# DESIGN OF BEAM-COLUMN JOINTS FOR SEISMIC RESISTANT REINFORCED CONCRETE FRAMES

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## SYNOPSIS

This paper describes a design procedure for beam-column joints of reinforced concrete moment-resisting frames. The procedure will assure adequate shear strength, anchorage and confinement in the joint to permit development of the flexural capacity of beams in a structure subjected to several cycles of severe lateral loads.

Moment-rotation relationships for two-full size test specimens containing joint hoop reinforcement are presented. These test results show that joints reinforced according to the design procedure will carry the column load and allow inelastic deformation of the beams during severe earthquakes.

## HIGHLIGHTS

In the design of tall buildings to resist severe earthquakes, considerable ductility must be provided in the structural frame. This requires that junctions between beams and columns be designed to ensure integrity when hinging occurs in the beams.

This paper presents details of a design procedure for beam-column joints in reinforced concrete frames. The procedure requires that three major design considerations be satisfied:

(1) Adequate confinement of the concrete must be provided to permit the column load to be transmitted through the joint should large deformations cause the concrete cover to spall off.

(2) Shear Reinforcement must be provided to resist any internal shear force in excess of that carried by the concrete.

(3) Adequate anchorage of the flexural reinforcement must be provided.

As an example, design of a joint where two edge beams frame into a column is presented. Complete reinforcing details needed for proper detailing of this joint are given.

From a more extensive investigation, two representative tests are described, one involving the edge joint selected for the design example, the other an isolated joint where a single beam frames into a column. The edge

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joint specimen was provided with 100 percent of the required shear reinforcement. However, the single beam specimen contained shear reinforcement for only about 80 percent of the shear in the joint. Reversed cycles of inelastic deformations in excess of those calculated to occur in a 24-story building during two separate catastrophic earthquakes were applied to both specimens. To test the specimen with two edge beams, one beam was deflected up while the other was deflected down an equal amount.

Measured moment versus rotation relationships for the test specimens are presented. It is shown that, even after application of deformations representing the second catastrophic earthquake, both test specimens had considerable reserve strength and ductility. It is therefore concluded that reinforcement provided by the design procedure is satisfactory for use in seismic regions.

## BACKGROUND

Ground motions during an earthquake may induce large lateral forces in highrise buildings. All structures in regions where major earthquakes may be expected to occur should be designed to resist moderate ground motions without damage. However, it may be uneconomical to proportion a structural frame to resist the most severe earthquake reasonably predictable during the life of a building without the frame undergoing some inelastic deformations.

Modern earthquake-resistant structures are designed on the assumption that, during the infrequent catastrophic earthquake, strong ground motions may cause local overstressing. This approach to design requires that adequate energy absorbing capacity be available to dissipate the motions of the structure without danger of its collapse.[1] To obtain this energy absorption, adequate ductility must be provided.

Recommendations for design of reinforced concrete frame buildings to resist earthquake motions are given in a book by Blume, Newmark, and Corning.[2] These recommendations primarily concern details of reinforcement in the spans of beams and columns, but reinforcement details to be used in the vicinity of joints are also discussed.

More recently, the Structural Engineers Association of California (SEAOC) has updated procedures for the design of seismic resistant frames.[3] On the basis of tests[4] conducted to verify previously recommended design procedures,[2] additional details for joint reinforcement were included. These provisions have served as the basis of the seismic design provisions of the Uniform Building Code (UBC).[5] Both of these documents[3,5] use the Ultimate Strength Design criteria of the 1963 ACI Building Code[6] as the basis for the structural design of reinforced concrete.

This paper describes a design procedure that will assure integrity of the joint when large rotations occur at hinging regions on each side of the beam-column junction. This method of design forms the basis for both the

SEAOC[3] and UBC[5] provisions. In this paper, recommendations are made for the extension of the provisions to structures containing high yield stress reinforcement in the flexural members. A design example is given and results of tests of two full-size specimens proportioned by this method are presented.

## DESIGN PROCEDURE

In the design of seismic resistant frames, the joint regions must be proportioned to accommodate the maximum forces that can occur. Forces that can be applied to a joint at the edge of a structural frame are shown in the free body diagram of Fig. 1. When subjected to lateral loads, the forces include tension, $f_sA_s$, and equal and opposite compression, C, due to bending in the beams; story, shear, H; beam shear, $\Sigma F$; column axial load, N; and column moment, M.

