The experiments were extensively instrumented, as reported elsewhere [6, 7, 8]. A notable feature of the instrumentation was the device for measuring shear deformation by means of diagonally mounted clip gages, as illustrated by the photos of Fig. 5. The gridwork seen in these photographs was used to obtain photogrammetric data on deformation. By using a comparator, the movements of targets located at the intersections of grid-lines, can be measured with considerable accuracy from photographic glass plates.

## Concluding Remarks

The main results of this study are summarized in Table 2. From an analysis of the results, it was concluded that:

(1) In each case, failure was due to the buckling of the main longitudinal bars, which occurred after a considerable amount of flexural yielding took place. To protect against the premature buckling of the main reinforcing bars, present code provisions on the spacing of lateral supports should be made more stringent. Test results and analytical studies indicate that the spacing of the lateral supports should be reduced to at least 6 to 8 bar diameters of the main bars [9].

(2) A considerable increase in the energy absorption and dissipation capacities of the beams occurs if equal amounts of longitudinal steel are used in both the top and bottom layers of the critical region. The behavior of specimens having less reinforcement in the bottom than in the top was poorer. Therefore, it would be advisable to use positive reinforcement in an amount greater than the 50% of the negative reinforcement currently allowed by the ACI (71) code provisions.

(3) Since the slab contributes substantially to the increase in the initial stiffness and the negative moment capacity of a specimen, energy absorption and energy dissipation capacities of a critical region are also increased (see Fig. 6). The strengthening effect of the slab should be included in the analytical models used for predicting the behavior of this type of critical region.

(4) Despite the special precautions taken in anchoraging the main bars (see Fig. 4), some significant pull-out of the bars occurred. In some cases, its contribution to the total tip deflection amounted to more than 40%, as shown in Fig. 7, where it is designated as $\delta_{FIXED\ END}$. Pull-out of the longitudinal bars is a result of the degradation of bond with repeated reversals of inelastic straining. Considering that the anchorage provided to the main bars of the specimens tested exceeds the one that can usually be provided in actual structures, it is clear that pull-out of the longitudinal bars can be one of the controlling parameters in the overall degradation of stiffness observed in real structures.

(5) Although there is some degradation of stiffness in the initial range of reloading during a reversal cycle (Fig. 6), this becomes accentuated only after a high displacement ductility ($\mu_\delta \geq 4$) takes place. The hysteresis loops for cycles of moderate reversals between the same peak deformations are quite stable. These results confirm previous findings [3].

(6) Most of the observed degradations in stiffness were a result of the Bauschinger effect and the bond deterioration, while some were a result of the presence of shear (most notably in the cycles with extremely large peak deformations).

Analytical Studies (Mathematical Model)

The hysteretic behavior of critical regions controlled by flexural deformations have been studied analytically. A mathematical model has been formulated, based upon the mechanical characteristics of the materials under this type of strain reversal [8].

Since the behavior of reinforcing steel under cyclic loading plays a dominant role in the overall performance of a member, an accurate analytical formulation of the behavior of steel under cyclic loading is essential. For this purpose, specimens were machined from pieces of rebars from which the reinforcement was fabricated to the dimensions given in Fig. 8. The cyclic strain history observed in one reinforcing bar of a selected beam experiment was then applied to specimens in experiments performed on MTS equipment. One such experimentally obtained diagram for a reinforcing bar is shown in Fig. 9a. A Ramberg-Osgood representation of a stress-strain curve having a modified Masing hypothesis to account for the strain-hardening and the cyclic effects was then used to develop suitable equations. Using these equations, the computer output of the cyclic stress-strain diagram (Fig. 9b) was obtained. As shown in Fig. 9, the comparison between the experimental and analytical results is excellent.

A somewhat cruder cyclic stress-strain diagram was adopted for concrete. In Fig. 10, the experimentally obtained results are indicated by dashed lines; the analytical results, by solid lines. From this figure it can be seen that the concrete mathematical model does not compare as favorably with the experimental curve. However, it is believed that for cyclic loading, this relatively simple concrete model may be sufficiently accurate, since the inelastic behavior of ductile reinforced concrete is controlled by the steel.

