

Use of High-strength Steel for Simply Supported Beams

MARIUS B. WECHSLER

As stated by AISC,¹ high-strength steels have proven to be economical choices where lighter members, resulting from use of higher allowable stresses, are not penalized because of instability, local buckling, deflection or other similar reasons. The reduction of dead load and associated savings in shipping costs can be significant factors. However, higher strength steels are not to be used indiscriminately. Effective use of all steels depends on thorough cost and engineering analysis.

The purpose of this paper is to find out the cases when high-strength steel, simply supported beams may be economical. The paper refers to High-strength, Low-alloy Steel A572-Gr. 50 only, and assumes its cost is 10% higher than Carbon Steel A36.

Only the lightest shapes of each group contained in the *Allowable Stress Design* Selection Table and in the *Moment of Inertia Selection Table* are considered.¹ Shapes with the same weight, but with a smaller depth and, at the time, with a smaller bending capability, were not considered. The shape W8×10 was also not considered, since its section modulus superiority versus the shape M10×9, is handicapped by reasons of compactness and/or flexibility.

The further developed method leads to two charts: Chart I for roof purlins and Chart II for floor beams. These charts indicate, with respect to the span, the allowable bending moments in excess of those due to the weight of the considered W and M shapes of $F_y = 36$ ksi and $F_y = 50$ ksi.

The allowable moments are determined for adequately braced beams which have maximum limiting values of allowable stress as permitted by the AISC Specification. For beams not meeting these bracing requirements, the designer should decide if it is reasonable to revise the bracing system, or to select a stronger shape.

Marius B. Wechsler is Senior Engineer, Bechtel Power Corporation, Norwalk, California and a Professional Member of AISC.

OUTLINE OF METHODOLOGY

The total bending moment capability M_t of a beam, expressed in kip-in., is limited by the equations:

$$f_b = \frac{M_t}{S} \leq F_b \quad (1)$$

$$\Delta = \frac{5}{384} \frac{p\ell^4}{EI} \leq \Delta_a \quad (2)$$

Replacing $p\ell^2 = 8 M_t$, $E = 29,000$ ksi, and $I = S d/2$, where d is the depth of the shape expressed in inches, it follows:

$$M_t \leq S F_b \quad (3)$$

$$M_t \leq \frac{139,200 S d \Delta_a}{\ell^2} \quad (4)$$

In accordance with the guiding rule in Sect. 1.13.1 of the *Commentary on AISC Specification (11/1/78) Structural Steel for Buildings*,¹ the allowable bending stress for beams governed by deflection should be²:

$$F_b' = \frac{1000 d F_b}{\ell F_y} \text{ for roof purlins} \quad (5)$$

$$F_b' = \frac{800 d F_b}{\ell F_y} \text{ for floor beams} \quad (6)$$

Implementing Eqs. 5 and 6 in Eqs. 3 and 4, it follows:

$$\Delta_a = \frac{\ell F_b}{139.2 F_y} \text{ for roof purlins} \quad (7)$$

$$\Delta_a = \frac{\ell F_b}{174 F_y} \text{ for floor beams} \quad (8)$$

Expressing the span in ft and moments in kip-ft, it follows from Eqs. 3, 4, 7 and 8:

$$M \leq \frac{S F_b}{12} - \frac{w \ell^2}{8000} \text{ for any beam} \quad (9)$$

$$M \leq \frac{62.5 S d F_b}{9 \ell F_y} - \frac{w \ell^2}{8000} \text{ for roof purlins} \quad (10)$$

$$M \leq \frac{50 S d F_b}{9 \ell F_y} - \frac{w \ell^2}{8000} \text{ for floor beams} \quad (11)$$

Equation 9 governs over Equations 10 and 11 respectively, as long as:

$$\ell [\text{ft}] \leq \frac{250}{3} \frac{d}{F_y} \text{ for roof purlins} \quad (12)$$

$$\ell \leq 62.5 d/27 \text{ when } F_y = 36 \text{ ksi} \quad (13)$$

$$\ell \leq 5d/3, \text{ when } F_y = 50 \text{ ksi} \quad (14)$$

as well as:

$$\ell [\text{ft.}] \leq \frac{200}{3} \frac{d}{F_y}, \text{ for floor beams} \quad (15)$$

$$\ell \leq 50d/27, \text{ when } F_y = 36 \text{ ksi} \quad (16)$$

$$\ell \leq 4d/3, \text{ when } F_y = 50 \text{ ksi} \quad (17)$$

For all these cases, the corresponding moments are:

$$M = 2S - \frac{W\ell^2}{8000}, \text{ when } F_y = 36 \text{ ksi} \quad (18)$$

$$M = \frac{1}{4} S - \frac{w\ell^2}{8000}, \text{ when } F_y = 50 \text{ ksi} \quad (19)$$

In both Charts I and II, the 50-ksi steel as well as the 36-ksi steel shapes are shown as governing until they meet a stronger but less expensive shape. As shown in the Introduction, the 36-ksi steel shapes are considered less expensive if their weight is smaller than 1.1 times the weight of the 50-ksi steel shape to which they are compared; while the 50-ksi steel shapes are considered as less expensive if their weight is smaller than 1/1.1 times the weight of the 36-ksi steel shape to which they are compared.

