SEISMIC BEHAVIOR OF REINFORCED CONCRETE MOMENT-RESISTING FRAMES

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SUMMARY

At severe random cyclic loadings, bond failure may occur and slippage may take place between the reinforcing bars and concrete. Some experimental evidence of this kind of behavior at interior joints of moment-resisting reinforced concrete frames is examined. Selected test results on one-half scale cruciform subassemblages of normal and lightweight aggregate concrete specimens are presented. Practical means of avoiding the bar slip in an interior column joint by designing for plastic hinges to occur away from the column faces are then described. A discussion of a mathematical model for the analysis of reinforced concrete frames when the main beam bars slip in a joint and plastic zones extend along a beam concludes the paper.

INTRODUCTION

Moment-resisting reinforced concrete frames are widely used as prime elements or in conjunction with structural (shear) walls for resisting seismic forces. The analysis of such frames has not been fully perfected because of several complicating features in the behavior of reinforced concrete. The most difficult aspects of the problem pertain to shear transfer across severely cracked sections and bond failure accompanied by bar slippage. Analytical models for predicting such behavior under cyclic loading are far from being satisfactory. This paper addresses itself primarily to the question of main beam bar slippage in an interior joint and a possibility of including in the analysis the zones of alternating plasticity along the beams.

SPECIMEN DESIGN

A 20-story, four-bay reinforced concrete frame of an office building designed as a ductile moment-resisting space frame in accordance with the most severe requirements of the 1970 UBC [1], 1971 ACI Code [2], and 1971 SEAOC Recommendations [3] served as the prototype for this study [4]. A strong column-weak beam design approach, which meant that under gravity and code seismic lateral loadings yielding would occur only in the beams, was adopted.

The location of the selected subassemblage at the third floor level of a 20-story frame prototype is shown in Fig. 1 [5]. The subassemblage beams were hinged at mid-span, since the inelastic behavior of the third floor beams is influenced primarily by the lateral rather than gravity forces. The columns were hinged at mid-height. The geometry and reinforcement of the half-scale typical test specimen are shown in Fig. 2 [5]. Typical beams were 9×16 in. (230×400 mm) with #6 (19 mm) main bars at the top and #5 (16 mm) bars at the bottom. For some specimens four #6 (19 mm) bars were used both at the top and at the bottom of the beams. For all specimens $L = h$ was 72 in. (1.8 m).

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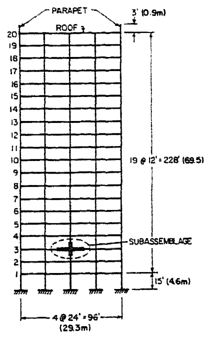

Fig. 1 20-Story Prototype Frame and Selected Subassemblage [8]

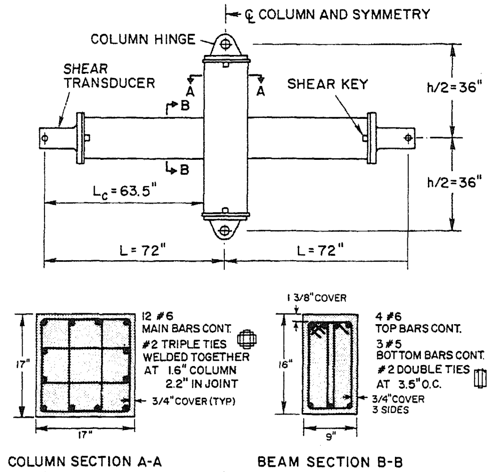

Fig. 2 Beam-Column Test Specimen [5]

EXPERIMENTAL PROCEDURE AND RESULTS

On placing the specimens into a test frame, the columns were loaded axially to 470 kips (2090 kN), and the beam ends were deflected downward developing reactions of 3.5 kips (16 kN). The application of these forces simulated gravity effects. A horizontal double-acting jack at the bottom hinge of the specimen simulated the effect of seismically induced forces by applying specified displacements in a quasi-static manner. A free-body diagram for a subassemblage for these conditions is shown in Fig. 3. At large

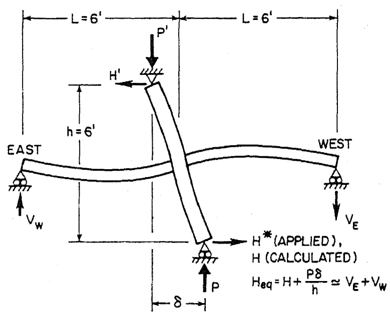

Fig. 3 Free-Body Diagram for Subassemblage [5]