Assuming that the plane sections of flexural members remain plane, the analytical expressions for steel and concrete may be combined to yield a moment-curvature behavior of beams under cyclic loading. A comparison of the experimental results shown in Fig. 11a with the analytical ones obtained on the above basis, shown in Fig. 11b, is very encouraging.

## BEHAVIOR OF FLEXURAL CRITICAL REGIONS WITH HIGH SHEAR

General

To minimize nonstructural and structural damage in tall buildings, it is necessary to control the story drift of the structure. This is achieved by providing the required lateral stiffness of the structure. To achieve this objective in moment-resisting frames, it is usually more effective to increase the stiffness of the girders by increasing their size rather than that of the columns. Since this increase in stiffness is usually accompanied by the girders' increase in bending strength, care should be taken to see that this strength does not exceed certain limits. The limits are imposed by the need to avoid subjecting the critical regions of the girders to shears which are higher than those desired.

Data available on the behavior of critical regions subjected to reversals of combined flexure and high shear are very limited. However, when the available results are compared with those obtained under monotonic loadings, it becomes apparent that as the shear increases, stiffness and strength deteriorate with the increasing number of load reversals. At the same time, a considerable reduction in ductility takes place [3].

A good example of the effect of high shear is presented in Fig. 12. Here, the results obtained from one of the half-scale cantilever specimens, R-6, are compared with those obtained in an identical specimen, R-5, for which the length was 38.5 in. (978 mm) rather than the 63.5 in. (1613 mm) shown in Fig. 4a. From Fig. 12, it can be seen that there is greater degradation of stiffness and energy dissipation in the case of R-5 (high shear) than in the case of R-6. This is even more clearly illustrated in Fig. 13 where some selected hysteresis loops corresponding to similar displacement ductility ratios are compared. In the case of high shear (R-5), the loops are pinched. This pinching effect is due to the large degradation in shear resistance and shear stiffness during the initial stage of reloading (see Fig. 14). The deformation pattern of the short cantilever specimen, R-5 (Fig. 15b), clearly displays the importance of shear distortions and the rotation of the beam as a rigid body. This rotation occurs as a result of the pull-out of the top bars from the anchor block, causing a fixed end rotation, $\theta_{FE}$, which contributed $\delta_{FIXED\ END}$ to the tip deflection (Fig. 15a).

<u>Large-Short Cantilevers [6, 7, 10, 11]</u>

To study in greater detail the effect of high shear reversals on the flexural behavior of critical regions, a series of tests were performed on the cantilever specimens having the design shown in Fig. 16a. These beams had a 15 x 29 in. (281 x 737 mm) cross section and a 78 in. (1981 mm) clear span. Thus, their shear-span ratio was about 3. In each case, six well-anchored #9 (29 mm) bars, at both the top and the bottom, provided the main longitudinal reinforcement. Web reinforcement was varied. In the first three experiments, single closed stirrup-ties were used, consisting of either #3 (9.5 mm) bars at 4.5 in. (114 mm) spacing, #4 (12.7 mm) bars at 6 in. (152 mm) spacing, or #4 (12.7 mm) bars at 3 in. (76 mm) spacing [10]. Double (overlapping) #3 (9.5 mm) stirrup-ties at 3 in. (76 mm) spacing were used in three other cases [11]. In one experiment, completed to date, heavy diagonal bars forming an internal truss were used in addition to the variable spacing of double #3 (9.5 mm) stirrup-ties. Grade 60 steel was used throughout the experiments. The main specimen characteristics of this series of tests are summarized in Table 3.

Typically, the cyclic load was applied at the tip of the cantilevers, inducing the deflections of the type illustrated in Fig. 16b. The main results obtained from this series of tests are summarized in Table 4. After considerable flexural yielding, failures in the first three beams were of predominantly shear type. Beam 35, with a web reinforcement roughly corresponding to the requirements of the ACI (63) Code, performed poorly, as shown in Fig. 17. Beam 46, following the same code, but assuming that the total shear was carried by the web reinforcement alone, also behaved poorly. However, as may be seen by comparing the results of Fig. 17 with those shown in Fig. 18, a dramatic improvement in hysteretic behavior can be achieved by increasing the amount of web reinforcement and decreasing the spacing of the stirrups as it was done with Beam 43. The contribution of concrete to the shear resistance was again neglected in this beam, and the stirrups were designed to resist the shear corresponding to the maximum possible moment which the beam could resist. This required the use of the actual maximum strength of the longitudinal bars.

