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Discussion of iterative limit load analysis of tall frames, By N. C. Lind, ASCE 90, ST2, p. 103, April 1964 and Journal of the Structural Division, ASCE, Vol. 90, No. ST4, August 1964

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### Recommended Citation

Lay, M. G., "Discussion of iterative limit load analysis of tall frames, By N. C. Lind, ASCE 90, ST2, p. 103, April 1964 and Journal of the Structural Division, ASCE, Vol. 90, No. ST4, August 1964" (1964). *Fritz Laboratory Reports*. Paper 1793.  
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## DISCUSSION OF "ITERATIVE LIMIT LOAD ANALYSIS OF TALL FRAMES"

by N. C. Lind, Proc. ASCE, 90(ST2), p. 103 (April 64)

Discussion by M. G. Lay

The writer wishes to congratulate Professor Lind on his excellent paper, which properly emphasises the importance of axial load moments in the design of tall unbraced frames. However, the author's approach to the design of the columns in such frames must be viewed with some caution. The comments given below apply only to this aspect of the paper.

Lind advances the interesting concept of generating and correction moments which are undoubtedly useful in solving beam-column problems of the type given in Examples 1&2. Unfortunately the same comment does not apply to the design of unbraced tall frames (Ex3). In Ex3 the correction moments  $M^C$  are seen to be insignificant. Lind's assumption concerning the behavior of the columns in such frames is illustrated in Fig. D1.

The fact that  $M^C$  is usually small, might introduce an unnecessary complication into design by requiring that the small quantity be considered, but it does not invalidate the process. The difficulty arises in those cases where  $M^C$  is significant and where it should, therefore, be of importance. In these cases the relation between Lind's assumed curves and the actual curves <sup>D1</sup> will normally be as shown in Fig. D2. Not only does the actual curve have a lower moment-capacity than assumed, but it also has a lower rotation capacity <sup>D2</sup>. The problem is not the size of  $M^C$  but the form of the actual curve.

The reason for the difference between Lind's assumed curve and the actual curve can be readily explained. In such cases the column hinges do not form at the ends of the members <sup>D3</sup>, as Lind assumed, but somewhere within the

length. This behavior is illustrated in Fig. D3 for a beam-column with one end pinned. The writer has shown<sup>D3</sup> that this limit between Figure D1 and D2 is approximated by (for 36 ksi steel)

$$\frac{P}{P_y} = \frac{1 - \lambda_x}{1 + \lambda_x} \quad (D1)$$

where

$$\lambda_x = \frac{L}{r_x} \cdot \frac{\pi}{\sqrt{G/E}} \quad (D2)$$

and

$$P_y = A \sigma_y \quad (D3)$$

Thus the zones in which the correction should be significant are also the zones in which its validity is questionable. It may be noted, however, that most columns in tall buildings will behave as shown in Fig. D1; Lind's method does not cover columns in single curvature in any instance<sup>D2, D4</sup>.

One further comment concerns the question of design for deformations. If a sway deflection limitation of (storey height/300) is adopted, and Stevens' hypothesis<sup>D5</sup> with respect to deflections at collapse is also used, the limiting deflections at collapse are  $4 \times 144/300 = 1.92''$ . Now Lind's solution to Heyman's problem<sup>12,13</sup> gives the maximum story deflection as 6.37'' (Table 3, Column 4) or 1/23 of the storey height. This is quite large.

From these calculations one might conclude that, in a practically designed structure, a much greater lateral stiffness would be employed and that the  $P\delta$  moments in the columns would be less critical than indicated by Example 3. In other words, the necessity of limiting the lateral deflections of a tall building will likewise limit the effect of  $P\delta$  moments.

The writer has intended his above comment to add to, rather than detract from, Professor Lind's approach. The writer fully concurs with the philosophy behind Lind's approach to the problem.

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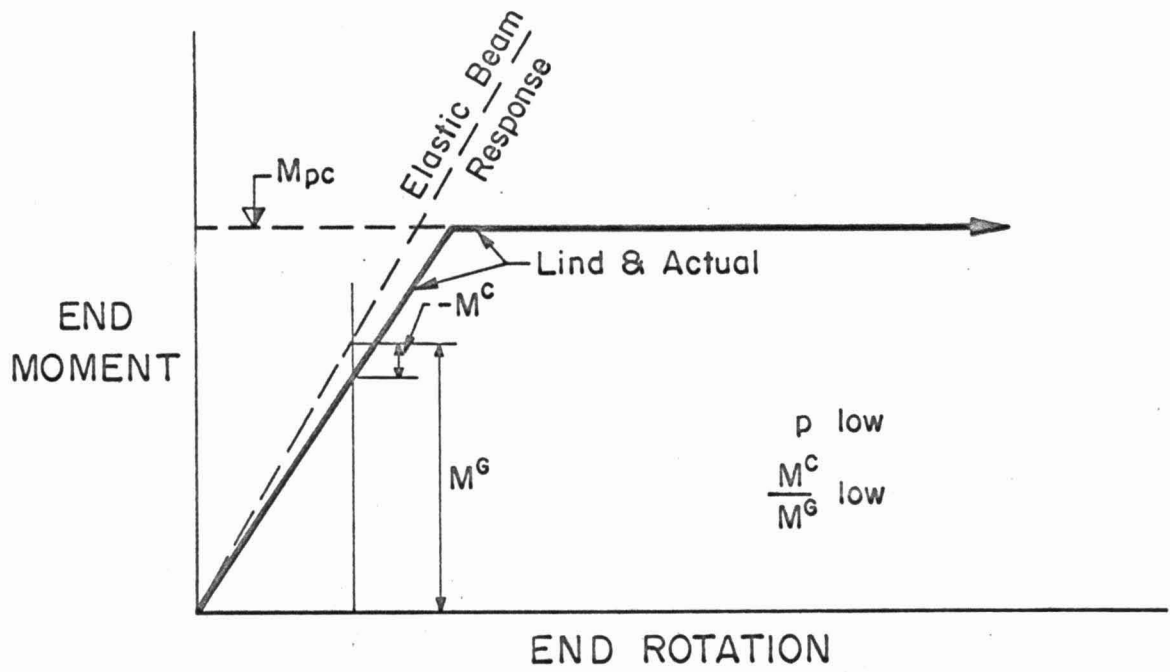


