ON SEISMIC DESIGN OF R/C INTERIOR JOINTS OF FRAMES

by E.P. Popov[I], V. V. Bertero[I], B. Galunic[II] and G. Lantaff[II]

SYNOPSIS

Two alternative designs for beam reinforcement at the interior joints of moment-resisting reinforced concrete frames are discussed. The proposed details, verified by experiments, avoid the danger of bond degradation of the main beam bars within the column.

INTRODUCTION

A rational design of R/C frames for successfully resisting seismic loadings is one of the more important problems in earthquake engineering. Such frames are widely used either as the principal structural system or in conjunction with infill or structural walls. To assess the behavior of such R/C frames, a good deal of experimental work was done on individual beams and columns. The behavior of these elements was studied under cyclic loadings simulating seismic disturbances. The separate study of these members, however, is not sufficient to determine the behavior of whole frames, as a very complex action develops in the joints. Improperly designed joints may change dramatically the behavior of the whole frame. Therefore, in addition to studying the beams and columns as separate entities, these elements must also be studied interconnected, forming typical frame subassemblages.

At Berkeley, a series of half-scale subassemblages of a tall R/C building was tested according to a procedure described later in this paper, and after some damage and repair, was re-tested [1]. The model simulated the basic subassemblage of a third floor level of a 20-story office building. The overall dimensions of the specimens were kept constant while changing the amount and type of reinforcement details in the joints and beams. The test arrangement was such that a Pδ effect could be induced. The specimen design was based on the concept of strong columns-weak beams, i.e., the inelastic action was to take place primarily in the beams.

In a previous test on specimen BC3, the beams were designed with four #6 bars on the top and three #5 bars or 50 percent of negative steel on the bottom. Since the bars were continuous, the beam plastic hinges formed at both faces of the column. The maximum shearing stresses in the beams were on the order of $3\sqrt{f'_c}$ (psi). This resulted in failures of the beams in the flexural mode[2,3]. The expected degradation of beam stiffness and strength during reversals of large shear did not occur, although the subassemblage showed severe degradation due to the bond failure of the main beam bars within the column. To remedy this objectionable condition in the joint, two alternative designs were made and tested, and the results are presented in this paper.

DESIGN OF NEW SPECIMENS

Both of the new specimens are so designed as to force the formation of plastic hinges away from the column faces. It is believed that by this scheme, the bond failure of the main beam bars within the column will be delayed or prevented. This should result in a superior ductile behavior of the whole subassemblage. The details of the specimens are shown in Fig. 1. For both specimens, the reinforcement of the beams at the face of the column consists of four #6 bars on the top and bottom. Thus, unlike the previous

I Professor of Civil Engineering, Univ. of Calif., Berkeley, California.

II Research Assistant, Univ. of Calif., Berkeley, California.

specimens, 100 percent of the negative steel is placed on the bottom of the beams. The plastic hinge was designed using two different schemes. In one specimen, BC5, the two top interior main bars were bent downward, and the two corresponding bottom bars were bent upward intersecting 16 in. away from the face of the column as shown in Fig. 1. The bars were inclined 60° from the horizontal. After additional bending, the bars extended to the end of the beam. All corner bars are both straight and continuous. In the second specimen, BC6, half of the main bars on the top and bottom of the beams were cut off at 24 in. away from the face of the column. The location of the plastic hinge is determined by the requirement that the steel at the column face begins to yield just shortly before the critical section reaches ultimate condition, where the bars are at their ultimate strength. The plastic hinge in BC5 was closer to the column because the large amount of diagonal steel at that section gave a substantial increase to the moment capacity of the section. In BC6 the bars were cut off at a point just slightly beyond where analysis indicated the plastic hinge would occur, because it was anticipated that the plastic hinge would gradually move toward the column. In both specimens, the beams were reinforced against shear with #2 double stirrups placed 3-1/2 in. apart except in the plastic hinge region where a more conservative stirrup spacing of 4.5 times the bar diameter (or 2 in.) was used to avoid any buckling of the main reinforcement. The columns were reinforced with twelve #6 bars and #2 bar triple ties at 1.6 in. on center. In the column joint the ties were spaced at 2 in. on center. Grade 60 steel was used for all reinforcement.

