# REINFORCED CONCRETE BEAM-TO-COLUMN CONNECTIONS SUBJECTED TO EARTHQUAKE-TYPE LOADING

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SUMMARY

Twelve full-size exterior reinforced concrete beam-column subassemblies were tested. The primary variables were the ratio of the column flexural capacity to that of the beam, the joint shear stress, the transverse reinforcement ratio in the joint, and the presence of transverse beams and slab. The results were combined with the results of other studies to develop a simplified design chart for the selection of the required amount of transverse reinforcement in a joint.

## INTRODUCTION

Since the early 1960's, many researchers have studied the behavior of reinforced concrete beam-column subassemblies subjected to cyclic inelastic loading. The results of tests performed in the U. S. (Ref. 1-3), Canada (Ref. 4), Japan and New Zealand (Ref. 5) were used by the ACI Committee 352 to develop design guidelines (Ref. 6). These guidelines have been under revision to incorporate the results of more recent studies (Ref. 7).

## EXPERIMENTAL STUDY

The draft recommendations of ACI Committee 352 (Ref. 7) still, in many cases, result in congested joints which are difficult to construct. The primary objective of this study was to show that in some cases joints reinforced with a lower number of hoops than that suggested by the draft recommendations will perform satisfactorily.

### Primary Variables

The effect of the following four parameters on the overall behavior of the beam-column subassemblages were studied: (1) the flexural strength ratio, defined as the ratio of the sum of the flexural strengths of the columns to that of the beam, (2) the percentage of transverse reinforcement in the joint, (3) the shear stress in the joint, as a multiple of $\sqrt{f'_c}$ and (4) the presence of transverse beams and slab.

The advantages of having stronger columns than beams at every connection has been recognized by most building codes. Previous tests of specimens with flexural strength ratios greater than 3 (Ref. 8) have indicated the primary advantage of a higher flexural strength ratio is moving the plastic hinging

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region away from the column. The flexural strength ratio for this study was varied between 1.1 and 2.0 to represent a more realistic range of flexural strength ratios.

The column and the joint of the specimens were confined by hoops. Each set of hoops consisted of a closed square tie enclosing all column longitudinal steel and a closed diamond-shaped tie enclosing only the intermediate column longitudinal steel. The joints of the specimens were designed with either two or three sets of hoops. Except for Specimen 4, the specimens had a smaller amount of joint transverse reinforcement than that of the draft recommendations (Ref. 7).

The draft recommendations (Ref. 7) limit the shear stress in an exterior joint to $12\sqrt{f'_c}$, psi. For the specimens tested, the joint shear stress varied between $10\sqrt{f'_c}$, psi and $14\sqrt{f'_c}$, psi. The joint shear stresses were calculated assuming that strain hardening will increase the stresses in the beam longitudinal steel by 10% over the measured yield stress. For specimens with transverse beams and slab, the joint shear stresses were calculated assuming that only the two slab longitudinal bars on each side of the main beam contribute to the joint shear forces.

Specimens were constructed in pairs. For each "bare" specimen, a companion specimen was constructed with transverse beams and slab, while the other primary variables remained the same.

Due to changes in the material properties and physical constraints, the final values for the primary variables in the specimens were slightly different than the original design values. The design values for the primary variables as well as the actual values are listed in Table 1.

## Dimensions of the Specimens

The column cross section for Specimens 1-8 was about 12 in. by 12 in., and had either eight or ten No. 6 longitudinal reinforcing bars. The column cross section for Specimens 9-12 was 13.4 in. by 13.4 in. Column longitudinal reinforcement in Specimens 9 and 10 consisted of ten No. 8 bars, while that for Specimens 11 and 12 consisted of eight No. 6 bars. The beam in each specimen was always 1.5 in. narrower than the column and had total depths of 17.3 in. or 18.9 in. The beams in the bare specimens had equal amount of tension and compression reinforcement which consisted of three No. 7 and three No. 6 bars. The slabs, which were 40 in. wide, were reinforced in both directions with No. 4 bars. The transverse beams had the same cross sectional area as the main beam and had a total of six No. 6 bars for reinforcement. Complete details of the design of the specimens is given in Ref. 9.

The design concrete compressive strength was 4,000 psi. The longitudinal and transverse reinforcement for all the columns was Grade 60 steel, and that for the beams and slabs was Grade 40.

## Test Setup and Instrumentation

Specimens were tested in the frame shown in Fig. 1. The frame consists of a four-hinge frame with its bottom beam bolted to the support frame. The specimen was tested with the column portion remaining vertical. An axial load, which was less than 40% of the column balanced axial load, was applied to the column and was kept constant during the test. The free end of the beam or beam and slab was also tied down to the four-hinge frame by the means of a force link which allowed rotation of the beam at that point.