To properly design the joint, it is necessary to provide adequate shear capacity, axial load capacity, and anchorage for the flexural reinforcement. Requirements for shear and axial forces can be met by providing either closed hoop or helical reinforcement through the beam-column junction. When bars are terminated within the joint, hooks may be used to provide adequate anchorage length for the bars.

Joint Shear - From Fig. 1 it can be seen that the net shear in the joint is given by the expression

$$V = (f_sA_s)_1 + (f_sA_s)_2 - H \qquad \ldots.(1)$$

Based on Ultimate Strength Design requirements for shear and diagonal tension as given in the 1963 ACI Building Code,[6] the maximum shear force that may be attributed to the concrete is

$$V_c = 3.5\ bd\ \phi \sqrt{f'_c\ (1 + 0.002\ \frac{N}{A_g})} \qquad \ldots.(2)$$

where b and d are column dimensions shown in Fig. 1, $\phi$ is the strength coefficient for shear ($\phi = 0.85$) and $A_g$ is the gross concrete area for the column. With the coefficients given, all quantities must be expressed in pounds and inches.

In columns with low axial load, Eqs. (17-2) and (17-3) of the 1963 ACI Code will govern, so that

$$V_c = bd\ \phi \left[1.9 \sqrt{f'_c} + 2500 \frac{pVd}{M-N\ \frac{(4t-d)}{8}}\right] \qquad \ldots.(3)$$

Where V is the joint shear given by Eq. (1), M is the maximum column moment, and p is the tension steel reinforcement index for the column considered as a flexural member. Dimensions and section properties are those of the joint, as shown in Fig. 1. A simplified graphic solution for Eq. (3) is given elsewhere.[7]

Shear reinforcement must be provided for the difference between the net shear applied to the joint and that attributed to the concrete according to

Eq. (2) or (3). The cross sectional area of reinforcement, $A_v$, needed to resist this force is

$$A_v = \frac{s\,[V - V_c]}{\phi\, f''_{yh}\, d} \qquad \ldots.(4)$$

where s is the stirrup or hoop spacing, $f''_{yh}$ is the yield stress of the joint reinforcement, $\phi$ is the strength coefficient for shear, and other terms are as previously defined.

Confinement - To ensure integrity of the joint during reversals of application of high overload, adequate confinement must be provided to permit the column load to be transmitted should the cover spall off. This can be accomplished by designing the joint as a spirally reinforced column. The ratio, p", of the volume of spiral reinforcement to total volume of core measured to the outside of the spiral should be taken not less than

$$p'' = 0.45\left(\frac{A_g}{A_c} - 1.0\right)\frac{f'_c}{f''_y} \qquad \ldots.(5)$$

In Eq. (5), $A_c$ is the area of the core of the column measured to the outside of the closed confinement reinforcement and $f''_y$ is the yield stress of the confinement reinforcement.

Rectangular hoops can also be used for confinement.(2) The required center-to-center spacing, s, of hoop reinforcement is given by the expression:

$$s = \frac{3.5\, D^2\, f''_{yh}}{\left(\frac{A_g}{A_c} - 1\right) f'_c\, h''} \qquad \ldots.(6)$$

In Eq. (6), D is the hoop bar diameter, h" is the length of the longer side of the hoop and $f''_{yh}$ is the yield stress of the hoop reinforcement. Only one-half the reinforcement required by Eqs. (5) and (6) need be provided for joints confined on four sides by beams at least three-fourths the width of the column. This reduction recognizes the confinement due to the crossing beams.

In addition to meeting the requirements of Eqs. (5) and (6), a minimum volumetric ratio of reinforcement of 0.008 for cold-drawn wire, 0.010 for hard-grade bars, or 0.012 for intermediate-grade bars should be provided as recommended in Ref. 2. Center-to-center spacing between hoops or spirals should not exceed 4 in. If intermediate grade or hard-grade steel is used, the minimum diameter of hoop reinforcement should be 3/8 in. Recent tests (8) have shown that considerable ductility in the concrete can be obtained when these minimum requirements are met.