FIG D1

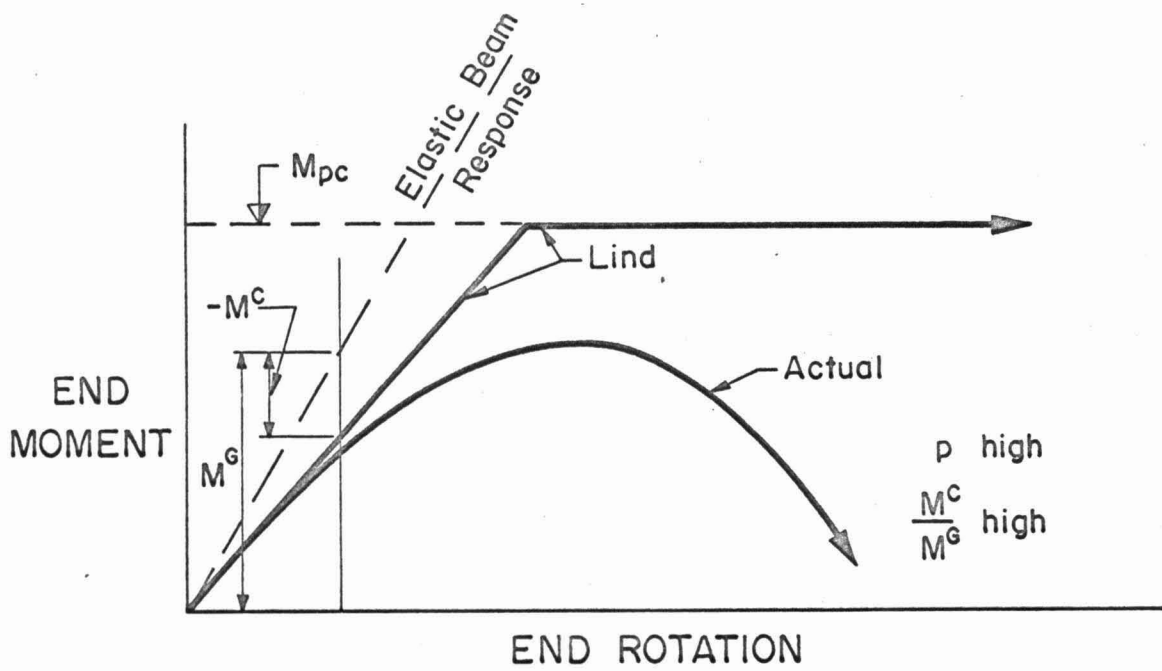


FIG D2

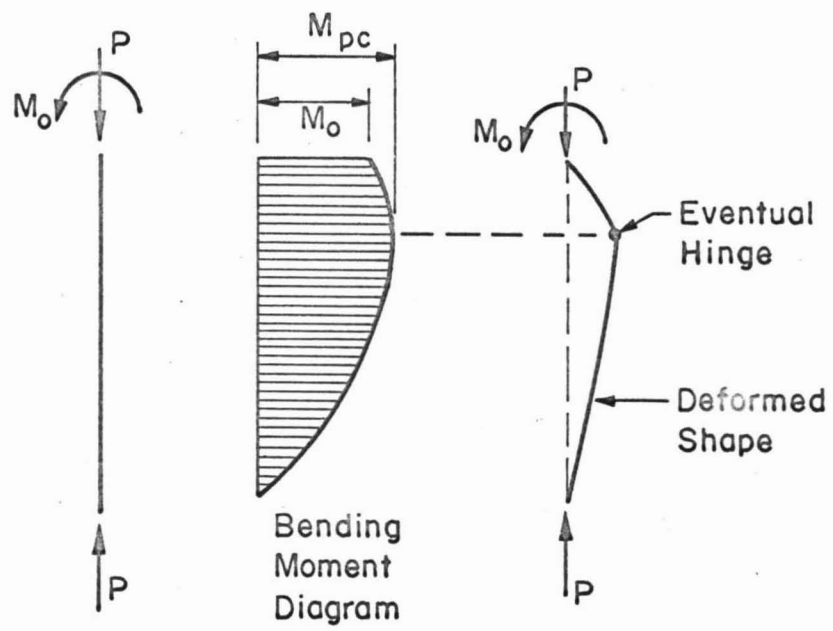


FIG D3