Reinforcing Loaded Steel Compression Members

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Theoretically, it does not seem plausible that a column which is reinforced under load will have the same ultimate capacity as a column reinforced in its unloaded state. Yet there is a tendency to use the allowable stresses for the geometry of the reinforced column for both cases. This question is examined in greater detail in this paper, where the analysis shows that both the geometry of the reinforcement and initial load can affect column capacity.

The paper develops a method to determine the capacity of a column reinforced under load based on AISC requirements. The analysis is applicable to any kind of column and reinforcement, but has not been verified by testing. Tests have been conducted by Nagaraja and Tall[2] on a W8×31 column with an L/r ratio of 48, which showed that welding flange plates—while the column is subjected to a load of 91.2 kips—produced results comparable to the same column reinforced under no load. This paper would confirm this, but shows this is not necessarily true for larger L/r values of the same reinforced column.

If the effects of residual stresses from welding are ignored, the following examples illustrate how the location of the new reinforcement influences capacity.

If a column is already carrying its full dead and live load, only the live load can be removed during reinforcement. The dead load stresses will be frozen in the column core after welding and can influence its capability to carry additional load. The original column, hereafter is referred to as the *core*.

The effects of residual stresses from welding have not been considered in this paper and the reader should consult other publications on the subject.[2,4,7]

METHOD OF ANALYSIS

Inelastic Analysis for Buckling

The analysis will be based on the assumption a plastic hinge will first form in the core column and the reinforced column has a post-buckling capacity after the core has failed. The additional capacity will be provided by the stiffness of the reinforcement. This results in a critical column length L_1. The alternate assumption the reinforcement will buckle first will result in a critical column length L_2. The equations for determining L_1 and L_2 are developed in the paper, together with the way to calculate the capacity of the reinforced column.

A typical reinforced column is shown in Fig. 1.

The reinforced column, modeled for analysis purposes, is in Fig. 2.

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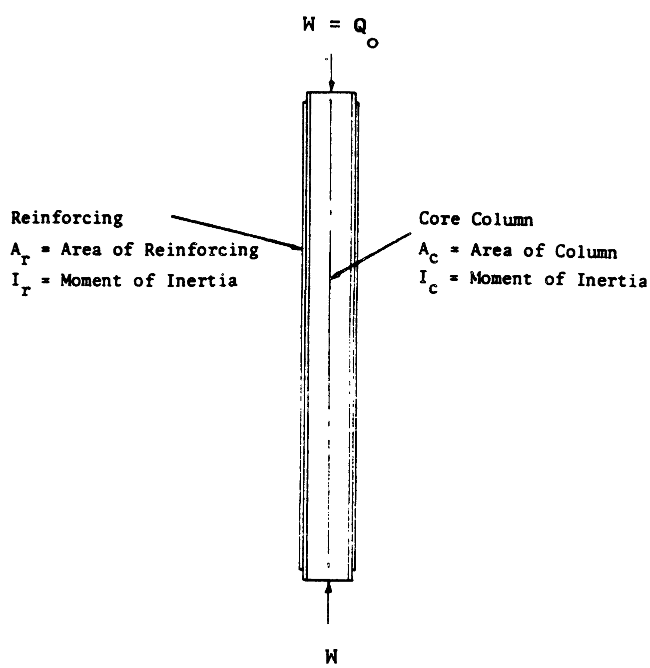


Figure 1

The model consists of two separate columns connected by rigid links. This insures that both the core and the reinforcement follow the same axial deformation and have the same deflection curves.

The model will have three stages of loading:

Stage 1 The reinforcement is welded to the loaded core.

Stage 2 A plastic hinge forms in the core at G.

Stage 3 Failure occurs when the reinforcement is unable to stabilize the core.

The loading in the core and reinforcement at Stage 1 is:

$Q = Q_o$	(core column)	(1a)
$P = Q$	(reinforcement)	(1b)
$W = P + Q$	(combined)	(2)

As the load W is increased, the core reaches its buckling load of Q_{cr}, with both the core and the reinforcement carrying loads up to Stage 2.

When Stage 2 is reached, the core has reached its maximum load capacity. If strain-hardening is ignored, any further lcad will be carried by the reinforcement. The mechanism against failure at this stage is shown in Fig. 2b.

