Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Research on earthquake resistant capacity of composite structures using high strength steel

Minoru Wakabayashi
General Building Research Corporation of Japan, Osaka, Japan

ABSTRACT: This paper describes the problems to be resolved for enhancing the use of high strength steel in SRC structures and major results obtained from a comprehensive research program to quantify the characteristics of SRC members combined with high strength steel. Procedures to estimate such SRC members are also presented.

1 INTRODUCTION

Concrete encased steel (referred to as SRC) structures have been extensively used in Japan, particularly since this type of structures exhibited good earthquake-resisting performance under the Great Kanto earthquake of 1923. Advantages of SRC structures are manyfold; large ductility can be achieved thanks to the ductile steel components, local buckling of the steel plates are retarded significantly because of the confinement provided by the surrounding concrete, and the cross section can be reduced because of the high strength provided by the steel, all of which provide SRC structures with high earthquake-resisting performance. At the present time, in Japan, steels allowable for used in SRC structures are limited to 520 MN/m^2 in strength (the Architectural Institute of Japan 1987), but use of steels having a higher strength can make the construction of SRC structures more economical and lead to more flexible structural planning. To realize the construction of SRC structures with high strength steels, we should answer the following fundamental problem: i.e., whether or not combining a steel having a larger yield strain with concrete whose yield strain is rather limited can still ensure full composite action in SRC structures? In order to enhance the use of high strength steels in SRC structures, a three year comprehensive research project was conducted. The writer served as Chairman of the committee, with nine research organizations participated in this project. This paper presents the background and objectives of the project, outline of the research program, and major results obtained from the study. This paper includes design procedures proposed for the design of SRC structures with high strength steels.

2 AREAS OF RESEARCH NEEDED

Based on a preliminary survey, the following problems were identified to be investigated to promote the use of high strength steels in SRC structures.

2.1 Method to estimate ultimate strength

It is known that the ultimate strength of a SRC member with a mild-steel can be estimated accurately, if the stress versus strain relationships of the steel and concrete are given and the assumption that the plane section remains plane after deformation is employed. We should examine if this method is still practicable for a SRC member with a higher strength steel. In Japan, the superposed strength method has been employed to estimate the ultimate strength of a SRC member because of its simplicity and reasonable accuracy (Wakabayashi 1987). In this method, the strengths of the steel and reinforced concrete components of a SRC member are computed separately, and the sum of the two strengths is taken as the strength of that member. Since a high strength steel is significantly larger in the yield strain than a normal mild-steel, it is imperative to investigate if this method is still applicable without causing an unsafe estimate of the strength when a high strength steel is used. In Japan, the superposed strength method is also used for estimating the shear strength of a SRC member, and this procedure should also be calibrated

2.2 Evaluation of ductility

Ductility is an important index in earthquake-resistant design, and use of a higher strength steel is likely to change the ductility characteristics of SRC members. Effects of the yield stress and yield ratio (defined as the yield stress divided by the ultimate strength) of steels, axial force imposed, width-to-thickness ratio of steel plates on the ductility of SRC members should be quantified.

2.3 Reinforcing effective to SRC members

When a high strength steel is used, the ductility required to reinforced concrete increases if a full composite action is to be achieved. Reinforcing details that ensure sufficient ductility but still are constructable should be established.

2.4 Connections effective to SRC members

Deformation capacity of a SRC member is likely to be reduced at its connections because of a relatively large yield ratio of high strength steel. Reinforcement to avoid premature failure at such connections should be established.

2.5 Long term deflection and creep of SRC beams

When a high strength steel is employed for a SRC beam, the beam is likely to become more slender, which may make its long-term deflections more critical in design. Range of long-term deflections and crack widths expected in such SRC beams should be evaluated.

3 OUTLINE OF TESTS

Steels shown in Table 1 were chosen in this study. The ultimate strength was either 590 MN/m^2 (the target in this study), 390 MN/m^2 (a mild-steel used extensively), and 780 MN/m^2 (a very high strength steel). The latter two were selected for comparison purposes. Further, three yield ratios: 0.7, 0.8, and 0.9, were selected. Table 1 lists the combination of the ultimate strength and yield ratio of steels used in this study, also denoting the designations of respective steels. Figure 1 shows the stress versus strain curves of these steels, obtained from coupon tensile tests. Normal concrete was used with the strength of either 20 or 29 MN/m^2. Table 2 summarizes the test programs, in which, in addition to tests of SRC members and connection, tests of steel and reinforced concrete members and connections were performed for the sake of comparison. This table also indicates the number, shape, and dimensions of the specimens tested.