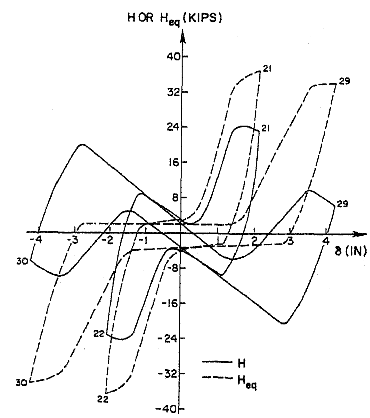

Fig. 4 Comparison of H-δ with H_{eq}-δ for Specimen BC2 [5]

column displacement δ, due to the presence of a large axial column force P, the P-δ effect becomes important and must be considered.

A portion of a hysteretic diagram for the horizontal force H applied by a jack vs. displacement δ for Specimen BC2 is shown in Fig. 4. In the same diagram an equivalent horizontal force H_{eq} vs. δ is also shown. The force H_{eq} was found by adding to the applied horizontal force H a term $P\delta/h$ which accounts for the P-δ effect. The H_{eq}-δ diagram shows the actual strength demands placed on a joint. Because of attaching an eccessive number of strain gages to the beam bars within the joint, the bar anchorage length in Specimen BC2 was reduced resulting in an early bond failure generating very poor hysteretic loops. A similar situation may develop in actual construction due to poor workmanship. Because of steel congestion at the joints of ductile moment-resisting frames, the occurrence of rock-pockets in such locations is a distinct possibility. In these cases significant beam fixed-end rotations can take place at large ductilities.

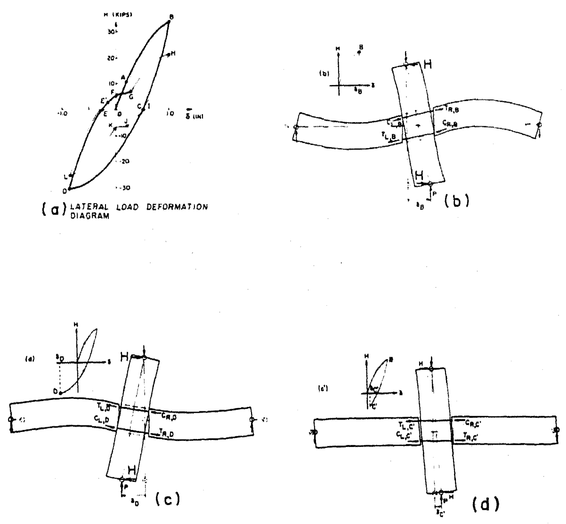

Fig. 5 Mechanism of Stiffness Degradation [6]

The mechanism of stiffness degradation at a joint is illustrated in Fig. 5 [6]. On complete load reversal cracks are formed on both sides of a column, and, due to plastically strained steel, these cracks can remain open and the beam bars can become subjected simultaneously to pull <u>and</u> push. This imposes severe demands on the bar anchorage within a joint. Vestiges of this behavior can occur in specimens with no construction defects. Such an example is shown in Fig. 6 for Specimen BC3 [5]. In this case some pinching of the hysteretic loops can be noted at relatively low values of ductility, and a progressive breakdown in bond within a joint was observed. Of course bond deterioration does not occur as rapidly nor is it as extreme in the example first cited. Nevertheless it should be remarked that although the beam fixed-end rotations for BC3 are substantially smaller than those for Specimen BC2, at large displacements they contribute 20 to 35% to the total horizontal displacement of the subassemblage.

The tendency for an anchorage failure at the beam bars within an interior joint can be greatly reduced or even completely eliminated. This is most easily done by requiring a larger amount of reinforcement at the bottom of a beam at a joint than is customary. Thus, instead of merely complying

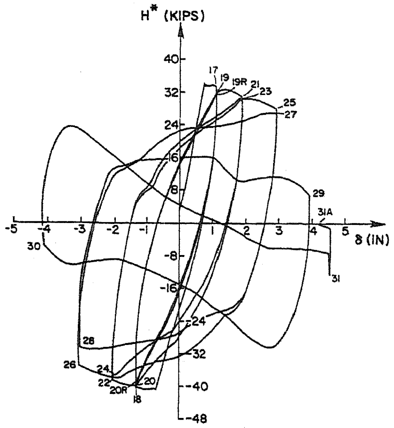

Fig. 6 H*-δ Diagram for Specimen BC3 [5]