As the deflection of point G increases because of the increased load of W, the capacity of the core will approach Q_{cr}, while its resistance to rotation at G approaches zero.

As W increases, a plastic hinge will form at E and the reinforcement loses its stiffness. This is the failure stage of the reinforced column. The analysis will start at Stage 1 with the assumed forces and displacements shown in Fig. 3.

No real column is perfectly straight and its initial crookedness can be given as,

$$y_o = a_1 \sin(\pi x/L) + a_2 \sin(2\pi x/L) + \ldots + a_n \sin(n\pi x/L) \quad (3)$$

The first term of the series is predominant and will be used in the analysis to follow. The column CD, in Fig. 3a, is assumed to have an initial bow. Using this assumption, then,

$$y_o = a_1 \sin(\pi x/L) \quad (4)$$

As the load Q increases, the initial bow will take the deflected shape given by y. This deflection will result in a load q on the reinforcement, shown in Fig. 3b. The reinforcem. nt will have a transverse loading of the same form as the deflected core.

$$q = F_o \sin(\pi x/L) \quad (5)$$

where

q = the load per unit length.

F_o = maximum force acting at $L/2$

The deflection[6] for the beam CD is,

$$y = Qa \sin(\pi x/L)/(\pi^2 EI_c/L^2 - Q) \quad (6)$$

The term $\pi^2 EI_c/L^2$ is Euler beam-column buckling load. If this is replaced by Q_{cr}, where $Q_{cr} = F_{cr} A_c$, then Eq. (6) becomes,

$$y = Qa \sin(\pi x/L)/(Q_{cr} - Q) \quad (6a)$$

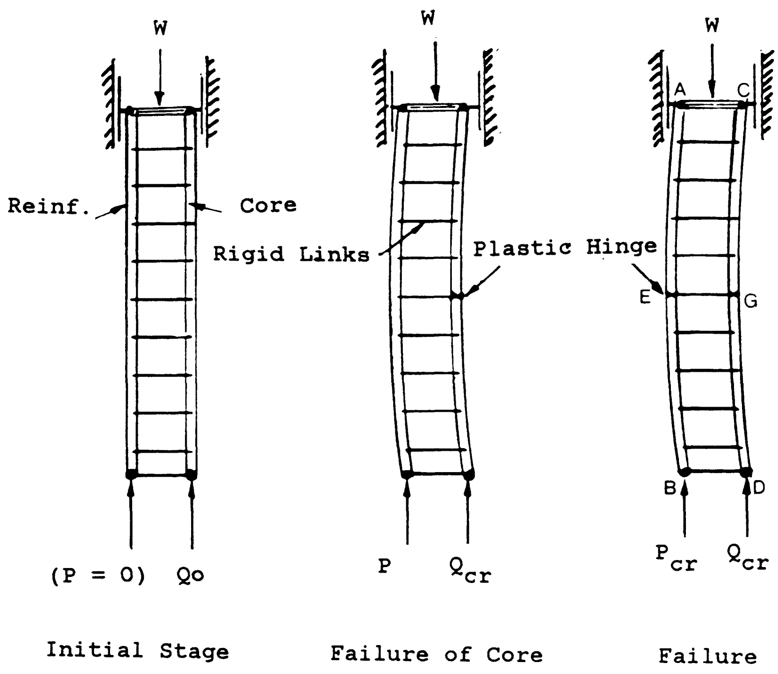


Figure 2a *Figure 2b* *Figure 2c*

The solution[5] to the beam-column shown in Fig. 3b is,

$$w = F_o(L/\pi)^2 \sin(\pi x/L)/(\pi^2 EI_r/L^2 - P) \quad (7)$$

Similarly, if $\pi^2 EI_r/L^2$ is replaced by P_{cr}, where $P_{cr} = F_{cr} A_r$ and F_{cr} is based on the stiffness of the reinforcement, then Eq. 7 becomes:

$$w = F_o(L/\pi)^2 \sin(\pi x/L)/(P_{cr} - P) \quad (7a)$$

Equating $y = w$, then,

$$F_o = \pi^2 Qa(P_{cr} - P)/L^2(Q_{cr} - Q) \quad (8)$$

As $(Q_{cr} - Q) \rightarrow O$, $(P_{cr} - P) > O$, (from initial assumption). Rewriting Eq. (8) as,

$$F_o = C\,(P_{cr} - P)/(Q_{cr} - Q) \quad (9)$$

As $Q \rightarrow Q_{cr}$, $F_o \rightarrow \infty$, whatever the value of C. So it can be concluded a plastic hinge will form at G in column CD, with a mechanism similar to that shown in Fig. 4b. The force F acts at the center of column CD, preventing it from collapsing.

