TESTS AND ANALYSES OF SRC BEAM-COLUMN SUBASSEMBLAGES

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SYNOPSIS

Tests of nearly full size SRC subassemblages provided data of the process of destruction of beam-column connections due to excessive shear in transmitting antisymmetric moments in the members. Ultimate strength of connections was satisfactorily predicted, but it was also found that the prediction of ultimate strength of SRC members, which dictates the ultimate external force to the connections, was more difficult than RC members. A simplified method to utilize subassemblage test data in the earthquake response analysis of prototype SRC frames was presented.

INTRODUCTION

SRC, referring to a type of construction of steel frames encased in the reinforced concrete, is the most typical type of construction in Japan for medium-rise buildings. As a part of the Technical Development Project organized by the Ministry of Construction, tests of SRC subassemblages were carried out at the University of Tokyo using two specimens of nearly full size.

Generally speaking, beam-column connections should be designed for: (1) transmission of vertical load, (2) resistance against shear stress associated with the transmission of moment in the members, and (3) anchorage of flexural reinforcement in the members. In case of SRC structures, the second item under horizontal loading becomes the most important problem.

The purpose of testing was, first, to relate the observed behavior to the more fundamental parameters of material, and second, to use the observed behavior as a fundamental unit of knowledge in analyzing SRC frames for earthquake response.

OUTLINE OF TESTS

Fig. 1 shows the specimens, which were meant to represent frame subassemblages at interior spans of about the 10th story from the top of buildings. Two specimens were identical except for the steel plate thickness at the connection panel, 9 mm for No.1 and 16 mm for No.2. Steel frame was made of built-up H shape members using SS41 plates with yield strength of 2530-2800 kg/cm^2. Yield strength of main reinforcing bars, D25, was 3840 kg/cm^2, while that of web reinforcement, D13, was 2710 kg/cm^2. Normal weight concrete with 28 day strength of 248 kg/cm^2 was used for specimen No.1, and the strength for No.2 was 192 kg/cm^2.

Special loading devices were attached to top and bottom of columns by HT bolts, and the specimens were mounted to a loading frame placed in the structural testing machine. Constant axial load of 245 t (N/bD = 50

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kg/cm^2) was applied by the testing machine, and reversal of simulated horizontal loading (antisymmetric loading at beam ends) was applied by hydraulic jacks as shown in Fig. 2(a). For the specimen No.2 after the completion of reversal of antisymmetric loading, symmetric loads as shown in Fig. 2(b) were applied in order to ascertain the strength of beams in bending. Deflections and strains were extensively measured at numerous locations taking advantage of large scale specimens.

Relations of load and deflection at beam ends under antisymmetric loading are shown in Figs. 3 and 4 together with occurrence of first cracks and crushing of concrete. Specimen No.2 with thicker panel plate showed higher strength and energy absorption regardless of lower concrete strength.

Both specimens finally failed at connections. Shear cracks were observed at about 30 t, steel panel of No.1 yielded at 30-40 t, and No.2, 35-45 t. Web reinforcement of No.1 yielded at 54-58 t, while about half of web reinforcement of No.2 yielded and others were close to yield at 62 t. Concrete outside web reinforcement was crushed at 50 t (in the third cycle) for No.1 and 62 t for No.2, and spalled off during reversal. Concrete within web reinforcement was also crushed but was retained within hoops, which were noticeably bent outward at the ultimate stage.

Under the maximum load, columns had a few cracks but no yielding was observed. At the critical sections of beams, both steel flanges and outside reinforcing bars were close to yield in tension, or in some places slightly beyond yielding. Some local crushing of concrete was also observed. However this does not necessarily mean that the beams almost reached their ultimate capacity. Table 1 shows measured and calculated strengths. The strength of beams and columns were calculated by two methods, one by superposition of steel full-plastic moment and RC ultimate moment, and another by linear strain distribution over the entire section (perfect bond). These values are generally higher than the maximum load in the test. Further, the beam strength under symmetric loading test of No.2 finally reached 94.2 t, which could only be explained by the strain hardening of steel.

