Proposed Criteria for Load and Resistance Factor Design

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The previous paper by Pinkham and Hansell outlined the basis and the background for Load and Resistance Factor Design (LRFD) criteria for steel building structures. The present paper describes a document entitled *Proposed Criteria for Load and Resistance Factor Design of Steel Building Structures.*[1] This document represents the end product of a research project conducted at the Civil Engineering Department of Washington University during the period 1969 through 1976. The project was sponsored by the Structural Steel and Steel Plate Producers Committees of the American Iron and Steel Institute, and it was guided by a Task Force chaired by Ivan M. Viest. The principal investigators were T. V. Galambos and M. K. Ravindra. The work involved the cooperation and help of many individuals and groups, and the proposed document, which is the subject of this paper, is the result of many iterations and trials. The design criteria have been tested in design office trials (see the paper by Wiesner, which follows in this issue).

DESCRIPTION OF LRFD CRITERIA

Reference 1 consists of two parts: Part 1—Criteria and Part 2—Commentary. Part 1 contains the detailed design provisions for the various strength limit states, such as overall and local instability, formation of a plastic mechanism, section plastification, yielding, and fracture. The Commentary (Part 2) contains guidelines for cases of loading not covered in the Criteria, and suggestions for designing for serviceability limit states. Since loads and load factors are treated differently in the LRFD method than in Allowable Stress Design, the Commentary has an expanded section on this subject. Enough of the background of the first order probability based method is presented to permit the determination of load factors and resistance factors for the unusual cases not covered in the Criteria (Part 1), provided the necessary statistical data are available or can be reliably estimated. The Commentary also contains charts, maps, and tables for use as design aids. For example, design aids are given for the determination of wind and snow loads, charts are provided for ponding loads, and tables are listed for the easy computation of the stability limit state resistance of columns and beams.

The proposed LRFD Criteria are not a complete set of structural steel design specifications which stand on their own; they are meant to be subordinate to many parts of the AISC Specification,[2] and they concern design for the same types of structures for which the present Specification applies, i.e., building structures fabricated from hot-rolled shapes and plates. This means that the criteria do not apply to structural components fabricated from cold-formed elements, nor do they concern the design of bridges, nuclear structures, etc. The applicable portions of the current AISC Specification are listed in Table 1. Most of these provisions are generally applicable, or they concern details which are the same regardless of the design methodology used.

The provisions in the AISC Specification which concern design criteria for strength limit states have been replaced in the LRFD Criteria, and Table 2 lists the sections which are superseded.

Table 1. Applicable Portions of AISC Specification

1.1	Plans and drawings	1.19	Camber
1.2	Types of construction	1.20	Expansion
1.4	Material	1.21	Column bases
1.7	Fatigue	1.22	Anchor bolts
1.12	Simple and continuous spans	1.23	Fabrication
1.14	Gross and net sections	1.24	Shop painting
1.15	Connections	1.25	Erection
1.16	Rivets and bolts	1.26	Quality control
1.17	Welds	2.2	Structural steel
1.18	Built-up members	2.10	Fabrication

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Table 2. Replaced Portions of AISC Specification

1.3	Loads and Forces	2.1	Scope
1.5	Allowable Stresses	2.3	Vertical Bracing System
1.6	Combined Stresses	2.4	Columns
1.8	Stability and Slenderness Ratios	2.5	Shear
1.9	Width-Thickness Ratios	2.6	Web Crippling
1.10	Plate Girders and Rolled Beams	2.7	Width-Thickness Ratios
1.11	Composite Construction	2.8	Connections
1.13	Deflection, Vibration, and Ponding	2.9	Lateral Bracing

The organization of the LRFD Criteria is given in Table 3. The arrangement and sequence of topics is different from the AISC Specification; the provisions are grouped according to structural components, i.e., members and connections. All criteria for one particular member type are located in one place. For example, Sect. 2.3.3, Flexural Members, contains design rules for web shear, web buckling, flexure, lateral bracing, width-thickness ratios, etc. The designer can thus find all the requirements for the design flexural members (e.g., rolled beams, plate girders, composite beams, etc.) in the same place. The organization of the rules was arranged in accordance with the guidance provided by the research of S. Fenves on the rational structuring of specifications.[3] This work also provided decision tables for further alleviating the burdens of the design engineer.

Table 3. Organization of the LRFD Criteria (Part 1)

Sect. 1. GENERAL PROVISIONS
- 1.1 SCOPE
- 1.2 DEFINITION OF LRFD
  - 1.2.1 Limit State: Strength
  - 1.2.2 Limit State: Serviceability
- 1.3 LOADS AND LOAD COMBINATIONS
  - 1.3.1 Load Types
  - 1.3.2 Load Combinations
  - 1.3.3 Load Factors for Strength Design

Sect. 2. DESIGN CRITERIA FOR THE LIMIT STATE OF STRENGTH
- 2.1 TYPES OF STRUCTURES
- 2.2 STRUCTURAL ANALYSIS
- 2.3 THE DESIGN OF MEMBERS
  - 2.3.1 Tension Members
  - 2.3.2 Compression Members
  - 2.3.3 Flexural Members
  - 2.3.4 Members Under Combined Flexure and Axial Force
  - 2.3.5 Members Under Combined Stress
- 2.4 THE DESIGN OF CONNECTIONS
  - 2.4.1 Definition
  - 2.4.2 Design of Connecting Elements
  - 2.4.3 Connectors
  - 2.4.4 Bearing Stresses on Contact Areas
- 2.5 FATIGUE

