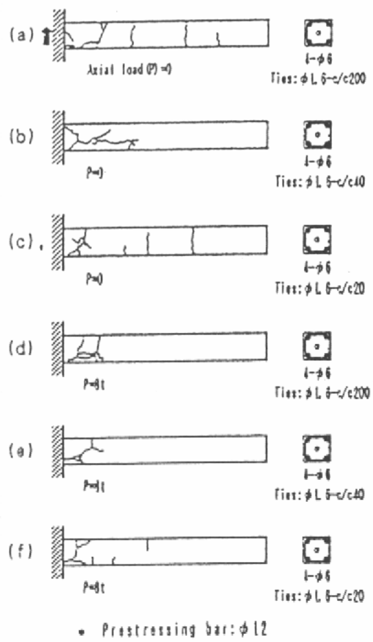


Fig. 4 Crack patterns

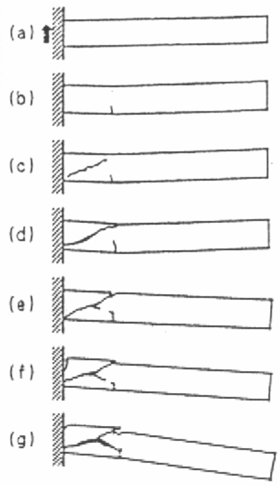


Fig. 5 Destruction patterens

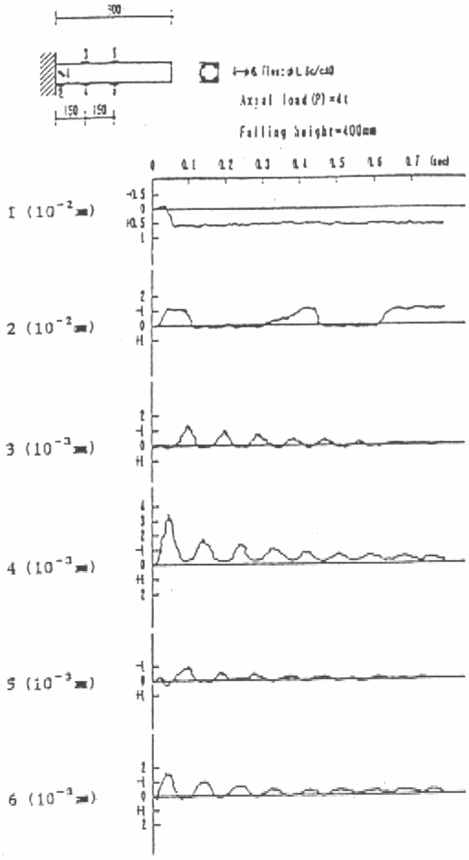
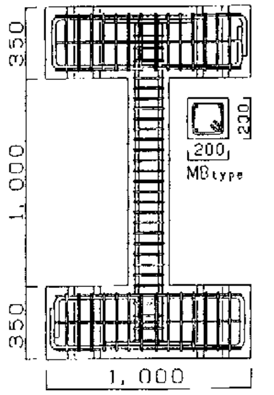
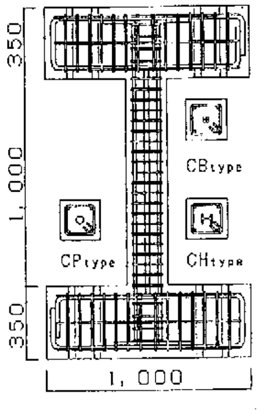


Fig. 6 Strain time relations



(a) Ordinary reinforcing type



(b) Central reinforcing type

Fig. 7 Test specimen (unit:mm)