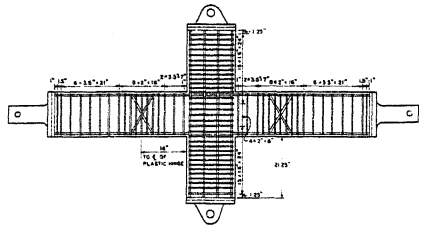

Fig. 7 Subassemblage with Inclined Bars in the Plastic Hinge Region. Specimen BC5 [8]

with the current practice [2] of providing as little as 50% of the top steel area in the bottom bars of a beam, one could specify the same amount of top and bottom reinforcement. Experiments have shown this to be very effective [7,8]. A somewhat costlier but superior method of detailing for avoiding bond failure within a joint is shown in Fig. 7. In this scheme [8] some of the beam bars are bent at points of the anticipated plastic hinges. The hysteretic loops for a specimen made in this manner are shown in Fig. 8. Note particularly the slow rate of deterioration of the loops in the important inelastic range of displacements from 0.75 to 2 or 3 in. (20 to 50 or 75 mm). This can be compared with the poorer performance of BC3 shown in Fig. 6. The cracking of Specimen BC5 was remarkably well distributed along the beam resulting in narrow cracks.

A comparison of hysteretic behavior between two dimensionally identical subassemblages with the same amount of reinforcement, but made with concrete having different aggregates is shown in Fig. 9 [9,10]. Specimen

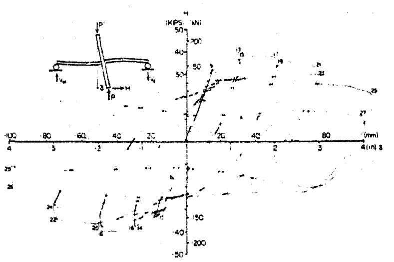

Fig. 8 H-δ Diagram for Specimen BC5 [8]

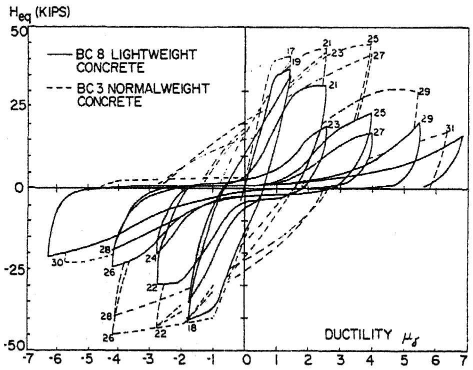

Fig. 9 Comparison of Hysteretic Behavior of Normal with Lightwaight Aggregate Specimens [10]

BC8, which was made of lightweight concrete, deteriorated under the application of cyclic loading much more rapidly than BC3 made of a normal weight concrete. No such difference in behavior was noted for monotonic application of the loads [10]. This clearly points to rapid bond deterioration in lightweight concrete under cyclic loading.

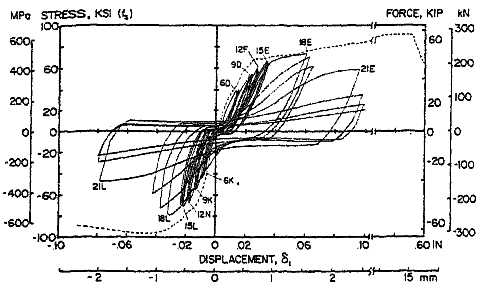

Fig. 10 End Displacements of a Bar under Cyclic Loadings [11]

Because of the importance of bond behavior under cyclic loading, this problem was isolated and investigated in some detail [11]. By applying different patterns of loading to a bar embedded in a reinforced concrete block simulating a column, numerous hysteretic loops were obtained. One such case for a #8 (25 mm) bar embedded in a column stub 25 in. (635 mm) deep is shown in Fig. 10. In this experiment pull at one end of a bar was simultaneously applied with a push at the other end. This corresponds to the most severe loading condition a bar may experience during a cycling process. It is significant to note that some slip of a bar occurs from the earliest stages of loading. Progressively this slip becomes larger until the bar pulls through the column stub. Some success was achieved in modeling cyclic behavior analytically [12]. A much simpler model for bar pull-out leading to satisfactory results has been proposed for monotonic application of loads [13]. Further work remains to be done to determine the interaction between parallel bars.