LOAD FACTORS

The design criterion is expressed by the following relationship:

$$\phi R_n \geq \gamma_o \sum_{i=1}^{n} \gamma_i c_i Q_i \qquad (1)$$

The various parts of this relationship have been discussed in the previous paper (by Pinkham and Hansell), where the probabilistic and statistical input for the determination of the resistance factor, ϕ, and the various load factors, γ, has also been described.* Only the operational application of the design criterion will be discussed here.

The left side of Eq. (1) concerns the resistance of the element, and the right side defines the load effects which must be resisted. A satisfactory design results when the resistance, modified by the resistance factor ϕ, is larger than the factored load effects. The load effects are the forces acting on the member due to the loading: the axial force, the bending moments, the shear force, or the torque. These are obtained from the loads or load intensities, Q, by structural analysis, and they are related to the loads Q through the influence coefficient, c. The loads may be due to various sources: dead loads, occupancy loads (designated as live loads), environmental loads, etc. Each type of load effect is multiplied by a given load factor, γ. This load factor reflects the uncertainties inherent in the determination of the loads and the load effects. The summed load effects are then multiplied by a load factor, γ_o, which takes care of the uncertainty inherent in structural analysis. The resistance factor ϕ accounts for the variation in the determination of the nominal resistance, R_n.

Following is an example of the application of Eq. (1) for a simply supported uniform "compact" beam subjected to a uniformly distributed dead and live load:

$$0.86 Z F_y \geq 1.1 \left[1.1 \frac{w_{DL} l^2}{8} + 1.4 \frac{w_{LL} l^2}{8} \right] \qquad (2)$$

In this equation, w_{DL} and w_{LL} are the mean distributed dead and live loads, respectively [Q_i in Eq. (1)]; $l^2/8$ is the influence factor by which load is transformed into bending moment [c_i in Eq. (1)], where l = span length, in.; the numbers 1.1 and 1.4 within the brackets are, respectively, the dead and the live load factors; the number 1.1 outside the brackets is the analysis factor, γ_o ; 0.86 is the resistance factor, ϕ; Z is the plastic section modulus; and F_y is the specified minimum yield stress.

For any given member there may be several load combinations which need to be checked. The main load combinations are summarized below:

Dead and Maximum Lifetime Live Load [$(DL) + (LL)$]:

$$1.1[1.1(DL) + 1.4(LL)] \qquad (3)$$

* *See also the report by Ravindra and Galambos[4] for a more detailed derivation.*

Dead plus Maximum Instantaneous Live plus Maximum Lifetime Wind Load $[(DL) + (LL)_I + W]$:

$$1.1[1.1(DL) + 2.0(LL)_I + 1.6W] \tag{4}$$

Dead plus Maximum Instantaneous Live plus Maximum Lifetime Snow Load $[(DL) + (LL)_I + S]$:

$$1.1[1.1(DL) + 2.0(LL)_I + 1.7S] \tag{5}$$

The LRFD Criteria and Commentary list the most common load factors and load combinations, and enough background is given to enable a design engineer to determine a load factor for unusual cases of loading. The load intensities to be used are all mean values, and the Commentary provides guidance on how these can be determined. A formula, based on research performed at MIT,[5] is provided for live load intensity for office occupancy:

Mean Maximum Lifetime Live Load:

$$(LL) = 14.9 + \frac{763}{\sqrt{A_I}} \leq 60 \text{ psf} \tag{6}$$

Mean Maximum Instantaneous Live Load: 12 psf

The term A_i in Eq. (6) is the influence area, and it is equal to twice the tributary area for beams and four times the tributary area for columns.

The Commentary also contains guidelines for other types of occupancy loads, for construction loads, etc., and it provides a method whereby wind loads given by ANSI can be transformed for use in the LRFD Criteria. Formulas, maps, and charts are also given for snow loads.

The treatment of loads and load factors is different from the AISC Specification in the following ways:

1. Mean loads are used instead of nominal loads
2. Instantaneous, annual, and lifetime loads are combined, rather than using only lifetime loads modified by a "probability" factor.[6]

Table 4 shows the comparison of the treatment of load combinations for LRFD, the new Canadian Limit States Criteria,[7] and the AISC Specification. The latter two specifications use the probability factor [0.7 on (LL) and W only in the Canadian specification, and 1/1.3 and 3/4 in the AISC plastic design and allowable stress design criteria, respectively], whereas the LRFD criteria combine the maximum lifetime wind with the instantaneous live load.