4 TEST RESULTS

4.1 Beams

Tests were carried out for simply supported beams (shown in Table 2), with one way (monotonic) loading. Load versus deflection curves of SRC beams with a high strength steel were found as stable as those of SRC beams with a mild-steel, exhibiting large deformation capacity. The correlation of the curves with those obtained by numerical analysis, in which stress versus strain relationships of the steel and concrete were combined with the assumption that the plane section remain plane after deformation, were also satisfactory.

Table 1 Material Properties of Steel

Yield Ratio		0.7	0.8	0.9
Ultimate	390	47		
Strength	590		68	69
(MN/m^2)	780			89

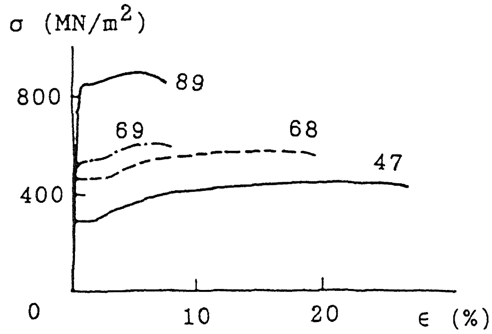


Fig.1 Stress vs. Strain Curves of Steel

Because significant portion of the steel component along the length remained elastic at the deflection corresponding to the maximum strength, the strength estimated by the superposed strength was larger than the experimental strength, but the discrepancy was not more than 10 percent.

4.2 Columns in compression

4.2.1 Columns subject to concentric loading

The test results indicated that the superposed strength method is useful if the ultimate strength of steel is 590 MN/m^2 or smaller but provides an unsafe estimate when it is 780 MN/m^2. Strength degradation after reaching the maximum strength was observed less significant for columns with a higher strength steel. Further, it was found that the width-to-thickness ratio of steel plates on the load versus deflection curves affected only in the unstable range.

4.2.2 Columns subject to eccentric loading

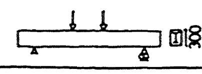
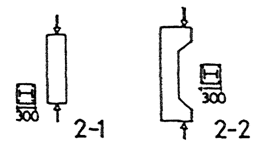
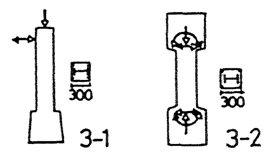
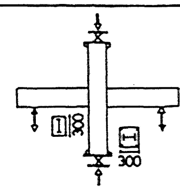
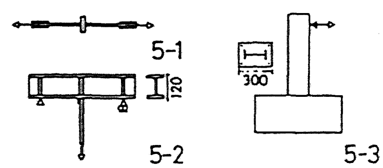
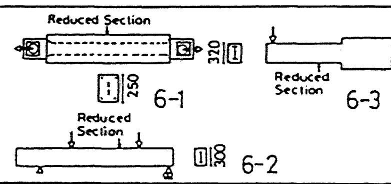
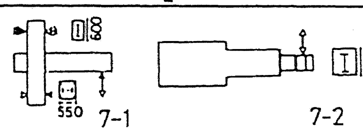
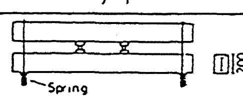
Force versus deflection curves of SRC columns subjected to eccentric compression were approximately the same regardless of the type of steel, but strength degradation in the unstable range was more conspicuous for columns with a lower strength steel. As shown in Fig.2, the ultimate strength estimated by considering the compatibility of strains between the steel and concrete components is always smaller than the experimental strength, but the strength estimated by the superposed strength method is larger than the experimental strength for columns with a steel whose ultimate strength is 780 MN/m^2. It was found that the superposed strength method requires amendment if it is to be employed for estimating the strength of a SRC column whose steel component has 780 MN/m^2 or larger in the ultimate strength.

