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Computer Study of Long Columns in Frames

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An extensive computer study of long column action in frames is summarized for both unbraced and braced building frames. This summary paper (1) indicates the weakness of the *R*-method in caring for column buckling in certain frames subject to sidesway, (2) compares the accuracy of the moment magnifier and *R*-methods in this and other situations, and (3) gives the background for the modified *R* equations mentioned as alternates in the explanation of changes of the Building Code.

Keywords: buckling; columns (supports); frames; long columns; reinforced concrete; structural design.

THE CURRENT REVISION of the ACI Building Code has introduced the moment magnifier method for designing long reinforced concrete columns, to replace the empirical *R*-method of ACI 318-63. One of the studies available to Committee 318 was that reported here. Computer methods which have proved accurate in checking results of column frame tests were used in this study for the analysis of 204 columns in unbraced frames and 367 columns in braced frames.

This summary paper (1) indicates the weakness of the *R*-method in caring for column buckling in certain frames subject to sidesway, (2) compares the accuracy of the moment magnifier and *R*-methods in this and other situations, and (3) gives the background for the modified *R* equations mentioned as alternates in the Building Code.

An earlier paper[1] has pointed out the significant influence of beam reinforcement and beam cracking on column moments and eccentricities. For braced (but not for unbraced) columns, consistency of the results has been improved by using *R* as the ratio of long column strength to $P_{h/t=1}$, a specially calculated short column strength at $h/t=1$ which reflects the cracked beam stiffness and an increased (more realistic) initial e/t different from the nominal e/t of simple moment distribution.

Many variables have been included in these analyses, including (in 1963 ACI Code notation) nominal r', nominal e/t, p_t of columns, p of beams (with p' equal in amount), f_c' (same for beam and column), f_y (60 ksi, 4340 kg/cm², for beams but varying for columns), and to some extent the distance $d-d'$ between the column steel layers. Only square columns with equal steel on two opposite faces were considered. No creep effect was included.

Many theoretical combinations of variables proved totally impractical. In an unbraced frame only a heavily reinforced beam can brace a large column with an eccentricity of $0.1t$; the beam moment will typically be as large as the column moment plus the beam vertical load moment. In a braced frame only a very heavily reinforced beam can put a large moment into the column because the beam capacity must then exceed (usually by several times) the moment it delivers to the column.

Columns in unbraced frames—Shear loads

All the practical sidesway data are compared in Fig. 1 with the computer evaluations of P_u

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plotted vertically and the value of RP_{short} horizontally. The inadequacy of the 1963 Code value for R for transient loads is shown in Fig. 1(a). On casual inspection the possibility of using Code Eq. (9.4) in Fig. 1(c) appears quite good, but closer study (in Fig. 2) indicates a very low special test value* and other low instability failures.

The moment magnifier provisions in the Code revision are similarly evaluated in Fig. 1(b) and 1(d), based on the two proposed approximate equations for EI. The approximate EI used in Fig. 1(b), generally thought to be the less accurate evaluation, shows to be the safer of the two and sometimes a little on the wasteful side. The other approximation, Fig. 1(d), indicates more unsafe values, especially at low eccentricities (large P_u values). ACI-ASCE Committee 441, Reinforced Concrete Columns, has shown that these unsafe values would be correctly evaluated if the net moment of inertia of the cracked beam sections were used to establish the relative column stiffness; but this is not required in the Code revision.

All these data are again plotted against h'/t and h'/r in Fig. 2. The upper curve, specified in the 1963 ACI Code for transient loads like wind, again shows as inadequate. The slightly lower curve, although not intended in the 1963 Code for transient loads, is reasonably usable up to h'/r of 50, if the beam steel ratio is at least $p = 0.01$:

$$R = 1.07 - 0.008\,h'/r \leq 1.0$$

For $h'/r \leq 50$ this R equation is definitely safe only if the beams have at least 1 percent of reinforcement p.