The characteristic stiffness and strength degradations from one cycle to the next, for the same deflection amplitude of the three beams, are illustrated in Fig. 19.

Since shear failures of beams are abrupt, they are undesirable, and for this reason several attempts were made to improve the web reinforcement. To date, three different schemes were tested to achieve this objective. In the case shown in Fig. 20, pairs of narrower overlapping #3 (9.5 mm) closed stirrups, 3 in. (76 mm) apart, were used instead of a single #4 (12.7 mm) stirrup as in Beam 43. Furthermore, in Beam 33 #2 (6 mm) supplementary cross-ties were provided near the top and bottom of the beam, and #3 (9.5 mm) supplementary cross-ties were bent around two #4 (12.7 mm) longitudinal bars in the middle of the beam. Fabrication of this reinforcing cage was somewhat time consuming, but was made to determine the beneficial effects of improving basketing of the concrete. In another experiment performed on Beam 33M, only pairs of narrower #3 (9.5 mm) stirrups, 3 in. (76 mm) apart, were used; and in the most recently completed test, heavy diagonal bars were provided.

Although considerable improvements in delaying and reducing the degrading effects of the reversals of high shear have been accomplished by using closely spaced vertical ties (see Figs. 18 and 21), it was not possible to completely eliminate this undesirable effect. Typically, after a few reversals, the diagonal tension cracks produced by the shear, not only cross similar cracks resulting from the reversals of the shear, but also intersect and combine with the vertical flexural cracks. After severe reversals, one or two nearly vertical cracks remain open at zero load throughout the cross section of the beam. At these cracks, the initial shear can be resisted only by the dowel action of the main bars which is most inefficient. It accelerates the failure of the beam which occurs in a mode that has been classified by Paulay as "Sliding Shear Failure" [12]. Because it occurs after considerable flexural yielding of the main reinforcement, its mechanism is designated herein as a Flexure-Shear Mechanism.

The effect of using heavy diagonal bars as the web reinforcement can be seen in Fig. 22. It is clear from the analysis of the results presented in this figure, that the beam exhibited remarkably stable spindle-type hysteretic loops near to failure, which occurred after a displacement ductility ratio greater than 5 took place. These hysteresis loops resemble those obtained in the case of compact structural steel beams.

The effective or equivalent plastic hinge rotations corresponding to the maximum peak deflection, LP 93 in Fig. 22, has been evaluated at 0.035 radians [11]. For the full load reversal, LP 91 to LP 93, the actual maximum plastic rotation developed was nearly twice this value (0.067 radians). The values of maximum plastic hinge rotation obtained from similar tests with W24 x 76 steel cantilever beams ranged from 0.021 to 0.054 radians. Comparison of these values with the value of 0.035 radians obtained for the concrete beam of Fig. 22, shows that by the use of proper web reinforcement, it is possible to achieve a maximum plastic hinge rotation in reinforced concrete regions similar to that available in equivalent compact steel regions. This is the best that can be achieved.

From the results discussed above, it is clear that with the inclined bar web reinforcement it is possible to virtually eliminate the degrading effects of high shear. However, the economic feasibility of placing this type of reinforcement in the field is questionable. Thus, simpler and more economical reinforcing schemes for minimizing the pinching shear effect require further study. Such schemes should also be designed so as to minimize the significant rigid body rotations of the beam caused by the slippage (pull-out) of the main bars along their anchorage in the column joint (Figs. 7, 15a, 15b, and 22b).

Although the main bars used in the large-short cantilevers were welded to a steel channel that was cast with the column (Fig. 16a), significant slippage of the main reinforcing bars was observed. This slippage increased as the severity of loading and the number of reversals increased, giving rise to the rigid rotation of the beam which in some cases accounted for nearly 20% of the total tip deflection (Fig. 22b).

The new reinforcing schemes under study are illustrated in Fig. 23. As the figure shows, it consists of controlling the formation of the critical flexure-shear region at a selected distance from the face of the column. This approach of resolving the problem appears to be very advantageous.

## Concluding Remarks

From the results obtained to date on the effect of high shear reversals, the following conclusions can be drawn:

(1) Increase in the shear forces at the critical regions of flexural members, reduces their energy absorption and energy dissipation capacities.

