Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.IV)

6-7-6

STRENGTH AND BEHAVIOR OF SRC BEAM-COLUMNS USING HIGH-STRENGTH STEEL

Keigo TSUDA[1], Chiaki MATSUI[2] and Guanhua JIANG[3]

[1-3] Department of Architecture, Faculty of Engineering,
Kyushu University 36, 6-10-1 Hakozaki, Higashi-ku, Fukuoka-shi 812, Japan

SUMMARY

In order to study the strength and behavior of steel-reinforced concrete (SRC) beam-columns using high-strength steel, twelve SRC cantilever columns are tested under constant vertical and cyclic horizontal loads, with varying values of vertical load ratio, amount of hoops and steel area ratio. The paper presents test results and discusses load-deflection behavior of SRC beam-columns with a high-strength steel, by comparing the test results with the results of elasto-plastic analysis.

INTRODUCTION

While steel grades used in SRC structures in Japan are 40 and 50 kg / mm^2 in the ultimate tensile strength (Ref. 1), use of higher strength steel is expected for a more economical design of composite tall building frames subjected to combined gravity and earthquake or wind loads. There are several problems, however, to be solved in application of high-strength steel to SRC structures.

In Japan, the method of superposed strength as a sum of the strength contributed from reinforced concrete portion and steel portion is used for strength calculation (Ref. 1). High-strength steel portion will be supposed to remain elastic at the state of maximum strength of reinforced concrete portion, and then the method of superposed strength may yield errors on the unsafe side, although this method gives a lower-bound value for a column composed of very ductile materials. Moreover, in SRC Standard of AIJ(Architectural Institute of Japan), a limiting value of vertical load on a column is specified for beam-columns which require the deformation capacity. The limiting value which is obtained from test results ensures the deformation capacity 1/100 of rotation angle of a member. It is not clear, however, whether the limiting value is valid or not for the SRC beam-columns using high-strength steel.

From this point of view, the paper presents the results of experimental studies on the behavior of SRC beam-columns using high-strength steel under the constant vertical and alternating horizontal loads, and discusses elasto-plastic behavior , maximum strength and strain behavior by comparing the test results with the results of theoretical analysis.

TEST PROGRAM

The shape and loading conditions of the specimen are shown in Fig. 1. The bottom of the specimen is fixed to rotation and the top is free to rotation and sway displacement. A H-shape steel is encased in a square concrete section of 100×100 mm with 4 deformed bars of a diameter 6 mm. Ends of H-shape and longitudinal reinforcing bars are welded to base plates, and round bar hoops of a diameter 2.6 mm are placed with a pitch of 100 mm or 50 mm. The length of the specimen from fixed end to the point of hinge is 600 mm. All specimens are designed so that they fail in flexure.

