FULL SCALE TESTS OF BEAM-COLUMN JOINTS

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SUMMARY

Four full-scale beam column slab assemblies were tested under reversed cyclic loads. Two specimens, one interior and one exterior, were identical to beam column joints in the second story of the full-scale seven-story structure tested in Japan (Refs. 1, 2). In two specimens the longitudinal reinforcement in the beams and columns was increased to provide a variation in beam-to-slab strength. The behavior of the four specimens under reversed cyclic loads was excellent up to story drift levels estimated to correspond to the maximum deflection level imposed on the seven-story structure. The influence of the slab on the strength of the floor system under imposed deformations was significantly greater than anticipated.

SPECIMEN DETAILS

The dimensions of the test specimens are shown in plan and elevation views in Figs. 1 and 2. The nonshaded portion indicates the geometry of the exterior joint. The cross-sectional dimensions of the longitudinal and transverse beams and the column are given in Fig. 3. The reinforcement details (Fig. 3) for the prototype specimens, the first interior and exterior joint assemblies, were similar to the seven-story structure. The slab reinforcement consisted of two mats of #3 bars (see Fig. 4). The bottom layer of the longitudinal slab steel was continuous over the transverse beam. The detail is not normally used in U.S. practice.

Each specimen was cast in two stages. The lower column, joint region, beams and slab were cast in the first operation. Following a four day curing period, the forms were stripped and the specimen was lifted from the platform and set on a steel frame. The upper column was cast in the second operation. The average cylinder strengths at the time of testing were 4.0 to 4.9 ksi. Grade 60 reinforcing steel was specified in all four specimens.

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TEST PROCEDURE

The test setup is shown in Fig. 5. Pin connections were provided to simulate zero moment conditions at the top and bottom of the column. Racking loads were applied at the end of the longitudinal beam with double rodded hydraulic rams. In the case of the interior joint assemblies equal and opposite displacements were imposed at the ends of the longitudinal beam.

The specimens were subjected to predetermined displacement levels typical of the interstory drift levels imposed on the seven-story structure. A series of cycles with increasing magnitude of deflection levels were applied.

LOAD-DEFLECTION BEHAVIOR

A beam load versus the beam end deflection plot for the interior prototype specimen is shown in Fig. 6. Only the first cycles at each deflection level are plotted for the sake of clarity. The figures show the deflection level estimated to correspond to the maximum deflection level achieved during PSD4 in the seven story structure tests. A maximum beam end deflection of 4.8 in., about four times the maximum deflection level in PSD4, was imposed on each of the four specimens. The beam load versus the beam end deflection envelopes for the four specimens are shown in Fig. 7.

The maximum load reached by the exterior prototype specimen in the strong direction (slab in tension) was about 2/3 of that attained by the interior prototype. The mode of failure in the case of the exterior prototype was different from the other three specimens, which failed due to flexural hinging. The controlling failure mode for the prototype exterior specimen was anchorage failure. It is important to note that anchorage failure occurred at high deflection levels. For deflection levels estimated to correspond to the maximum deflection level imposed in the seven-story structure tests, the performance of the exterior prototype specimen was adequate. The poor performance of one component would not be alarming in a continuous multibay, multistory structure due to the redistribution of the forces in a highly redundant system.

Due to the large size of the columns, shear problems in the joint were not anticipated and none of the four specimens tested showed any shear distress.

INFLUENCE OF SLAB ON STRENGTH OF THE FLOOR SYSTEM

The measured loads are compared with the predicted loads in in Fig. 8. Since the exterior prototype specimen exhibited an anchorage failure, it cannot be compared directly but is included in the figure.

The flexural capacity of the longitudinal beam was computed for three different assumed flange widths. The total flexural capacity was computed as the sum of the flexural capacity of the longitudinal beam and the flexural capacity of that portion of the slab which was not assumed to act integrally as an effective width of the slab (see Fig. 8). It was assumed that the shallow slab section reached and maintained its maximum capacity before the specimen failed. This is a conservative assumption as far as determination of the mode of failure is concerned because it leads to overestimation of available strength. In Fig. 8 the magnitudes of beam and slab strengths for the three different assumed effective flange widths are compared. The sections analysed to predict the total flexural capacity are also indicated in Fig. 8.