A 50-ksi steel shape governed by deflection becomes weaker than a 36-ksi steel shape of the same weight but governed by stress when:

$$\ell [\text{ft}] \geq \frac{1}{6} d, \text{ for roof purlins} \quad (20)$$

$$\ell \geq \frac{13.75}{6} d, \text{ for floor beams} \quad (21)$$

while the corresponding moments for both cases are:

$$M = 2S - \frac{w\ell^2}{8000} \quad (22)$$

Considering the moments due to small differences of shape weights as negligible, and using the subscript 1 for the lighter and the subscript 2 for the heavier shape, it follows that a 50-ksi steel shape governed by deflection becomes weaker than a lighter 50-ksi steel shape governed by stress when:

$$\ell [\text{ft}] \geq \frac{5}{3} \frac{S_2}{S_1} d_2, \text{ for roof purlins} \quad (23)$$

$$\ell \geq \frac{4}{3} \frac{S_2^2}{S_1} d_2, \text{ for floor beams} \quad (24)$$

while the corresponding moments for both cases are:

$$M = \frac{1}{4} S_1 - \frac{W_1 \ell^2}{8000} \quad (25)$$

Keeping the same assumption as above, it follows that a 50-ksi steel shape governed by deflection becomes weaker than a heavier 36-ksi steel shape governed by stress when:

$$\ell [\text{ft}] \geq \frac{13.75}{6} \frac{S_1}{S_2} d_1, \text{ for roof purlins} \quad (26)$$

$$\ell \geq \frac{11}{6} \frac{S_1}{S_2} d_1, \text{ for floor beams} \quad (27)$$

while the corresponding moments for both cases are:

$$M = 2S_2 - \frac{W_2 \ell^2}{8000} \quad (28)$$

Equations 9 through 28 are the basis of the Charts I and II presented here.

Chart I for roof purlins is drawn up for bending moments larger than 3.0 kip-ft and is limited to the shape W24×76 and the span $\ell = 70.0$ ft. Chart II for floor beams, is drawn up for bending moments larger than 4.0 kip-ft and is limited to the shape W36×300, and the span $\ell = 120$ ft.

DESIGN PROCEDURE

Both Charts I and II are self explanatory. If the intersection point of the lines corresponding to the span ℓ (ft) of a beam, and of its bending moment in excess of that due to the shape weight M (kip-ft), lies in the shaded area of the chart, then use the 50-ksi steel shape listed. If that point lies in the white area, then the 36-ksi steel shape listed above it should be used. In the transition part of the chart, where cost differences are questionable, the kind of steel to be used will be an engineering judgement.

ILLUSTRATIVE EXAMPLE

Seven 31-ft span beams are subjected to different bending moments, namely 105, 115, 125, 140, 170, 200 and 220 kip-ft respectively. Chart II indicates the use of 36-ksi steel W18×35; 50-ksi steel W18×35; 36-ksi steel W18×40; 36-ksi steel W21×44; 50-ksi steel W21×44; 36-ksi steel W21×50 and 50-ksi steel W21×50, respectively. However, since it is a matter of the transition part in the chart, the design will probably lead to some digressions from the chart output.

CONCLUSIONS

The two charts presented in this paper may constitute an important design aid for selecting the appropriate structural steel for roof purlins and floor beams. However, since the charts are based only on a guide,¹ the most satisfactory solution must rest on the sound judgement of the engineer, in the light of specifications concerning vibration, ponding and plaster ceilings. When it is a matter of responding to rigidity considerations, the design aid

