progressive damage reduced the transfer of forces across inclined cracks and led to loss of strength and stiffness. Consequently, this mode was denoted as an eventual shear failure in spite of the yielding of the reinforcement. That is, flexural yielding of longitudinal reinforcement occurred initially followed by a degradation of the shear strength eventually resulting in a shear failure. It should not, however, be confused with the abrupt shear failure of monotonically-loaded beams.

Other types of failures observed included buckling, rupture, loss of anchorage and splitting. The buckling failures involved local instability of longitudinal reinforcing bars in compression due to loss of lateral restraint when the concrete cover spalled. Premature rupture of longitudinal reinforcement in tension was observed in three specimens; it was associated with embrittlement of the steel at locations where instrumentation had been welded to the bar. One specimen underwent an anchorage failure, and another specimen, with a tee cross section, failed due to splitting along the interface between the slab and stem. In figures which indicate mode of failure, the rupture, anchorage and splitting failures are lumped into a single category entitled 'other'.

RANGES OF STRENGTH AND DEFORMATION CAPACITY ACHIEVED DURING THE TESTS

Figure 3 shows the ratio of maximum force (V_{test}) to calculated or nominal strength (V_n, which was the smaller of V_{mn} or V_{vn}) plotted with respect to the minimum available deformation capacities quantified in terms of the ductility (μ_1) and chord rotation (θ). Though not all of the specimens were deemed failed when they attained the maximum displacement (Δ_{max}), they were deformed to a sufficiently large extent to develop their nominal strengths ($V_{test}/V_n > 1$). Of the three specimens which did not attain their nominal strengths, only two failed. These two specimens were loaded with axial loads approximately equal to 50 and 100% of the load corresponding to balanced strain conditions and failed by buckling of longitudinal bars in compression.

Ductilities exhibited by the test specimens (Fig. 3a) ranged from 1.5 to 18, with more than 90% of the data falling within ductility values of 2 and 9 (average ductility was approximately 5). With the exception of tests which indicated ductilities less than 3, the type of failure did not appear to affect either the ratio of observed to calculated strength (V_{test}/V_n) nor the ductility (μ) exhibited by the specimens. Nine of ten specimens exhibiting ductilities less than 3 failed in shear; the tenth failed prematurely due to rupture of longitudinal reinforcement at an instrument weld location. Corresponding chord rotations are shown in Fig. 3b. If chord rotation is taken as a crude approximation of interstory drift, then the deformation levels achieved by the specimens represent interstory drifts in excess of 1.4%. In some cases the deformations exceeded values consistent with 10% drift.

Figure 4 shows the relation between chord rotation and ductility for all specimens in this study. Even though these two quantities exhibit an approximate degree of correlation, they are not identical. In Fig. 4, it can be seen that two data points with equal ductilities may have chord rotations that differ by as much as twice the smaller value. Both ductility and chord rotation are based on the maximum displacement (Δ_{max}), but they are normalized by vastly different quantities; span length (a) and yield displacement (Δ_{y1}). In fact, both measures of deformation capacity were included in this study because they represent different aspects of response.

It is of interest that two specimens (B4 and C1) exhibited deformation capacities that far exceeded those of all other specimens in this study (Table 5). These two specimens along with C3, which also exhibited a very large ductility, were reinforced with Grade 40 steel. All specimens except those in series "B" and "C" were reinforced with Grade 60 steel.

DISCUSSION OF PARAMETERS WHICH AFFECT DEFORMATION CAPACITY

Several factors affect the amount of deformation achieved by beam elements. These factors include: amount of longitudinal reinforcement, shear span-to-depth ratio, amount of transverse reinforcement, presence or absence of axial load, presence or absence of slab, and type of load history applied (magnitude and number of cycles, as well as loading direction). The effect of these parameters is investigated in the following sections.

