yield at 105 ksi (7382.6 kgf/cm²) but made up only a total of 1.01% of the cross-section area. Load was applied at eccentricities of 1.5, 3.5 and 5.5 inches (3.81, 8.89 and 13.97 cm). The average value of P_{test}/P_{code} at ultimate strength was calculated to be 1.005 while the average P_{test}/P_{theory} using the general theory was 1.024. Both methods predicted that the steel was well below yield when failure occurred.

Use of Steel with no Definite Yield Point

Deformed wire with no definite yield point was used for reinforcement for columns of Group II and Type "W" steel in Group III. Code predictions of ultimate load were made using a hypothetical stress-strain curve with a sharp yield at 73.9 kis (5196 kgf/cm²) which corresponded to a strain of .0035 on the actual stress-strain curve, according to the Code requirements. Theoretical interaction curves as determined by the Code provisions and the general theory, representative of the columns reinforced with deformed wire, are shown in Fig. 6. The general theory interaction curves are smooth, reflecting the smooth nature of the deformed wire stress-strain curve. Again, a deviation between the curves is noted at high eccentricities where the Code predicts failure before the hypothetical yield point of the steel is reached in any bar, while the general theory allows the full potential of the materials to be realized.

Code predictions of ultimate load for Group II were generally higher than the experimental results as shown in Table II. Various degrees of slip were evidenced in most of these columns, probably the result of the poor bond characteristics of the deformed wire and inadequate anchorage. Discounting the columns which obviously failed in bond, however, the overall correlation with the code is close enough to verify the Code provisions for reinforcement of this nature. The increase in strength predicted by the general theory for angles of eccentricity of $22\frac{1}{2}°$ and 45°, as shown in Fig. 6 were not obtained.

The type "W" columns of Group III exceeded the Code prediction of ultimate strength, except for column DW 2, which suffered an obvious bond failure. Columns loaded at an eccentricity of 2.5 inches (6.35 cm) correlated with the general theory predictions as represented in Fig. 6. However, equal success in reaching the general theory strength was not obtained at 5 inch (12.54 cm) eccentricity. The most likely reason is that relatively high strains must be developed in the tension reinforcement at large eccentricities, and the poor tension bond characteristics of the deformed wire therefore had a greater effect on ultimate load in this area.

CONCLUSIONS

1. The equivalent rectangular stress block with a limiting concrete strain of .003, as recommended in the ACI Code, is an adequate representation of the concrete stress block for calculation of ultimate capacities of square, corner reinforced tied columns, loaded at any angle of eccentricity. Where major deviations from the Code predictions occur, the Code is conservative. For steels of high yield

stress, the Code imposed limiting concrete strain does not allow full development of the steel strengths at certain eccentricities. As a result, the full potential of the steel is not realized because of the artificial strain constraint. Therefore, the general theory or actual ultimate loads are somewhat higher than those predicted by the Code.

2. No apparent simple modification of the Code provisions can be applied to predict the greater ultimate column strengths occurring with high strength reinforcement, since the relative influence of the reinforcement on the ultimate column capacity has a great bearing on the ability to develop high yield stresses. An alternative method is the application of the general theoretical method as described in this paper. Although the general theory is somewhat more complex to apply than the simple Code provisions, the use of high speed electronic computers minimizes the difficulties involved.

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