Anchorage - Adequate anchorage must be provided to assure development of the beam reinforcement. This may be accomplished conservatively by providing an anchorage length that will permit development of the yield force of the bar within the requirements of Chapter 18 of the 1963 ACI Code. For deformed bars conforming to ASTM Designation A305 and placed so that more than 12 in.

of concrete is cast below the bar, this length is the greater of 24 in. or

$$L = \frac{D^2 f_y}{\phi\, 27 \sqrt{f'_c}} \quad \text{but not less than} \quad \frac{D f_y}{\phi\, 2240} \qquad \ldots.(7)$$

For similar bars with less than 12 in. of concrete cast below, this length is the greater of 24 in. or

$$L = \frac{D^2 f_y}{\phi\, 38 \sqrt{f'_c}} \quad \text{but not less than} \quad \frac{D f_y}{\phi\, 3200} \qquad \ldots.(8)$$

In Eqs. (7) and (8), the required anchorage length, L, and the nominal diameter of the bar, D, are expressed in inches. All other terms are in pounds and inches. The strength coefficient, $\phi$, is taken equal to 0.85.

If the bar is anchored entirely within the column, the anchorage length is measured along the bar from the column face where the bar is in tension. If the bar cannot be anchored within the column, but passes through the column, the anchorage length shall be measured starting from the face opposite that where the bar is in tension and extending along the beam.

Limitations - Since present physical requirements for reinforcement do not specify an upper limit for yield stress, it may be expected that the yield stress of bars used in construction may sometimes be higher than that assumed in design. Large excesses in yield stress of flexural reinforcement may increase the shear forces within the joint enough to cause a premature loss of stiffness during high overload. Consequently, knowledge is needed of the range of excess in yield stress that can be accommodated.

Tests have shown(4) that when reinforcement is provided to resist about 80 percent of the shear in the joint, adequate strength and ductility are provided. Consequently, actual yield stress can exceed the design value by as much as 25 percent without reducing the ductility of the structure below that needed. For steel with strain hardening similar to that in Ref. 4, and a design yield stress for the flexural reinforcement of $f_y$ = 40 ksi, no problems would be encountered if its actual yield stress were 50 ksi while yield stress of the hoops and strength of the concrete were equal to the design values. Similarly, an actual yield of 75 ksi in the flexural reinforcement could be accommodated if the design was for 60 ksi.

Unlike the yield stress of the flexural reinforcement, increased yield in the hoop reinforcement and overstrength of the concrete have a beneficial effect. For this reason, no upper limit is needed on these strengths. However, design yield stress of the hoop reinforcement should be limited to 60 ksi. This corresponds to the maximum design yield stress permitted for stirrup reinforcement by the 1963 ACI Code.

## EXAMPLE PROBLEM

The forces on a beam-column junction located at the edge of a frame subjected to lateral loads are shown in Fig. 1. In this example problem, the column is 15x15 in. and is reinforced with 8 No. 11 bars. The beams are 12-in.

wide by 20-in. deep and are reinforced with 4 No 9 bars as top steel and 2 No. 9 bars as bottom steel. The column reinforcement is ASTM Designation A432 (60,000 psi yield) and both the beam steel and hoop reinforcement are ASTM Designation A15 intermediate grade (40,000 psi yield). The concrete has a design strength of 5000 psi.

## Joint Shear

The maximum shear in the joint is given by Eq. (1). The steel used in this design has a well defined yield stress and has strain hardening similar to the steel in Ref. 4. Therefore, the design steel stress is $f_s = f_y$ = 40,000 psi In this example, the story shear is H = 36,000 lb. Consequently, the joint shear is

$$V = 4\ (1.0)\ (40{,}000) + 2\ (1.0)\ (40{,}000) - 36{,}000$$
$$= 204{,}000 \text{ lb.}$$

The maximum shear that may be attributed to the concrete is determined from Eq. (2) With the minimum column axial load, N, calculated to be 640,000 lbs, shear carried by the concrete is

$$V_c = 3.5\ bd\ \phi \sqrt{f'_c\ (1 + 0.002 \frac{N}{A_g})} \qquad \ldots.(2)$$
$$= 3.5\ (15)(12.75)(0.85) \sqrt{5000\ (1 + 0.002 \frac{640{,}000}{225})}$$
$$= 104{,}000 \text{ lbs}$$

A check by Eq. (3) for the maximum shear at low axial loads requires M, the column bending moment in the joint. In this example, M = 1,800,000 lb-in. at the top and bottom of the joint. Substituting in Eq. (3)

$$V_c = 15\ (12.75)(0.85) \left[ 1.9 \sqrt{5000} + 2500 \frac{0.24\ (204{,}000)\ (12.75)}{1{,}800{,}000 - 640{,}000\ (\frac{4(15) - 12.75}{8})} \right]$$

Since the denominator of the second term is negative, Eq. (3) does not apply, the shear capacity from Eq. (2) governs.