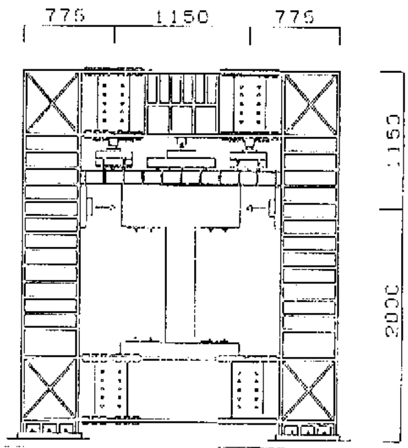


Fig. 8 Loading setup (unit:mm)

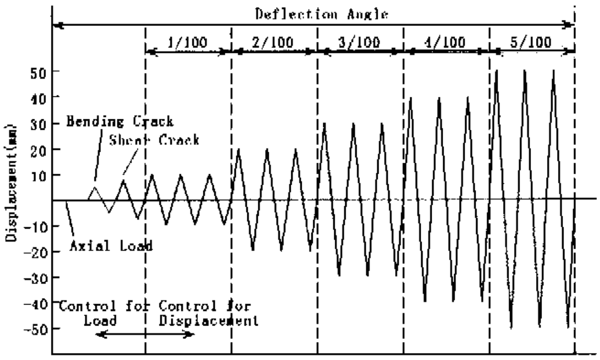


Fig. 9 Loading process

Table 1. Properties of specimens

Specimen	Strength of concrete(*Mpa*)		Axial load ratio	Axial load	Tie ratio	Tie pitch	Tensile steel ratio	Central bar ratio	Section steel ratio
	Compression	Tension	*(%)*	*(kN)*	*(%)*	*(mm)*	*(%)*	*(%)*	*(%)*
MB-1.28-0.2	35.4	3.03	0.2	354	1.28	50	1.44	-	2.88
CB-1.28-0.2	35.4	3.03	0.2	356	1.28	50	1.00	0.97	2.97
CP-1.28-0.2	35.4	3.03	0.2	351	1.28	50	1.00	1.08	3.08
MB-0.56-0.4	37.5	3.07	0.4	740	0.56	100	1.44	-	2.88
CB-0.56-0.4	37.5	3.07	0.4	745	0.56	100	1.44	0.97	2.97
CP-0.56-0.4	37.5	3.07	0.4	735	0.56	100	1.00	1.08	3.08
MB-1.28-0.4	33.4	3.02	0.4	676	1.28	50	1.44	-	2.88
CB-1.28-0.4	33.4	3.02	0.4	680	1.28	50	1.00	0.97	2.97
CP-1.28-0.4	33.4	3.02	0.4	672	1.28	50	1.00	1.08	3.08
CP-1.28-0.4*	24.5	2.57	0.4	549	1.28	50	1.00	1.08	3.08
CH-1.28-0.4	24.5	2.57	0.4	613	1.28	50	1.00	2.2	4.20

CP-1.28-0.4*; a steel pipe filled with inside mortar

Table2. Experimental results

Specimen	Shear losf(*kN*)										
		Displacement/Length									
	Maximum	1/100		2/100		3/100		4/100		5/100	
MB-1.28-0.2	102.90	95.06	-90.16	102.90	-100.94	87.22	-91.14	59.78	-60.76	33.32	-33.32
CB-1.28-0.2	96.04	86.24	-88.20	80.36	-86.24	66.64	-75.46	56.84	-57.82	53.90	-55.86
CP-1.28-0.2	102.90	93.10	-1086.24	95.06	-92.12	75.46	-91.14	64.68	-66.64	57.82	-58.80
MB-0.56-0.4	108.78	91.14	-108.78	48.82	-35.28	-	-	-	-	-	-
CB-0.56-0.4	103.88	92.12	-102.90	54.88	-64.68	-	-	-	-	-	-
CP-0.56-0.4	107.80	107.80	-96.04	55.86	-54.88	-	-	-	-	-	-
MB-1.28-0.4	99.96	94.08	-97.02	75.46	-89.18	64.68	-75.46	58.80	-32.34	-	-
CB-1.28-0.4	93.10	89.18	-86.24	71.54	-65.66	60.76	-61.74	55.86	-47.04	-	-
CP-1.28-0.4	98.00	97.02	-87.22	78.40	-65.66	62.72	-56.84	62.72	-57.82	-	-
CP-1.28-0.4*	96.04	96.04	-88.20	83.30	-70.56	69.58	-57.82	64.68	-52.92	57.82	-46.06
CH-1.28-0.4	101.92	100.94	-93.10	90.16	-76.44	74.48	-55.86	69.58	-46.06	-	-