The ratios of measured capacity to that predicted by ACI (Ref. 3) effective flange width show that the use of ACI flange width results in underestimating the strength by about 6% for the exterior modified specimen to 32% for the interior prototype specimen. The assumption that the full slab width acts as a flange of the T-beam would result in a considerable overestimation (between 17% for the interior prototype to 23% for the exterior modified) of the available strength. Thus, it is clear that a greater slab width than that prescribed by ACI is effective as the flange of the T-beam at ultimate.

The participation of the slab in resisting deformation is a function of the level of deformation. The profiles of the top and bottom slab steel strains along the transverse beam under the action of a negative moment are shown in Fig. 9. It can be observed that at larger deflection levels, the strains in the slab steel are higher and the strain profile indicates a large participation of the slab in resisting the applied deformations.

It should be noted that if the strength of the floor system is to be determined, it is "conservative" to consider a small contribution from the slab. However, if deformations are imposed on the structure and the moment imposed on the column by the floor is to be determined, it is no longer conservative to consider a small slab contribution.

CONCLUSIONS

1. The behavior of the four specimens under reversed cyclic loads was excellent up to story drift levels estimated to correspond to the maximum deflection level imposed on the seven-story structure. The behavior of all four specimens up to this drift level was governed by flexure.

2. The influence of the slab on the strength of the floor system under imposed deformations was significantly greater than would be anticipated using the ACI effective slab width as a flange for T-beam analysis. Underestimation of floor system flexural capacity could result in hinges developing in the column rather than in the flexural members if large deformations are imposed on the structure.

3. No shear distress was observed in the joint region during any stage of testing. Large columns resulted in low shear stresses in the joint.

4. Although the failure of the exterior prototype specimen was governed by anchorage, the failure occurred at deflection levels much higher than the maximum deflection level experienced by the seven-story structure. An improved confinement detail (additional transverse cross ties) in the modified exterior joint greatly improved its performance.

ACKNOWLEDGMENTS

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REFERENCES

1. "Recommendations for a U.S.-Japan Cooperative Research Program Utilizing Large-Scale Testing Facilities," by U.S.-Japan Planning Group, Report No. UCB/EERC-79-26, September 1979.

2. Okamoto, S.; Nakata, S.; Kitagawa, Y.; Yoshimura, M.; and Kamimosono, T., "A Progress Report on the Full-Scale Seismic Experiment of a Seven-Story Reinforced Concrete Building--Part of the U.S.-Japan Cooperative Program," B.R.I. Research Paper No. 94, Building Research Institute, Ministry of Construction, Japan, March 1982, 92pp.

3. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-77)," American Concrete Institute, Detroit, 1977.

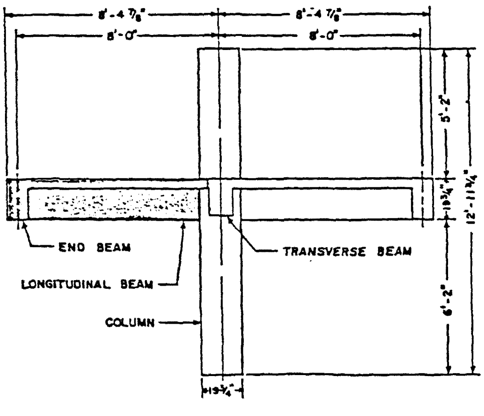

Figure 1. Elevation view of joint specimen.

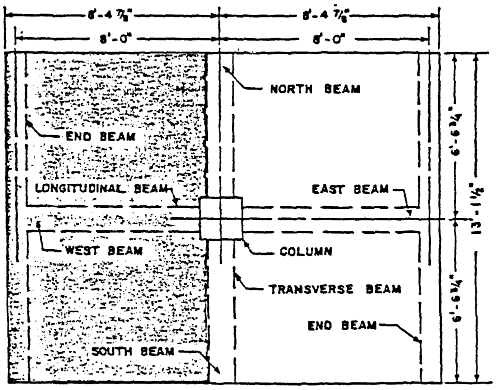

Figure 2. Plan view of joint specimen.