CASTING PROCEDURE

The specimens were cast in a vertical position in the course of the same day, in three separate lifts with a stopover after each to allow for the shrinkage of the concrete. The lower section of the column was cast first, up to the bottom of the beams. Later, the beams were cast and, finally, the top part of the column was cast to complete the process. Because of an error in the mixing process, the concrete for specimen BC5 was not of uniform quality. The design called for a 28-day concrete strength of 4000 psi, but the actual strength obtained in the beams was only 2100 psi. However, since the tests were conducted primarily to study the behavior of the subassemblages after the steel has yielded, the steel had a greater influence on the specimen's performance in the inelastic range than the concrete, and the overall results were not strongly affected. The concrete strength in specimen BC6 was uniform throughout. The material properties are listed in the table below. After the specimens were cured and the forms removed, metal hinge assemblies were attached at all four ends of the specimens. These were made of steel for the column and of aluminum for the beams. The aluminum hinge assemblies were designed to act as transducers for measuring the reaction forces acting on the beams during the experiments.

Subassemblage Material Properties

SPECIMEN	CONCRETE ULTIMATE STRENGTH	
	ksi	MPa
BC5	2.1	14.4
BC6	4.0	27.5

REINFORCING STEEL				
BAR SIZE	YIELD STRENGTH		ULTIMATE STRENGTH	
	ksi	MPa	ksi	MPa
#2	65.0	448	106	731
#6	64.0	441	106	731

EXPERIMENTAL SET-UP AND TESTING PROCEDURE

The specimens were tested in a large steel testing frame. The top hinge of the column was connected to a pin fixed to the frame and prevented from translation. The hinges in the beams were restrained from vertical translation but were free to displace horizontally. The lateral load was applied at the bottom hinge of the column by means of a double acting hydraulic cylinder which moved the hinge back and forth. Also, a large axial load was applied to the bottom hinge by means of a hydraulic jack which was supported by a movable cart. The axial load of 470 kips represents the dead and live loads acting on the specimen and it was kept constant in these experiments. The loads applied and the beam reactions are shown schematically in Fig. 3. As can be seen from the figure, the reactions in the beams are caused not only by the applied horizontal force, but also, by an extra $P\delta$ moment caused by the vertical force, P. The additional beam reaction at each end caused by the vertical loads is equal to $P\delta/(L)$, where P is the total vertical load, δ is the bottom hinge displacement measured from the top of the column, and L is the distance between the two beam hinge supports. In the lower stories of a structure where the axial load is large, the $P\delta$ effects are very significant for large story displacements.

The applied displacement program is shown in Fig. 2. The specimens were first subjected to a few loading reversals in the working stress range. After the first yield, the specimens were subjected to increasing displacements in a stepwise manner. Two cycles were made at each particular displacement. The cyclic loading applied to BC5 and BC6 was very similar to that of the previously tested BC3.

Extensive instrumentation was used to record the specimens' behavior. Strain gages were placed at various locations on the main beam reinforcement to study the variation of strain and to check for yielding. Clip gages were used on the top and bottom of the beams to measure curvature. Shear distortion was measured by a set of clip gages diagonally placed at two adjoining locations near the plastic hinge on each beam. Measurement of beam reactions was made by the aluminum transducers in the hinges. Photogrammetric measurements were made of the beams to study the deformation pattern.

EXPERIMENTAL RESULTS

In representing the overall performance of the members, the H_{eq} vs. δ graphs were used instead of the H vs. δ, because they give a better measure of the main beam's strength. A comparison of the H_{eq} vs. H curves was made for a previously tested specimen and is shown in Fig. 4. The H value is seen to decrease after L.P. 17 with increasing displacement. However, because of the $P\delta$ effect, the equivalent horizontal force, H_{eq}, actually increases, indicating that the beam gains strength after first yield. Similar behavior was evidenced in specimens BC5 and BC6 (Figs. 5 and 8).

Excellent results were obtained from specimen BC5 in terms of overall member performance as well as of solving the problem of anchorage loss. Slippage of the bars through the column was completely eliminated and smooth stable hysteretic curves were recorded throughout the duration of the experiment. The steel at the face of the column yielded at a ductility ratio of approximately 4.5, which represents a ratio of maximum displacement to displacement at yield. Large plastic rotations in the plastic hinge of .058 rad. (.110 rad. for a complete cycle) were observed during the last few cycles of the test. The large increase in strength of the specimen after yielding is caused by the strain-hardening of the continuous bars and by the increasing effectiveness of the inclined bars due to the large strains in that region. From Fig. 5, it can be seen that strength increases to a maximum with increasing displacement and then begins to decrease. The lateral resistance and stiffness decrease with repetition of a reversal cycle with same peak displacement. Shear distortions were very small due to the excellent reinforcement provided by the inclined bars. The deformation characteristics of the beam in the area of the plastic hinge is shown in Fig. 6.