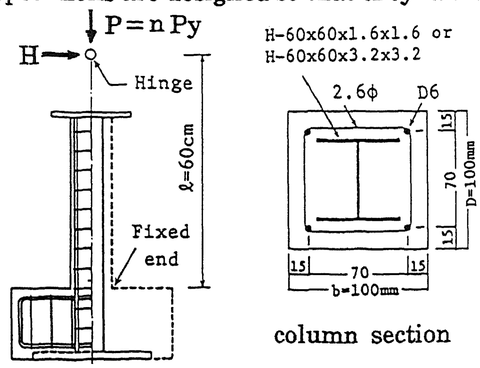


Fig. 1 Test specimen

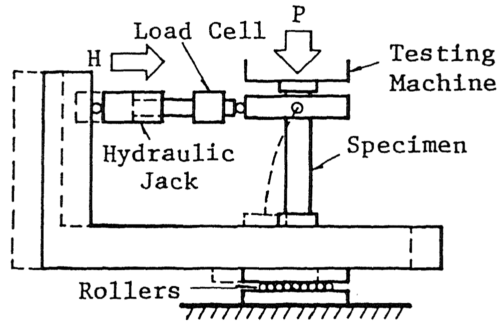


Fig. 2 Loading apparatus

Table 1 Test program

No.	Name	n	sA/A (%)	wp (%)	P (ton)	Nℓ (ton)	P/Nℓ	Fc (kg/cm²)
1	A1M	0.1	2.8	0.1	6.8	27.4	0.25	435.8
2	A3M	0.3	2.8	0.1	20.3	27.4	0.74	440.1
3	A5M	0.5	2.8	0.1	33.8	27.3	1.24	436.2
4	A1N	0.1	2.8	0.2	6.7	27.3	0.25	436.8
5	A3N	0.3	2.8	0.2	20.3	27.3	0.74	438.8
6	A5N	0.5	2.8	0.2	33.2	27.0	1.23	424.2
7	B1M	0.1	5.6	0.1	6.3	29.3	0.22	312.8
8	B3M	0.3	5.6	0.1	20.2	30.6	0.66	353.3
9	B5M	0.5	5.6	0.1	33.2	30.4	1.09	342.3
10	B1N	0.1	5.6	0.2	6.4	29.4	0.22	313.4
11	B3N	0.3	5.6	0.2	19.3	29.6	0.65	329.7
12	B5N	0.5	5.6	0.2	32.8	30.0	1.09	337.3

n : Vertical load ratio
sA: Area of steel portion
A : Area of cross section
wp: Area of shear reinforcement within a distance x
x : Spacing of shear reinforcement
P : Constant axial load of a specimen
Nℓ: Limiting value of axial load

Table 2 Material properties of steel

		σy (t/cm²)	σu (t/cm²)	σy/σu	εy (%)	εst (%)	εb (%)
Steel plate	1.6	6.42	6.98	0.92	0.31	2.04	11.8
	3.2	5.03	6.09	0.83	0.24	0.63	16.3
Steel bar	D6	4.73	6.93	0.68	—	—	14.9
	2.6φ	1.86	3.12	0.60	—	—	23.9

σy : Yield stress
σu : Tensile strength
σy/σu : Yield ratio
εy : Yield strain
εst: Strain at start of strain hardening
εb : Maximum elongation

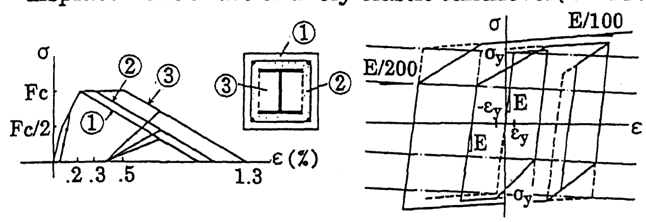
The parameters of test selected are the vertical load ratio n (=P / Py), amount of hoops $_{w}p$ ($=_{w}a$ / (b·x)) and steel area ratio $_{s}p$ ($=_{s}A$ / (b·D), ratio of steel area to total area of cross section) (see Fig. 1), and they vary as follows: n=0.1, 0.3, 0.5, $_{w}p$=0.1 and 0.2%, $_{s}p$=2.8% (type A) and 5.6% (type B). Test conditions of specimens are shown in Table 1, together with cylinder strength of concrete Fc. Table 2 shows yield stress σ_y and tensile strength σ_u of steel plates and reinforcing bars obtained from coupon tests. Yield stress of the high-strength steel plate for H-shape is 5.0 or 6.4 t/cm², and the ratio of yield stress σ_y to ultimate stress σ_u is 0.8 or 0.9. Total twelve specimens are tested.

The experimental apparatus is shown in Fig. 2. The vertical load P was applied to a specimen by the testing machine and kept constant value assigned in the test program during horizontal loading process. The horizontal load was applied to the top of a specimen by the hydraulic jack. As to the horizontal loading program, specimens are tested under the cyclic loading, where the amplitude of horizontal displacement δ is increased by 1/200 of column height ℓ in a stepwise manner every two cycles of loading completed.

THEORETICAL ANALYSIS

A theoretical analysis is performed to obtain the load-deflection curves of cantilever beam-columns under the cyclic bending.