Ultimate strength of connection panel in terms of beam end load was calculated by Eq. (1), which is an extension from the equation for RC beam-column connections.[1] Eq. (2) had been empirically obtained from tests of RC connections.[1,2]

$$P_u = \frac{{}_s\tau_u \cdot {}_sV}{(1-{}_su-{}_sv)L} + \frac{({}_c\tau_u + p_w \cdot {}_w\sigma_y)\,{}_cV_e}{(1-{}_ru-{}_rv)L} \qquad (1)$$

where ${}_s\tau_u$ = yield shear stress of panel = $\sigma_y/\sqrt{3}$

${}_sV$ = volume of shear panel = $t \cdot {}_sj_b \cdot {}_sj_c$, ${}_sj_c = {}_su.L$, ${}_sj_b = {}_sv.H$

${}_c\tau_u$ = ultimate shear stress of concrete as in eq. (2)

p_w = web reinforcement ratio in the panel zone = $a_w/b_e \cdot s$

${}_w\sigma_y$ = yield stress of web reinforcement

${}_cV_e$ = effective concrete volume in the panel zone = $b_e \cdot {}_rj_b \cdot {}_rj_c$

b_e = average width of column and beam, ${}_rj_c = {}_ru.L$, ${}_rj_b = {}_rv.H$

$$ {}_c\tau_u = (0.65 - 0.0014F_c)F_c \qquad \text{for } F_c \leq 232\ \text{kg/cm}^2 $$
$$ {}_c\tau_u = 75.4\ \text{kg/cm}^2 \qquad \text{for } F_c > 232\ \text{kg/cm}^2 \qquad (2) $$

As shown in Table 1, strength of connection panel calculated by eqs. (1) and (2) agreed fairly well with test results. It had been found that this set of equations generally give satisfactory prediction for full size or nearly full size SRC connections.[3)] It should be noted, however, that the exact evaluation of connection capacity alone may not lead to a satisfactory design, as the capacity of connecting members may become much higher than the calculated ultimate strength.

Besides the load vs. deflection relations shown in Figs. 3 an 4, contribution of beam, column, and connection panel deformation to the overall deflection was separately evaluated, by displacement measurements on the side surface of specimens. The results were unbelievable for two reasons. First, beams and columns showed remarkable nonlinear behavior in terms of load vs. deformation, while they did not yield completely until the end of testing as mentioned previously. Second, shear strain of panel determined from displacement measurement did not agree with shear strain from wire strain gages. Figs. 5 and 6 show these shear strains for the first two cycles.

It was concluded that three-dimensional deformation of SRC panel must have caused smaller shear deformation on the surface of column. To confirm this, flexural deformation of beams and columns were calculated assuming plane strain distribution over each section, and they were subtracted from the overall deflection of Figs. 3 and 4, to obtain panel shear strain. Figs. 5 and 6 also show this strain, which agreed fairly well with that from wire strain gages. Contribution of the evaluated shear deformation of connection to the overall deflection was about 23 percent for No.1 and 21 percent for No.2 in the elastic range, and it increased to about 70 percent for both specimens at the peaks of first two cycles.

ESTIMATION OF DYNAMIC RESPONSE

As an example of using the subassemblage test data in the earthquake resistant design, two prototype buildings, of which the weaker specimen No.1 would constitute a part, were subjected to earthquake response analysis. The specimen was thought to represent a typical bay of reasonably large and uniform building, either the first story of ten-story building, or the 10th story of twenty-story building. The axial load on the subassemblage of 245 t would correspond to about 30 to 40 m^2 of tributary area on each floor.

The observed load vs. deflection curves in Fig. 3 could be readily transformed into column shear force vs. story drift relation. The curves were then idealized into degrading trilinear hysteresis.[4,5)] Yield strength was determined from connection panel strength. Load after yielding was assumed to be constant. Initial stiffness determined by elastic analysis was reduced to about 75 percent to make a better fit to the larger portion of primary load-deflection curve. Deflection at yielding was calculated considering inelastic deformation of members and connection panel, based on a set of empirical equations.[1,2)] Resulted yield deflection was about 0.01 rad. in terms of column rotation angle, and the secant stiffness at yielding was about 50 percent of the initial stiffness, a little higher than the usual values for RC structures. Cracking strength was determined by fitting the idealized hysteresis to the measured curves by means of least square method. The resulted effective cracking strength was about 40 percent of yield strength, and the euqivalent viscous damping, which is constant regardless of amplitude by the nature of degrading trilinear hysteresis rule, was about 13 percent. Fig. 7 shows the comparison of measured vs. idealized hysteresis curves.