(2) When the average shear stress at the critical region reaches

values in excess of $3.5\sqrt{f'_c}$ (psi), the degradation in stiffness with a reversal of load becomes considerably greater than that for flexural critical regions with low shear stresses. This also applies to the energy dissipation capacity of each cycle with an increase in the number of cycles. Degradation in strength occurs as the number of similar loading cycles increases. Failure of these regions results only after some significant flexural yielding of the main steel has taken place.

(3) The ability of the critical regions to maintain load and energy dissipation capacity (hysteresis loop stability) is significantly improved by reducing the stirrup-tie spacing. For cases where the shear stress reached values on the order of $5\sqrt{f'_c}$ (psi) or greater, and where the web reinforcement was designed according to present code recommendations, the degradations in strength, stiffness, energy absorption and energy dissipation per cycle, increased considerably as the number of cycles and the magnitude of deformations also increased. To minimize these degradations, the design of web reinforcements must be made by: (a) neglecting any contribution of the concrete in resisting shear stress; and (b) proportioning the web reinforcement for the shear corresponding to the actual maximum bending capacities of the critical regions, i.e. these capacities should be based on the actual strength of the main reinforcing steel, including strain-hardening, and not on the minimum specified yield strength.

(4) To avoid excessive damage which leads to degradation, the shear force that can be developed at the critical sections when a region is subjected to severe repeated loading reversals, should be limited in magnitude to a value lower than the one recommended by the ACI Building Code, which is approximately $10\sqrt{f'_c}$ (psi). Unlike the case occurring in pure flexure, the degradation of the critical regions for such members cannot be predicted by consideration of the bond deterioration and the Bauschinger effect alone.

(5) By using inclined web reinforcements in combination with vertical ties, it is possible to minimize the degradation of stiffness and to obtain stable hysteresis loops for a ductility displacement exceeding 4.

(6) The use of inclined web reinforcements permitted the development of a maximum effective plastic hinge rotation of 0.035 radians; 0.067 at a full load reversal. These values are similar to those found for compact steel beams.

(7) The shear effect in a member cannot be noted from an analysis of the measured moment-average curvature relationship alone, since the contribution of shear deformations is not detected by simply measuring rotation at the critical region. The use of diagonal clip gages proved to be an effective method of approximately determining the shear distortion that occurred in the critical regions. The photogrammetric technique also proved to be useful for evaluating the shear distortion. However, this procedure was especially valuable for studying both the degrading and the failure mechanism of the critical regions. The Flexure-Shear Mechanism observed in these experiments will develop only under the reversal of inelastic deformations well beyond the flexural yielding of the beam. This mechanism develops only in regions where yielding of the main reinforcement takes place. Thus, special

web reinforcements against the shear are needed only in regions where flexural yielding can occur. Along the remainder of the member, closely spaced vertical ties would provide adequate web reinforcement, although the nominal average shear stress is the same as that at the critical region.

(8) Significant pull-out of the main bars gives rise to an increase in tip deflection. This pull-out increases with the severity of the loading and the number of reversals. In practical cases where the main bars cannot be anchored mechanically by welding, as in this series of specimens, the observed bond deterioration can lead to severe anchorage problems at the column joints.

## BEHAVIOR OF BEAM-COLUMN SUBASSEMBLAGES

### General

As pointed out in the Introduction, to predict the overall behavior of a structure, one needs to know the behavior of individual subassemblages. Such studies are presently being conducted. Using the third-floor framing in the 20-story moment-resisting reinforced concrete building of Fig. 3 as the prototype, a half-scale subassemblage with an interior joint was designed (Fig. 24). In the lower stories of such buildings, the beams are designed primarily to resist lateral forces. Since the gravity loads have only a small influence on the behavior of these beams under severe seismic loadings, it was possible to terminate the beams of the subassemblage at their respective midspans with a hinge. The columns were also terminated in hinges at their respective midheights.