Rearranging Eq. (4) to solve for required spacing of shear reinforcement gives

$$s = \frac{\phi\ A_v f''_{yh}\ d}{V - V_c}$$

After several tries, the spacing is calculated for No. 4 bar hoops with 4 legs in each layer.

$$s = \frac{0.85\ (4)(0.2)(40{,}000)(12.75)}{(204{,}000 - 104{,}000)}$$
$$= 3.47 \text{ in.}$$

The length of joint subject to the high shear force, V, is approximately equal to the distance between the centroids of top and bottom beam steel, $d_s$, therefore the number of stirrup layers, $n_s$, equals the beam steel separation, $d_s$ divided by the calculated stirrup spacing, s. In this case:

$$n_s = \frac{d_s}{s} = \frac{15.88}{3.47} = 4.58$$

Thus, 5 layers of doubled No. 4 hoops should be evenly spaced in horizontal layers in the joint between the beam top and bottom main reinforcement.

Confinement

The spacing of joint hoops required for confinement is calculated by Eq. (6) assuming No. 4 hoops with a maximum side length of 6.5 in.

$$s = \frac{3.5\ D^2\ f''_{yh}\ \ z}{(\frac{A_g}{A_c} - 1)\ f'_c\ h''}$$

$$= \frac{3.5\ (0.5)^2\ 40{,}000}{(\frac{225}{169} - 1)\ 5000\ (6.5)} \qquad \ldots.(6)$$

$$= 3.27 \text{ in.}$$

The hoops used as shear reinforcement can also be considered for confinement. One additional hoop layer at the top and bottom of the joint combined with the 5 layers of hoops provide the required spacing through the joint height.

Anchorage of Beam Steel

Top Bar Anchorage - The No. 9 bar anchorage length for top bars is calculated by Eq. (7).

$$L = \frac{(1.13)^2\ (40{,}000)}{(0.85)(27)\sqrt{5000}} \qquad \ldots.(7)$$

$$= 31.3\text{-in.}$$

This is greater than the lower limits of either 24-in. or:

$$\frac{(1.13)(40{,}000)}{(0.85)(2240)} = 23.7\text{-in.}$$

Therefore use 31.3-in. for top bar anchorage.

Bottom Bar Anchorage - By Eq. (8)

$$L = \frac{(1.13)^2\ (40{,}000)}{(0.85)(38)\sqrt{5000}} \qquad \ldots.(8)$$

$$= 22.3\text{-in.}$$

This is greater than the lower limit of

$$\frac{(1.13)(40{,}000)}{(0.85)(3200)} = 16.6\text{-in.}$$

but is less than the minimum anchorage length of 24 in. Therefore, use 24 in. for bottom bar anchorage.

Anchorage of the tension reinforcement may be provided from the near face of the column, unless this anchorage length cannot be developed inside the column. In this case, the column width is less than the anchorage length required. Consequently, the anchorage length must be provided in the beam measuring from the far face of the column. In this example, the bars are continuous, therefore adequate anchorage is provided.

## TESTS

Specimens similar to the one shown in Fig. 2 have been tested under simulated seismic loading at the PCA Laboratories to determine the behavior of joints between beams and columns of cast-in-place reinforced concrete frames. The 1,000,000-lb capacity testing machine shown was used to maintain the vertical load in the 10-ft long pin ended column, while cycles of up and down loads were applied to the beams at points 10 ft from opposite column faces. The hydraulically applied loading at each of the two beam ends was controlled so that, as one beam end was deflected downward, the other beam end was deflected an equal amount upward. Other details of the test rig are described elsewhere.(9)

In a building subjected to an earthquake, the magnitude of deflections depends on the building stiffness and the earthquake motion. To simulate the effect of a catastrophic earthquake similar to the ElCentro 1940, the specimen was loaded to yield of the tensile reinforcement, twice in each direction. In each of these loadings to yield of the reinforcement, the applied inelastic rotational deformation in the hinging region was several times that corresponding to the elastic limit. This ratio is defined as the ductility factor, $\mu$. Loading cycles shown graphically in Fig. 3 were chosen to represent the effects of both the ElCentro earthquake, designated earthquake No. 1, and another highly severe earthquake No. 2.