A hydraulic actuator was used to apply shear forces to the top of the column. The specimens were subjected to displacement controlled loading cycles which were intended to simulate the displacements in a mild or severe earthquake. Each specimen was loaded to its yield displacement during the first cycle of loading. It was then unloaded and loaded in the opposite direction to the yield displacement and unloaded to the original position. The maximum displacement for each of the next five cycles of loading was increased by 0.25 times the yield displacement observed during the first cycle of loading. Approximately 30 electrical resistance strain gages were attached to the reinforcing bars in and around the joint region of the specimens. At selected points during each cycle of loading, the loading was temporarily stopped and the strain gage measurements were automatically recorded.

## EXPERIMENTAL RESULTS

Flexural cracks were observed in the beam and slab near the column. The region of flexural cracking extended over a distance equal to 1.5 times the depth of the beam away from the face of the column. There were very few flexural cracks in the columns except in Specimens 5 and 6 which experienced flexural hinging in the column due to low flexural strength ratios. In the specimens with transverse beams and slab, torsional cracks were observed in the transverse beams during the second cycle of loading. As shown in Fig. 2, these cracks crossed the slab, and following a spiral path, terminated in the upper column half.

Plots of the applied load vs. the load point displacement, similar to that shown in Fig. 3 for Specimen 12, were obtained for each specimen during the test. All specimens showed a loss of stiffness indicated by a "pinching" of the applied load vs. displacement curve near the zero displacement region. The severe pinching of the hysteresis curves started during different cycles of loading and was an indicator of the amount of the cracking of the concrete and the slippage of the reinforcing bars through the joint.

The damage in the joint region had a significant effect on the load carrying capacity of the specimens. A plot of the ratio of the maximum load carried by the specimen during each cycle to the maximum load carried during the first cycle of loading is given in Fig. 4. Due to premature failure, results for Specimens 8 and 10 are not shown in this figure. It can be seen that the load carrying capacity of Specimens 1, 2, 3 and 9 reduced after the first cycle of loading, while the remaining specimens were able to maintain their maximum first cycle load at least through the fourth cycle of loading.

### Bar Slippage and Pullout

A major cause of the loss of stiffness for the beam-column subassemblages is the slippage of column longitudinal bars and the pullout of the beam longitudinal reinforcement from the joint. Data from the strain gage measurements were used to determine bar slippage or pullout. Because the maximum displacement increased during each cycle of loading, it was expected that the measured strains on the column and beam longitudinal reinforcement near the joint would also increase. If the maximum strain during two consecutive cycles of loading remained the same or decreased, bar slippage or pullout had occurred.

In all specimens tested, the column longitudinal reinforcement on the face of the column where the main beam framed in, slipped. This was attributed to the excessive damage in the concrete in this region due to the formation of the plastic hinges in the beam near the face of the column. However, the column longitudinal reinforcement in the side opposite where the beam framed in either did not slip or started to slip during the latter cycles of loading.

Beam bar pullout was observed in all bare specimens except for Specimen 11, where due to low joint shear stresses, the bar maintained adequate anchorage throughout the test. In Specimen 4, which had a flexural strength ratio of 1.41, the bar pullout did not take place until the fourth cycle of loading. In specimens with transverse beams and slab, due to the improved confinement of the joint provided by the transverse beams, no pullout of the main beam longitudinal reinforcement was recorded.

## Effect of the Primary Variables

Flexural strength ratio is the major parameter determining the location of plastic hinges. Formation of the flexural hinges in the joint significantly impairs the anchorage of the beam bars in the joint. Therefore, in such specimens, column bar slippage and beam bar pullout start during the earlier cycles of loading.

In the specimens with transverse beams and slab, the flexural strength ratio was calculated assuming that only the first two longitudinal reinforcing bars on each side of the main beam contribute towards the flexural capacity of the beam. However, during the tests it was observed that all longitudinal reinforcing steel in the slabs yielded. This resulted in lower flexural strength ratios than the design values. For Specimens 5 and 6 which had design flexural strength ratios of 1.1, the actual flexural strength ratios were 0.89 and 0.87, respectively. Thus, flexural hinges were formed in the columns of these specimens just above the slab. Although the hysteretic behavior of Specimens 5 and 6 was satisfactory, formation of plastic hinges in the column may cause structural stability problems.

Among the bare specimens, the hysteretic behavior of Specimen 4, which had a flexural strength ratio of 1.41, was satisfactory. This specimen maintained its maximum first cycle load through the first four cycles of loading and the flexural hinge formed in the beam.

Transverse reinforcement in the joint is needed to resist the joint shear forces and to provide confinement for the concrete in the joint. An increase in the number of hoops placed in the joint region caused a significant improvement in the behavior of the specimens for which the flexural hinges were formed outside of the joint. In the specimens with transverse beams and slab, however, due to the confinement provided by the unloaded transverse beams, an increase in the amount of joint reinforcement did not make as great an improvement in the overall behavior of the subassemblage as in the bare specimens.