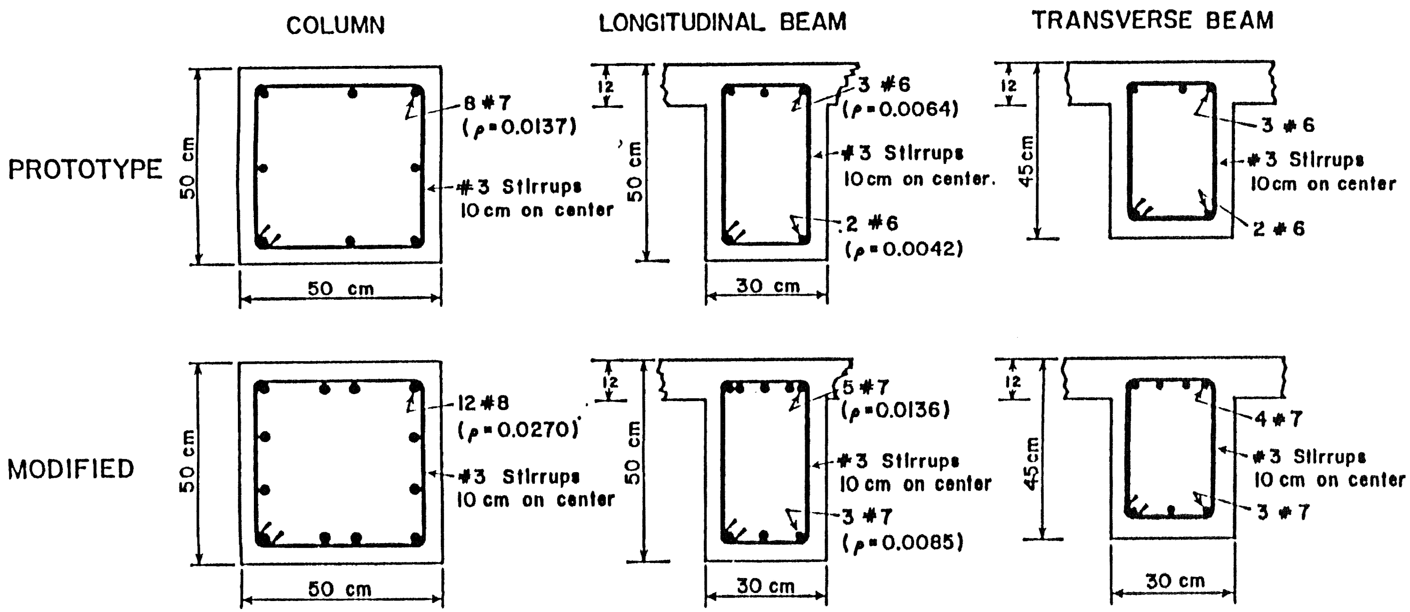

Figure 3. Cross-sectional details of the test specimens.

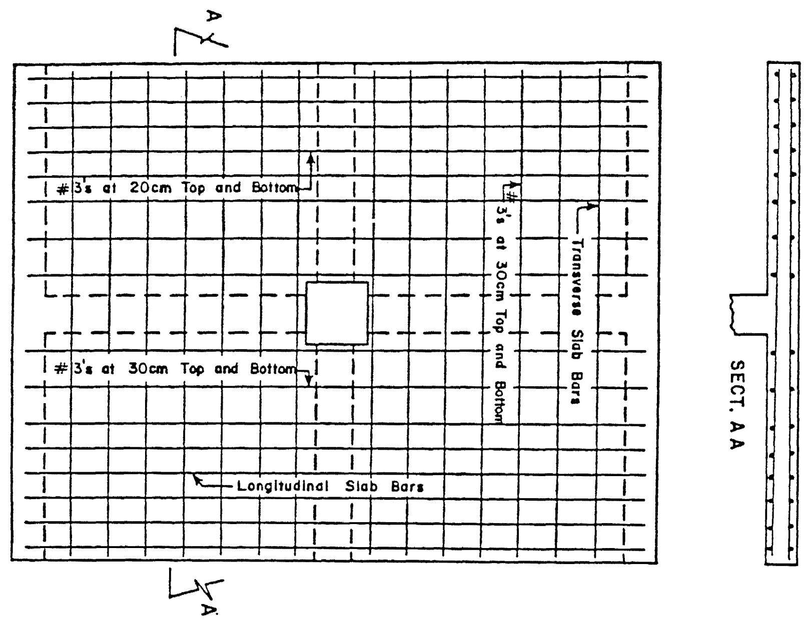

Figure 4. Slab reinforcement detail.