The force F also acts at the center of the reinforcement, creating the beam-column shown in Fig. 4a.

The equation[7] for the deflection center of the column AB is,

$$w = LF\,[\tan(uL/2)/(uL/2) - 1]\,/4P \quad (10)$$

where

$$u^2 = P/EI_r$$

An approximation of Eq. 10 is:

$$w = FP_e L^3/48EI_r(P_e - P) \quad (11)$$

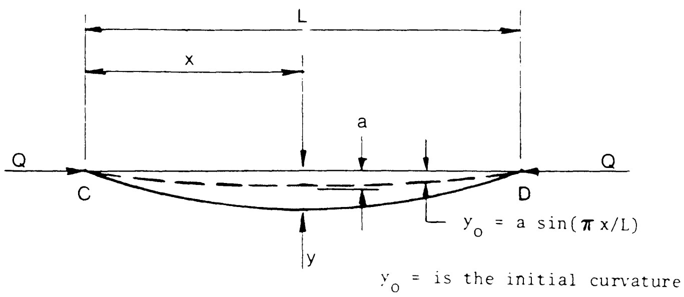


Figure 3a

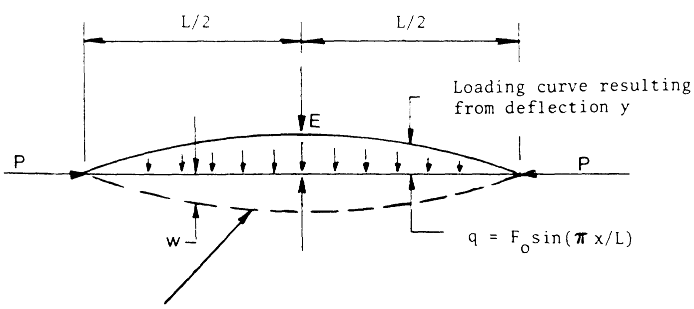


Figure 3b

where

$$P_e = \pi^2 EI_r/L^2 \quad (12)$$

The corresponding deflection at the center of column CD after the plastic hinge has formed, shown in Fig. 4b, is,

$$y = FL/4Q_{cr} \quad (13)$$

Critical Load on Reinforcement

If the deflection y and w are equated, the maximum load P can be determined using Eqs. 10 and 13 and equating $y = w$, then,

$$P = Q_{cr}\,[\tan(uL/2)/(uL/2) - 1] \quad (14)$$

The solution of Eq. 14 is best solved by computer or by successive approximations.

A simpler but less accurate method for finding P is to use the approximation given by Eq. 13 and Eq. 11. Equating deflections,

$$y = w$$

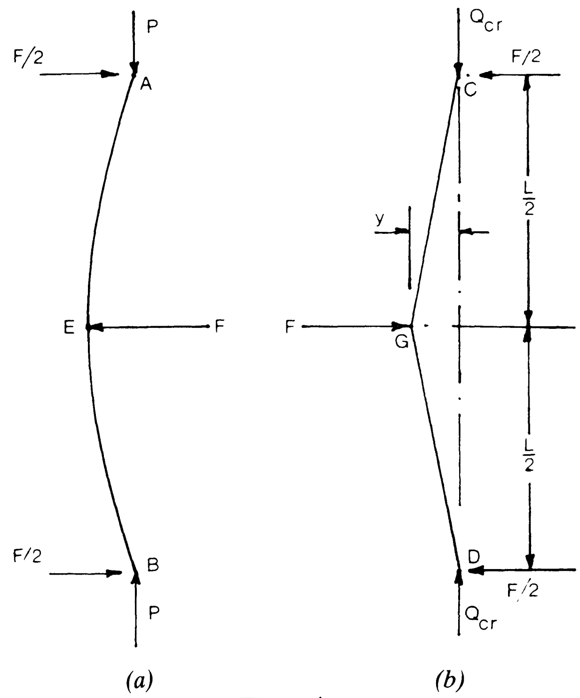
then

$$P_{max} = P_e(1 - Q_{cr}L^2/12EI_r) \quad (15)$$

$$P_{max} = P_e(1 - Q_{cr}\pi^2/12P_e)$$

$$P_{max} = P_e - 0.822\,Q_{cr} \quad (16)$$

$$P_{max} \leq P_{cr}$$



(a) *(b)*

Figure 4

Critical Length of Reinforcement

If Eq. 16 is equated to P_{cr}, a critical length L_1 results. This is the maximum column length for which the properties of an unloaded reinforced column can be used, given as,