ANALYTICAL STUDIES OF FRAME BEHAVIOR

In order to evaluate the contribution of the inelastic beam fixed-end rotations as well as the effect of finite length plastic hinges at beam ends on the cyclic behavior of subassemblages and frames, two computer programs were written [4,14]. One of these programs was for the static analysis of frames; the other, for the dynamic analysis. The principal features for these two programs which are common to both are briefly outlined next.

Since the developed computer programs are intended for the analysis of reinforced concrete frames designed on the basis of the strong columns - weak beams concept, the columns were assumed to remain elastic throughout a time-history analysis. However, in order to allow for the formation of a sidesway mechanism, rotational springs with a yielding feature were provided at the column bases.

The beams were idealized as shown in Fig. 11 [14]. To account for the fixed-end rotations of the beams at the column faces during the inelastic cyclic excursions, rotational springs were provided at the beam ends. Finite lengths of Zones of Alternating Plasticity (ZAP) were assumed to extend over the end portions of the beams. This assumption contrasts with the

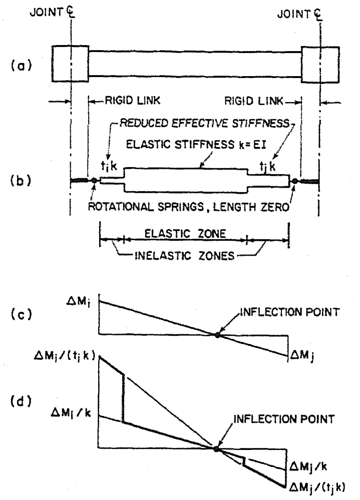

Fig. 11 ZAP Model Characteristics: (a) Actual Member; (b) Idealized Member; (c) Incremental Moments; (d) Incremental Curvatures

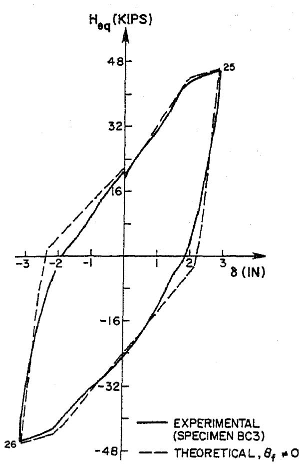

Fig. 12 Experimental and Theoretical H_{eq}-δ Diagrams [14]

usual practice of taking plastic point hinges at the ends of beams. Time-step analyses of a frame show that the lengths of the inelastic regions of a beam (ZAP) vary with time. The incremental beam curvatures were taken as shown in Fig. 11(d) in order to symmetrize the stiffness matrices. A degrading moment-curvature model, which included stiffness degradation and strain hardening features, was used to relate the beam moments with their curvature.

The developed computer program for the static analysis of frames subjected to cyclic loading was used to compare the analytical with the experimental results. Such a comparison for a hysteretic loop for Specimen BC3 is shown in Fig. 12 [14]. The agreement between the two is seen to be excellent. If the fixed-end rotation of the beam ends is not permitted in the analysis, the agreement of the analytical results with the experimental ones is poorer.

The dynamic computer program developed on the same basis was used to analyze the prototype structure shown in Fig. 1. For some severe earthquakes this analysis indicates a number of interesting results. As to be expected, the nonlinear behavior of the frames reduces the story shears by a factor on the order of three, but even then significantly exceeds the level of the lateral loads currently prescribed in the codes [1]. The increase in displacements caused by fixed-end rotation of beams appears to be surprisingly small, being approximately 8% for the derived Pacoima earthquake. For the same earthquake the base shear decreases about 10% if the beam ends rotate. These tentative results need further verification, and it must be recognized that the developed programs do not include a provision for a complete bar pull-through in the joints. In some situations the latter condition may be the most critical.

ACKNOWLEDGEMENTS

The findings reported are a result of research extending over several years of joint effort with Professor V. V. Bertero, and were supported by the National Science Foundation (current Grant No. PFR-7908984). Any opinions, findings, and conclusions expressed in the paper are those of the author and do not necessarily reflect the views of the Foundation. A number of graduate students participated in the projects described, and several of them are co-authors of the cited papers; Philip Filippou assisted with the preparation of this paper. It is a pleasure to acknowledge all of this help with gratitude.

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