Table 4. Load Combinations: Wind and Gravity Loads

Specifications	Gravity	Wind and Gravity
LRFD[1]	$1.1\,[1.1(DL) + 1.4(LL)]$	$1.1\,[1.1(DL) + 2.0(LL)_I + 1.6W]$
Canadian Limit States[7]	$1.25(DL) + 1.5(LL)$	$1.25(DL) + 0.7[1.5(LL) + 1.5W]$
AISC (Part 2)[2]	$1.7[(DL) + (LL)]$	$1.3[(DL) + (LL) + W]$
AISC (Part 1)[2]	$\frac{5}{3}[(DL) + (LL)]$	$\frac{4}{3}[(DL) + (LL) + W]$

RESISTANCE FACTORS

While the Commentary on the LRFD Criteria concerns itself in great detail with loads, load factors, and load combinations, the design criteria (Part 1 of Ref. 1) deals mainly with the resistance side of Eq. (1), i.e., with the term ϕR_n. Formulas for the nominal resistance, R_n , and corresponding resistance factors ϕ are provided for tension, compression, and flexure members, for beam-columns, and for connectors. The resistance factor ϕ is based on the following formula:[4]

$$\phi = \frac{R_m}{R_n} e^{-0.55\beta V_R} \tag{7}$$

where R_m is the mean resistance, R_n is the nominal resistance represented by a specific formula in the LRFD Criteria, β is the safety index, and V_R is the coefficient of variation of the resistance. The statistical properties of the resistance, R_m and V_R , depend on the material properties, the dimensional tolerances, and on the theory used to predict the ultimate capacity of the member or component. In the course of the research project, all the available relevant research results were examined, and the collected data was catalogued in a series of reports (Ref. 4, and Refs. 8 through 12). The safety index, β, is a measure of the reliability of the member design, and it was developed from a first-order probabilistic model due to C. A. Cornell[13] and from calibration,[4] as discussed in the preceding paper by Pinkham and Hansell. The following values of β were selected for use with LRFD Criteria: $\beta = 3.0$ for members and $\beta = 4.5$ for connections. The larger value was chosen for connections, so that connection performance is made more reliable in accordance with established design custom.

Various resistance factors from the proposed LRFD Criteria are listed in Table 5 for members and connections. It should be realized that these factors represent values from research results and that they have not yet been rounded off. It may well be that the AISC Specification Advisory Committee will consider round-offs and unified groupings, and so these factors should not be taken as final values.

Table 5. Resistance Factors

Member	Limit State	ϕ
Tension member	Yield in net section	0.88
Tension member	Fracture in net section	0.74
Column	Buckling	0.86–0.65
Beam, plate girder	Maximum moment capacity	0.86
Plate girder	Maximum shear capacity	0.86
Composite beam	Maximum moment capacity	0.84
Beam-column	Instability	0.86
Fillet welds	Shear	0.80
A325 h.s. bolts	Shear	0.86
A490 h.s. bolts	Shear	0.82

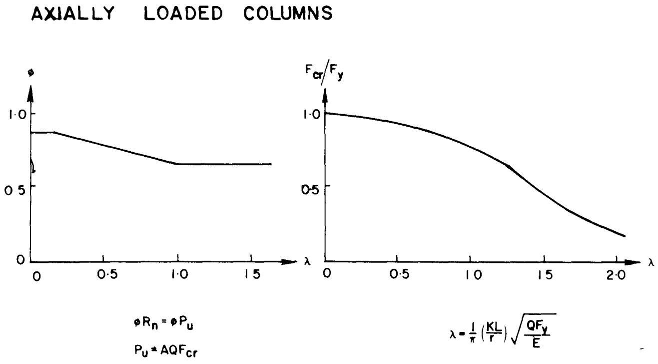

Fig. 1. Variation of the resistance factor ϕ and the critical stress with the slenderness parameter λ for axially loaded columns

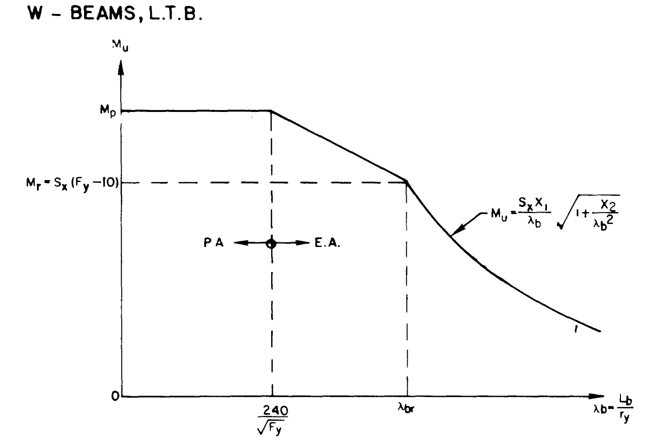

Fig. 3. Design criteria for wide-flange beams, limit state lateral-torsional buckling

EXAMPLES FROM THE LRFD CRITERIA

The application of the proposed LRFD Criteria is best illustrated by several examples.

The first example (Appendix A) shows the application of the new method to the design of a compact two-span continuous beam. Column design is illustrated in Fig. 1 and in Appendix B. The curves in Fig. 1 show the variation of the resistance factor and the critical stress with the slenderness parameter λ, and the details of the calculation are given in Appendix B. The basic column curve used in the LRFD Criteria is the same as that which serves as the basis for column design in the 1977 AISC Specification.