Assumptions of Theoretical Analysis Prime assumptions are as follows : 1) Plane section remains plane after bending. 2) The local buckling of steel and the buckling of reinforce do not happen. 3) The tensile strength of concrete is neglected and the stress-strain relationships for compression are assumed as shown in Fig 3 (a) (Ref. 2). Here, the strain at the maximum stress is assumed to be $\varepsilon_0=0.2\%$, the ascending portion of the curve is represented by two lines, and the falling branch of the curve will be assumed to be three types because of the containment effect of the steel flanges and hoops. The hysteretic stress-strain relationship of steel is assumed to take Bauschinger effect into account, as shown in Fig. 3 (b). 4) The mathematical model of a column for the deflection analysis is assumed to be the one as shown in Fig.4, being composed with a bending part and a rigid part (Ref. 3). In the bending part, a curvature ϕ, which corresponds to the bending moment at the fixed end, is assumed to be uniformly distributed along the length ℓ_p. The value of ℓ_p would be given as $\ell_p=\ell/3$, which is so determined that the horizontal displacement on tip of this model ($\delta=\phi\cdot\ell_p\cdot\ell=M/EI\cdot\ell_p\cdot\ell$) coincides with the exact tip displacement of the entirely elastic cantilever($\delta=M/3EI\cdot\ell^2$).



(a) Concrete (b) Steel

Fig. 3 Stress-strain relationships of materials

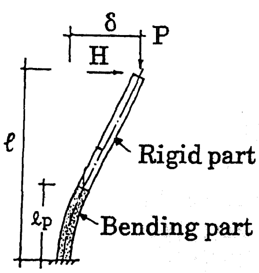


Fig. 4 Model of member

Method of Theoretical Analysis Based on the above assumptions, a procedure for obtaining the load-deflection relation is as follows : 1) A cross section is idealized as a geometric assemblage of fibers under a uniaxial state of stress (Ref. 2 and 3). On the basic assumption that plain section remains plane, the relation among the increments of the axial force (dP) and the bending moment (dM), the strain increment on the central axial ($d\varepsilon_0$) and the curvature increment ($d\phi$), can be defined as the equation (1), where A_i is the area of a fiber (i), E^t_i is the tangent modulus of the individual fiber (i), y_i is the distance from the central axial to the fiber (i), m is the number of fibers. 2) On the model of cantilever, the relation of the horizontal load H and the horizontal tip displacement δ can then be calculated in the equations (2) and (3).

$$\begin{Bmatrix} dM \\ dN \end{Bmatrix} = \begin{bmatrix} \sum_{i=1}^{m} A_i E_i^t y_i^2 & \sum_{i=1}^{m} A_i E_i^t y_i \\ \sum_{i=1}^{m} A_i E_i^t y_i & \sum_{i=1}^{m} A_i E_i^t \end{bmatrix} \begin{Bmatrix} d\phi \\ d\varepsilon_0 \end{Bmatrix} \qquad (1)$$

$$\delta=\phi\cdot\ell_p\cdot(\ell-\ell_p/2) \qquad (2)$$

$$H=(M-P\cdot(\delta-\phi\cdot\ell_p^2/4))/(\ell-\ell_p/2) \qquad (3)$$

RESULTS AND DISCUSSION

Inelastic Behavior Figure 5 shows the relations between the horizontal load H and the horizontal deflection δ at the tip of the cantilever specimen. In each figure the dash-dot(a), broken(b), and dotted line(c) indicates the plastic collapse mechanism line which is obtained by assuming a plastic hinge forming at the base of the beam-column. Full plastic moment used for obtaining the mechanism line (a), (b) and (c) are corresponding to the stress distribution (a),(b),(c) of cross section shown in Fig. 6, respectively. In the case of (a), compressive strength "Fc" of concrete and yield stress "σ_y" of steel is assumed for the stress distribution. In the stress distribution (b) and (c) of the concrete, the magnitude of compressive stress is reduced by multiplying the compressive strength Fc by the factor ${}_{c}r_{u}$ to compensate the error owing to the method of superposed strength (Ref. 1). The magnitude of stress (c) in the steel is reduced as such a way performed for steel structure