A loading cycle on the test specimen started with a download on one beam end. Deflections were increased until moment versus beam rotation at the face of the column indicated the proper multiple of original yield rotation was reached. The beam on the far side was simultaneously deflected upward an equal amount. Following return to zero load, the near beam end was deflected upward to produce the desired ductility factor while the far beam was deflected downward an equal amount. The loading cycle was completed by return to zero load.

A test specimen representing the edge-joint described in the example problem was one of 11 that have been tested. Reinforcement details are shown in Fig. 4. The column reinforcement had a yield stress of 65 ksi while the beam and hoop reinforcement had a yield stress of 44 ksi and 45 ksi, respectively. The joint hoops are correctly detailed for shear but purposely do not meet the requirements in both directions for full confinement. To meet the confinement requirements, it would be necessary to have additional ties at the mid-length of the long side of the hoops.

Measured bending moment versus beam rotation at the hinging region is shown in Fig. 5. It can be seen that the flexural capacity of this beam-column specimen increased at increasing rotations. This increase was the result of strain hardening of the intermediate grade reinforcement. There were no indications of deterioration of the joint even after more than the nine scheduled cycles of the test had been applied.

Strain measurements on the beam steel through the column indicated complete loss of anchorage within the column by load Cycle 3. Anchorage of the flexural reinforcement in the beam concrete at the far side of the column was adequate to provide the increase in strength.

A previously reported test[4] was performed on a single beam that framed into a column with the joint reinforced for 82 percent of the design shear calculated on the basis of measured properties of the materials. The column reinforcement had a yield stress of 70 ksi while the beam and hoop reinforcement had a yield stress of 48 ksi and 53 ksi, respectively. Measured moment-rotation curves for this specimen are shown in Fig. 6. This joint also showed increasing strength with increased inelastic rotation. Anchorage of the beam steel provided within the joint in this specimen was adequate. This test indicates that the design procedure will provide satisfactory details even when the yield stress of the reinforcement actually provided exceeds that used in design by as much as 25 percent.

## CONCLUDING REMARKS

This paper presents a procedure for design of beam-column joints in structures subjected to earthquakes. Major recommendations of this paper are given in the section on HIGHLIGHTS.

## ACKNOWLEDGMENTS

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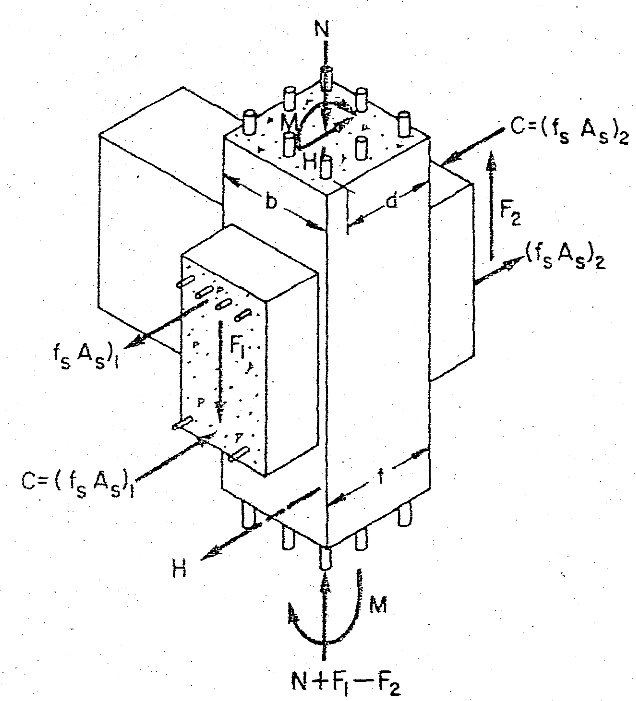