While compact rolled beam and column design is quite comparable in the AISC Specification and in the proposed LRFD Criteria, the design of beams for the stability limit states has been streamlined. Three stability limit states are accounted for: (1) lateral-torsional buckling, LTB, (2) flange local buckling, FLB, and (3) web local buckling, WLB. The maximum-moment-versus-slenderness domain (Fig. 2) is divided into three ranges: (1) plastic range, where the maximum flexural capacity equals the plastic moment M_p, (2) the inelastic range, where buckling takes place after some yielding, and (3) the elastic range, where the elastic buckling governs. A uniform resistance factor, $\phi = 0.86$, applies to beams, and the slenderness parameter λ depends on the type of stability limit state. A straight-line transition is used from the elastic to the plastic range (Fig. 2). The moment-capacity-versus-slenderness parameter curves for the three types of stability limit states are presented in Figs. 3 through 5. In Fig. 3 the slenderness parameter for the stability limit state of lateral-torsional buckling is the minor axis slenderness ratio of the unbraced length. The elastic buckling formula is a function of the elastic section modulus S_x, the slenderness ratio λ_b, and the cross-sectional constants X_1 and X_2, which are tabulated in the Commentary, Table C2333-1.[1] The end of the

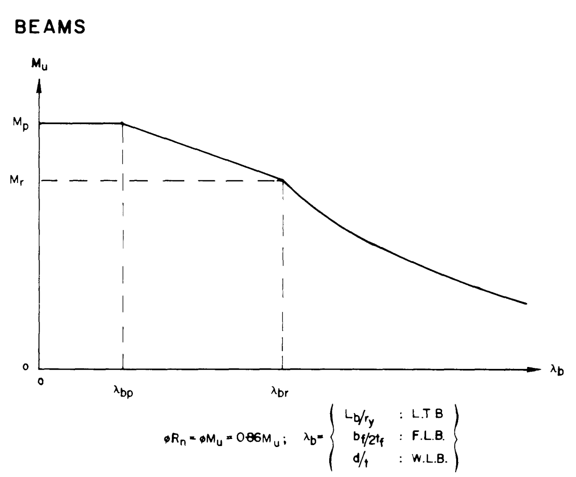

Fig. 2. Beam design criteria

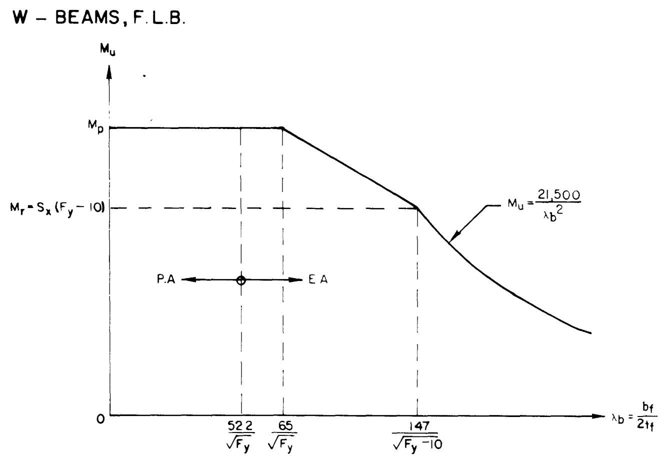

Fig. 4. Design criteria for wide-flange beams, limit state flange buckling

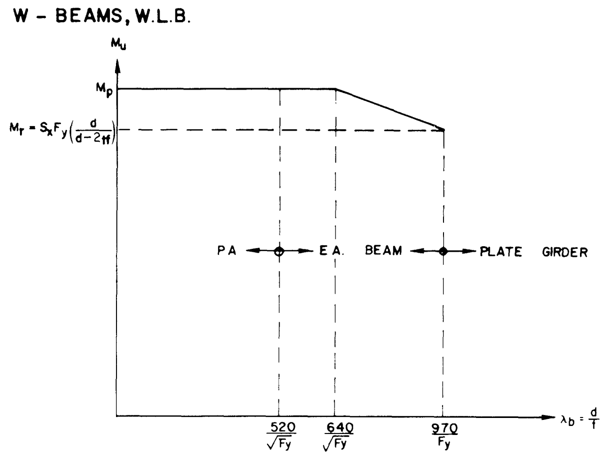

Fig. 5. Design criteria for wide-flange beams, limit state web local buckling

elastic range occurs when the elastic flexural stress is equal to $(F_y - 10)$, where 10 ksi is the mean residual stress in the compressed flange. The limit of plastic behavior corresponds essentially to the same limit as in Part 2 of the AISC Specification, and plastic analysis (P.A.) may be used in this region. Elastic analysis (E.A. in Fig. 3) must be used in the inelastic and the elastic regions for the determination of the moment distribution in indeterminate beams. The curves in Figs. 4 and 5 show the corresponding regions for the limit states of flange and web local buckling, FLB and WLB, respectively. Both of these curves show that a dual limit exists for the plastic range: one limit up to which plastic analysis may be used, and another more liberal limit up to which the capacity is M_p. This duality is carried over from the AISC Specification, where a similar situation exists. No curve is shown in Fig. 5 for the elastic range, because beyond $d/t = 970/\sqrt{F_y}$ the plate girder rules apply.