4.3 Members subjected to constant axial force and repeated bending

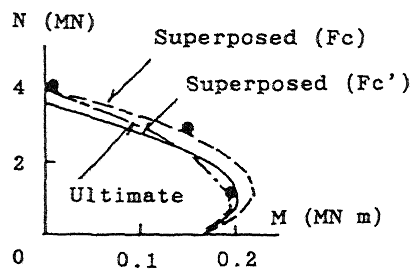
4.3.1 Members failed in flexure

The tests showed that force versus deflection curves are

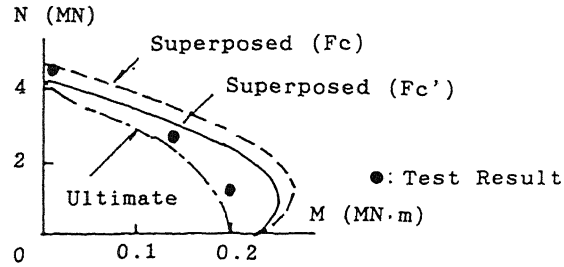
Table 2 Summary of Test Programs

ID of Test	Outline of Tests		Number of Specimens SRC	S(RC)	Dimensions of Specimens (unit:mm)
0	Material Behavior		-	60	
1	Beams in Flexure		20	4	
2-1	Columns in Compression	Concentric Loading	20	11	
2-2		Eccentric Loading	30	-	
3-1	Columns in Axial and Shear	Failed in Flexure	22	12	
3-2		Failed in Shear	30	-	
4	Beam-to-Column Connections		9	7	
5-1	Ductility of Welded Connections	+ - Shaped Connections (S)	-	12	
5-2		T - Shaped Connections (S)	-	15	
5-3		SRC Connection	12	-	
6-1	Ductility of Members in Reduced Section	in Tension	24	12	
6-2		in Uniform Moment	11	4	
6-3		in Moment Gradient	8	-	
7-1	Ductility of Connections with Reduced Section	Ductility Behavior	6	-	
7-2		Effect of Reinforcement	6	-	
8	Creep Behavior of Beams		6	(2)	

stable as shown in Fig.3 unless the axial force imposed was very large. In Fig. 4, the skeleton curves obtained from corresponding tests for steel and SRC members are compared for steels: '47' and '89' (see Table 1 for notation). In steel: '89', the steel component yielded at the deflection twice as much as that corresponding to the crushing of concrete, which had resulted in a large portion of stable range designated as B in Fig.4(b). The superposed strength method was found applicable to those with steels having 390 and 590 MN/m^2 but provided an unsafe estimate for those with a steel having 780 MN/m^2.



(a) Steel: 68



(b) Steel: 89

Fig.2 Columns Subject to Eccentric Loading

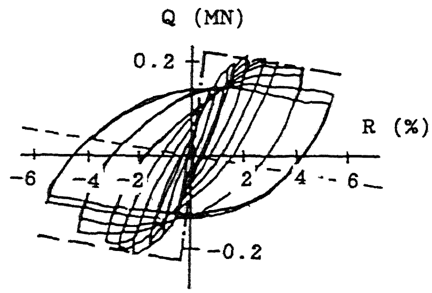


Fig.3 Response of Column Failed in Flexure

4.3.2 Members failed in shear

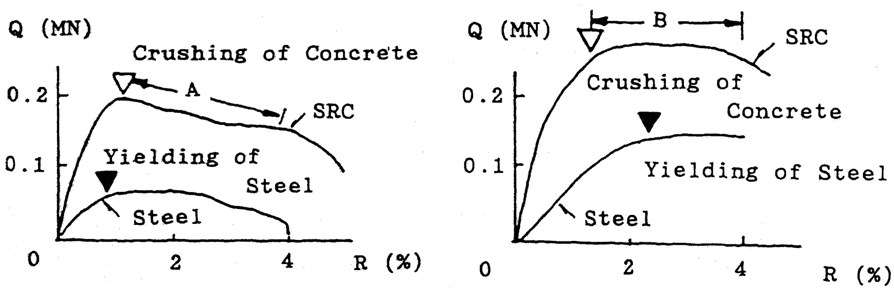
Force versus deflection curves of SRC members failed in shear are of typical Masing-type as shown in Fig.5. Figure 6 shows the skeleton curves of SRC members with steel: '47', '69', and '89'. In this figure, $_{SRC}Q$ denotes the strength of the SRC member (obtained from the test), $_{S}Q$ the strength resisted by the steel component (which was estimated from the measured strains), and $_{RC}Q$ the difference between $_{SRC}Q$ and $_{S}Q$ and taken as the strength resisted by the reinforced concrete component. The black circle shows the value of $_{RC}Q$ at the deflection corresponding to the maximum $_{SRC}Q$. This figure indicates that the steel component had yielded when the member reached the maximum strength. At the maximum strength, $_{RC}Q$ also reached its maximum for steel '47', but exceeded the maximum for steels '69' and '89'. This observation indicates that the maximum strength of the SRC member is smaller than the sum of the maximum strengths of the steel and reinforced concrete components, thus invalidating the superposed strength method. To amend this discrepancy, a reduction factor (ϕ_r) may be introduced for estimating the strength of the reinforced concrete component. In this study, it is proposed that the value of this factor should be set at 1.0, 0.9, and 0.7 for steels with 390, 590, and 780 MN/m^2 in ultimate strength respectively. In some tests, L-shaped hoops were used instead of typical rectangular hoops. No clear difference in behavior was observed between them, and full bonding was achieved up to the deflection corresponding to 8 x 10^{-3} in the drift angle.