With $h'/r > 50$ the low values (and many others plotting above the lower curve) represent stability failures. The R-method cannot fully cope with stability failures without becoming quite wasteful. The point marked "test" represents a frame

*A physical frame test with low beam steel by German Blomeier at The University of Texas at Austin; report in process of preparation.

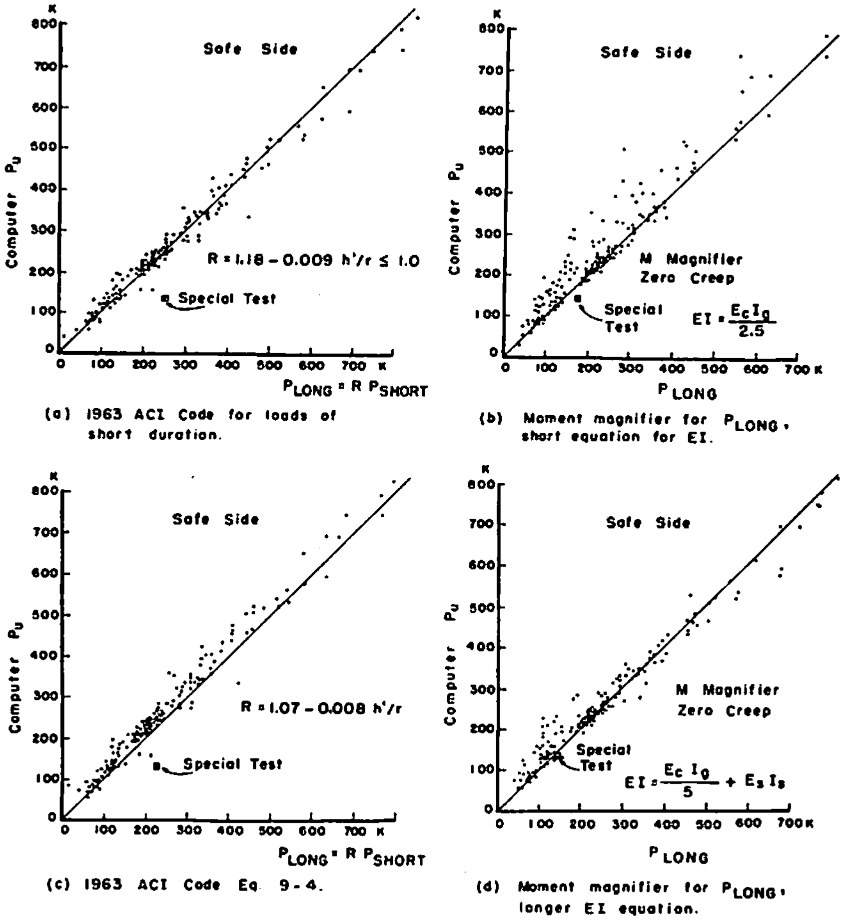

Fig. 1 — Columns with *sidesway*. Comparative safety of different methods. All data are for 10 x 10 in. columns (25.4 x 25.4 cm) without allowance for creep. Multiply kips by 0.44 to get metric tons

tested specifically to show the ill effects of small percentages of reinforcement in beams.

Columns in braced frames—Single curvature

All practical computer values of R for single curvature columns have been plotted in Fig. 3 and the lower bound is sketched. These data clearly indicate that at h/r of 67 ($h/t = 20$) or more an increased value for R (marked proposed) is justified. A study of points in the region of h/t of 5 or 10 indicated that all the extremely low values were for e/t of 0.3. A further study of the other points there ($e/t = 0.1$) is shown in the upper right-hand corner of the figure, with most curves terminating at an extreme practical value marked with a small circle. If one accepts a 5 percent deficiency (6 percent in 2 cases) the liberalized value:

$$R = 1.23 - 0.008\, h/r \leq 1.0$$

is adequate for cases where $e/t \leq 0.1$, this still leaving, above an h/r of 50, at least the allowances noted by the percentages in Fig. 3 for creep deflection. Creep allowance must increase with h/t, but below an h/r of 50 creep can create only a small excess deflection. For $e/t > 0.1$ the more restrictive equation of the 1963 ACI Code is still adequate and is recommended.