$$L_1 = \sqrt{[\pi^2 EI_r/(P_{cr}+0.822Q_{cr})]} \tag{17}$$

Similarly, if Eq. 15 is equated to zero, a critical length L_2 results. This is the minimum length of column for which the reinforcement provides additional stiffness to the core. For column lengths greater than L_2, the column must be designed for the elastic range as given by the next section.

$$L_2 = \sqrt{(12EI_r/Q_{cr})} \tag{18}$$

Since Eqs. 17 and 18 give approximate values of L_1 and L_2 sometimes it is necessary to make a second trial calculation.

Elastic Analysis

When the core column length is greater than L_2, the column must be designed for elastic stresses only, with both the core and reinforcement working below the critical buckling stress of the reinforced column.

The following analysis derives the ultimate load for a reinforced column in this state.

Eqs. 21 and 22 are given for the factor of safety and Eqs. 23 and 24 give the critical buckling stresses.

The larger value of the reinforced or unreinforced L/r should be used to calculate the factor of safety.

The factor of safety (F.S.) for Eq. 1.5-1[3] is,

$$\text{F.S.} = 5/3+3(KL/r)/8C_c-(KL/r)^3/8C_c^3 \tag{21}$$

and for Eq. 1.5-2[3] is,

$$\text{F.S.} = 23/12,\ KL/r > C_c \tag{22}$$

The critical stress F_{cr} for Eq. 1.5-1 is,

$$F_{cr} = [1-(KL/r)^2/2C_c^2]F_y,\ KL/r<C_c \tag{23}$$

$$F_{cr} = \pi^2 E/(L/r)^2,\ KL/r > C_c \tag{24}$$

where

$$C_c = \sqrt{(2\pi^2 E/F_y)}$$

EXAMPLES

Three cases will be considered, each using $L = 126$ in. Initial dead load $Q_o = 91.2$ kips and the same W8×31 section with the equivalent area of reinforcement of 2 – 7-in. × ⅜-in. plates with $F_y = 36$ ksi.

Strain in the core column CD at load Q_o is ϵ_o and the strain at load Q_{cr} is ϵ_{cr}. Strain in reinforcement at load Q_{cr} is:

$$\epsilon_{cr} - \epsilon_o$$

Load in reinforcement at load Q_o is $P = O$. Load P in the reinforcement at load Q_{cr} is:

$$P = (\epsilon_{cr} - \epsilon_o) A_r E$$
$$P = (Q_{cr} - Q_o) A_r / A_c \tag{19}$$

The ultimate load P_u of the reinforced column is:

$$W = P_u = P + Q_{cr} \tag{20}$$

In no case should W exceed the buckling strength of the reinforced column, with $Q_o = 0$.

FACTOR OF SAFETY AND CRITICAL BUCKLING STRESS

The factor of safety and critical inelastic buckling stress should be based on AISC Specification Eq. 1.5-1.[3] This equation can be broken down into two parts: one is the factor of safety and the other the critical buckling stress.

Properties of core column W8×31.

$A_c = 9.12$ in.^{2}

$I_y = 37.0$ in.^{4}

$r_y = 2.02$ in.

$L/r = 126/2.02 = 62.38$

Determine critical stress F_{cr} by Eq. 23.

$C_c\ (F_y = 36\text{ ksi}) = 126.1$, K = 1.0

$F_{cr} = [1-(62.38/126.1)^2/2]\times 36 = 31.60$ ksi

$Q_{cr} = 31.60 \times 9.12 = 288.19$ kips

Case 1.

2 – 7-in. × ⅜-in. plates welded to the flanges.

Reinforcement properties:

$A_r = 2 \times 7 \times 0.375 = 5.25$ in.^{2}

$I_r = 2 \times .375 \times 7^3/12 = 21.44$ in.^{4}

$r = \sqrt{21.44/5.25} = 2.02$ in.