Longitudinal Reinforcement Ratio

The ratio of longitudinal reinforcement area in tension to effective cross-sectional area, ρ, is plotted with respect to ductility and chord rotation in Fig. 5 for specimens with equal top and bottom reinforcement ratios. Data points connected with lines represent results from similar specimens for which the primary variable was the reinforcement ratio. No conclusions may be drawn from the data for specimens A1 and A3 because the tests were terminated before the specimens failed; however, these data points do provide an indication of the lower bound minimum available deformation capacities that may be expected. The data for A2 and A4 indicate that although the reinforcement ratio does not have much of an effect on chord rotation, it varies inversely with ductility. A 40% decrease in ductility was observed when the reinforcement ratio was increased from 1.5 to 2.7% (Fig. 5a).

An increase in the amount of longitudinal reinforcement is generally believed to limit the deformation capacity of beam cross sections. For monotonic loading, an increase in the amount of tension steel shifts the neutral axis towards the tension face of the beam, decreasing the strain of tension reinforcement and thus curvature at failure; where failure is defined when the limit concrete compression strain is reached (13). However, not all phenomena associated with response to monotonic loading can be extrapolated to behavior under cyclic loading. Specimen A2 failed in accordance with the shear failure mode for cyclic loading as described earlier, and specimen A4 suffered an anchorage failure due to bond deterioration under reversing loads. Consequently, neither specimen could achieve the magnitude of strain in tension reinforcement that is possible under monotonic loading, and a decrease in maximum deformation was not observed as the reinforcement ratio was increased (Fig. 5b). The decrease that was observed in ductility with increasing reinforcement ratio (Fig. 5a) was due, in its entirety, to larger yield deformations associated with larger forces required to yield the specimen as the amount of tension reinforcement increased.

These results do not agree with experimental observations made by Brown and Jirsa (14) on specimens reinforced with Grade 40 steel. Results from these tests led to the conclusion that larger values of reinforcement ratio resulted in larger shear forces which, in turn, hastened failure. The tests conducted by Brown and Jirsa (14) were quite similar in configuration, dimensions, materials and loading history to those tested by Gosain (1). The detrimental effect of increasing the reinforcement ratio was observed as a reduction in the number of load cycles which a test specimen could resist before failure.

It is difficult to extrapolate a trend in energy dissipation capacity because as reinforcement ratio increases, yield force and yield deformation increase and two counteracting effects are mobilized: The vertical ordinate (force) of the force-deformation relation is increased, but the magnitude of post-elastic deformation is reduced.

Unsymmetrically reinforced beams, that is beams with more longitudinal reinforcement on one face than the other, have a high propensity for failure due to buckling of longitudinal bars in the less heavily reinforced face (8). Bars in the less heavily reinforced face are more prone to buckling because they yield in both tension and compression during cyclic flexural loading (15). Bars on the other face yield only in tension. Cyclic (tension-compression) yielding leads to a reduction in the steel tangent modulus at stresses less than f_y due to the Bauschinger effect; thus making buckling more likely. The likelihood of buckling also depends on the spacing of the transverse reinforcement.

The effect of unsymmetrical reinforcement (ρ_{bot}/ρ_{top}) is shown with respect to minimum available deformation capacity in Fig. 6. For the specimens that were loaded to failure (H3 and H6), an unsymmetrical reinforcement arrangement led to a buckling failure, while the symmetrically reinforced beam failed in shear. These specimens were similar except for the imposed displacement history (Table 4) and a small variation in the web reinforcement detail (Table 2). However, the symmetrically reinforced specimen (H6) exhibited a slightly lower deformation capacity than the specimen which had different amounts of top and bottom steel (H3). One possible explanation is that specimen H3, which had less longitudinal reinforcement in one of its faces than did specimen H6, attracted smaller shear forces in one of the loading directions and underwent less damage than did specimen H6.

Due to the high propensity for buckling of bars in unsymmetrically reinforced beams, the capacity for dissipating energy by hysteresis is also affected. Bertero and Popov (16) and Ma et al. (8) indicated that when equal amounts of top and bottom reinforcement are present in the hinge region, a considerable increase in energy dissipation may be obtained. Ma et al. (8)

reported increases in energy dissipation ranging from 27 to 54% over that for unsymmetrically reinforced alternatives, when top and bottom reinforcement ratios were equal.