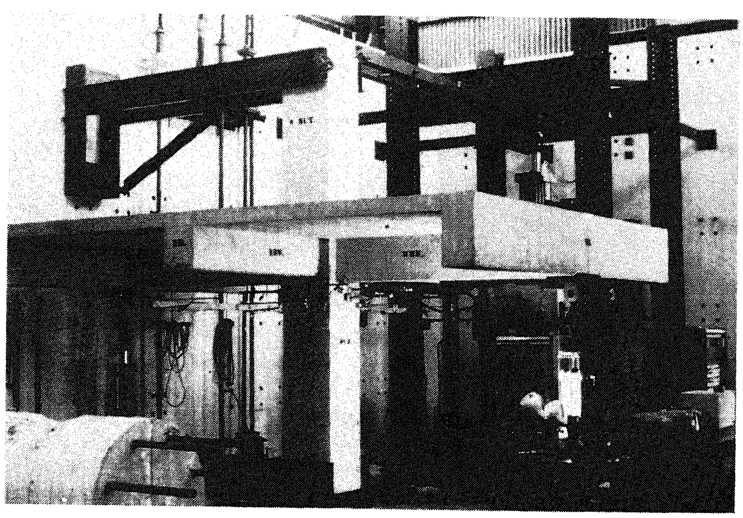

Figure 5. Test setup.

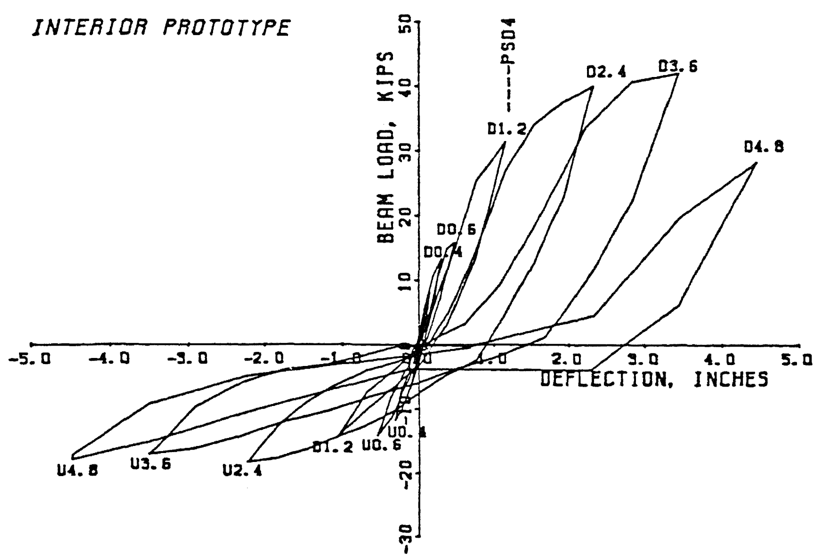

Figure 6. Measured load-deflection curve, interior prototype.

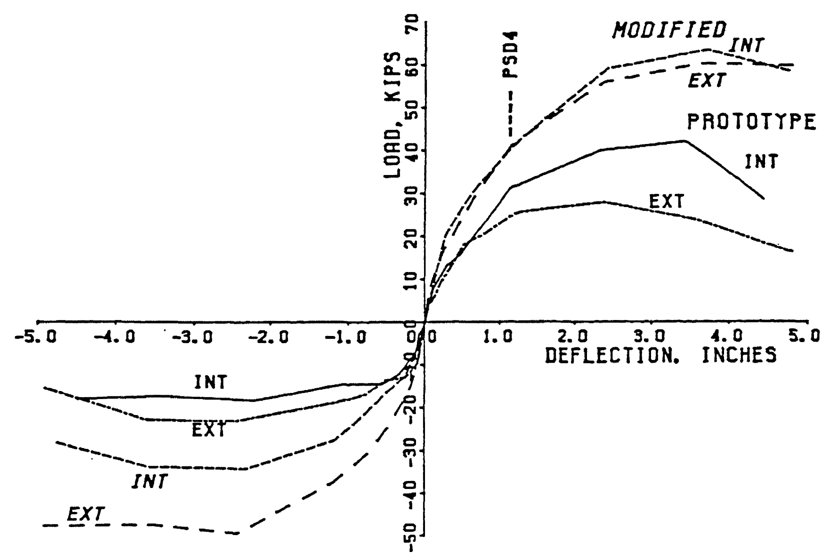

Figure 7. Envelopes of l. l-deflection response.