Formulas for the maximum moment capacity and the various limiting slenderness parameters are tabulated in the LRFD Criteria[1] for easy reference for the following cross sections: doubly symmetric W-shapes, singly symmetric I-shapes, channels, tees, box beams, solid bars, and hybrid sections.

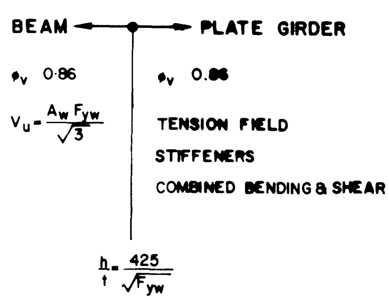

Fig. 6. Design criteia for beam webs in shear

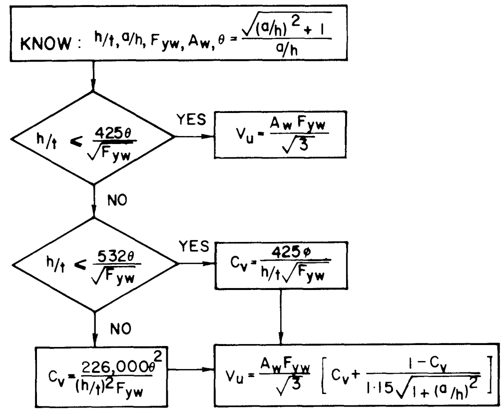

Fig. 7. Design criteria for plate girder webs in shear

The design for the shear capacity of plate girders is illustrated in Figs. 6 and 7. It is shown in Fig. 6 that the limiting web slenderness separating beam behavior in shear (limit state: full web yielding) and plate girder behavior (transverse stiffeners are needed, tension-field action may be included, and combined bending and shear check must be made) is $h/t = 425/\sqrt{F_{yw}}$. A different resistance factor, ϕ_v, applied for beam and plate girder webs. The flow diagram of Fig. 7 illustrates the determination of the maximum shear capacity, V_u.

A complete design example for a plate girder (Appendix C) illustrates the application of the LRFD Criteria discussed above.

SUMMARY

This paper is intended to give a very brief review of the *Proposed Criteria for Load and Resistance Factor Design Criteria of Steel Building Structures.*[1] The criteria and the supporting papers (Ref. 4 and Refs. 8 through 12) contain much additional material, not discussed here, on beam-columns, composite beams, and connections. Details of this work will appear in future publications and lectures.

The main differences between the proposed LRFD Criteria and the current AISC Specification are in the format (resistance factors and load factors, load combinations) and in the organization of the contents. The LRFD Criteria are ordered according to component design. Furthermore, the LRFD Criteria treat cases explicitly (e.g., channels, etc.) and the designer can find all the required rules for one design situation in the same place.

The advantages of LRFD are:

1. It is a rational and consistent design methodology.
2. It can be updated and modified as more data becomes available from research.
3. It may result in more economical designs, especially when dead load dominates.
4. It permits an integrated treatment of loads and load effects where two material specifications interface.

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NOTATION

A_F = Flange area
A_I = Influence area
A_w = Web area
a = Stiffener spacing
b_f = Flange width
c_i, c = Influence coefficient
C_v = Factor defined in Fig. 9
(DL) = Mean dead load
e = Eccentricity
E = Modulus of elasticity
F_{cr} = Critical stress
F_y = Minimum specified yield stress
F_{yw} = F_y of web
h = Web depth
I_x = Moment of inertia about x-axis
K = Effective length factor
L = Span length, ft
L_b = Unbraced length, ft
(LL) = Mean maximum lifetime live load
$(LL)_I$ = Mean maximum instantaneous live load
l = Span length, in.
M_D = Factored design moment
M_p = Plastic moment
M_u = Maximum moment capacity
Q_i, Q = Load effect
r = Radius of gyration
R_m = Mean resistance
R_n = Nominal resistance
S_x = Elastic section modulus
S = Mean maximum lifetime snow load
t, t_f = Web and flange thickness, respectively
V_R = Coefficient of variation of resistance
V_u = Maximum shear capacity of web
w = Mean distributed load
W = Mean maximum lifetime wind load
Z = Plastic section modulus
β = Safety index
θ = Factor defined in Fig. 9
ϕ = Resistance factor
γ = Load factor
λ = Slenderness parameter

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APPENDIX A: BEAM DESIGN EXAMPLE

Given:

Design a 2-span continuous beam in an office building. See Fig. A1.

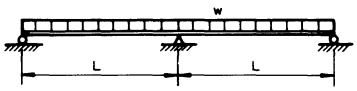

Figure A1

Spans: $L = 30'\text{-}0''$
Beam spacing: $20'\text{-}0''$
Joist spacing: $2'\text{-}0''$ (= spacing of braced points along beam)
Loading:

Mean uniformly distributed dead load; including weight of beam, w_b: $(DL) = 1.20$ kips/ft

Tributary area: $A_{trib} = 30 \times 20 = 600$ sq ft/span

Influence area: $A_I = 2A_{trib} = 1200$ sq ft/span

Mean office live load intensity:

$$(LL) = 14.9 + 763/\sqrt{A_I} = 14.9 + 763/\sqrt{1200}$$
$$= 37 \text{ lbs/sq ft} = 0.037 \text{ kips/sq ft}$$