$L/r = 126/2.02 = 62.38$

Combined column properties

$A = 9.12 + 5.25 = 14.37$ in.^{2}

$I = 37 + 21.44 = 58.44$ in.

$r = \sqrt{I/A} = \sqrt{(58.44/14.37)} = 2.02$ in.

$L/r = 126/2.02 = 62.38$

$F_{cr} = [1 - (62.38/126.1)^2/2] \times 36 = 31.60$ ksi

Ultimate capacity with $Q_o = O$.

$P_u = 31.60 \times 14.37 = 454.09$ kips

$F_{cr} = [1 - (62.38/126.1)^2/2] \times 36 = 31.60$ ksi

$P_{cr} = 31.60 \times 5.25 = 165.9$ kips

Check critical length L_1 for post buckling by Eq. 17,

$$L_1 = \sqrt{[\pi^2 \times 29000 \times 21.44/(165.9+.822 \times 288.19)]}$$
$$= 123.4 \text{ in.}$$

Check critical length L for the elastic buckling by Eq. 18,

$$L_2 = \sqrt{(12 \times 29000 \times 21.44/288.19)} = 160.9 \text{ in.}$$

The reinforced column is in the transition stage, since $L_1 < L < L_2$. Use Eq. 16 to calculate P.

$$P = P_e - 0.822\, Q_{cr}$$

$$P_e = \pi^2 \times 29000 \times 21.44/126^2 = 386.49 \text{ kips}$$

$$Q_{cr} = 288.19 \text{ kips}$$

$$P = 386.49 - .822 \times 288.19 = 149.60 \text{ kips}$$

$$P \leq P_{cr}$$

$$W = P_u = Q_{cr} + P = 288.19 + 149.96 = 438.15 \text{ kips}$$

The factor of safety is given by Eq. 21 using the L/r ratio for the combined column properties,

$$\text{F.S.} = 5/3+3 \times 62.38/(8 \times 126.1) - .125 \times (62.38/126.1)^3$$
$$= 1.84$$

ASD design load on reinforced column

$$\text{F.S.} = 438.15/1.84 = 238.5 \text{ kips}$$

$$F_a = 238.5/14.37 = 16.60 \text{ ksi}$$

Welds to reinforcement

Use E70XX electrodes

Shear flow $\tau b = V_y Q_x / I_x$

$$Q_x = (7 \times .375) \times (.75+8)/2 = 11.5$$

$$I_x = 110 + [(.75+8)/2]^2 \times 2.63 \times 2 = 210.7 \text{ in.}^4$$

$$S_x = 210.7/4.375 = 48.2 \text{ in.}^3$$

Allowable moment, $M = 24 \times 48.2 = 1156$ kip-in.

Equivalent shear, $V_y = 2M/L$
$= 2 \times 1156/126 = 18.4$ kips

Shear flow/in. $= V_y Q_x / I_x$
$= 18.4 \times 11.5/210.7 = 1.0$ kips/in.

Spacing welds $= 127/\sqrt{F_y} = 127/\sqrt{36} = 21.2$ in.

Limit spacing to 12-in.

Capacity of 3⁄16-in. fillet weld $= 36 \times 0.4 \times 3/16 =$ 2.7 kips/in.

Length of weld $= 12 \times 1/2.7 = 4.44$ in.

Use 2 − 3⁄16-in. fillet × 2⅜ long at 12-in. c. to c.

End welds

Shear on end of plate $= MQ_x/I_x$
$= 1157 \times 11.5/210.7$
$= 63.0$ kips

Length of 3⁄16-in. fillet weld $= 63.0/2.7 = 23.3$ in.

Use 12-in. of 3⁄16-in. fillet weld each side of plate with 1-in. return weld

Case 2.