CP-1.28-0.4*; a steel pipe filled with inside mortar

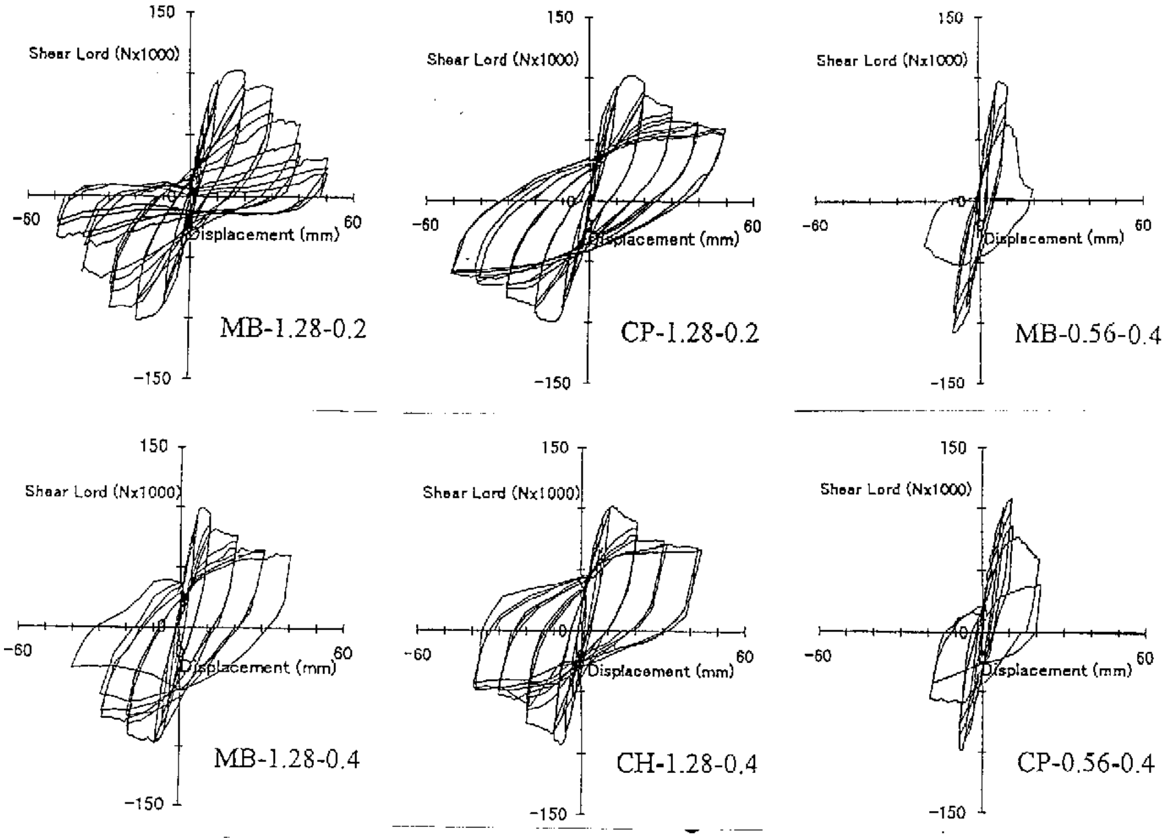


Fig. 10 Shear load-deflection relationship

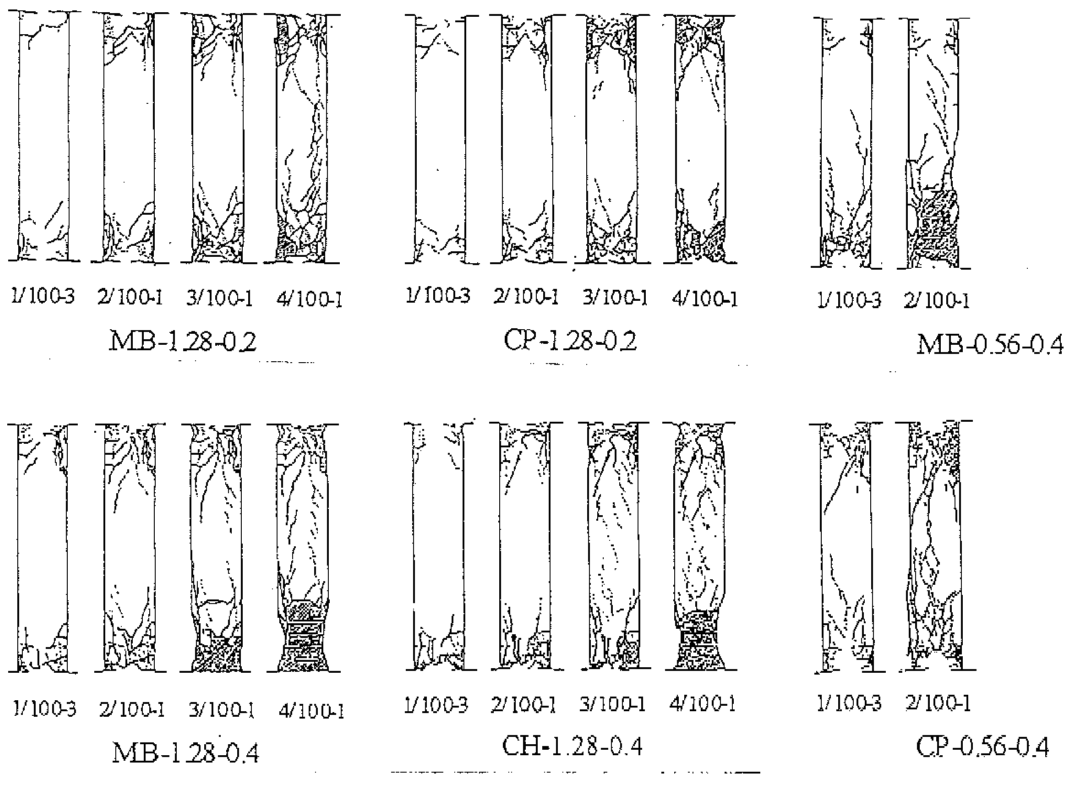


Fig. 11 Crack patterns

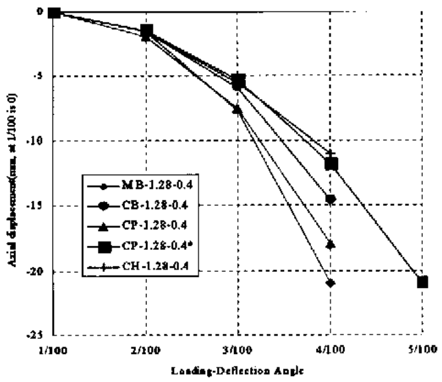