In this subassemblage, inelastic action was to develop in the beams, i.e. the design philosophy of strong columns-weak girders was followed. The 17 x 17 in. (432 x 432 mm) column with twelve #6 (19 mm) main bars was conservatively selected. Moreover, at the joint region, overlapping hoops of #2 (6 mm) deformed bars at 1.5 in. (38 mm) apart were used. The beams were reinforced in exactly the same manner as specimens R-3 and R-4 of the half-scale cantilever series of experiments (Fig. 4 and Table 1). The testing arrangement for the cross-shaped specimen was such that an axial column force, as well as vertical forces at the ends of the beams, could be applied to it. Whereas the top hinge of the subassemblage remained fixed in position, the other three hinges could be displaced horizontally upon application of a horizontal force at the lower hinge. At large displacements of the lower hinge, the P-$\delta$ effect caused by the vertical load in the column was significant.

Four similar subassemblages have been tested to date. A brief discussion of the major results follows. One of the specimens was tested under a monotonically increasing load. The lateral load-deformation relationship (H vs $\delta$) is shown in Fig. 25. From this figure, it can be seen that the curve is of the softening rather than the strain-hardening type. This is as to be expected from the results obtained with the beams, Fig. 6, together with the added P-$\delta$ effect. The significance of the P-$\delta$ can be noted from the comparison of the two curves shown in Fig. 25. Besides the H-curve, there is another one for the equivalent story shear, $H_{eq}$, which is related to the measured story shear by the relationship, $H_{eq} = H + P\delta/h_{col}$.

For a lateral displacement ductility ratio of 4, the P-δ effect causes a decrease in the story shear capacity greater than 40%. This effect cannot be neglected in either the analysis or design as it may lead to premature lateral instability.

In Fig. 26a, a comparison is made between the analytically predicted H vs δ hysteresis loop which includes the P-δ effect, and one which neglects it. The loops corresponding to just one cycle of full deformation reversal are based on a bilinear elastic-linear strain-hardening moment-curvature model. The comparison of these loops shows that if the hysteretic behavior of the subassemblage consisted only of loops with full reversals of displacement, the P-δ effect would not affect the energy dissipation capacity of the subassemblage. On the other hand, if its deformation response were of the incremental type, not having complete reversals of displacement, the P-δ effect would considerably decrease the energy absorption and energy dissipation capacities of the subassemblage.

In Fig. 26b, the experimental hysteretic loop is compared with the analytical one of Fig. 26a. The agreement for the monotonically increasing story shear is excellent. However, large discrepancies can be noted during the loading in the reverse sense and these discrepancies become greatly magnified during the initial reloading of the second cycle. The following questions therefore arise:

(1) Why is there such a sharp degradation in strength during the first reversal, after just the first loading to a displacement ductility ratio of 4.5?

(2) Why is there such a pronounced degradation in stiffness during the first reloading, after just one cycle of a full reversal?

Since the amount of nominal shear stress developed in the beams was small [on the order of $3\sqrt{f'_c}$ (psi)], similar to that induced in the cantilever beams of Figs. 6 and 12b, it is clear that the observed degradation was not the result of shear in the beams. The main reason for this behavior was the slippage (pull-out) of the beams' main longitudinal reinforcement along the column joint. This is clearly shown in Fig. 27 where the sum of the measured pull-out and push-in of the steel bars is plotted.

The effect of repeated load reversals can be seen from the results presented in Fig. 28. These results were obtained from tests conducted on the specimen used in obtaining the results of Fig. 25 after it was repaired by injecting epoxy into the cracks.

The results for the first four cycles of load reversal are illustrated by the curves shown in Fig. 28a. Although it was possible to achieve the strength attained during the first loading of the virgin specimen, this strength was achieved at a considerably greater deformation. During the second cycle, there was a large drop in strength at the first peak deformation reached of the initial loading. The main reason for this decrease in strength was a sudden decrease in stiffness. As can be seen from the curves of Fig. 28b, this decrease in stiffness became further accentuated as the peak deformations and the number of cycles of load reversals increased.

During the initial loading, after the beams have been repaired (region OA in Figs. 28a and 29a), the behavior of the repaired specimen resembles that of the specimen in its virgin state. This demonstrates that the epoxy injection technique is an efficient way of repairing cracks and regaining most of the original stiffness at a working or service load range. However, as soon as new cracks start to develop (around A) there is a decrease in the stiffness of the specimen (region AB), in comparison with its behavior in the virgin state. It is believed that this decrease in stiffness is due to the slippage of the beams' main reinforcing bars along a large part of the column width (pull-out and push-in) and between beam cracks.