4.4 Beam-to-column connections

Beam-to-column connections were tested in a repeated loading condition. Hysteresis curves obtained exhibited stable behavior regardless of the type of steels. Present design provisions for SRC connections (the Architectural Institute of Japan 1987) were found applicable to those with a high strength steel.

4.5 Ductility of members with reduced cross section

Figure 7 shows force versus deflection curves of SRC beams sustaining uniform moment and having a section where the cross sectional area of the steel component was reduced to 50 percent. If the section was reinforced by reinforcing bars so that 80 percent of the strength that



(a) Steel: 47

(b) Steel: 89

Fig.4 Skeleton Curves of Columns Failed in Flexure

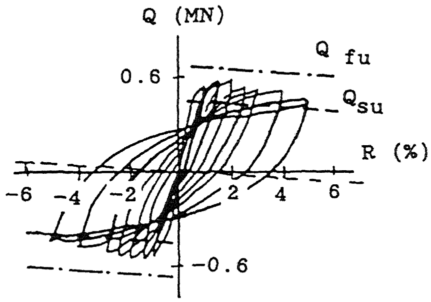


Fig.5 Response of Column Failed in Shear

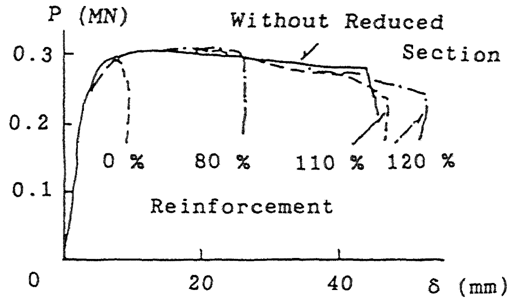


Fig.7 Ductility of Members in Reduced Section

should have been resisted by the steel area removed was compensated for, strength of the section was fully recovered, and if it was reinforced by 110 percent, ductility of the section was also recovered. Furthermore, if the moment applied varied along the length, reinforcement by 120 percent ensured full recovery of both strength and ductility even under load reversals. When the steel section was corped at the beam-end, it was found difficult to move the critical section where a plastic hinge occurs even with additional reinforcement. Figure 8 shows hysteresis curves of beam-to-column connections whose ends were corped. Ductility decreased when a high strength steel was employed. If the corped section was reinforced by stirrups and additional U-shaped bars, ductility improved (Fig.9), but the member still failed in that section.

4.6 Creep behavior of SRC beams

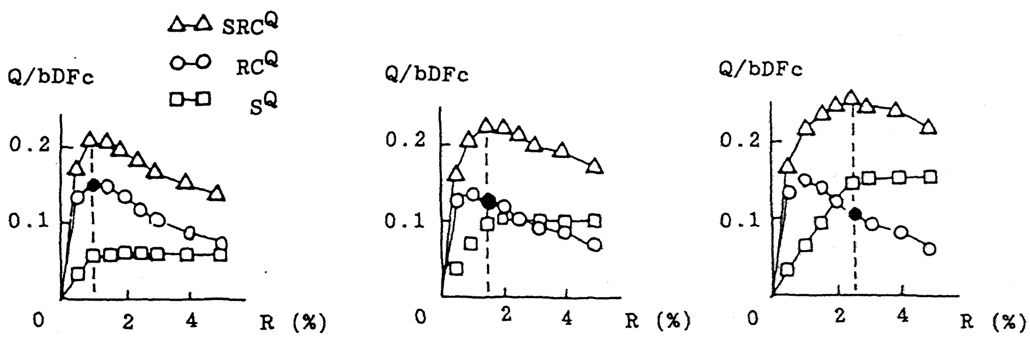
Creep tests for SRC beams indicated that the creep coefficient remained unchanged even if a high strength steel was employed. The long-term deflection of a SRC beam can be estimated by assuming it as an equivalent RC beam. As an alternative, the deflection can be estimated by assuming that the SRC beam has a stiffness given as the sum of the stiffness of the reinforced concrete component (considering the effect of creep) and the stiffness of the steel component (without the effect of creep).

5 AMENDMENT OF SRC DESIGN PROVISIONS

The superposed strength method was found to underestimate the strength of SRC members whose steel is 590MN/m^2 or larger in ultimate strength. In this study, the following procedure is proposed for applying this method to estimate the strength of such SRC members. The strength of a SRC member is given as the superposed strength obtained by assuming that the concrete strength in compression is β (a value less than unity) times the nominal concrete strength (that used for SRC members with a mild-steel), or the superposed strength obtained by assuming that the ultimate strength of the steel in compression is limited to 390 MN/m^2, whichever is greater. Figure 10 shows an example of the results obtained by this amended procedure.