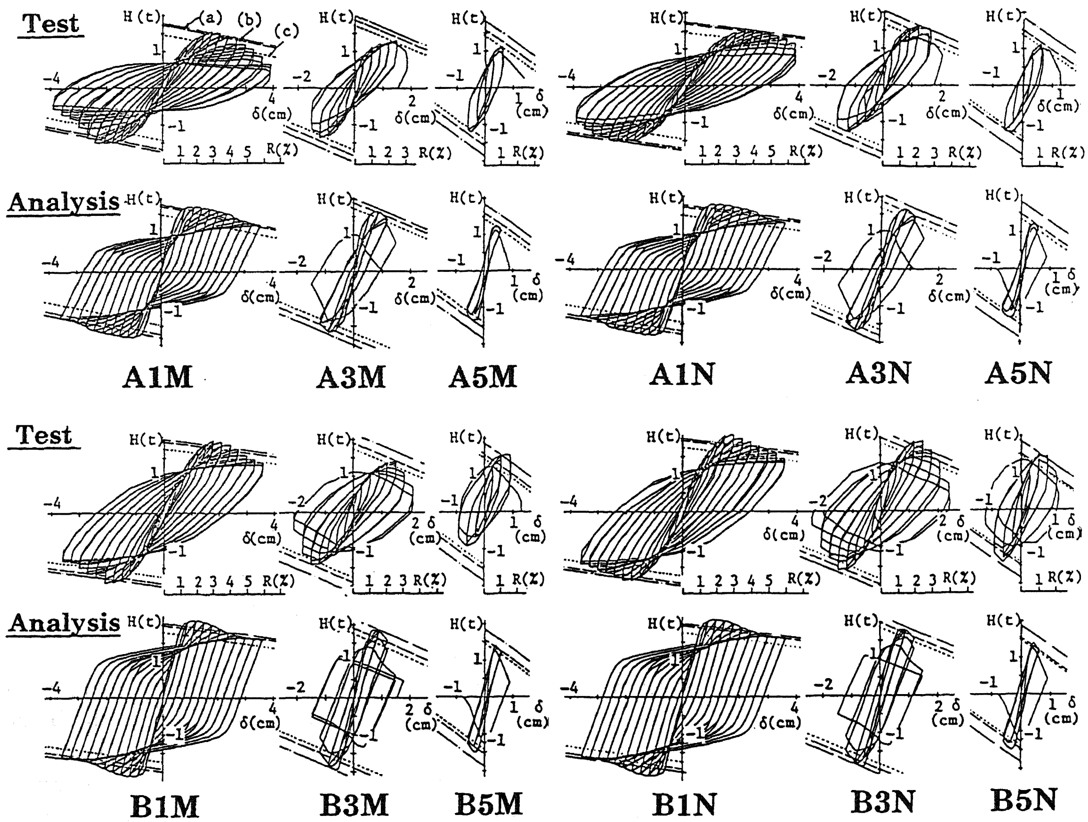
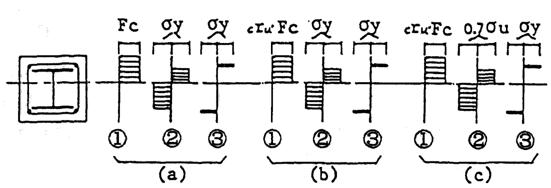


Fig. 5 Horizontal load-deflection relations



①: Stress of concrete
②: Stress of steel
③: Stress of reinforcement
${}_{c}r_{u}=0.85-2.5{}_{s}p_{c}$
${}_{s}p_{c}$: The ratio of the compression flange area to the total area of the concrete

Fig. 6 Assumed stress of cross section

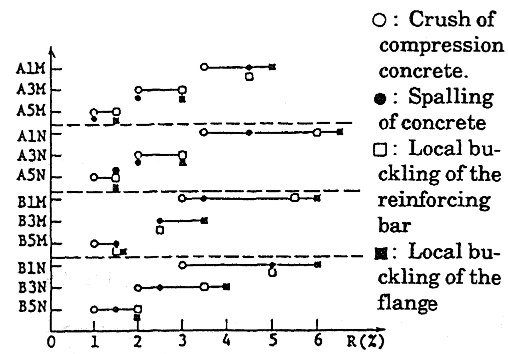


Fig. 7 Process of failure

in Japan (Ref. 4), taking the effect of high yield ratio σ_y/σ_u into consideration.Figure 7 shows the process of the failure of the specimen which is observed during the test.