The accuracy to be expected when R is used as indicated is given in Fig. 4(a) for all practical cases investigated. The corrected e/t values mentioned earlier are not used in Fig. 4(a), but when used with a selected wide range of variables in Fig. 4(c) they give a better and safer correlation. For the moment magnifier method in the Code revision, the accuracy to be expected is comparable [Fig. 4(b) and 4(d)] if a creep factor R_m is as the revision suggests.

CONCLUSIONS

Unbraced frames

1. The 1963 ACI Building Code Eq. (9.5) must be abandoned as unsafe.

2. The moment magnifier provisions of the ACI Code revision are necessary for columns having h'/r values in excess of 50 and for columns braced by beams having a negative moment steel ratio lower than 1.0 percent. Accuracy is improved if the relative column stiffness is based on the cracked section stiffness of the beams.

3. For columns with h'/r values up to 50, supported by beams having a negative moment steel ratio of at least 1.0 percent, Eq. (9.4) of the 1963 Code would be adequate:

$$R = 1.07 - 0.008\, h'/r \leq 1.0$$

Beams bracing these columns should be designed at the joint for increased lateral load moment.

Braced frames

4. *Short* column designs should typically be made for increased (corrected) eccentricities reflecting a better frame analysis.

5. For columns in single curvature the R equations below would be as satisfactory as the moment magnifier method in the Code revision.

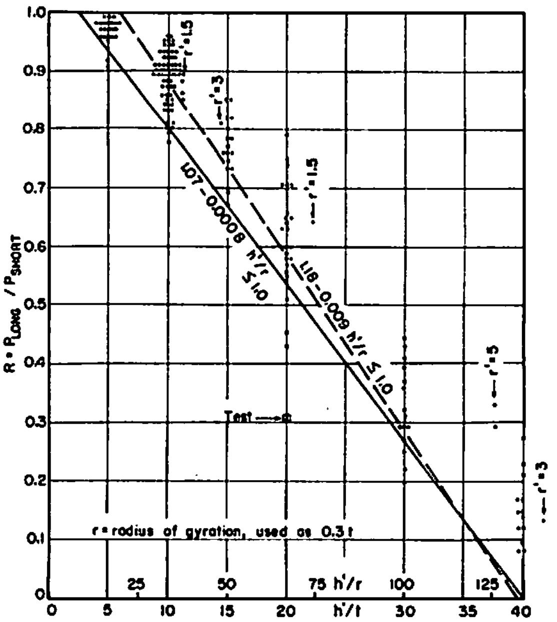

Fig. 2 — All practical data compared to ACI 1963 Code values of R. Unbraced frame, no creep

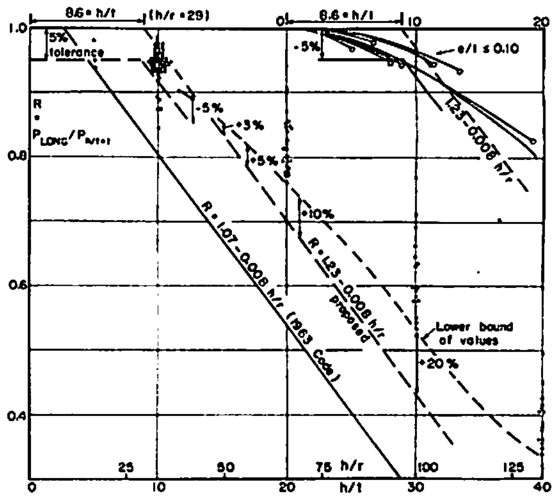

Fig. 3 — All practical R values for single curvature case compared to present and proposed column reduction equations. Upper corner traces all critical (low) values with $e/t \leq 0.1$ to practical limit marked by circles