W8×31 with 2 − 10.5-in. × ¼-in. flange plates

Properties of reinforcement

$$A_r = 10.5 \times .25 \times 2 = 5.25 \text{ in.}^2$$

$$I = 2 \times 0.25 \times 10.5^3/12 = 48.23 \text{ in.}^4$$

$$r = \sqrt{(48.23/5.25)} = 3.03 \text{ in.}$$

$$L/r = 126/3.03 = 41.57$$

$$F_{cr} = [1 - (41.57/126.1)^2/2] \times 36 = 34.04 \text{ ksi}$$

$$P_{cr} = 34.04 \times 5.25 = 178.71 \text{ kips}$$

Combined properties

$$A = 14.37 \text{ in.}^2$$

$$I = 37 + 48.234 = 85.23 \text{ in.}^4$$

$$r = 2.44 \text{ in.}$$

$$L/r = 126/2.44 = 51.64$$

$$F_{cr} = [1 - (51.64/126.1)^2/2] \times 36 = 32.98 \text{ ksi}$$

$$P_u = 32.98 \times 14.37 = 473.92 \text{ kips}$$

Check critical length L_1 by Eq. 17

Using $L_1 = 182.25$, $Q_{cr} = 244.21$ kips,

$$P_{cr} = 167.49 \text{ kips}$$

then

$$L_1 = \sqrt{[\pi^2 \times 29{,}000 \times 48.23/(167.49+.822 \times 244.21)]}$$
$$= 193.6 \text{ in.}$$

$L_1 \geq L$, so the column can be designed for the reinforced column properties with $Q_o = O$.

then,

$$P_u = 473.92 \text{ kips}$$

Case 3.

W8×31 with 2 − 6-in. × 7⁄16-in. web plates

Properties of reinforcement

$$A_r = 5.25 \text{ in.}^2$$

$$I_r = 2 \times 6 \times .438^2/12 + 5.25 + (.438+.288)^2/2$$
$$= .774 \text{ in.}^4$$

$$r = \sqrt{(.774/5.25)} = .384$$

$L/r = 126/.384 = 327.8$

$F_{cr} = \pi^2 \times 29000/327.8^2 = 2.66$ ksi

$P_{cr} = 2.66 \times 5.25 = 14.0$ kips

$A = 14.37$ in.^{2}

$I = 37.324$

$r = \sqrt{37.324/14.37} = 1.61$ in.

$L/r = 126/1.61 = 78.16$

$F_{cr} = [1 - (78.16/126.1)^2/2] \times 36 = 29.08$ ksi

Ultimate capacity assuming $Q_o = 0$.

$P_u = 29.08 \times 14.37 = 417.88$ kips

Check critical length for elastic buckling using Eq. 18.

$L_2 = \sqrt{(12 \times 29{,}000 \times .776/288.19)} = 30.6$ in.

The column is elastic; use Eq. 19

$$P = (288.19 - 91.2) \times 5.25/9.12 = 113.4 \text{ kips}$$
$$P_u = Q_{cr} + P = 288.19 + 113.4 = 401.59 \text{ kips}$$

If the column had been designed with the reinforced properties and load $Q_o = 0$, then the column would be deficient by,

$$(417.88 - 401.59) \times 100/401.6 = 4.06\%$$

If the same column was carrying its full load at the time of reinforcement, then

$L/r = 62.38$

$Q_o = 17.2 \times 9.12 = 156.86$ kips

and

$$P_u(Q_o{=}156.86) = (288.19{-}156.86){\times}5.25/9.12{+}288.19 = 363.1$$

% overstress = $(417.88{-}363.1){\times}100/363.1 = 15\%$

COMPUTER ANALYSIS AND RECOMMENDATIONS

A computer program was written to investigate the characteristics of a W8×31 section reinforced as previously described for Cases 1, 2 and 3 in the Example section. The behavior of the three cases is shown in Figs. 5 and 6. Also, the computer analysis was used to check the validity of using the approximate Eq. 16 instead of the more exact Eq. 14.

Figure 6 shows the curves have a distinct discontinuity when a critical length L_1 is reached for each type of reinforcement. This is best described by examining the curve for Case 1.

From points a to b, the curve gives the ultimate capacity of the column, based on the reinforced column properties. The initial load on the column has no effect on the ultimate capacity P_u of the column for column lengths less than L_1. For lengths up to point b, the column can be designed as a normal column and the effects of the residual load Q_o can be ignored.

From points b to c, there is a change in the curve as the reinforcement provides post-buckling strength to the core column. This is the transition stage between inelastic buckling to elastic buckling and the column should be designed using Eq. 16.

From points c to d, the column is completely dependent on the initial loading Q_o and the strain of the core cannot go beyond the first buckling load. The column should be designed as outlined by the section on Elastic Analsyis.