A reduction in the joint shear stress significantly improved the load carrying capacity of the specimens. For the bare specimens, when the joints could be visually inspected, the damage in the joints of specimens with lower joint shear stresses was considerably less than that in the joints of specimens with higher joint shear stresses. As a result of this, in specimens with lower joint shear stresses, the slippage of column longitudinal reinforcement and the pullout of the beam bars was delayed or eliminated.

The transverse beams in this study were subjected to a combination of indirect bending and torsional loading. Although this caused additional cracking near the joint, the presence of the transverse beams and the additional confinement these elements provided for the joint had a net positive effect on the overall behavior of the specimens.

## DESIGN RECOMMENDATIONS

The load vs. displacement hysteretic behavior is perhaps the best means for judgment of the overall performance of the subassemblage. A survey of all exterior reinforced concrete beam-column subassemblies reported in Ref. 1-5 and 8-10 was carried out and each specimen was plotted with the appropriate symbol in Fig. 5. This figure contains information about three of the primary variables. The effect of the transverse beams and slab are not included in the figure due to relatively insufficient information on the behavior of such specimens.

The maximum load and displacement at the end of each cycle for each specimen was recorded. The yield load and displacement for each specimen was also calculated. Cyclic displacement ductilities, defined as the ratio of the maximum displacement during each cycle of loading to that at the yield, was calculated and recorded for each cycle of loading (Ref. 9). The cyclic displacement ductilities were added up for all cycles of loading as long as the maximum load carried by the specimen in that cycle of loading was higher than the yield load for the specimen. This sum for each specimen was recorded as the total displacement ductility.

A total displacement ductility of 10 was selected as the lower limit for satisfactory behavior under moderate to severe earthquake loading (For example, specimens with total displacement ductilities of 10 can sustain their yield load for four cycles of loading at a displacement ductility of 2.5). Based on this criterion, the behavior of the surveyed specimens was either accepted or rejected. A solid symbol was used to indicate the accepted specimens in Fig. 5, while open symbols were used to show the rejected specimens.

In order to develop a simplified design chart, a lower limit of 1.4 for the flexural strength ratio and an upper limit of $12\sqrt{f'_c, \text{psi}}$ for the joint shear stress were selected to define the boundaries for the acceptable region of the design chart (Fig. 5). NOTE: The joint shear stress is defined as the horizontal shear force in the joint divided by the joint (i.e. column) gross area. Line A was drawn through the data points for the two specimens with joint transverse reinforcement ratios less than or equal to 0.7% which had exhibited satisfactory behavior. The second line, B, was drawn parallel to Line A and through the data point for the accepted specimen furthest away from Line A which had joint transverse reinforcement ratio between 0.7% and 1.0%.

The design chart has been thus divided into three regions with maximum required joint transverse reinforcement ratios specified for each region to achieve a maximum total displacement ductility of 10. The design chart is very easy to use. Flexural strength ratio and the joint shear stress can be calculated readily. Once these values are calculated for a particular connection, the required amount of joint transverse reinforcement can be read directly from the chart.

## CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to investigate the effect of four parameters on the behavior of external beam-to-column connections and to determine if in certain cases the existing design guidelines could be relaxed. Based on this study, the following can be concluded:

1. To avoid formation of flexural hinges in the joints, the flexural strength ratio should be no less than 1.4 for bare joints.

2. A minimum flexural strength ratio of 1.2 is recommended for connections where slab and transverse beams are present. In calculated the flexural strength ratio for such cases, the slab longitudinal reinforcement over a region at least equal to the width of the main beam on each side of the beam should be considered effective.

3. The maximum joint shear stress in an exterior connection should be limited to $12\sqrt{f'_c}$, psi to reduce excessive joint damage, column bar slippage and beam bar pullout.

4. Although slippage of column longitudinal reinforcement and pullout of beam bars are undesirable, the need to eliminate this problem completely is unwarranted. Specimens with minor bar slippage or pullout in the later cycles of loading showed a very good overall behavior.

5. Unloaded transverse beams improved the joint confinement considerably.

6. The presence of transverse beams helped eliminate the beam bar pullout. However, slippage of column bars was observed in specimens which either had or did not have transverse beams and slab.

7. An increase in the amount of joint transverse reinforcement in the specimens which had transverse beams did not improve the overall behavior of the specimen as much as it did for specimens which did not have transverse beams.

8. The presented design chart can be used as an effective and simple guide for selection of the amount of transverse reinforcement for a joint.

9. In cases where low joint shear stresses and high flexural strength ratios exist, a joint may be designed with considerably smaller amounts of transverse reinforcement than that recommended by ACI-Committee 352 (Ref. 7) without jeopardizing the overall behavior of the connection.