FIG. 1 FORCES ON BEAM-COLUMN JOINT

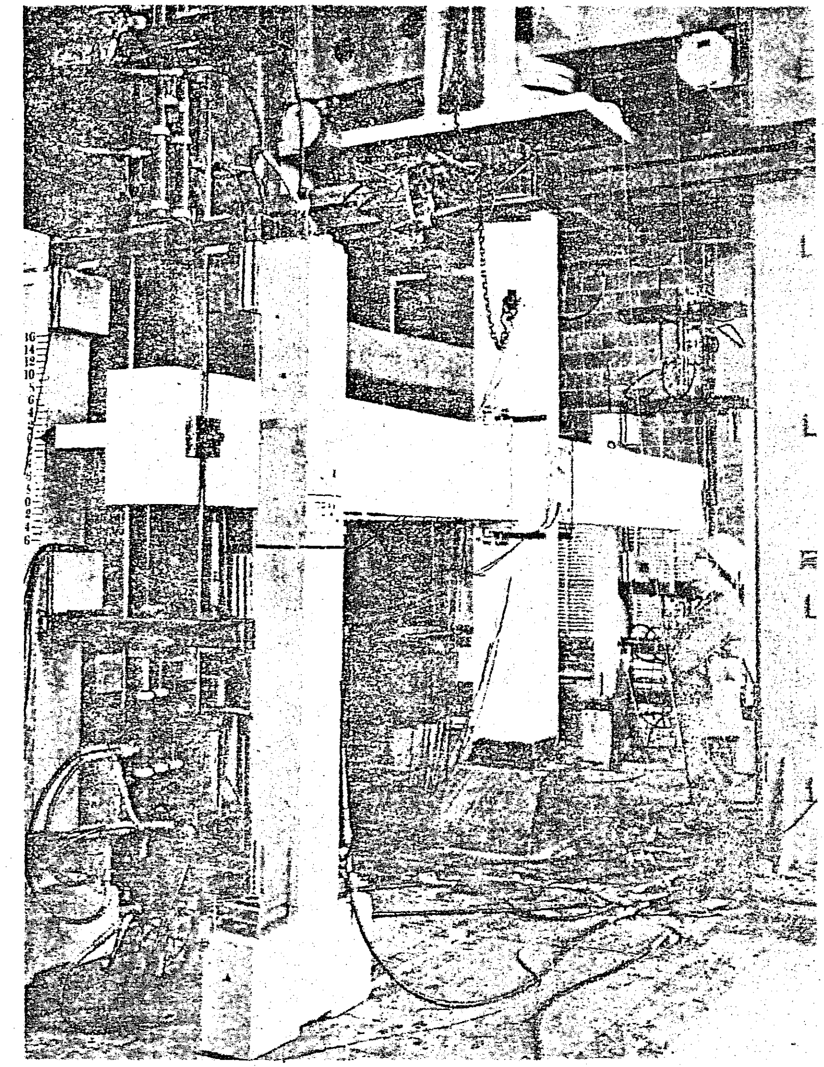

FIG. 2 BEAM-COLUMN JOINT TEST

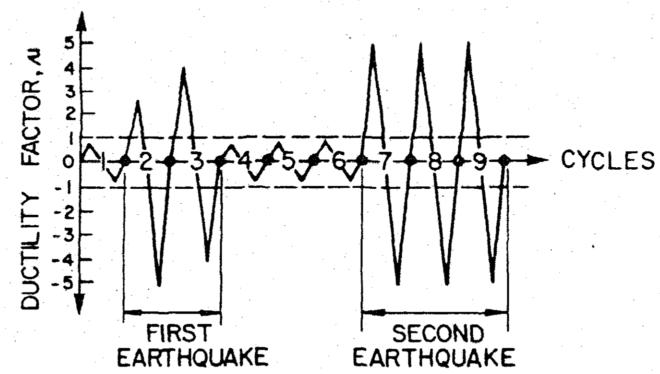