Uniformly distributed live load:
$20 \times 0.037 = 0.74$ kips/ft

Construction load (acting before joists provide lateral bracing): 5 kips concentrated load at center of each span, plus weight of beam, w_b, between supports

Material: A36 steel, $F_y = 36$ ksi

Solution:

A. Design for maximum capacity:

Use plastic design. For a mechanism to form in each span and at the interior support:

$$M_{p(req)} = wL^2/11.66$$
$$= (L^2/11.66)[\gamma_o(\gamma_{DL}w_{DL} + \gamma_{LL}w_{LL})]$$
$$= \frac{30^2 \times 12}{11.66}[1.1(1.1 \times 1.20 + 1.4 \times 0.74)]$$
$$= 2400 \text{ kip-in.}$$

LRFD design criterion (Sect. 2.3.3 of Ref. 1):

$$\phi Z F_y \geq M_{p(req)}$$
$$\phi = 0.86$$
$$Z_{req} = \frac{2400}{0.86 \times 36} = 77.5 \text{ in.}^3$$

Try W18 $\times$ 40, $Z = 78.4$ in.^{3}

Verify limit state $M_u = M_p$:

1. LTB (lateral torsional buckling):
   $L_b = 2$ ft–0 in. = 24 in.
   $L_b/r_y = 24/1.27 = 18.9$
   Plastic action, $\lambda_p = 240/\sqrt{F_y}$
   $= 40.0 > 18.9$ **o.k.**

2. FLB (flange local buckling):
   $b_f/2t_f = 6.015/(2 \times 0.525) = 5.73$ (use new 1978 section properties)
   Plastic action, $\lambda_p = 52.2/\sqrt{F_y}$
   $= 8.70 > 5.73$ **o.k.**

3. WLB (web local buckling):
   $d/t = 17.9/0.315 = 56.8$
   Plastic action, $\lambda_p = 520/\sqrt{F_y}$
   $= 86.7 > 56.8$ **o.k.**

Therefore, $M_u = M_p$ **o.k.**

B. Check construction load (see Fig. A2):

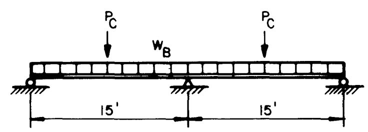

Figure A2

Concentrated load: $P_C = 5$ kips; $\gamma_C = 1.4$
Weight of beam: $w_b = 0.040$ kips/ft; $\gamma_{DL} = 1.1$
M_{max} occurs in interior of span under P_C:

$$M_{max} = 1.1[1.4(5P_CL/32) + 1.1(w_bL^2/16)]$$
$$= 1.1\left[\frac{1.4 \times 5 \times 5 \times 30 \times 12}{32} + \frac{1.1 \times 0.04 \times 30^2 \times 12}{16}\right]$$
$$= 466 \text{ kip-in.}$$

Moment capacity, M_u:

When $\lambda_b = L_b/r_y > \lambda_{br}$ or, equivalently, $M_u < M_r = S_x(F_y - 10)$:

$$M_u = S_xF_{cr}$$
$$F_{cr} = \frac{C_bX_1}{\lambda_b}\sqrt{1 + \frac{X_2}{\lambda_b{}^2}}$$

$C_b = 1.0$ (conservative)
$L_b = 30$ ft = 360 in.
$\lambda_b = L_b/r_y = 360/1.27 = 283$
$X_1 = 2552$ (Table C2.3.3.3-1, Ref. 1)

$X_2 = 28{,}260$ (Table C2.3.3.3-1, Ref. 1)

$$F_{cr} = \frac{1 \times 2552}{283}\sqrt{1 + \frac{28{,}260}{(283)^2}}$$
$$= 10.49 \text{ ksi}$$

$M_u = 68.4 \times 10.49 = 717$ kip-in.

$M_r = 68.4(36 - 10)$
$= 1778$ kip-in. > 717 kip-in.

$\phi M_u = 0.86 \times 717$
$= 617$ kip-in. $> M_{max} = 466$ kip-in. **o.k.**

$\therefore$ Use W18 $\times$ 40

APPENDIX B: COLUMN DESIGN EXAMPLE

Given:

Design a pin-ended column in a braced frame.

Length: 15′-0″

Axial loads: Mean dead load = 200 kips
Mean live load = 250 kips

Material: A36 steel, $F_y = 36$ ksi, $E = 29{,}000$ ksi

Solution:

Factored axial load:

$P = 1.1[(1.1 \times 200) + (1.4 \times 250)] = 627$ kips

$P_u = AF_{cr}$

$F_{cr} = F_y(1 - 0.25\lambda^2)$ for $\lambda \leq \sqrt{2}$
$= F_y/\lambda^2$ for $\lambda \geq \sqrt{2}$

$$\lambda = \frac{KL}{r}\left(\frac{1}{\pi}\sqrt{\frac{F_y}{E}}\right)$$

Design Criterion (Sect. 2.3.1.1 of Ref. 1):

$\phi P_u \geq P$

$\phi = 0.86$ for $\lambda \leq 0.16$
$= (0.90 - 0.25\lambda)$ for $0.16 < \lambda \leqslant 1.0$
$= 0.65$ for $\lambda > 1.0$

Try W12 $\times$ 96 (new 1978 section):

$A = 28.2$ in.^{2}, $r_y = 3.09$ in.