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TABLE 1. DESIGN AND ACTUAL VALUES FOR THE PRIMARY VARIABLES

| Specimen Number | Flexural Strength Ratio | Joint Shear Stress/$\sqrt{f'_c}$ | Transverse Reinforcement Ratio (%) | Transverse Beams and Slab |
|---|---|---|---|---|
| 1 | 1.1 (1.01) | 14.0 (14.2) | 1.0 (1.10) | No |
| 2 | 1.5 (1.35) | 14.0 (14.2) | 1.0 (1.23) | No |
| 3 | 1.1 (1.07) | 14.0 (12.8) | 1.5 (1.64) | No |
| 4 | 1.5 (1.41) | 14.0 (12.5) | 1.5 (1.85) | No |
| 5 | 1.1 (0.89) | 14.0 (10.9) | 1.0 (0.97) | Yes |
| 6 | 1.1 (0.87) | 14.0 (11.3) | 1.5 (1.46) | Yes |
| 7 | 1.5 (1.17) | 14.0 (13.5) | 1.0 (1.08) | Yes |
| 8 | 1.5 (1.16) | 14.0 (13.4) | 1.5 (1.64) | Yes |
| 9 | 2.0 (1.93) | 14.0 (15.2) | 1.0 (0.98) | No |
| 10 | 2.0 (1.58) | 14.0 (14.4) | 1.0 (0.86) | Yes |
| 11 | 1.5 (1.56) | 10.0 ( 8.8) | 1.0 (0.93) | No |
| 12 | 1.5 (1.17) | 10.0 ( 9.1) | 1.0 (0.86) | Yes |

NOTES: Numbers outside the parenthesis are the design values.
Numbers inside the parenthesis are the actual values.

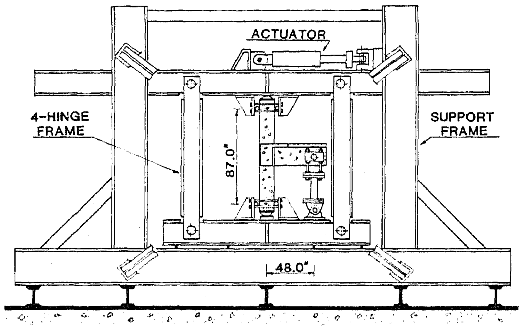


Fig. 1. Testing Frame

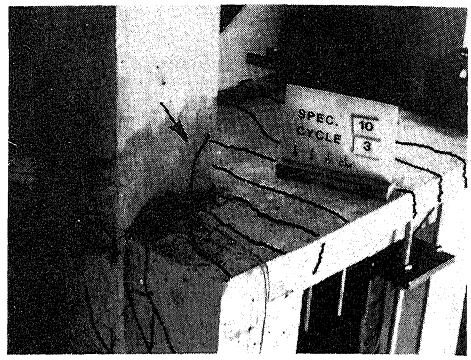


Fig. 2. Torsional Cracks in Specimens with Slab

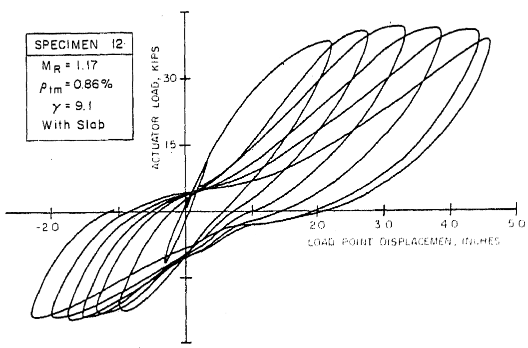


Fig. 3. Load vs. Displacement Response of Specimen 12

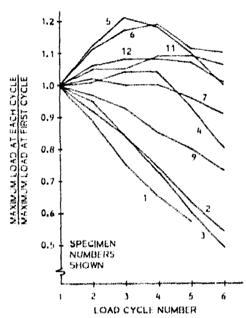


Fig. 4. Cyclic Load Carrying Capacity of the Specimens

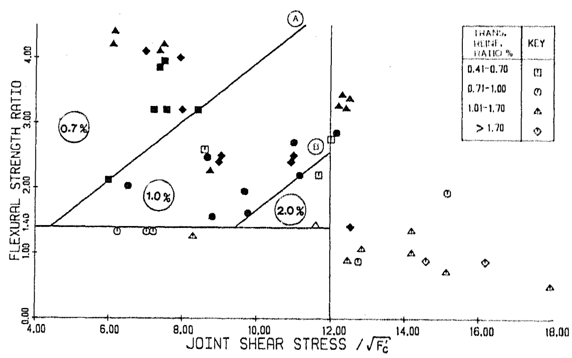


Fig. 5. Design Chart