The crush of concrete first occurs, and then the cover concrete spalls off. After that the reinforcing bar buckles, followed by buckling of the flange. Specimens (A3M,A5M,A3N,A5N) cannot sustain the vertical load in the ultimate state. It is shown from Figs. 5 and 7 that the effect of vertical load ratio n is quite apparent. Specimens with $n=0.1$ show large energy-dissipation capacity and stable hysteresis loops without a large strength degradation. On the other hand, specimens of $n=0.5$ collapse immediately at the occurrence of buckling of reinforcing bars induced after the spalling of the concrete.

As to the theoretical results, the experimental behavior can be predicted to some degree by the analysis. The effect of steel area ratio ${}_sp$ is observed in specimens with $n=0.3$, while in specimens of $n=0.1$ and 0.5 apparent difference between the behavior of type A and type B cannot be observed. Although the restoring force of specimens in type A becomes zero at the stage of rotation angle R=2.5%, that of B does not collapse because the vertical load can be supported by steel and core concrete. Actually the compressive strength of the bare steel portion ($={}_sA\cdot\sigma_y$) of type A cannot sustain the vertical load of $n=0.3$. This phenomenon is observed in the experimental behavior.

<u>Strain Behavior</u> Figure 8 shows the relations between strain of the center of cross section and rotation angle of the column. The strain is the value at the first reversed point of each displacement amplitude, taking the compressive strain as a positive sign. The strain is obtained from the data of strain gauges mounted on the center of the steel section 6 cm apart from the fixed end. From this figure, apparent difference between the behavior of $n=0.1$ and $n=0.3$, 0.5 is observed. The strain of specimens with $n=0.1$ decreases gradually in the small displacement range, and then increases in the large deflection range. In specimens with $n=0.3$, 0.5, the strain is always accumulated with the repetition of loading cycles. It is observed that the strain increases sharply in the vicinity of the point of crashing of the concrete. It is because that compressive force

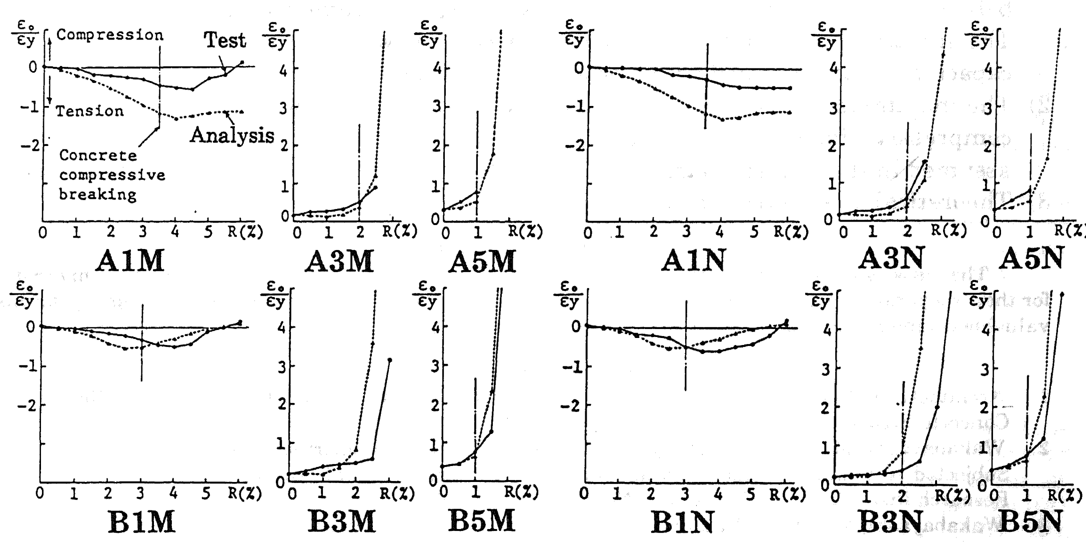


Fig. 8 Strain behavior

supported by the cover concrete transfer to the steel and core concrete. Theoretical results can predict fairly well such a behavior observed in the test.