*FIG. 3* EARTHQUAKE REPRESENTATION

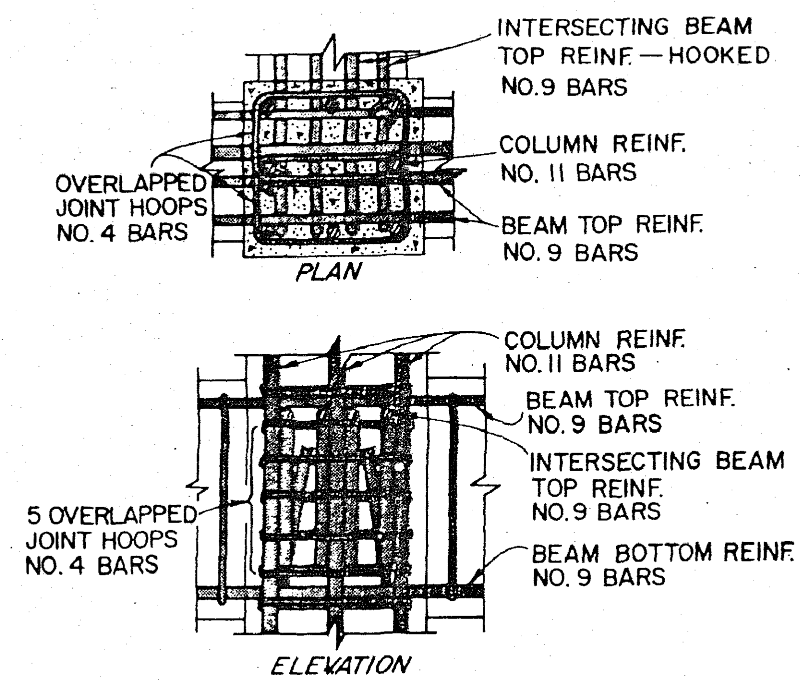


*FIG. 4* DETAIL OF REINFORCEMENT IN EDGE — JOINT SPECIMEN

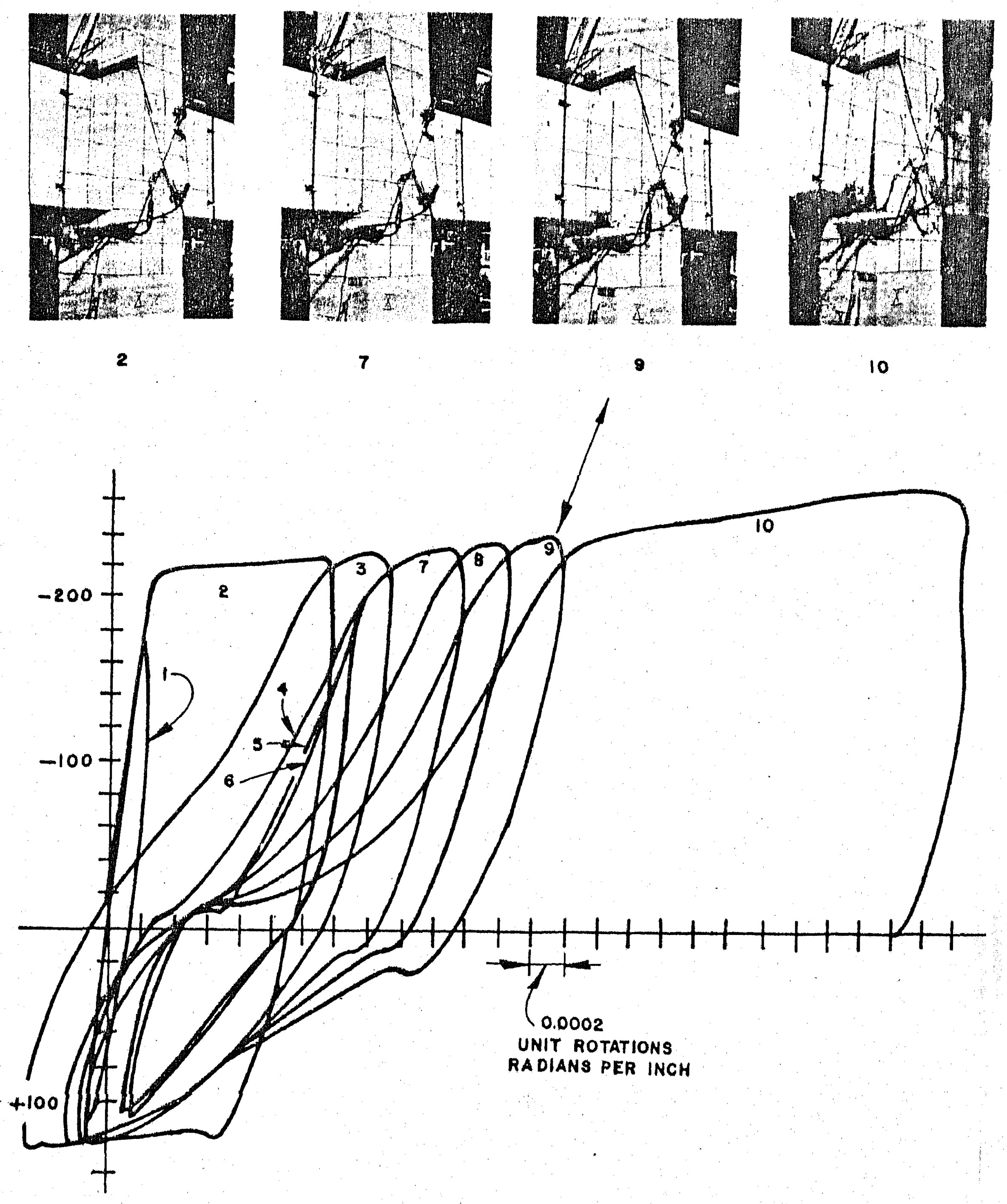