The epoxy injection technique appears to be ineffective in restoring bond. The repaired specimen developed a resisting force only slightly greater than the one developed in the original, but this occurred at a peak deformation (point B) about 50% greater than before. The observed increase in strength can be attributed to the fact that the main bars were strain-hardened during the first test on the original specimen.

The lateral stiffness during load reversal (range CD) was considerably less than the one observed during the first loading (range OA). This again indicates that once cracks begin to develop and propagate, greater deformations occur because the epoxy injection has failed to restore bond.

During the initiation of the second cycle (range EF) the stiffness was considerably less than the corresponding one in the previous cycle (range OA). Furthermore, at loading stage F there was a large drop in stiffness, and consequently, in resistance. It is believed that the main reason for this abrupt change in behavior was a sudden loss of anchorage of the beams' main reinforcing bars throughout the width of the column. The mechanism for this phenomenon is illustrated by the sequence of sketches in Fig. 29. Although at stage E, H = 0 (Fig. 29e), some internal forces in the beam remain because of the moment introduced by P-$\delta$. However, at loading stage E' (Fig. 29e'), the forces in the beams' main reinforcing bars are practically zero, and the beams are cracked throughout their depth at both faces of the column. Application of a lateral force, H, induces forces in the bars pulling the main bars from one side and pushing them from the other. Thus, at loading stage F (Fig. 29f), each of the three #5 (16 mm) beam bottom bars was pushed-in and pulled-out by a force,

$$T_{LF} \doteq \frac{7k \times 72''}{2 \times 72''} \times \frac{63.5''}{(16-3.31)''} \frac{1}{(3)} = 5.85k \doteq C_{RF}$$

Since the bond deteriorated during the first test of the original specimen, the simultaneous action of tensile and compressive forces acting on the same bar was large enough to overcome the friction and the epoxy restored bond; consequently, the bar slipped. Because the widths of the cracks at both faces of the column were small, the beam concrete quickly came in contact with the column, thereby offering restraint to the further slippage of the bar (Fig. 29g). At this loading stage (sketch g), the beams were able to offer further resistance to the applied force, H, and an increase in stiffness was observed. However, the resistance of the specimen at the same peak deformation as the previous cycle (stages H and B in Fig. 29a), was reduced by more than 50%. The above mechanism of stiffness degradation is confirmed by the results obtained from measuring the pull-out and push-in of the beams' bars at the face of the columns (Fig. 30).

It is clear from Fig. 28, that in order to achieve the original maximum lateral resistance of the specimen, it is necessary to deflect the subassemblage 50% more than that required during a first loading. It can also be noted that as the number of load reversal cycles increased, a considerable degradation in strength developed in comparison to the case of monotonically increased loading. The hysteretic loop tended to assume an N-shape, with an extremely significant reduction in energy dissipation. The main reason for this degradation was the slippage of the beams' main bars, as illustrated by the results presented in Fig. 30.

## Concluding Remarks

From the results obtained in the four subassemblage experiments completed to date, the following observations can be made:

(1) The P-$\delta$ effect can be extremely pronounced on the lateral load deformation response of the lower stories of tall buildings, and must be considered in the aseismic design of such buildings. To minimize or avoid the danger of an incremental (crawling) type of collapse (dynamic instability), limitations should be established for the story drift at yield, and for the maximum story lateral displacement ductility ratio.

(2) At an interior joint, significant degradations in strength and stiffness, and therefore, in energy absorption and energy dissipation capacities, take place under severe repeated reversals of lateral loadings. While under a monotonically increasing lateral load, it was possible to attain a ductility ratio of 5 without any decrease in strength of the specimen; under repeated reversals of increasing deformation, a considerable drop in lateral resistance was observed after a ductility ratio of 2.5. Drastic pinching of the hysteretic loops was noted after a displacement ductility ratio of 2.5.

(3) The early drop in resistance and the drastic pinching of the hysteresis loops are caused by the bond failure of the beams' main longitudinal bars along the width of the column. After starting to yield in tension at one face of the column and compression at the other face of the column, these bars finally pull-through.

(4) Epoxy repairing is very effective in restoring stiffness at the service limit states. However, its efficiency in restoring the stiffness beyond these limits is questionable. It appears that epoxy injection cannot redevelop bond. Strong evidence of premature bar slippage within an epoxy repaired interior joint was observed when the