Based on the fact that the specimens whose strain behavior is stable show a ductile behavior, it seems that the limiting value of vertical load on a column which requires the deformation capacity can be estimated on the basis of the strain behavior.

<u>**Maximum Strength**</u> Figure 9 shows the moment-axial force interaction curves. Moment M denotes the maximum moment, which is considered the effect of P-δ moment. The curves (a), (b) and (c) in the figure are obtained by assuming the stress distribution shown in Fig. 5 (a), (b), (c), respectively. A dash-dot line shows the limiting value N_ℓ specified in SRC standard in Japan. Symbols "•" and "o" denote the experimental and theoretical results, respectively. Strength curve (b) can estimate the strength conservatively for specimens of vertical load within the limiting value, while curve(a) cannot estimate the strength conservertively except for the specimen with n=0.1. Curve (c) can estimate the maximum strength of all specimens.

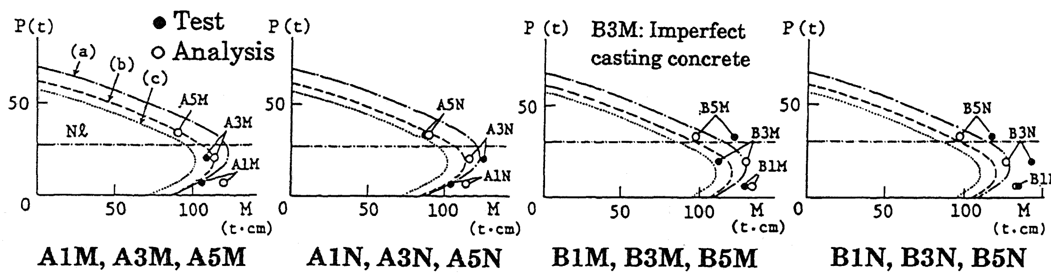


Fig. 9 Moment-axial force interaction curves

CONCLUSIONS

Conclusions derived from the experimental and the theoretical investigations on the behavior of SRC columns are as follows.

1) The effect of vertical load ratio on the inelastic behavior is apparent. When strain behavior of the centroid of cross section is stable, the column shows a ductile behavior. The limiting value of vertical load on a column which requires the deformation capacity can be estimated on the basis of the strain behavior.
2) The maximum strength of specimens which meet a code requirement of the limiting compressive force N_ℓ can be conservatively estimated by the stress distribution assumed in the SRC standard, even whose steel portion is made of high-strength steel.
3) Theoretical analysis can fairly well predict the experimental behavior.

ACKNOWLEDGMENT

The authors would to thank Messrs. T. Kimura and Y. Uwabe, former students of Kyusyu University, for their cooperation to the experimental works, and Mr. A. Kawano, Lecturer of Kyushu University, for his valuable comments and contribution to computer programming.

REFERENCES

1. Architectural Institute of Japan (AIJ), Standard for Structural Calculation of Steel Reinforced Concrete Structures.
2. Wakabayashi, M ., et al., A Study on the Ductility of Steel Reinforced Concrete Column Sections Subjected to Constand Axial Load and Repeated Bending Moment, Annuals, Disaster Prevention Research Institute, Kyoto University, No. 19-B, pp. 153-166, 1976.4.
3. Wakabayashi, M ., et al., A Study on the Behavior of Steel Reinforced Concrete Columns and Frames, IABSE, Québec. Preliminary Report, pp. 53-60, 1974.8.
4. Architectural Institute of Japan (AIJ), Design Standard for Steel Structures.