The computer showed Eq. 16 slightly overestimates the critical buckling of the reinforced column. If one desires to check the value of P by Eq. 14, then Eq. 16 can be used as a starting point.

The ultimate capacity of the column should not exceed the capacity of the reinforced column under no load condition, i.e., $Q_o = 0$.

The complexity of the failure modes is shown in Fig. 6 curve (3), where the buckling load after $L = 150$ is based on Eq. 24 with the properties of the reinforced column and $Q_o = 0$, rather than the method suggested in the Elastic Analysis of this paper. This is because the radius of gyration of the reinforced column is less than the radius of gyration of the core.

CONCLUSION

The method developed in this paper for calculating the ultimate capacity of a column reinforced under load is based on a rational analysis and is not substantiated by testing, although the method does provide the same answer as Ref. 2 for a W8×31 column reinforced with 2 ft. − 7 in. × 3/8 in. flange plates and an $L/r = .48$.

The paper treats the reinforced column as a frame with two members joined to each other by rigid links. The members are the reinforcement and the unreinforced column. The reinforced column of length L has two critical lengths L_1 and L_2 which define its behavior. The values of L_1 and L_2 are dependent on the relative stiffness of the reinforcement and column.

The design criteria for determining the capacity of the column is given by the following:

1. $L \leq L_1$, the properties of the reinforced column can be used.
2. $L > L_2$, the column should be designed for elastic behavior only, by Elastic Analysis section of this paper.
3. $L_1 < L < L_2$, the column uses the post-buckling strength of the reinforcement after the column core has failed. The critical load is given by Eq. 14.