Shear

It is generally accepted that shear forces have a detrimental effect on the deformation and energy-dissipation capacities of reinforced concrete beams. Shear failure for cyclically-loaded beams was the predominant mode of failure for the specimens in this study. The effect of shear force on beam behavior was a function of inclined cracking in two loading directions and localized damage adjacent to the inclined cracks. In this study the effect of shear forces is quantified in terms of three commonly-used parameters; shear span-to-depth ratio (a/d), measured shear stress (v_{test}) expressed in terms of multiples of $\sqrt{f'_c}$, and web reinforcement ratio (ρ_v). In addition, another parameter, web reinforcement efficiency ratio (WRER), was defined in the course of this study to quantify the efficiency with which the web reinforcement was used to resist shear forces.

Shear span-to-depth ratio--The ratio of shear span to effective depth (a/d) is probably the most common parameter for quantifying the effect of shear on the behavior of beams loaded with concentrated forces. It is shown plotted in Fig. 7 versus ductility and chord rotation for some of the specimens considered in this study. Data points connected by solid lines represent results from specimens in a series of tests that differed only in the shear span length, with values of this parameter varying from 2 to 6 times the effective depth.

Six of the 14 data points shown in the figures corresponded to tests in which the specimens were not taken to failure, including most of series "G". Consequently, any inferences must be made with care. Nonetheless, the data indicates that for some specimens an increase in the shear span-to-depth ratio led to a reduction in the ductility (Fig. 7a), this was especially true for the specimens in series "A" and "H". In general, as shear span-to-depth ratio increases, flexural stress increases in relation to shear stress, and enhanced deformation capacity is the expected outcome. However, the data for test series "A" and "H" indicate the opposite, that stocky beams undergo larger ductility factors than do slender beams. It has been shown elsewhere (15) that for a given displacement ductility factor, slender beams require a greater available curvature ductility factor in the plastic hinge region. This is attributed to the relative ratio of the plastic hinge length to member length which tends to be larger for the case of stocky beams.

Fig. 7b suggests a similar trend affects maximum chord rotation for the specimens in series "A", but the specimens in series "H" exhibited a modest increase in maximum deformation as shear span was lengthened.

For the specimens in series "H", shortening the shear spans augmented the shear response mechanisms, thus resulting in stiffer beams. The shorter and stiffer specimen (H5) had a smaller yield displacement and exhibited a larger ductility factor, even though the maximum deformation actually decreased a small amount over that exhibited by the more slender and flexible specimen (H6). The results of the specimens in series "A" must be qualified because the loading in these tests differed dramatically from the idealized displacement history. Rather than being loaded in groups of cycles for which the peak displacements increased incrementally, these specimens were loaded in cycles at an arbitrary peak deformation level which remained constant over the duration of each test. For the specimens with short shear spans (A13 and A14), the arbitrarily selected peak deformation level corresponded to a ductility factor of 8, while that for the specimens with longer shear spans (A1 and A5) was approximately one-half as large (Fig. 5). The specimens with shorter shear spans actually underwent greater distress under the larger shear forces and failed in shear in much fewer load cycles (Table 4).

From the foregoing, it is clear that shear has a measurable and detrimental effect on deformation capacity, at least for those specimens which were loaded to failure and for which the loading met the requirements of the idealized displacement history (series "H"). However, in some cases shear span-to-depth ratio does not distinguish between beams that suffered shear distress and those which did not. It does not serve to indicate the relative degree of shear distress in the web reinforcement and the concrete which is a function of the amount of transverse reinforcement present. It can be further concluded that ductility can provide misleading information (series "H"). Changes in shear span produce not only changes in deformation capacity, but also changes in stiffness and yield deformation (Fig. 7a). Furthermore, the increased contribution of shear deformation to tip displacement for specimens with shorter spans tends to inflate the values of ductility, when in reality, flexural deformations decrease. These shear deformations have also been observed to reduce energy dissipation (5,7).

Measured shear stress--The magnitude of shear is often expressed as an average shear stress, for which shear force at a section is divided by effective cross-sectional area. In order to determine the state of shear distress in concrete, it is useful to represent the measured shear stress (V_{test}/bd) at a section as a multiple of $\sqrt{f'_c}$. This parameter, at section of maximum shear, is plotted with respect to ductility (Fig. 8a) and chord rotation (Fig. 8b). Because none of the investigations included a series of tests in which average shear stress was varied explicitly, Fig. 8 includes data for all test specimens considered in this study.