Fig. 5 MOMENT ROTATION CURVES FOR EDGE—JOINT SPECIMEN

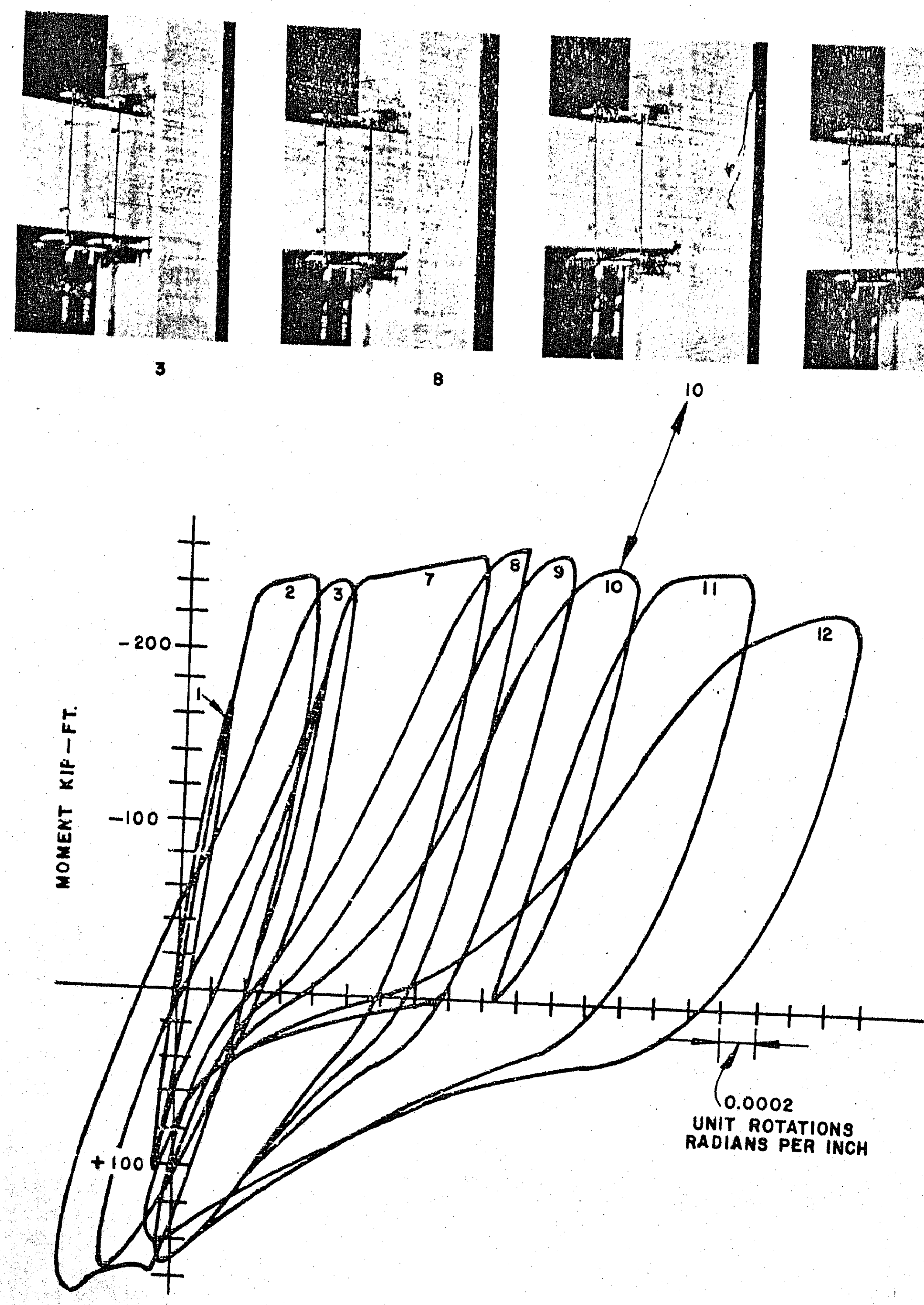


Fig. 6 MOMENT ROTATION CURVES FOR ISOLATED-JOINT SPECIMEN