Slippage of the bars through the column was also eliminated in BC6; however, there was a large amount of shear deformation at the plastic hinge. This plastic hinge formed as expected at the cutoff point of the main interior bars, but progressively moved toward the face of the column as the bond at the end of the cutoff bars deteriorated and the moment capacity of the severely cracked section decreased. The bars at the face of the column did not yield, although during the last several cycles the strains were very large. Under repeated application of displacement reversal in the inelastic range, the flexural-diagonal cracks at the top and bottom of the critical region of the beam increased in width and propagated until they crossed. At this stage, resistance to shear deformation decreased significantly because the only effective mechanisims of shear resistance that remained were the aggregate interlocking and friction along the crack faces and the dowel action of the main steel bars. All of these sources of resistance degrade with increasing severity of reversals. During shear reversal, at small shear force there is a small amount of slippage between the two adjacent sides of the main crack, which causes a drop in the overall stiffness. This shear distortion at the critical region reached 3/4 in. at the last cycle of the test. The deformation pattern at the plastic hinge was obtained by a photogrammetric technique and is shown in Fig. 7.

CONCLUSION

Both specimens were successful in avoiding the problem of bond loss by the main beam bars in the column core. However, to permit the development of large plastic rotations, special shear reinforcement should be provided to prevent an early shear failure. The crossing steel of BC5 offered excellent resistance to shear. In BC6 only vertical stirrups were used for shear reinforcement. Such vertical stirrups are very effective when the cracks in the concrete form at approximately 45°. But, after several cycles into the inelastic range, the plastic hinge of BC6 had many vertical cracks for which vertical stirrups were ineffective. Therefore, with little effective shear reinforcement, large amounts of shear deformation developed in the plastic hinge region. Diagonal shear reinforcement should be used to prevent this type of failure.

When the plastic hinge is forced to form away from the column face, larger amounts of rotation are required in the plastic hinge for the same story drift. Because of these large rotational demands, it is very important that the steel and its detailing be carefully selected so as to develop large strains.

ACKNOWLEDGEMENTS

This work was supported by the National Science Foundation Grants GI-36387 and ENV76-04263 for which the authors are most grateful. Several graduate students among whom D. Soleimani, S. Viwathanatepa, T. Y. Wang and Messrs. B.Lutz, D. Clyde, and R. Osborne were most helpful with the tests.

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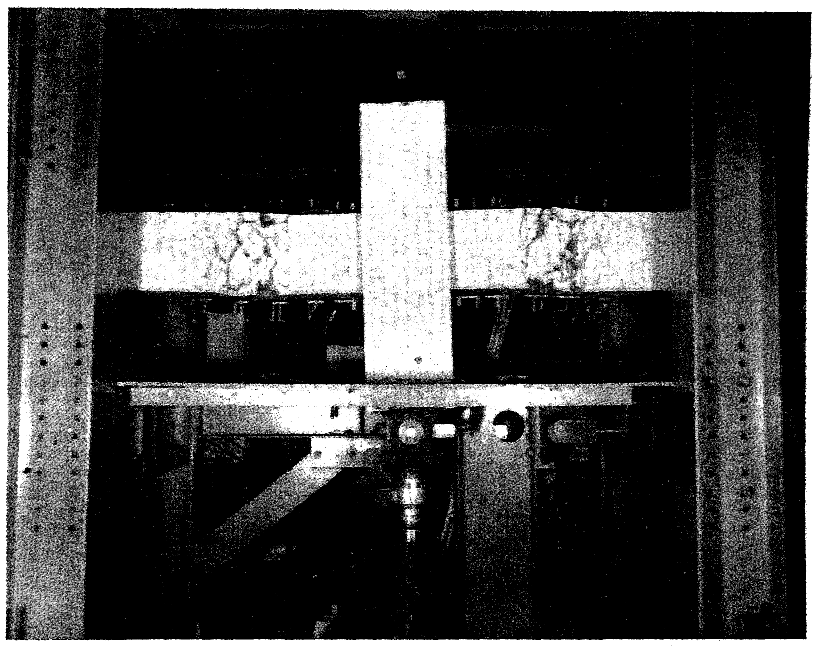


General Experimental Set-up

Specimen BC6 after last load cycle. Note the large shear slippage in the plastic hinge regions.