It can be seen from Fig. 8 that some specimens failed at measured shear stresses smaller than $2\sqrt{f'_c}$ (the nominal shear strength of concrete, v_c), but most of these were buckling failures. Most of the data correspond to stresses ranging between $2\sqrt{f'_c}$ and $7\sqrt{f'_c}$. It can also be seen that 26 of the 28 specimens that failed in shear were subjected to shear stresses that exceeded twice the nominal shear strength of the concrete ($4\sqrt{f'_c}$). These shear failures represented the majority of the failure types that occurred at shear stresses exceeding a value of $4\sqrt{f'_c}$. It appears that flexure-dominated reinforced concrete beams detailed to avoid premature failures (buckling, rupture and anchorage) have a very high likelihood of failing in shear, if these are cycled to exhaustion. But, those beams which suffer premature failures will seldom achieve imposed shear stresses in excess of $4\sqrt{f'_c}$.

While the measured shear stress can be used to indicate the mode of failure of the specimens, it is insufficient to indicate the magnitude of deformation capacity of the specimens. In Fig. 8, the three largest ductilities were exhibited by specimens with relatively low shear stresses ($2\sqrt{f'_c}$ to $3\sqrt{f'_c}$). However, for the remaining 66 specimens there is no clear correlation between measured shear stress and either ductility or chord rotation. This feature can be explained in part by the fact that parameters, particularly the amount of web reinforcement, were varied among the specimens represented in Fig 8. For many of the specimens, the limit state of concrete (inclined cracking) was reached early in the tests. By the time these specimens had attained their minimum available deformation capacities, a considerable amount of damage to the concrete core had taken place at inclined cracks. Clearly, web reinforcement plays a significant role in mechanisms of shear resistance for cyclically loaded beams, and measured shear stress alone does not provide much insight into the state of distress in web reinforcement. It is necessary to correlate the amount of web reinforcement available in the section to resist the measured shear.

High shear stresses also reduce the ability of a beam to dissipate energy by hysteresis as shear deformations induce progressive pinching of the load-deformation response. Tests by Ma et al. (8) indicated that beams with shear stresses on the order of $5.3\sqrt{f'_c}$ had half the energy dissipation capacity of a beam with shear stresses on the order of $3.5\sqrt{f'_c}$. The damage to the critical section associated with shear deformation and pinching may also lead to earlier shear failure.

<u>Web reinforcement ratio</u>--Reinforced concrete beams are most easily strengthened in shear with transverse or web reinforcement in the form of stirrups or hoops. The amount of web reinforcement in a beam is quantified in terms of a ratio, ρ_v, in which the cross-sectional area of the legs of the stirrups, A_v, is normalized by the effective area of concrete, namely the product of the beam width, b, and the stirrup spacing s. Web reinforcement ratio, expressed as a percentage, is plotted with respect to ductility and chord rotation in Fig. 9 for all tests. Only three data points were available for web reinforcement ratios between 1 and 1.75%. However, the scatter and range of ductilities for specimens with web reinforcement ratios exceeding 1.75%, for which data are numerous, is much the same as that for specimens with web reinforcement ratios less than 1%.

Data corresponding to three experimental series in which web reinforcement ratio was the primary variable are shown in Fig. 10. Explanatory notes should be made regarding two of the points in Fig. 10. The ductility and chord rotation for specimen E2 were lower than those for the preceding and succeeding specimens in series "E-F". This was the consequence of premature failure of specimen E2 due to rupture of a longitudinal reinforcing bar in tension near a weld used to attach instrumentation to the bar. Data point E3 appears to have a disproportionately high ductility factor in comparison with other specimens in series "E-F". Even though this specimen was intended to be similar to the others in this series, the longitudinal reinforcement had a relatively low yield strength (60 ksi) in comparison with the other beams in this series (67 ksi). This resulted in a lower yield deformation and consequently greater ductility in comparison with the rest of the specimens in this series.