$K = 1.0$

$$\lambda = \frac{1 \times 15 \times 12}{3.09\pi}\sqrt{\frac{36}{29{,}000}} = 0.653 < \sqrt{2}$$

$F_{cr} = F_y(1 - 0.25\lambda^2) = 32.16$ ksi

$\phi = 0.90 - 0.25\lambda = 0.74$

$P_u = 0.74 \times 28.2 \times 32.2 = 672$ kips > 627 kips **o.k.**

Similarly, for W12 $\times$ 87, $P_u = 608$ kips < 627 kips **n.g.**

$\therefore$ Use W12 $\times$ 96

APPENDIX C: PLATE GIRDER DESIGN EXAMPLE

Given:

Design a simply supported plate girder. See Fig. C1.

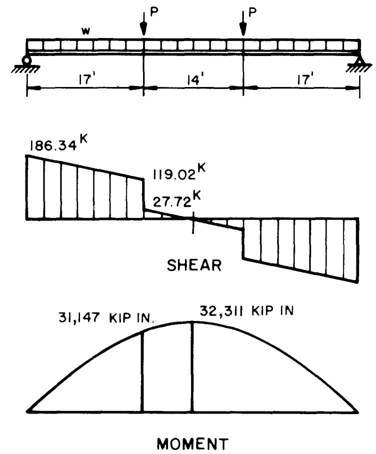

Figure C1

Material: A36 steel, $F_y = 63$ ksi

Max. depth: 72 in.

Loading:

$w = w_{DL} + w_{LL}$
Mean values: $w_{DL} = 2.0$ kips/ft
$w_{LL} = 1.0$ kips/ft

$P = P_{DL} + P_{LL}$
Mean values: $P_{DL} = 50$ kips, $P_{LL} = 20$ kips

Solution:

Factored loads:

$w = \gamma_o(\gamma_{DL}w_{DL} + \gamma_{LL}w_{LL})$
$= 1.1[(1.1 \times 2.0) + (1.4 \times 1.0)]$
$= 3.96$ kips/ft

$P = \gamma_o(\gamma_{DL}P_{DL} + \gamma_{LL}P_{LL})$
$= 1.1[(1.1 \times 50) + (1.4 \times 20)]$
$= 91.3$ kips

Trial section:
Web: $\frac{5}{16}$-in. $\times$ 70 in.
Flanges: $\frac{3}{4}$-in $\times$ 18.5 in.
Web area: $A_w = 21.88$ in.^{2}
One flange area: $A_f = 13.88$ in.^{2}
$I_x = 43{,}660$ in.^{4}, $S_x = I_x/35.75 = 1221$ in.^{3}
$b_f/2t_f = 12.33$, $h/t = 12.33$

Consider a tee section made of one flange plus $\frac{1}{6}$ of the web:

$I_T \approx I_y$ of the flange
$= 0.75 \times 18.5^3/12 = 396$ in.^{4}

$$r_T = \sqrt{\frac{396}{A_f + A_w/6}} = \sqrt{\frac{396}{13.88 + (21.88/6)}}$$
$$= 4.75 \text{ in.}$$

Assume lateral bracing at points of support and under the concentrated loads only:

$$L_b/r_T = 17 \times 12/4.75$$
$$= 42.9 \text{ for exterior unsupported span}$$
$$L_b/r_T = 14 \times 12/4.75$$
$$= 35.4 \text{ for interior unsupported span}$$

Design for flexure (Sect. 2.3.3.3.2 of Ref. 1):

$$\text{Web slenderness reqd.} = \left(\frac{h}{t}\right)_{req} \leq \left(\frac{h}{t}\right) \leq \left(\frac{h}{t}\right)_{max}$$

$$\left(\frac{h}{t}\right)_{req} = \frac{970}{\sqrt{F_y}} = 161.7 < 224 \textbf{ o.k.}$$

$$\left(\frac{h}{t}\right)_{max} = \frac{14{,}000}{\sqrt{F_y(F_y + 16.5)}} = 322 \text{ for } a/h \geq 1.5$$

where a = transverse stiffener spacing

$$\left(\frac{h}{t}\right)_{max} = 322 > 224 \therefore \text{ web slenderness } \textbf{o.k.}$$

Maximum moment capacity:

$$M_u = S_x R_{PG} (F_{cr})_b$$

$$R_{PG} = 1.0 - 0.0005 \left(\frac{A_w}{A_f}\right) \left(\frac{h}{t} - \frac{970}{\sqrt{(F_{cr})_b}}\right)$$

$$(F_{cr})_b = F_y \text{ for } \lambda_b \leq \lambda_{bp}$$
$$= F_y \left[1 - \frac{0.5(\lambda_b - \lambda_{bp})}{\lambda_{br} - \lambda_{bp}}\right] \text{ for } \lambda_{bp} \leq \lambda_b \leq \lambda_{bp}$$
$$= \frac{C_{PG}}{\lambda_b{}^2} \text{ for } \lambda_b \geq \lambda_{br}$$