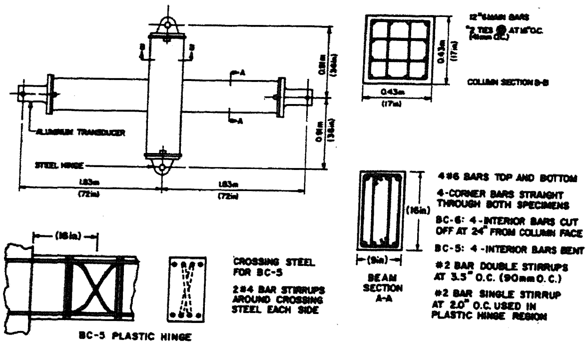


Fig. 1.

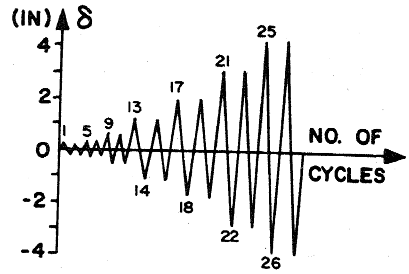


Fig. 2.

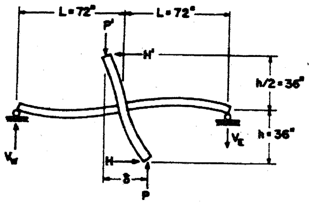


Fig. 3.

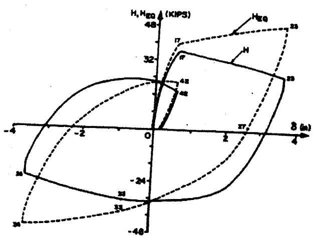


Fig. 4.

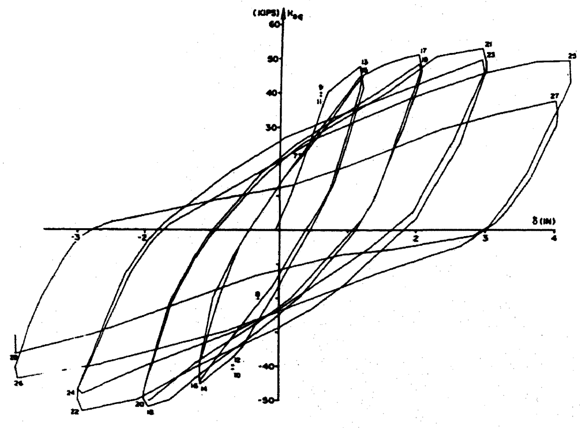


Fig. 5. BC5

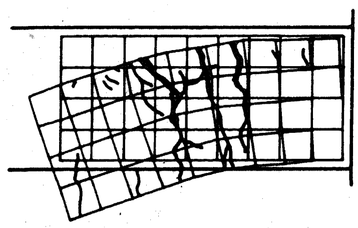
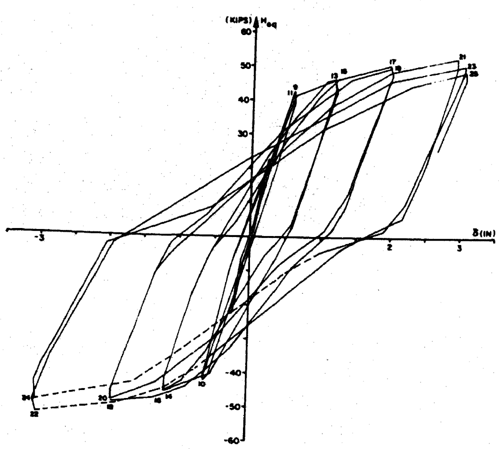


Fig. 6. BC5



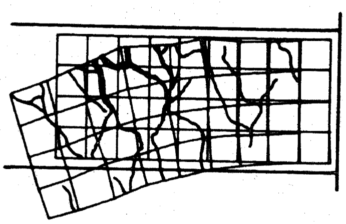


Fig. 7. BC6

Fig. 8. BC6