Fig. 12 Load cycle and axial displacement relationship

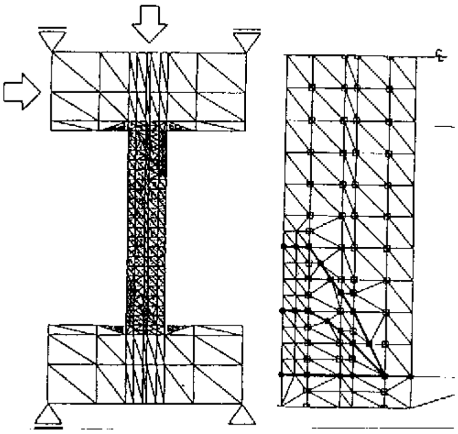


Fig. 13 Analytical model

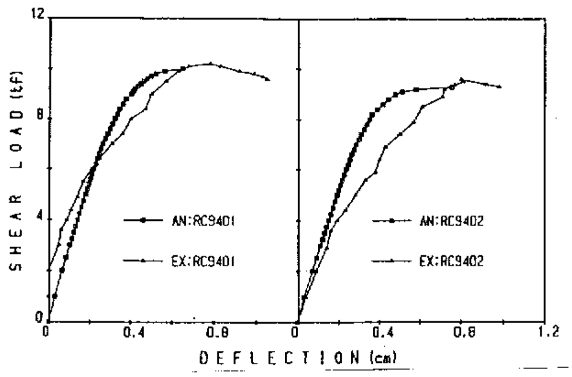


Fig. 14 Shear load-deflection relationship

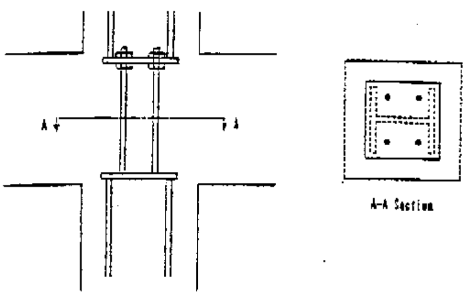
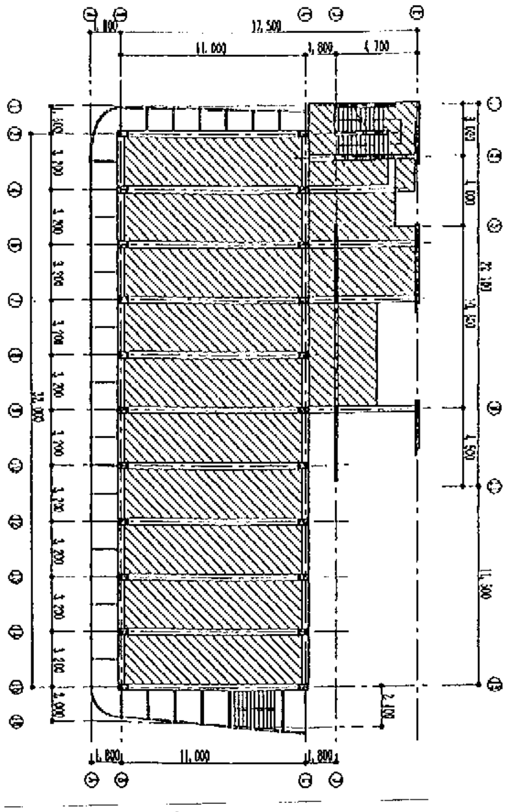
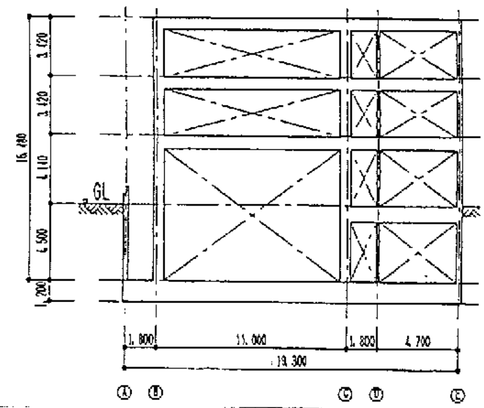


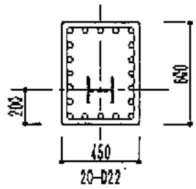
Fig. 15 Panel zone detail of HSRC column



(a) 2nd floor beam plan



(b) 6th row frame



H - 150×150×7×10

Hoops: D13−100@

(c) 1st story column section

Fig. 16 HSRC Waseda Laboratory (unit:mm)