Two kinds of shear-type buildings as shown in Fig. 8 were considered: Model A, a uniform mass-spring model, and model B, with stiffness distribution associated with linear first mode. Nonlinear springs with degrading trilinear hysteresis were assumed to change their stiffness simultaneously in all stories under the deformation distribution proportional to the first mode. After all parameters were determined, average for two models were taken, as the structural characteristics of actual buildings would usually lie somewhere between the two models.

Because of the approximate nature of the analysis, both 10- and 20-story buildings were analyzed as a single-degree-of-freedom model considering the first mode only. Three recorded earthquake motions, El Centro 1940 NS, Hachinohe Harbor 1968 NS and EW were chosen because of their relatively large "destructiveness" to nonlinear oscillators. Response of SDF models, excited by these motions with maximum acceleration of 0.3 g and 0.5 g, were interpreted into story drifts as shown in Fig. 9.

It is seen that the 10-story building would not yield when subjected to 0.3 g earthquakes, and it would probably deform beyond yield point under 0.5 g earthquakes, to the ductility factor not greater than 1.5. 20-story building would generally have smaller response, except that one might possibly have response beyond yield point under 0.5 g earthquakes depending on the type of waveform. However even this would be less than 1.3 in terms of ductility factor.

CONCLUDING REMARKS

The authors attempted in this paper, not only to present the test results and associated analyses of nearly full size SRC frame subassemblages, but also to describe a procedure to utilize test results in the dynamic analysis of prototype frame buildings. The analysis of this kind may be performed for the planning of experiment. It will be particularly useful when the specimens to be tested are the subassemblages of actual building to be designed.

The experimental work reported herein was carried out at the Engineering Research Institute, Faculty of Engineering, University of Tokyo. The authors wish to acknowledge valuable helps and assistance of many personnel involved in this project, and particularly those of Mr. Masumi Ito and Mr. Toshimi Kabeyazawa, graduate students.

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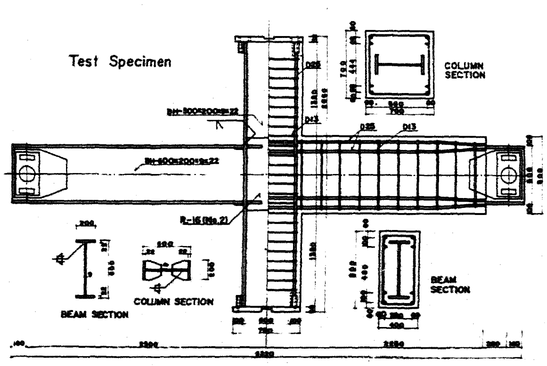

Fig. 1 Specimens

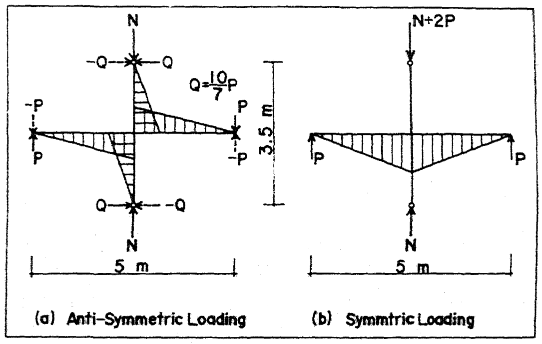

Fig. 2 Types of Loading

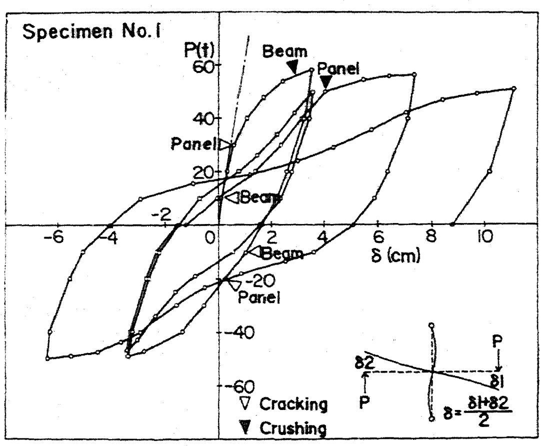

Fig. 3 Load vs. Deflection (No. 1)