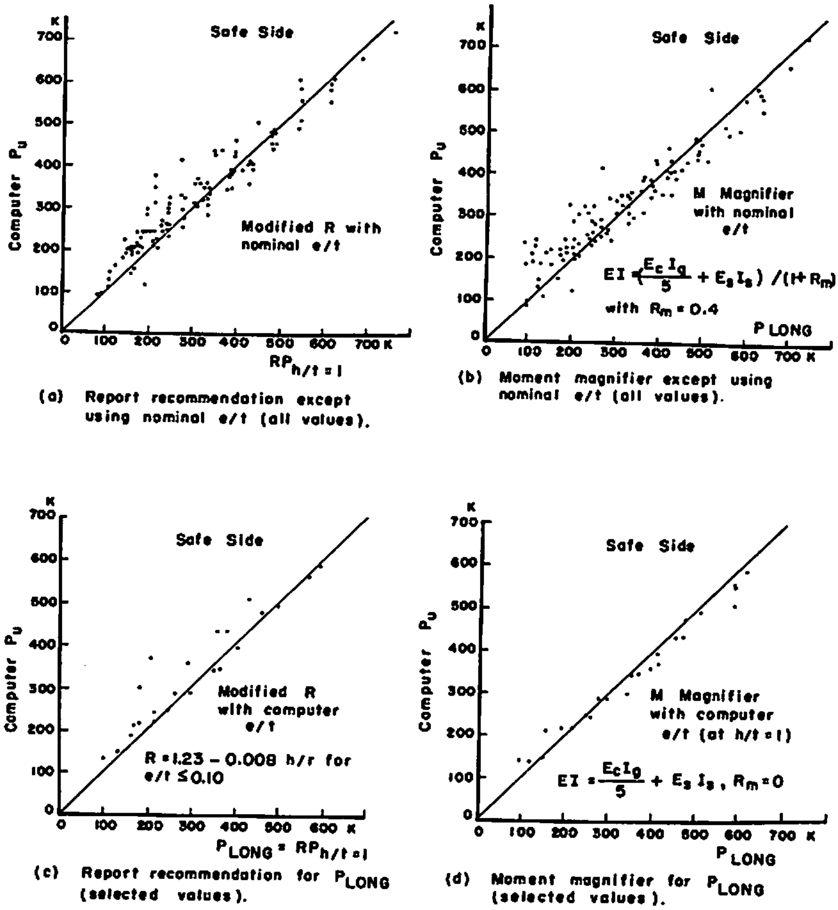

Fig. 4 — Comparative safety of two methods for single curvature columns in braced frames, nominal 10 x 10 in. columns (25.4 x 25.4 cm). Multiply kips by 0.44 to get metric tons

a. Length effects may be ignored for h/t values up to 8.6 (h/r up to 29), with errors less than 5 percent.

b. For nominal e/t values of 0.10 or less, a *relaxed* load reduction relation would be adequate:

$$R = 1.23 - 0.008\ h/r \leq 1.0$$

c. For nominal e/t values larger than 0.10, and especially for nominal e/t of 0.3, the 1963 Code value is still adequate:

$$R = 1.07 - 0.008\ h/r \leq 1.0$$

6. For columns in reversed curvature, the 1963 Code value of R is conservative, even when moment at one end is much less than at the other.

ACKNOWLEDGMENTS

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NOTE: This is an ACI summary paper. It presents the major important material from parts 2 and 3 of a study: "Long Columns in Frames—Computer Analysis," Part 2, "Columns in Braced Frames," and Part 3, "Columns in Unbraced Frames—Shear Loading." Copies of the full reports are available from Prof. Phil Ferguson, 212A Taylor Hall, Austin, Tex., 78712. Copies of the reports will be maintained on permanent file at American Concrete Institute Headquarters and when copies are no longer available, xerographic or similar copies may be obtained from there at a charge equal to the cost of reproduction and handling at the time of request.

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