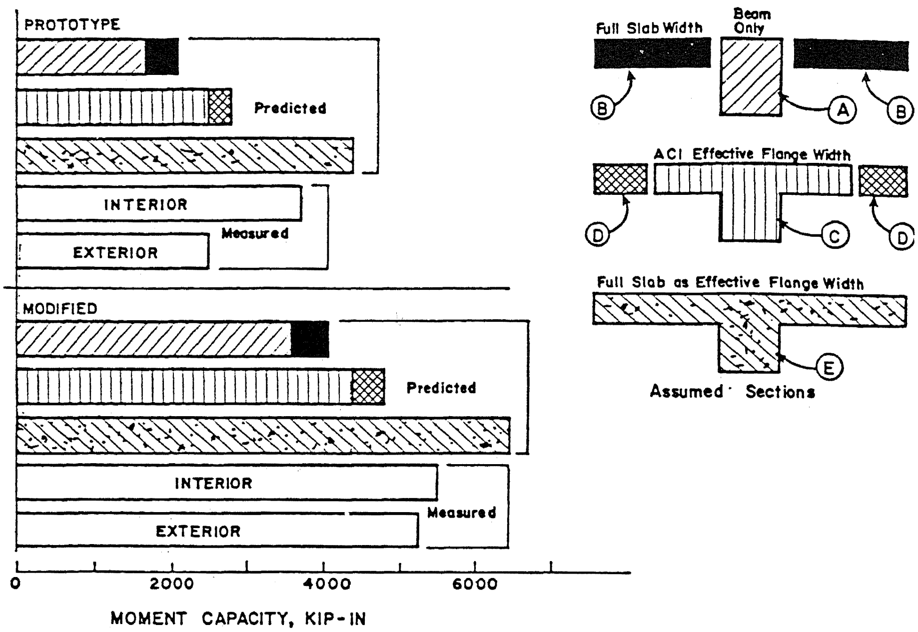

Figure 8. Influence of slab on floor strength, slab in tension.

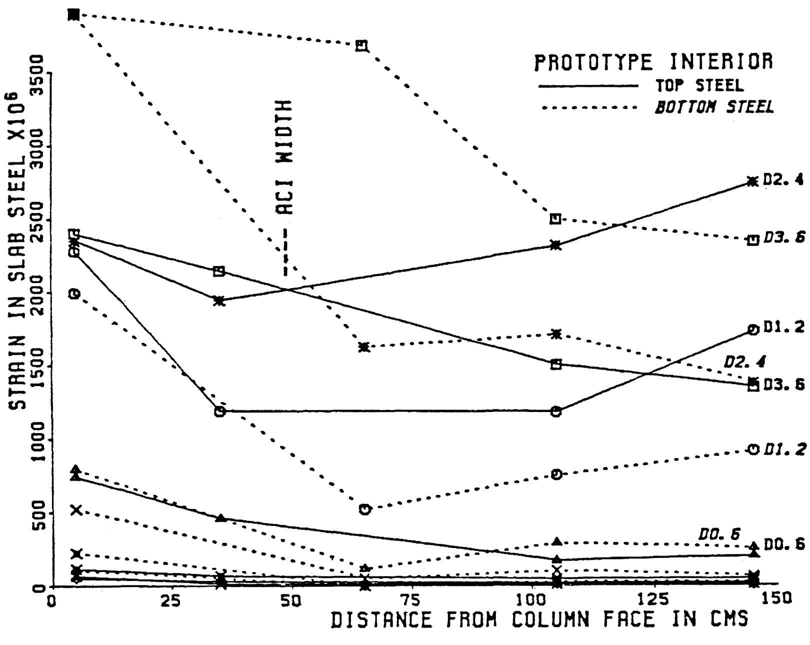

Figure 9. Profiles of slab steel strains, interior prototype.