L S Beedle

STABILITY CONSIDERATIONS IN THE DESIGN OF STEEL COLUMNS [a]

Discussion by Theodore V. Galambos

THEODORE V. GALAMBOS[1]. Professor Massonnet is to be congratulated on his excellent summary of the stability problems encountered in the design of steel columns.

Extensive reference is made by the author to a paper by Professor R. L. Ketter and the discusser (Ref. 15). This report has been published, and reference should be made to the printed version of the report as follows:

> Galambos, T. V.; Ketter, R. L.; COLUMNS UNDER COMBINED BENDING AND THRUST, ASCE Proc. Paper 1990, 85 (EM 2), 1, (April 1959).

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a) Proc. Paper 2163, September 1959, by Charles Massonnet

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The interaction curves of this reference have been reduced to analytical expressions by curve-fitting, and these formulas are the basis of column design as proposed in the recently published AISC PLASTIC DESIGN MANUAL (American Institute of Steel Construction, New York, 1959).

The following items of discussion are presented in order to supplement the author's treatise on the stability of steel columns:

1. The author correctly concludes that the hypothetical maximum stress due to the "secant" type solution of the problem of plane bending under axial load is the yield stress (Eq. 3). However, it is not pointed out that yielding of the column will commence before the the yield stress according to the elastic theory is reached in the most stressed fiber of the column. This is because of the presence of residual stresses, which can attain a magnitude of about 50% of the yield stress for rolled wide-flange columns. This negative effect counteracts part of the beneficial effects discussed by the author (e.g. reserve of strength due to plasticity), and in certain cases the "initial yield"

or "secant" theory may lead to unconservative designs. This was pointed out in Fig. 36 of a paper by Ketter, Kaminsky and Beedle.* In this figure, a curve for ultimate strength, including the effect of a compressive residual stress of 0.3 $_{y}$, is compared with a curve for an "initial yield" solution. The latter is seen to be unconservative in the region where the axial load is relatively high and the bending moments are small.

2. The discusser has recently completed a solution for the inelastic lateral-torsional buckling strength of rolled wide-flange columns subjected to axial forces and equal end bending moments causing single curvature deformation of the column axis (M_{equiv} = M in the author's Eq. 40)**. The influence of residual stresses is also included in the calculation. The procedure of solution is an Eigenvalue process, where the reduction of the various stiffnesses governing lateral-torsional buckling due to yielding is taken into account. No further

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* Ketter, R. L.; Kaminsky, E. L.; Beedle, L. S.; PLASTIC DEFORMATION OF WIDE-FLANGE BEAM-COLUMNS, ASCE Transactions, (120), 1028, (1955)

**Galambos, T. V.; INELASTIC LATERAL-TORSIONAL BUCKLING OF ECCENTRICALLY LOADED WIDE-FLANGE COLUMNS, Ph.D. Dissertation, Lehigh University, 1959.

description of the method is included here, since it is quite complicated. A comparison of interaction curves obtained by this method and by the author's interaction formula (Eq. 40) for lateral-torsional buckling is shown in Figs. 1 and 2. The solid lines refer to the interaction equation (author's Eq. 40, where P_o is taken as the weak axis buckling load of a pin-ended column, and M_o is the critical moment under pure bending, as computed by the author's Eqs. 45), and the dashed lines represent the discusser's "exact" solution. Figure 1 is for the 8WF31 shape, and Fig. 2 for the 14WF 142 shape. It can be seen from these figures that the interaction equation is in general conservative, except in some limited regions where it furnishes somewhat unconservative answers.

Although the correspondence between the "exact" method and the interaction equation is not as good as for buckling in the plane of the moments (Figs. 21 and 22 of author's paper), it is still sufficient to warrant the use of the interaction equation. Because of the extreme laboriousness of the "exact" method, the interaction equation proposed by the author may well represent the only practical solution for the design office.

3. In Fig. 3 a comparison of the author's test results with the "exact" method shows that the discusser's method can predict column behavior with fair accuracy.