Limit state LTB:

$$\lambda_b = L_b/r_T; \quad \lambda_{bp} = \frac{146}{\sqrt{F_y}}; \quad \lambda_{br} = \frac{757\sqrt{C_b}}{\sqrt{F_y}}$$

$$C_{PG} = 286{,}000\, C_b$$

Exterior span:

$$L_b/r_T = 42.9; \quad C_b = 1.75$$
$$\lambda_{bp} = 24.3; \quad \lambda_{br} = 166.9 \therefore \lambda_{bp} \leq \lambda_b \leq \lambda_{br}$$
$$(F_{cr})_b = 36 \left[1 - \frac{0.5(42.9 - 24.3)}{166.9 - 24.3}\right] = 33.65 \text{ ksi}$$

Interior span:

$$L_b/r_T = 35.3; \quad C_b = 1.0$$
$$\lambda_{bp} = 24.3; \quad \lambda_{br} = 126.2 \therefore \lambda_{bp} \leq \lambda_b \leq \lambda_{br}$$
$$(F_{cr})_b = 36 \left[1 - \frac{0.5(35.3 - 24.3)}{126.2 - 24.3}\right] = 34.06 \text{ ksi}$$

Limit state FLB:

$$\lambda_b = b_f/2t_f = 12.33; \quad \lambda_{bp} = 52.2/\sqrt{F_y} = 8.7$$
$$\lambda_{br} = 149/\sqrt{F_y} = 24.8; \quad C_{PG} = 11{,}140$$
$$\therefore \lambda_{bp} \leq \lambda_b \leq \lambda_{br}$$
$$(F_{cr})_b = 36 \left[1 - \frac{0.5(12.33 - 8.7)}{24.8 - 8.7}\right]$$
$$= 31.94 \text{ ksi} \quad \text{(controls)}$$

$$R_{PG} = 1 - 0.0005 \left(\frac{21.88}{13.88}\right) \left(224 - \frac{970}{\sqrt{31.94}}\right)$$
$$= 0.959$$

$$M_u = 1221 \times 0.959 \times 31.94 = 37{,}400 \text{ kip-in.}$$

Design criterion:

$$\phi M_u \geq M_{max}; \quad \phi = 0.86$$
$$0.86 \times 37{,}400 = 32{,}164 \text{ kip-in.}$$

M_{max} = 32,311 kip-in. **o.k.** (within 1%)

Design for shear (Sect. 2.3.3.2.2 in Ref. 1):

$\phi = 0.86$

End panel:

Assume stiffener at a = 42 in.

$$\left(\frac{a}{h}\right) = \frac{42}{70} = 0.60$$

$$\theta = \frac{\sqrt{(a/h)^2 + 1}}{(a/h)} = 1.94$$

$$\text{Max. shear capacity:} \quad V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_y C_v$$

$$C_v = \frac{425\theta}{(h/t)\sqrt{F_y}} \text{ for } \frac{425\theta}{\sqrt{F_y}} \leq \left(\frac{h}{t}\right) \leq \frac{532\theta}{\sqrt{F_y}}$$
$$= \frac{226{,}000\theta^2}{(h/t)^2 F_y} \text{ for } \left(\frac{h}{t}\right) \geq \frac{532\theta}{\sqrt{F_y}}$$

$$(h/t) = 224 > \frac{532\theta}{\sqrt{F_y}} = 172$$

$$\therefore C_v = \frac{226{,}000 \times 1.94^2}{224^2 \times 36} = 0.471$$

$$V_u = \left(\frac{1}{\sqrt{3}}\right) 21.88 \times 36 \times 0.471 = 214 \text{ kips}$$

Design criterion:

$$\phi V_u \geq V_{max}$$
$$\phi V_u = 0.86 \times 214 = 184 \text{ kips}$$

V_{max} = 186 kips **o.k.** (approx. 1%)

Interior panels:

Assume stiffeners at a = 81 in.

$$\left(\frac{a}{h}\right) = \frac{81}{70} = 1.16; \quad \theta = \frac{\sqrt{(a/h)^2 + 1}}{(a + h)} = 1.32$$

$$\frac{5320}{\sqrt{F_y}} = 117 < \left(\frac{h}{t}\right) = 224$$

$$C_v = \frac{226{,}000 \times 1.32^2}{224^2 \times 36} = 0.218$$

$$V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_y \left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right]$$

$$= 301 \text{ kips}$$

Design criterion:

$\phi V_u \geq V_{max}$

$(0.86)(301) = 259$ kips $>$ 186 kips **o.k.**

Space stiffeners as shown in Fig. C2.

Additional design steps not shown:

1. $M - V$ interaction check: **o.k.**

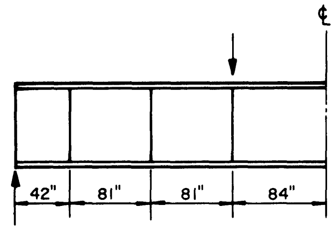

Figure C2

2. Transverse stiffener design:
   Use $\frac{1}{2} \times 6$ PL on one side only.
3. Bearing stiffener design at support and under load:
   Use two $\frac{3}{8} \times 5$ PL on both sides of web.