As is clearly evident, the addition of web reinforcement led to increases in minimum available deformation capacity for the specimens represented in Fig. 10. This is particularly true for the specimens in series "E-F" for which there was a large range of web reinforcement ratios and in which all specimens were loaded to failure. The effectiveness of the web reinforcement, however, tends to decrease with increased ratios of web reinforcement. This suggests that there is a limit to the effectiveness of web reinforcement. For the data in Fig. 10, specimens with web reinforcement ratios less than 1% exhibited marked improvements in minimum available deformation capacity as the amount of web reinforcement increased, but for specimens with web reinforcement ratios larger than 1%, ductilities were limited to approximately 5.

Energy dissipation capacity is improved by using closely-spaced stirrups (2,5,7,8,14,16-18). Tests by Popov et al. (5) showed a 40% increase in energy dissipation using closer spaced stirrups and ties. It was also found to improve rotational ductility.

<u>Web reinforcement efficiency ratio</u>--In an attempt to quantify the demands imposed upon the web reinforcement as the specimens were taken to their minimum available deformation capacities, a new parameter, web reinforcement efficiency ratio (WRER), is proposed. The web reinforcement efficiency ratio is intended to provide a relative measure of the stress level induced in the web reinforcement by shear force at a section. It is defined as

$$WRER = \frac{V_{test} - V_c}{V_s}$$

where V_{test} is the maximum shear force measured during a cyclic load test, and V_c and V_s are the nominal shear strengths attributable to concrete and web reinforcement, respectively. In defining this equation (WRER), it was assumed that the shear strengths attributable to the concrete (V_c) and the web reinforcement (V_s) are independent quantities and can be estimated accurately. It was further assumed that mechanisms of shear resistance not represented by V_c and V_s produce negligible shear strength contributions. The numerator represents the portion of the maximum shear force which is resisted by the web reinforcement. This quantity may be debatable as it is difficult to assess the amount of shear resistance attributed to the concrete in a cyclically loaded member; however, $V_c = 2\sqrt{f'_c}bd$ was chosen as it is the most conservative estimate provided in the ACI 318-89 Code (9). (Note that in the New Zealand Standard, NZS 3101 (19), the shear strength V_c attributed to concrete is assumed zero in plastic hinge regions of beams.) The amount of shear attributed to the reinforcement is then expressed as a fraction of the nominal strength of the web reinforcement, which is also calculated according to the recommendations of the ACI 318-89 Code (9). A ratio of unity indicates that the web reinforcement is stressed to its nominal capacity. It should be noted that because WRER is a relative measure of stress in web reinforcement, it cannot be used to distinguish between specimens with low or high shear stresses.

Ductilities and chord rotations are plotted with respect to WRER in Fig. 11. No single series of tests in this study held all variables constant except for WRER. Therefore, results for all specimens are shown in Fig. 11 even though a number of parameters other than WRER were varied among the specimens. While there was more scatter in chord rotation (Fig. 11b) than in ductility (Fig. 11a), the same trend is present for both parameters. In Fig. 11, it is evident that the minimum available deformation capacity of the specimens decreases as the reserve strength of the web reinforcement is exhausted. Concerning the data in Fig. 11, it is worth noting that WRER serves to indicate shear critical behavior: nearly 80% of all shear failures in the test data occurred when the web reinforcement efficiency ratio exceeded one-half (WRER > 0.5), and only three of the 25 specimens stressed to a value of WRER larger than 0.5 did not fail in shear. In fact, from comparison of Fig. 8 and Fig. 11, it can be concluded that WRER is a better indicator of shear distress than measured shear stress (v_{test}). It should also be noted that WRER exceeded unity for several of the specimens failing in shear indicating that for these specimens the web reinforcement was stressed beyond its nominal capacity. This is not surprising because the ACI 318-89 Code (9) expressions for the nominal shear strengths of concrete (V_c) and web reinforcement (V_s) are conservative in most cases.

The web reinforcement efficiency ratio (WRER) was defined for the specific purpose of determining the efficiency with which web reinforcement was used to resist shear forces. However, web reinforcement serves two fundamental purposes: it resists shear forces, and