4. The author places very much importance on the reduction of strength due to lateral-torsional buckling. This, however, is not the most serious drawback of this type of failure. In fact, maximum strength may often be quite well predicted by neglecting the effect of lateral-torsional buckling (see Figs. 12 through 15 of Ref. 15), except for columns pinned in both directions at the ends and subjected to equal or nearly equal end moments. The most serious consequence of lateral-torsional buckling is that it may reduce the rotation capacity of a column. Such an effect is precluded through the use of appropriate lateral bracing.*

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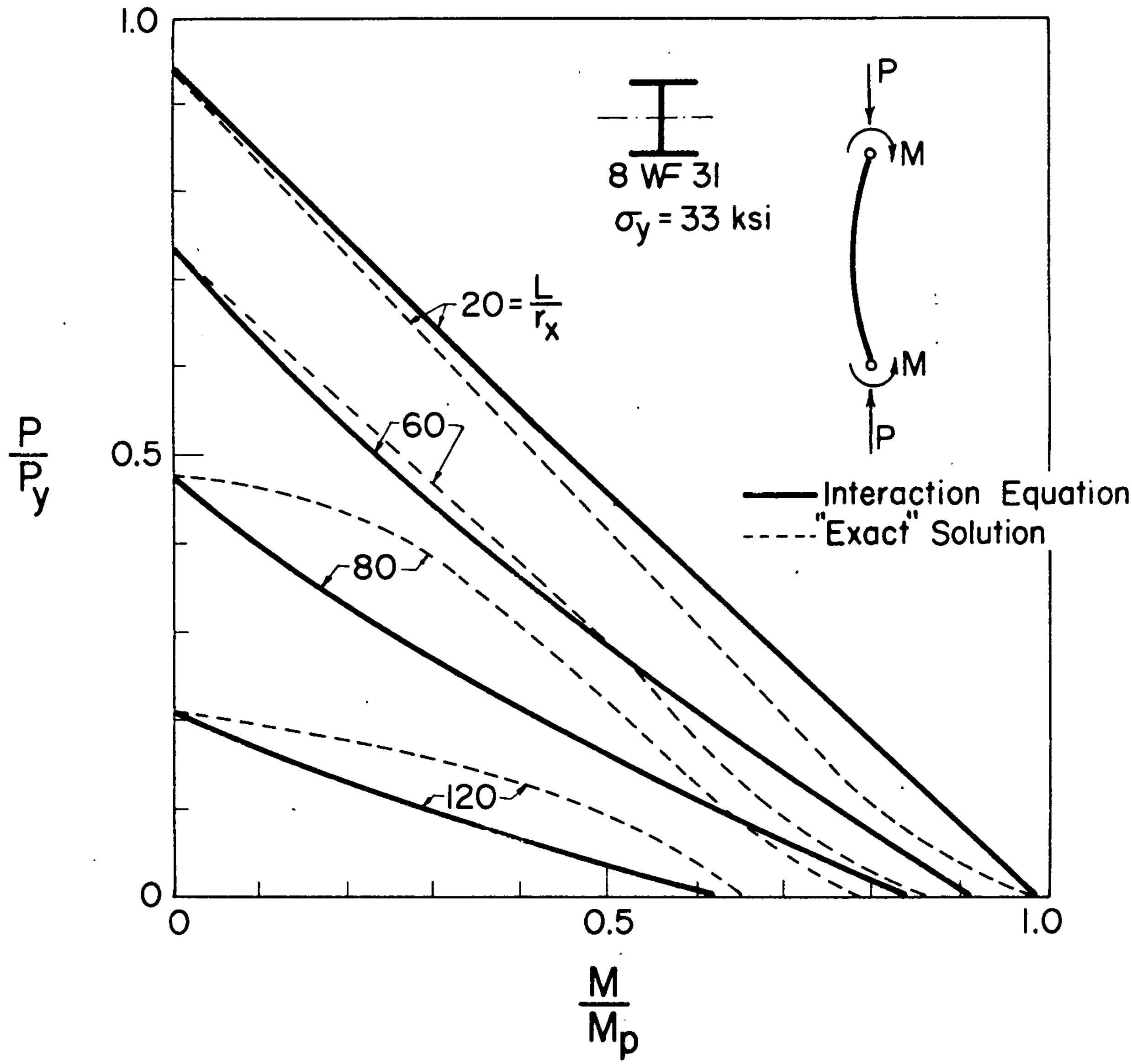
* ASCE-WRC COMMENTARY ON PLASTIC DESIGN - COMPRESSION MEMBERS, ASCE Proc. Paper, 86 (EM-1) Jan. 1960.