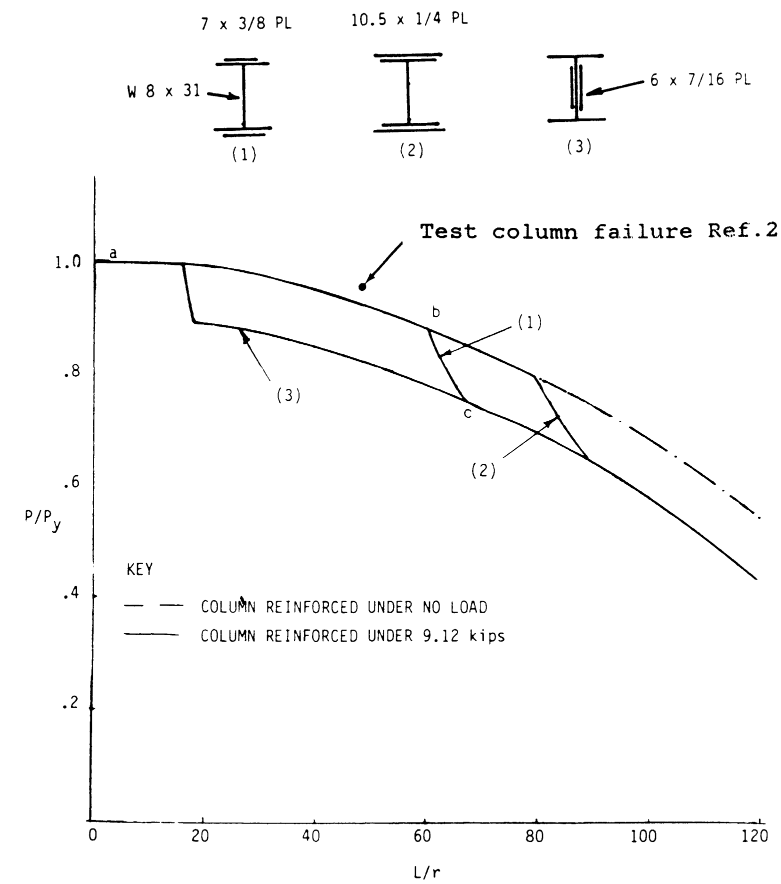


Figure 5

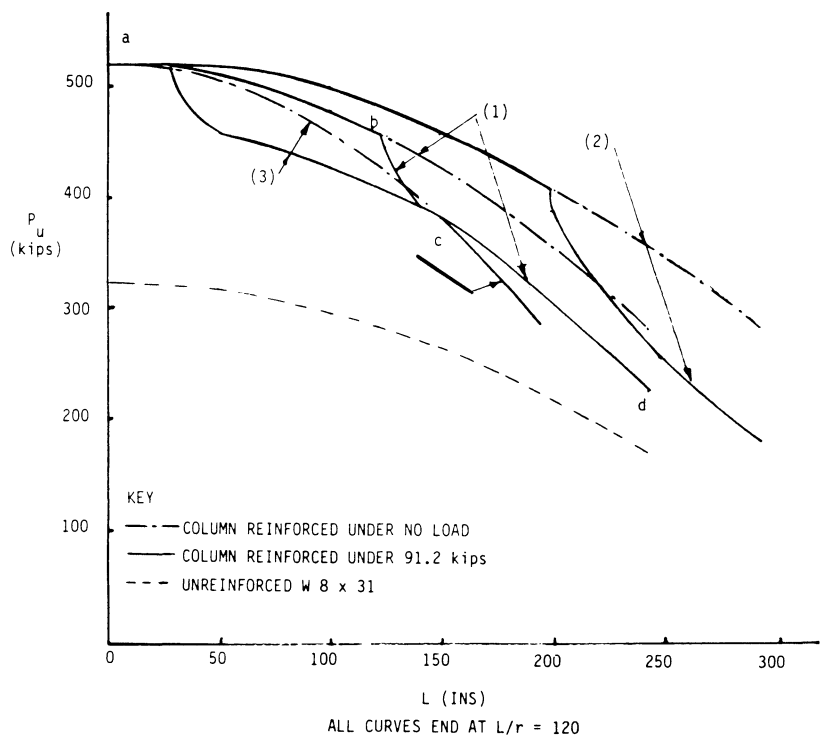


Figure 6

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NOMENCLATURE

A = Area of compression members
A_c = Area of core column
C = Constant
C_c = Column slenderness ratio separating elastic and inelastic buckling
E = Modulus of elasticity of steel
F = Lateral force acting at center of column, between core and reinforcement at failure
F_a = Allowable compressive stress
F_{cr} = Critical buckling stress
F_o = Maximum force between core and reinforcement, elastic range
F_y = Specified minimum yield stress
F.S. = Factor of safety
I_c = Moment of inertia of core column
I_r = Moment of inertia of reinforcement
K = Effective length factor for a prismatic member
L = Length of column
L_1 = First critical length, Eq. 17
L_2 = Second critical length, Eq. 18
M = Moment acting on reinforcement
P = Load on reinforcement
P_{cr} = Critical load on reinforcement
P_e = Euler equation, Eq. 12
P_u = Ultimate load on reinforced column
q = Lateral force between core column and reinforcement
Q = Load on core column
Q_o = Load on core column at time of reinforcing
Q_{cr} = Critical load on core column
r = Radius of gyration
W = Load on combined core column and reinforcement
w = Deflection of reinforcement, elastic range
x = Distance along column, measured from support
y = Deflection of core column
y_o = Initial curvature of core column
u^2 = P/EI_r
ϵ_o = Strain in core column at time of reinforcing
ϵ_{cr} = Strain in core column at buckling

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STATEMENT OF OWNERSHIP, MANAGEMENT AND CIRCULATION
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1A. TITLE OF PUBLICATION	1B. PUBLICATION NO.	2. DATE OF FILING
Engineering Journal, American Institute of Steel Construction	0 0 1 3 8 0 2 9	Dec. 2, 1987

3. FREQUENCY OF ISSUE	3A. NO. OF ISSUES PUBLISHED ANNUALLY	3B. ANNUAL SUBSCRIPTION PRICE
Quarterly	Four	$11.00

4. COMPLETE MAILING ADDRESS OF KNOWN OFFICE OF PUBLICATION *(Street, City, County, State and ZIP+4 Code) (Not printers)*
400 North Michigan Avenue, Chicago, Illinois 60611

5. COMPLETE MAILING ADDRESS OF THE HEADQUARTERS OF GENERAL BUSINESS OFFICES OF THE PUBLISHER *(Not printer)*
American Inst. of Steel Construction, Inc.
400 North Michigan Avenue, Chicago, Illinois 60611

6. FULL NAMES AND COMPLETE MAILING ADDRESS OF PUBLISHER, EDITOR, AND MANAGING EDITOR *(This item MUST NOT be blank)*

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400 North Michigan Avenue, Chicago, Illinois 60611

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