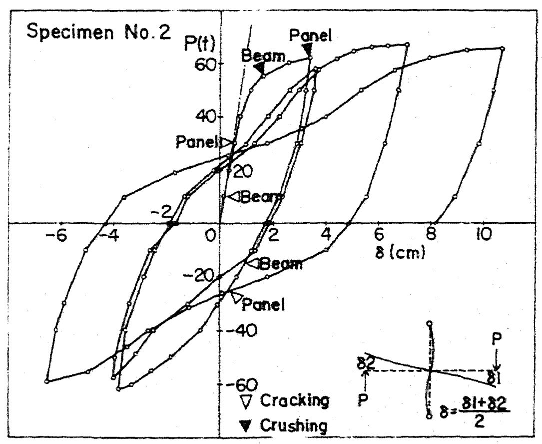

Fig. 4 Load vs. Deflection (No. 2)

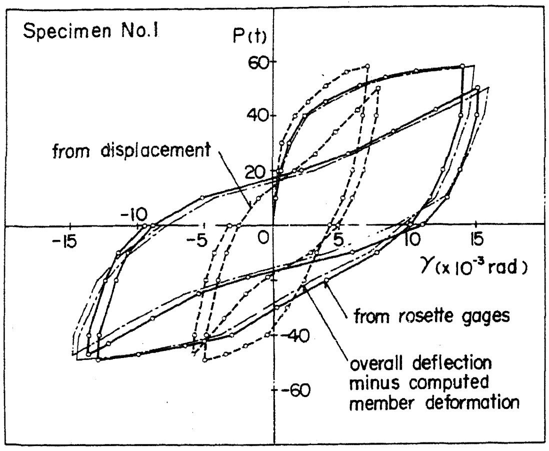

Fig. 5 Load vs. Panel Shear Strain
(No. 1)

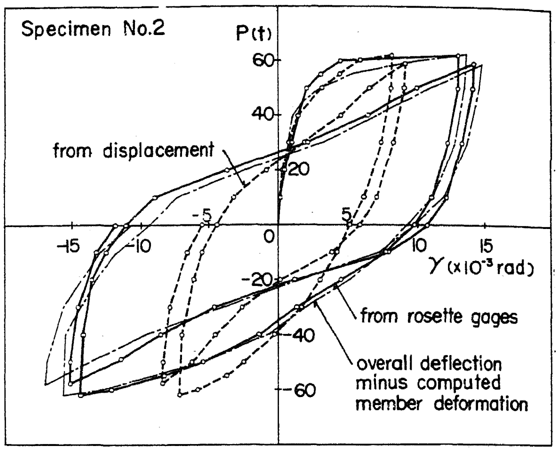

Fig. 6 Load vs. Panel Shear Strain
(No. 2)