1.0
0.5
0
0
0.5
1.0
P/P_y
M/M_p
8 WF 31
σ_y = 33 ksi
P
M
M
P
20 = L/r_x
60
80
120
Interaction Equation
"Exact" Solution

FIG. 1

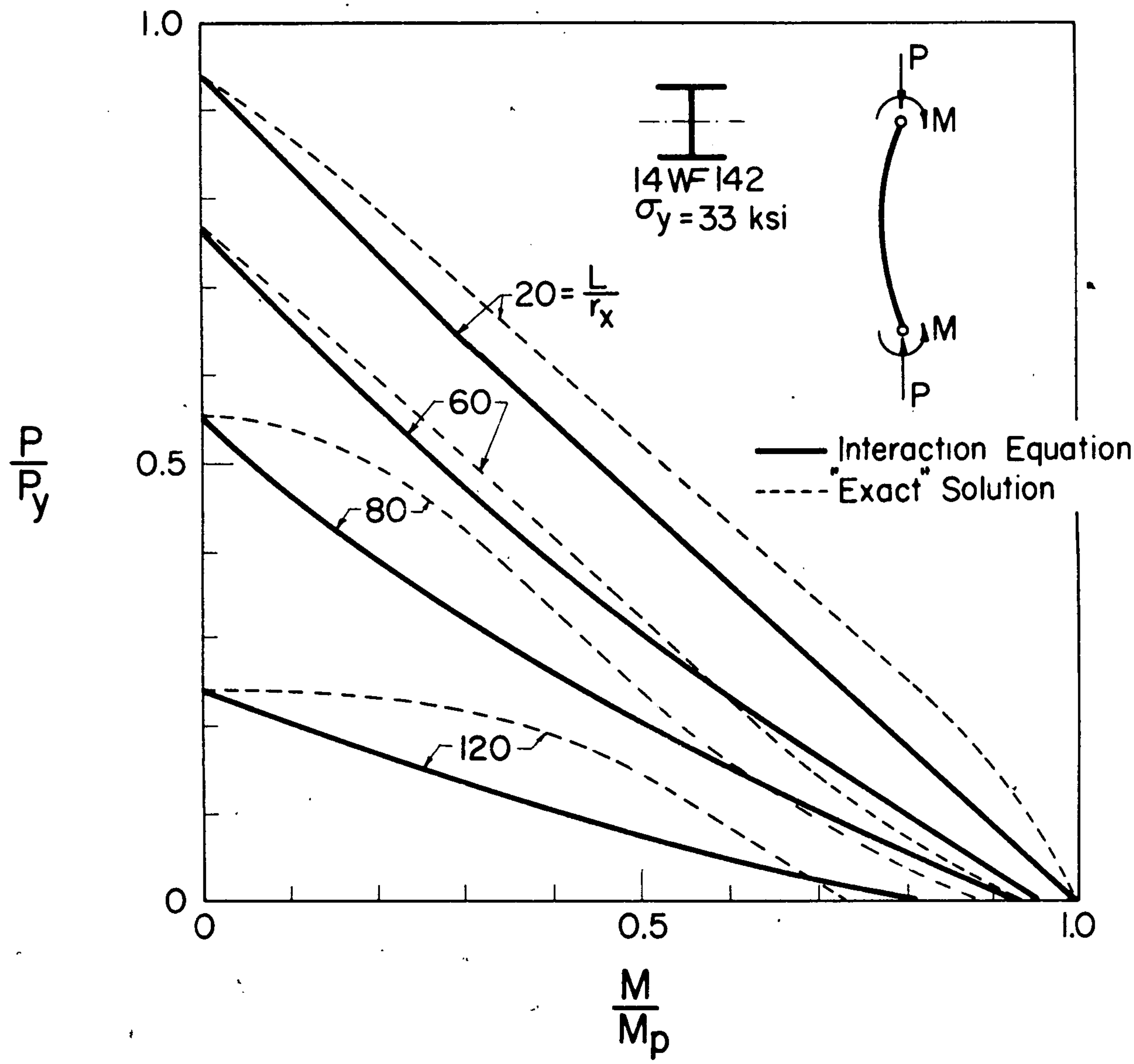
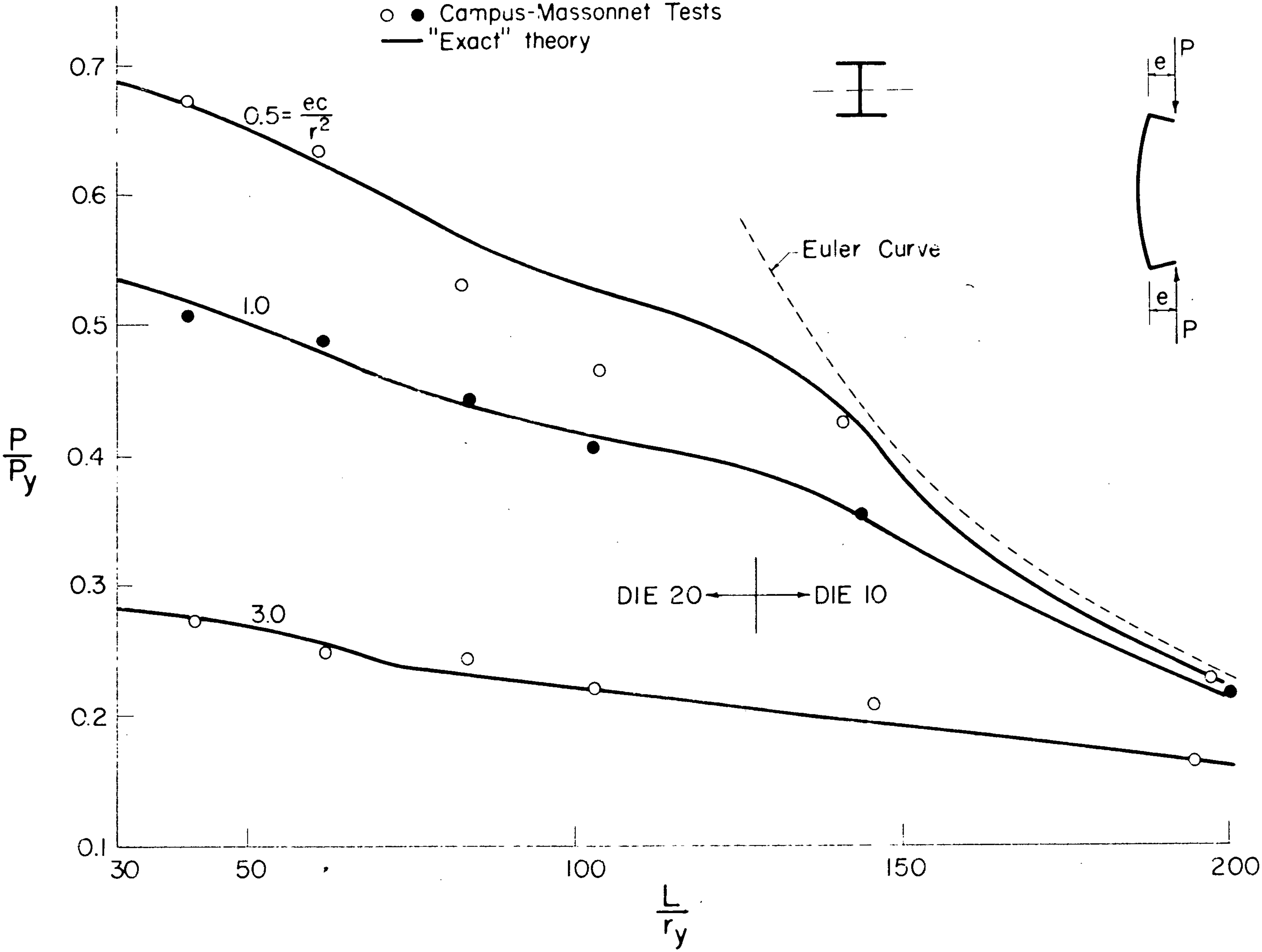

1.0
0.5
0
$\frac{P}{P_y}$
0
0.5
1.0
$\frac{M}{M_p}$
14 WF 142
σ_y = 33 ksi
P
M
M
P
20 = $\frac{L}{r_x}$
60
80
120
Interaction Equation
"Exact" Solution

FIG. 2



○ ● Campus-Massonnet Tests
"Exact" theory
0.7
0.6
0.5
0.4
0.3
0.2
0.1
$0.5 = \frac{ec}{r^2}$
1.0
3.0
Euler Curve
e
P
e
P
DIE 20
DIE 10
$\frac{P}{P_y}$
30
50
100
150
200
$\frac{L}{r_y}$

FIG. 3