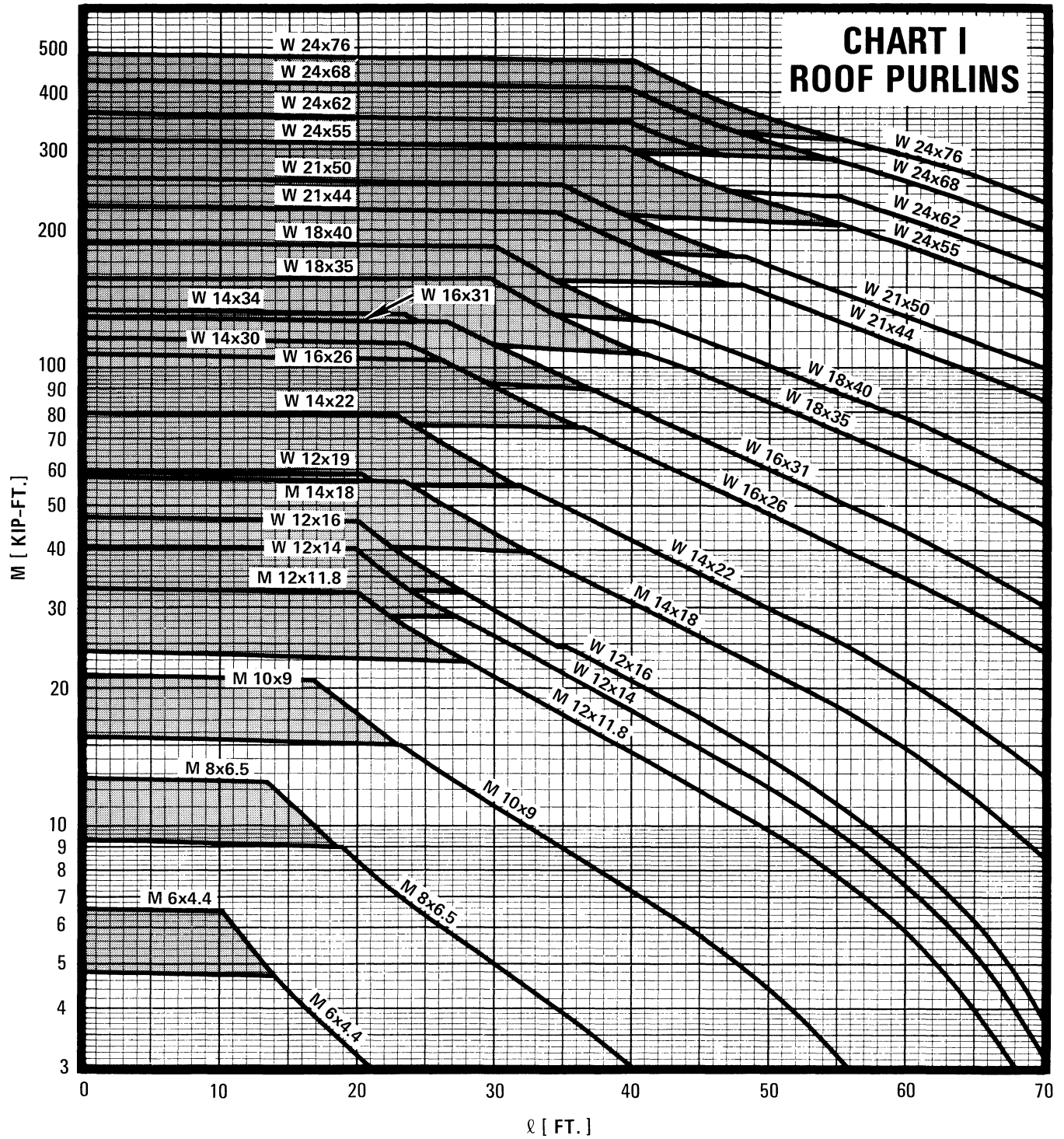


Chart I. Roof Purlins

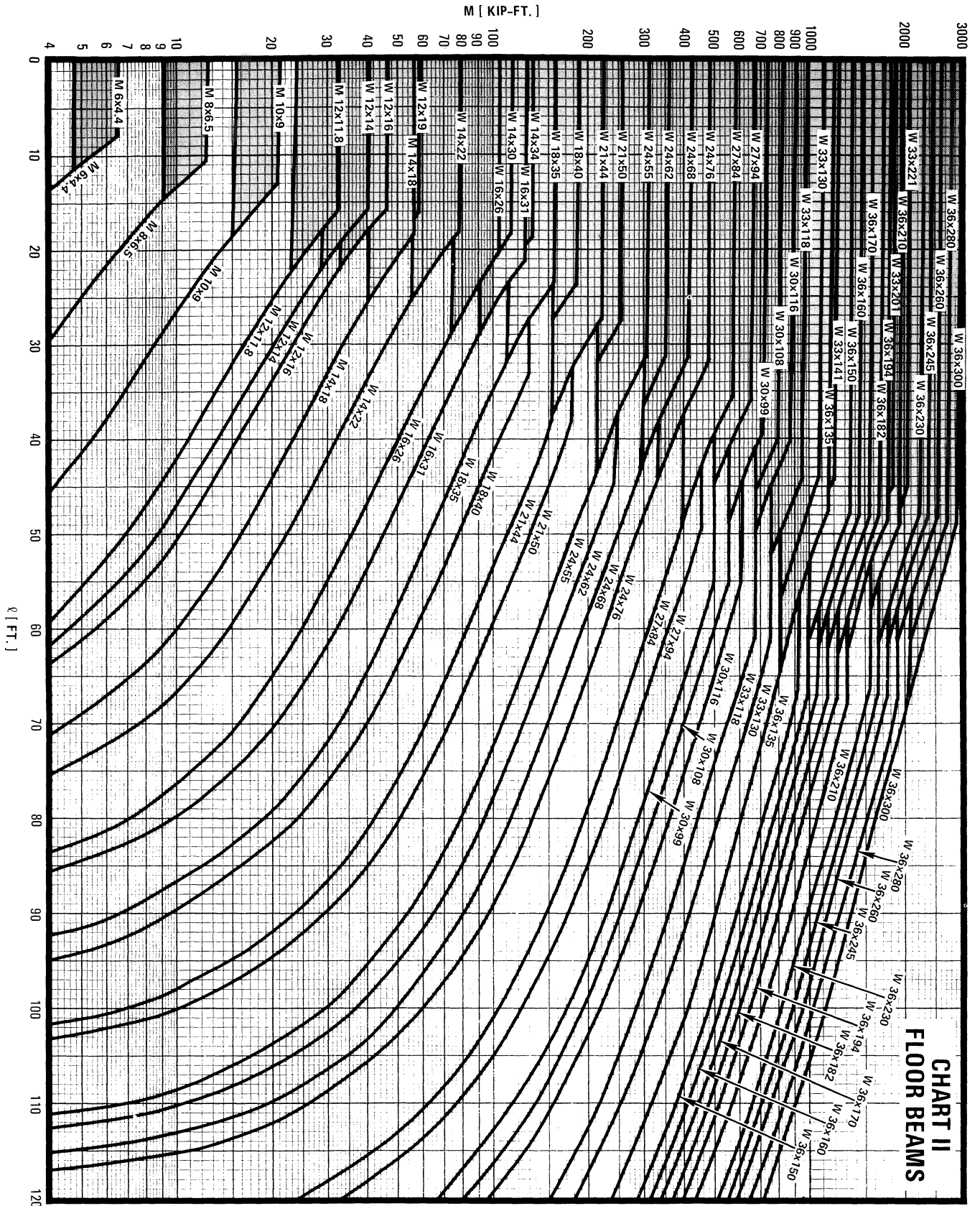


Chart II. Floor Beams

furnished by the two charts is superseded by procedures of determining the sources of damping and the selection of shapes with respect to the required frequency. In those cases, the design will lead mostly to the 36-ksi steel.

ACKNOWLEDGEMENT

The initial help and encouragement of Lucien G. Hersh and Kenneth Wagstaffe are gratefully acknowledged, as is the support of my employer, Bechtel Power Corporation. □

NOMENCLATURE

- E = modulus of elasticity of steel, 29,000 ksi
 F_b = allowable bending stress, ksi
 F'_b = reduced allowable bending stress (ksi) defined by Eqs. 5 and 6
 F_y = specified minimum yield stress of the type of steel being used, ksi
 I = moment of inertia of a section, in.⁴
 M = allowable bending moment in excess of that due to the weight of the shape, kip-ft
 M_t = Total bending moment capability of a beam, kip-ft
 S = Elastic section modulus, in.³

- d = Depth of a beam, in.
 f_b = Computed bending stress, ksi
 ℓ = Span length, ft
 p = Uniformly distributed equivalent of loading, kips per linear ft
 w = Nominal weight of a shape, lb./ft
 Δ = Actual deflection of the beam, in.
 Δ_a = Allowable deflection of the beam, in.

SUBSCRIPTS

- 1 = refers to the lighter beam
2 = refers to the heavier beam

REFERENCES

1. *American Institute of Steel Construction, Inc. Manual of Steel Construction 8th Ed., 1982, Chicago, Ill. (pp. 1-3, 2-6 through 2-9, 2-18, and 5-139).*
2. *Salmon C. G. and J. E. Johnson Steel Structures; Design and Behavior 2nd Ed., Harper & Row Publishers, New York, N.Y. (pp. 352).*