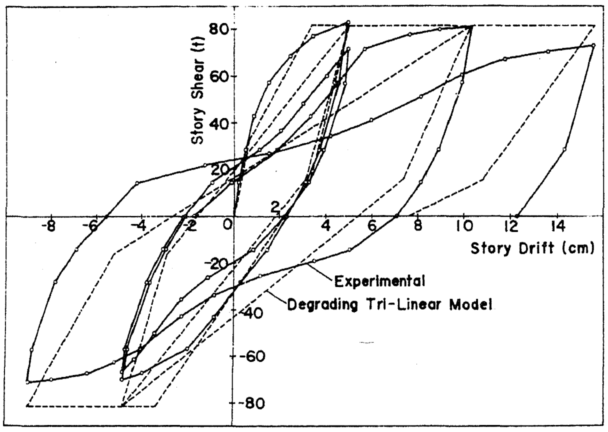

Fig. 7 Comparison of P-δ and D-Tri

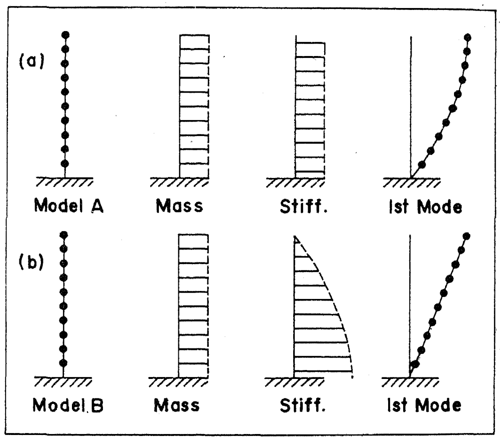

Fig. 8 Models A and B

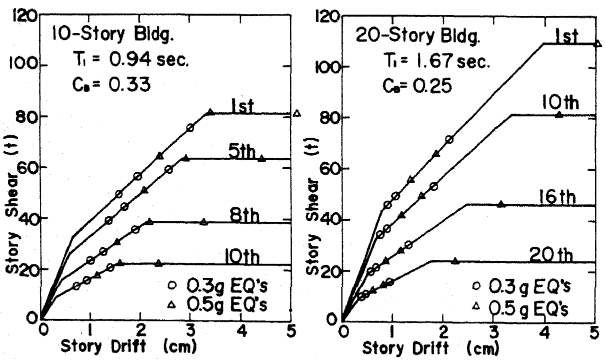

Fig. 9 EQ Response of Buildings

Table 1 Comparison of Strength
(Load at beam end in t)

		Test	Analysis	
			Superposition	Linear Strain
Beam	No.1 No.2	(94.2)	67.2 66.7	75.1 73.6
Column	No.1 No.2		95.5 92.6	99.0 94.4
Connect.	No.1 NO.2	57.9 67.0	56.6 62.0	

DISCUSSION

S. Otani (Canada)

The paper is quite valuable in a sense that the safety of the prototype structure was examined by nonlinear response analysis on the basis of the force-deflection properties found in the experiment.

The question is about the treatment of the gravity load in the response analysis. Is the entire resistance of the test structure assumed to resist the lateral earthquake loads ? Some part of the resistance capacity observed during the experiment must be used to resist stresses due to the gravity loads.

Koichi Minami (Japan)

The discussor would like to talk about the simple theoretical treatment to obtain the hysteretic response of SRC BEAM TO COLUMN CORNER CONNECTION failed in shear.

Shear resistant function of SRC connection may be divided into the four element as shown in Fig. 4.

That is, (a) steel web panel element (b) steel flange frame element (c) concrete panel element within steel flange and (d) reinforced concrete element.

According to the superimposed method, overall hysteretic response of SRC connection is obtained from the hysteretic response for each element computed by elastic-plastic technic.

An example of hysteretic response is shown in Fig. 2.

Measured response for 1st cycle at each shear strain amplitude is illustrated dotted line.

Measured fundamental response agree with calculated response.

Author's Closure

Dr. Otani asks how the gravity load effect was considered in the simplified earthquake response analysis. In the first place, the gravity load creates axial compression to vertical members. This was simulated by the constant compression force to the column of the subassemblage, and the amount of axial load was determined from the relative location of the subassemblage in the prototype building. Secondly, the gravity load will cause bending of beams. As the subassemblage represents an interior beam-to-column joint beam ends are subjected to negative moments, which decreases the remaining bending capacity available for earthquake loading on the one side, and increases it on the other side. Adding them up, the vertical loading does not affect the capacity to the earthquake loading, as long as yield hinges form at beam ends. Hence this was not considered in the testing.

Dr. Minami's comment on the simple theoretical treatment for the hysteresis of corner connections is highly valuable in that he showed how the behavior of simple elements was combined to represent the overall response of a complicated system. His effort to bring it out here is appreciated.