6 CONCLUSIONS

1 For the flexural strength of a SRC member, the superposed strength method is applicable if the steel does not exceed 590 MN/m^2 in ultimate strength. For the shear strength, the method provides an unsafe estimate when the ultimate strength of steel is 590 MN/m^2 or larger. Amendment of the superposed strength method has been proposed so that it can be utilized for SRC members with steels having a higher ultimate strength.
2 SRC members subjected to axial force and repeated

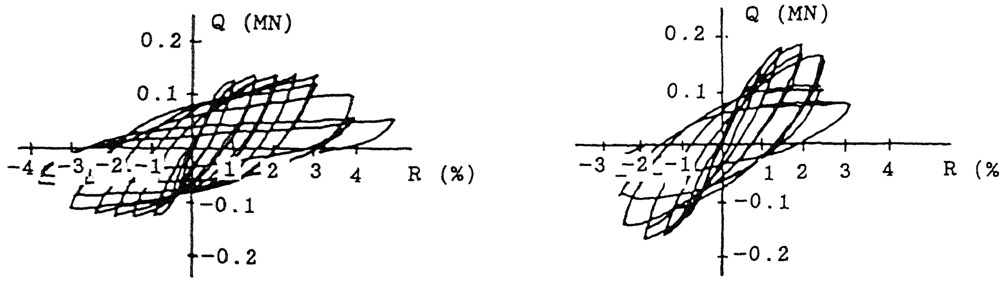


(a) Steel: 47

(b) Steel: 69

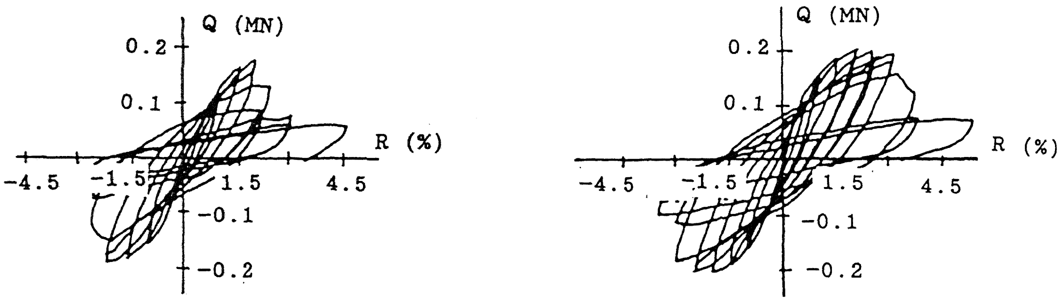
(c) Steel: 89

Fig.6 Skeleton Curves of Columns Failed in Shear



(a) Steel: 47 (b) Steel: 69

Fig.8 Hystereses of Beam-to-Column Connections



(a) Without Reinforcement (b) With Reinforcement

Fig.9 Effect of Reinforcement in Members With Reduced Section

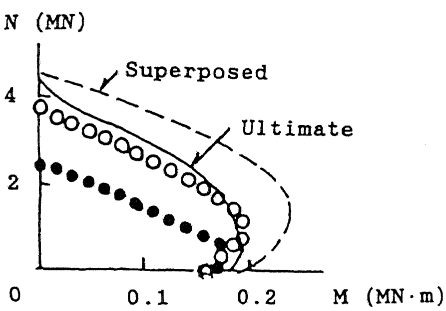


Fig.10 Amendment of Superposed Strength Method

shear, failed in either flexure or shear, are more ductile when the steel has a higher ultimate strength.

3 If a SRC member has a section whose cross sectional area of the steel component is reduced, ductility decreases particularly when the yield ratio is large. Reinforcement at such a section can improve ductility, making possible full recovery in ductility when the section is located within the span, but it is difficult to achieve full recovery of the ductility when the section is corped at a beam-to-column connection.

4 Hoops arranged in a L-shape are found to be effective for increasing ductility.

REFERENCES

Architectural Institute of Japan. 1987. *AIJ standards for structural calculation of steel reinforced concrete structures.*.

Wakabayashi, M. 1987. Japanese standards for the design of composite buildings. Proc. of Engineering Foundation Conference on Composite Construction in Steel and Concrete: 53-70. Henniker: New Hampshire.

ACKNOWLEDEMENT

The writer is grateful for the support and assistances given by the Kozai Club, Japan, and those participated in this research project.